LOAD BEARING BRICKWORK WALLS

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by

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LOAD BEARING BRICKWORK WALLS

ABSTRACT

The results of tests on sixteen storey height walls 4\frac{1}{2} ins thick loaded between reinforced concrete slabs to investigate the effects of brick and mortar strength, eccentric loading, rate of loading, strain distribution, secant modulus and the strength/age relationship are given and compared with the Code of Practice C.P.111 (1964).

A review of previous work on single leaf storey height walls is given together with a review of previous work on cavity walls with particular reference to the distribution of load and bending moment at the slab/wall junction.

The results of preliminary tests on two cavity walls carried out in a 600 ton testing frame are included and also the results of tests on brickwork cubes and piers to investigate the effect of the brickwork strength of mortar joint thickness.

From the test results and the literature review it would appear that the C.P.111 (1964) requires revision. The typical compressive failure of brickwork was by transverse splitting and the tensile strength of the brick and Young's modulus and Poisson's ratio of the mortar would appear to be of primary importance in determining the strength of brickwork. The tests on brickwork cubes and piers revealed a decrease in strength of 23\% when the mortar joint thickness was increased from \frac{2}{8} ins to 1 ins. An increase of 15\% was observed when the thickness was reduced from \frac{2}{8} ins to \frac{1}{4} ins. The load factors for the single leaf wall tests based on the code of Practice C.P.111 (1964) ranged between 6.3 and 12.3. For walls loaded between reinforced concrete floor slabs the reduction in strength of eccentrically loaded walls compared with those axially loaded was considerably less than
when loaded between knife edges and it appears that the bending moment due to eccentric loads is distributed to the wall and floor slabs in proportion to their stiffnesses.

Walls loaded over $1\frac{1}{2}$ hours failed at loads approximately 20% less than similar walls loaded over $\frac{1}{2}$ hour.

The C.P.111 (1964) stress reduction factors for eccentric loading are not supported by the results of wall tests and more conservative values would seem desirable.

The design and construction of a 600 ton multipurpose testing frame is given in Appendix 1, and a comparison of compression testing machines and test specimens given in Appendix 3. The results of individual brick crushing tests in three different machines are given in Appendix 2.
CHAPTER 1
INTRODUCTION

1.1 General

The use of load-bearing brickwork for structures is not new. The Babylonians and Assyrians built brick structures over 4,000 years ago and the elliptic brick vaults of Ctesiphon and the domes of St. Sophia are 1,400 years old. The roof and central stone lantern of St. Paul's Cathedral (1710) are supported on a brickwork cone. The Chicago Monadnock Building in America 1 completed in 1891 is 197 ft in height (16 storeys), of load-bearing brickwork construction (Fig. 1.1) and bears a plaque which commences with the sentence "The final triumph of traditional masonry construction". The walls were 12 ins thick in the uppermost storey and increased by 4 ins for every storey below the top, resulting in walls about 6 ft thick at the base. These buildings were not designed on a stress basis but intuitively, by trial and error or by rule of thumb. These design methods are unscientific, nearly always inaccurate and uneconomic, and sometimes dangerous.

Loading tests on brickwork piers were first carried out in America at the Watertown Arsenal in 1882, and brickwork testing was continued in that country until the mid 1930's with particular references to reinforced brickwork. 2

Extensive testing was carried out in Britain over the same period and culminated in the Code of Practice 3 C.P.111 being issued in 1948, A paper 4 summarising this research was published in 1950. It was now possible to design load-bearing brickwork on a stress basis which for normal domestic building such as housing, hostels, hotels, etc. where the
floor plan repeats at each floor level, provides a cheaper structure than steel or concrete frame construction which would in any case require brickwork party walls for fire protection and sound insulation.

This structural and economic potential of calculated brickwork remained virtually untapped in Britain, construction in load-bearing brickwork being limited almost entirely to 3, 4 and 5 storey buildings, until 1960 when a 12 storey block of flats at Birmingham was erected.

In Switzerland over 1600 wall tests have been carried out since 1946 at E.M.P.A. (Swiss Federal Material Testing and Research Institute) under the direction of Haller and the results of the first investigations were the basis for the design and construction of three 13 storey blocks of flats at Basle from 1951-1953. The construction of other blocks followed and in 1957 the tallest load-bearing brickwork building in the world was commenced at Schwamendingen near Zurich having 17 storeys in brickwork and the lowest storey in reinforced concrete. The external walls are 12ins to 15 ins thick and internal walls 5 ins to 10 ins. A plan and elevation are given in Figs. 1.2 and 1.3.

The impact on the British architectural and engineering professions of the tall Swiss Buildings and the 12 storey Birmingham block has been considerable over the last 4 to 5 years and there are now more than a dozen point blocks over 10 storeys in height in this country either completed or under construction. The tallest is 14 storeys at the Essex University site; a plan and elevation of which we show in Figs. 1.4 and 1.5.

1.2 Scope of Investigation

Advancement to greater heights and higher design stresses is held
back by lack of design information and load factors at present are about 6 for 9 ins walls and between 6 and 12 for 4½ ins walls when designed to C.P.111 (1964). These relatively high load factors include a large "factor of ignorance" which may be reduced as additional design information becomes available on the following:

i) design of axially and eccentrically loaded walls,

ii) Assessment of the structural interaction at the floor slab/wall junction,

iii) Distribution of load and bending moments into cavity walls.

iv) Influence on load distribution into the two leaves of a cavity wall when constructed of different strength bricks and/or mortar and/or materials.

v) Behaviour of brickwork subjected to concentrated loads.

vi) Frame action of slab and shear wall structures.

vii) Increase in strength of brickwork with age.

viii) Influence on brickwork strength of mortar properties.

This thesis is concerned with items i), ii) and iii). Details are given in Chapter 3 of a series of tests on single leaf storey height walls 4½ ins thick loaded between 4 ins concrete floor slabs and a review of relevant literature is included in Chapter 2. The design and specification of a 600 ton testing frame are given in Appendix 1 and the results of preliminary 10½ ins cavity wall tests in the 600 ton frame are given in Chapter 3.

A review of previous work on cavity walls is given in Chapter 4 and theory and design methods for brickwork are considered in Chapter 6.

Details are given in Appendices 2 and 3 of two series of tests to investigate the variations in crushing machines and also the influence of lubrication of the ball seat on the crushing strength of specimens.
FIG. 1.2 - Plan of Zurich Building - 17+1 storeys

FIG. 1.3 - Elevation of Zurich Building
FIG. 1.4 - Plan of Essex towers - 14 storeys

FIG. 1.5 - Elevation of Essex tower - Artist's impression.
CHAPTER 2

REVIEW OF PREVIOUS WORK ON SINGLE LEAF STOREY HEIGHT PANELS

2.1 General

The general influence on brickwork strength of brick and mortar strength is well known\textsuperscript{4, 8, 9, 10} and is summarised in Fig. 2.1. It is seen that brickwork strength increases as brick and mortar strength increase although not in direct proportion.

Failure of brickwork wall panels is normally by transverse splitting caused by the lateral strain of the mortar under load inducing horizontal tension in the brickwork. Post war research has revealed the relative importance of additional factors\textsuperscript{9} such as mortar properties, brick suction, etc. influencing the strength of brickwork which supplement the general relationship shown in Fig. 2.1.

2.2 Brick

Earlier investigators have endeavoured to relate brick compressive strength to wall strength for a given mortar in terms of the ratio of wall compressive strength to brick compressive strength. These ratios range between 10% and 50% for clay bricks. It would seem however, that since failure of brickwork is normally by transverse splitting a relationship between brick tensile strength and wall strength might be more reliable.

With many types of solid brick there is most likely a consistent relationship between the tensile and compressive strength of the bricks - the ratio of tensile to compressive strength being approximately 1 to 10. For perforated bricks, however, the tensile strength will be related to the minimum area of solid material through the brick cross-section and the
ratio of tensile to compressive strength might be higher than 1 to 15. Deeply frogged bricks may have a similar strength ratio.

The mode of failure of brickwork constructed of deep frogged bricks may differ from that of normal brickwork because of the large volume of mortar present. When the bricks have a solid mortar bedding at the edges only, due to the deep frog and furrowed bed joint, then crushing at the edges will occur before transverse splitting.

This edge bedding would be similar to when the bricks are laid frog down and in one series of tests the walls and piers with bricks laid frog down were approximately 30% weaker than when the bricks were laid frog up.

Haller has shown that the suction of the brick when laid can considerably influence the strength of brickwork especially when dense cement mortars are used such as a 1:3 cement/sand mix. Some results are given in Fig. 2.2 and it can be seen that there is a reduction in strength with increase in suction. This is most likely due to the high suction bricks absorbing an excess of water from the mortar and so preventing complete hydration of the cement. When cement/lime/sand mortars are used the loss in strength associated with high suction bricks is less because of the water retaining property of the lime.

Reductions in brickwork strength can result from variations in brick dimensions. For example the mortar bed joints will be thicker under certain bricks when bricks having varying depths are used and this may result in lower brickwork strengths as discussed in section 2.4. Variations of depth in individual bricks can result in eccentric loading as also can variations in length and width.
2.3 Mortar

It can be seen from Fig. 2.1 that the strength of brickwork increases with increase in mortar strength although not in direct proportion.

The paste(or cementitious material) to sand ratio to approximately fill the voids and to hence give optimum strength has been established as 1:3 but little work has been carried out on the influence on mortar strength of sand grading. In fact the recommended sand gradings for mortars in B.S.1200 cover a very wide range (Table 2.1). Stedham has observed the increase in mortar strength when a coarse sand is used and Monk has noted that some degree of optimum packing is possible through sand gradation and that the lowest void volume achieved experimentally is about 16.8% using a mixture of particles as follows:

<table>
<thead>
<tr>
<th>% of total volume</th>
<th>Relative diameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse 70</td>
<td>50.5</td>
</tr>
<tr>
<td>Medium 20</td>
<td>8.0</td>
</tr>
<tr>
<td>Fine 10</td>
<td>1.0</td>
</tr>
</tbody>
</table>

Sands for general purpose mortars have the volumes and relative diameters shown in Table 2.1 and it can be seen that the normal building sands for mortar have a particle size distribution considerably different to that required for optimum packing.

The use of a coarse sand would result in a loss of workability, but the gain in strength may well compensate for this.

Bessey has given a method of expressing the characteristics of sands in respect of their fineness and breadth of grading which might be developed and found useful in assessing the suitability of sands for mortars and designed mixes.
Since the particle size distribution of sands tends to be a logarithmic normal Gaussian distribution, plotting the sieve analysis on logarithmic probability graph paper should tend to give a straight line. For the greater part of the distribution this is generally found to be so, with sometimes a break or departure from linearity at the coarse end. In nearly all instances (except for blended sands) the bulk of the sand from 10% to 80% passing the sieves can be represented by a straight line. Since a straight line can be represented by two parameters this offers a means of comparing or specifying gradings which is much more meaningful than the present method of the limiting B.S. curves. The parameters we have used for expressing the grading in this way are:

1. **The median diameter** (Md) which is that diameter on the particle size distribution curve of the sand above and below which lies 50% by weight of the sand. It is read off simply as the 50% value on the curve (on whatever scale this is drawn).

2. **The Sorting Coefficient** (So) given by the formulae

   \[ So = \sqrt{\frac{Q_3}{Q_1}} \]

   Where \( Q_3 \) and \( Q_1 \) are the diameters below which lie 75% and 25% of the sand by weight (the upper and lower quartiles). The sorting coefficient is a measure of the breadth of grading and is also related to the log standard deviation of the particle size.

The possibility of application of this concept to specification of sands for mortars is shown in Fig. 2.10 where, by plotting these two parameters, a sand grading is represented by a single point. The area
enclosing possible gradings according to B.S.1199 is shown by the full lines, from which it may be seen that a wide variety of sorting coefficients, including low and palpably poor ones are permitted whilst others which may well be satisfactory are excluded. The dotted lines indicate the alternative approach to specification which, while effectively excluding the very poorly graded sands which are quite unsuitable, would permit use of some sands which are at present outside the British Standard but are known to be satisfactory. Areas on this diagram which represent unsuitable sands are clearly seen whilst one might expect improvement in properties on going toward the centre of the area marked Good, i.e. with a median size between 0.3 and 0.6 mm and sorting coefficients of at least 1.5.

The use of these two parameters alone as a specification does not quite cover the needs, for a limit is also required to the "tails" of the distribution curve. At the lower end this is covered by the requirement for clay and silt content, but at the upper end a limit for residue on the No. 7 and possibly 3/16 ins sieves will also be necessary.

Since the compressive failure is by transverse splitting of the brickwork caused by lateral strain of the less stiff mortar it follows that the action of the mortar will induce lateral compressive stresses within its volume. The mortar will be restrained by friction, aided by bond at the brick mortar interface and is therefore in a triaxial state of compression. Consequently the properties of the mortar in the joint are of particular importance, especially since they are likely to differ from the larger site and laboratory specimens such as 3 ins and 4 ins cubes, 4 ins x 1 in x 1 in prisms and 12 ins x 4 ins dia. cylinders.
Some typical examples of tensile strength of mortar are given in Table 2.2 and it can be seen that the tensile strength of mortar may be between $\frac{1}{5}$ and $\frac{1}{15}$th of the compressive strength and is therefore unlikely to contribute to any extent towards the tensile strength of brickwork.

Tests have been carried out in Germany to assess the strength of the mortar in the joint by measuring the penetration of cartridge fired pins and Graf has shown that the mortar from the joint can be 3 to 5 times as strong as that of the larger cube and prisms specimens. Anderegg came to similar conclusions. Haller found that for fairly dense cement mortars the cube strength $P$ is increased by an amount 4.1 times the value of the transverse stress $f$ when laterally restrained so that the restrained cube strength $= P + 4.1f$.

The modulus of elasticity and Poisson's ratio of mortar will influence the brickwork strength and one authority gives Poisson's ratio for a strong 1:2 cement/sand mortar (strength 3,500 lb/ins$^2$) as ranging between 0.138 and 0.166. Plowman investigated Poisson's ratio for a wide range of concrete strengths using 12 ins x 4 ins dia. cylinders. He concluded that the value was independent of strength and the mean value was $0.135 \pm 0.035$. These values might be applicable to 1:1:6 cement/lime/sand mortars and stronger. It is not yet certain whether Poisson's ratio is independent of the applied stress.

If the height of the bricks were increased to say 6 ins nominal instead of 3 ins nominal the cross-sectional area of brick resisting the transverse force from the mortar is doubled and the horizontal tensile stress halved. It would be expected that such an arrangement would show
an increase in brickwork strength and whilst few tests have been carried out on clay bricks to confirm this, it is supported by numerous tests on concrete block walls.

2.4 Joint thickness and joint materials

Earlier compression tests\(^\text{18}\) on brickwork piers and wallettes each 4 courses high and having mortar joints of various thicknesses indicated that for joint thicknesses up to \(\frac{3}{4}\) ins there may be little reduction in strength over similar brickwork having thinner joints. For joint thicknesses between \(\frac{3}{4}\) ins and \(1\frac{1}{8}\) ins there was a reduction in strength of up to 30%.

American compressions tests\(^\text{19}\) on single brick wallettes in stack bond similar to those above showed a reduction in strength of 46% for wallettes with mortar joints \(\frac{3}{4}\) ins thick over those with joints \(\frac{3}{8}\) ins thick. Flexural tests on wallettes having similar joint thicknesses showed a reduction in strength of 49%. These results are shown graphically in Fig. 2.3 and it can be seen that there is a linear relationship between strength and joint thickness for both the compressive and flexure tests.

Tests have been carried out on brick couplets to study the influence of the bed joint material on the compressive strength of brickwork, and the results are given in Table 2.3. Whilst these values indicate the influence of various bedding materials on the strength of the brick couplets it should be noted that failure of the couplets was by inclined shear and that normally brickwork wall panels fail by vertical splitting. The results may not therefore be representative of the influence of different bedding materials for wall tests.
2.5 Single leaf walls

The extensive tests carried out by Haller\(^9\) clearly show that a single leaf storey height wall is up to 15% stronger than a bonded wall under axial loading other factors such as brick and mortar strengths, slenderness ratio and workmanship being equal.

The strength of single leaf and bonded walls are compared in Fig. 2.4.

The inherent strength of single leaf walls has enabled 4\(\frac{1}{2}\) ins thick load bearing walls to be used for certain buildings up to 6 storeys in height and has prompted the development of a 7 ins thick brick which has the required density to provide sound insulation for party wall construction. The gain in strength of a single leaf wall over similar but bonded walls is most likely due to the larger volume of mortar in the bonded wall. Also the surface area of the exposed mortar bed face is less per unit volume for the bonded wall and these two factors will contribute towards increased lateral forces within the mortar of the bonded wall and hence increased tensile forces in the brickwork.

2.6 Slenderness ratio

2.6.1 Axial loading

The extensive brickwork pier tests carried out at the Building Research Station\(^4,8\) clearly show that the strength of brickwork decreases considerably as the slenderness ratio increases.

Tests on walls in this country are limited, most of the tests being on piers. It might be interpreted from one series of tests on 4\(\frac{1}{2}\) ins thick wall panels that when subjected to axial loading there is no decrease in strength with increase in slenderness ratio\(^4\). These results are shown graphically in Fig. 2.5. Extensive tests by Haller\(^9\) however indicate a decrease in brickwork wall strength with increase
in slenderness, as shown graphically in Fig. 2.4 and this has been confirmed by other workers\(^2\).

The O.P.\(^1\) reduction factors for slenderness ratio whilst agreeing quite well with those in the Swiss Norm\(^20\)\(^{113}\) make no allowance for the greater decrease in strength with increase in slenderness ratio of brickwork made up of weak bricks and weak mortar when compared with brickwork of stronger bricks and mortar. This effect has been observed by Thomas\(^8\) and Haller\(^9\) and the Swiss Norm 113 gives different reduction factors for slenderness ratio for the four grades of brickwork covered; greater reductions for S.R. being applied to the weaker brickwork. This reduction in strength with increase in slenderness may be connected with the greater probability of a weakness in the wall. The greater decrease in strength with increase in slenderness when weaker materials are used may be connected with the lower EI value which would result in a lower buckling load.

A comparison of the Swiss Norm 113 and C.P. 111 recommendations are given in Fig. 2.6.

2.62 Eccentric loading

It has been well established that there is a marked decrease in the strength of walls with increase in slenderness ratio when subjected to eccentric loading\(^4,8,9\) and this may be seen in Figs. 2.4, 2.5 and 2.6. These tests however, were carried out on piers and walls loaded between knife edges and hence take no account of end restraints met in normal building i.e. walls supporting reinforced concrete slabs.

One series of tests\(^18\) where the walls were loaded 1 ins eccentrically between 4 ins thick reinforced concrete floor slabs indicated a reduction in strength of only 17% compared with a similar axially load wall.
Tests between knife edges for 1 ins eccentricity by Davey and Thomas\textsuperscript{4} show a reduction in strength of over 75\% (Fig. 2.5) and similar tests by Haller\textsuperscript{9} showed a reduction in strength of over 60\% (Fig. 2.4).

It would appear that the restraint afforded by the 4 ins reinforced concrete slabs added considerably to the strength of the wall by reducing the effective height to about half that of the walls loaded between knife edges\textsuperscript{18}. Also the bending moment caused by the eccentric load is distributed in the wall and floor slabs in proportion to their stiffnesses.

The assessment of the degree of fixity at the slab/wall junction is complex and several factors require consideration.

For example, when an insitu reinforced concrete slab is designed to bear onto an end span brick wall and the shuttering is struck before the next lift of walling above is constructed, the slab will deflect under its own weight and form a hinge at the bearing. A similar situation is met when precast floor units are employed. If however, the next wall and slab are constructed before the floor shutters are struck there will be some fixity at the junction.

In general terms, the degree of fixity of the slab/wall junction will increase with increase in vertical load on the brickwork, other factors being equal. In the uppermost storeys of a load-bearing brickwork building any bending moments transferred to the wall would be most likely to develop tensile stresses because the direct stresses are low. Fortunately the degree of fixity of the slab/wall junction is less than at the lower storeys and hence the bending moment transferred to the wall is less.

The strength of the mortar at the slab/wall junction will also
influence the degree of fixity and a cement/lime/sand mortar rather than a straight cement/sand mortar is generally more able to absorb local high stressing by readjustments in the mortar. This subject has not been investigated previously although Sahlin has studied the slab/wall junction when the wall is supporting vertical load; this is discussed in Chapter 4. of this thesis.

2.63 Intersecting stiffening walls

Where load-bearing walls are stiffened by intersecting walls the C.P.111 (1964) allows an increase in permissible design stress only when the horizontal distance between restraints is less than the effective height. For this case, the horizontal distance may be used to determine the slenderness ratio. The increase in permissible stress by treating the intersecting walls as piers and increasing the effective thickness is usually negligible.

Haller has investigated the stiffening effect of intersecting walls and recommends that the permissible design stress be increased as below provided the wall is also tied at floor levels.

\[ p_{cc} = p_{sr} + (p_6 - p_{sr}) F \]  (2.6.1)

where \( p_{cc} \) = Revised permissible compressive stress in brickwork.

\( p_{sr} \) = permissible compressive brickwork stress in wall when slenderness ratio based on effective height.

\( p_6 \) = Basic permissible stress in brickwork having slenderness ratio of 6 or less from C.P. 1111 (1964) Table 3.

\( F \) = Reduction factor from Fig. 2.7.

For example consider a wall stiffened by intersecting walls where

\( t = 9 \) ins solid wall

\( h = 9 \) ft vertically between floor slabs
L = 12 ft horizontally between intersecting walls. 3000 lb/ins² crushing strength bricks laid 1:1:6 cement/lime/sand mortar.

Slenderness ratio = \( \frac{\text{effective height}}{\text{effective thickness}} = \frac{3}{4} \times \frac{9}{12} = 0.9 \)

(C.P.111, 1964 p.10, 12, and 13)

Reduction factor = 0.88 (C.P.111 1964, Table 4)

Ratio \( L/h = 12/9 = 1.33 \)

Now

\[
P_{cc} = P_{sr} + (P_6 - P_{sr}) F \quad \ldots\ldots(2.6.1)
\]

\[
P_6 = 190 \text{ lb/ins}^2
\]

(C.P.111, 1964 Table 3)

\[
P_{sr} = 190 \times 0.88 = 167 \text{ lb/ins}^2
\]

\[
F = 0.75 \text{ (from Fig. 2.7)}
\]

\[
P_{cc} = \text{Revised permissible brickwork compressive stress} = 167 + (190 - 167) 0.75 = 167 + 17 = 184 \text{ lb/ins}^2
\]

This is an increase of 10.2% over the C.P.111 (1964) method.

If wall is reduced to 4\(\frac{1}{2}\) ins nominal thickness

Slenderness ratio = \( \frac{3}{4} \times \frac{9}{12} = 18 \)

Reduction factor = 0.5

\[
P_{sr} = 190 \times 0.5 = 95 \text{ lb/ins}^2
\]

\[
P_{cc} = 95 + (190 - 95) 0.75 = 95 + 71 = 166 \text{ lb/ins}^2
\]

This is an increase of 75% over the C.P.111 1964 method.

2.7 Brickwork wall strength formulae

Many workers have produced formulae for brickwork wall strength in terms of brick and mortar strength.

Haller \(^{10}\) gives the following formulae based on wall tests which are shown graphically in Fig. 2.8.

\[
M = \left( \sqrt{1 + 0.15b} - 1 \right) \left( 8 + 0.057 m \right)
\]
where $M = \text{brickwork wall strength}$

$b = \text{brick strength}$

$m = \text{mortar strength}$

Hermann 22 gives

$$M = c \sqrt[3]{mb^2} \pm 25\%$$

where $c = \text{constant}$

Brocker 22 gives

$$M = c \sqrt[3]{m^3} \sqrt{b^4}$$

Klein 15 gives

$$M = c \sqrt[4]{m^4} \sqrt{b^4} \pm 20\%$$

It can be seen from the above that the strength of brickwork is proportional to the square root of the brick strength and the third or fourth root of the mortar strength.

2.8 Workmanship

Variations in materials and mix proportions can result in a considerable reduction in strength. Badly proportioned mortar materials can result in low mortar strength as for example an excess of water added to the mortar to improve workability.

The addition to mortars of plasticisers based on calcium chloride will reduce the bond between brick and mortar.

The use of many additives will result in lower mortar strengths when the amount of additive is in excess of the recommended quantity. The strength of cements from different works, whilst complying with the minimum requirements of B.S.12 23 may differ by as much as 100% and consequently a change of supplier part way through a contract may result in changes in brickwork strength.

The influence on strength of sand grading is considerable and is discussed under section 2.3 and that of variation in brick sizes under
section 2.2. Variations in joint thicknesses are discussed under section 2.4. Several attempts have been made to assess the effects on masonry strength of various kinds of workmanship and one test programme carried out in America at the National Bureau of Standards and summarised by Monk investigated two standards of workmanship - "Commercial" or ordinary unsupervised site brickwork was characterised by complete absence of vertical joint filling, deep grooving of horizontal joints and comparatively high speed laying. In the supervised workmanship these characteristics were avoided.

For wall specimens built with brick of strengths varying from 3,000 to 4,000 lb/ins² the supervised workmanship resulted in brickwork strength approximately 60 to 80% greater than for specimens built with ordinary workmanship.

For specimens built with an 8,700 lb/ins² brick the increase was only about 30%. This greater influence of workmanship on the strength of brickwork built with lower strength bricks reflected other research and is summarised in Fig. 2.9.

For brickwork subjected to vertical loading only the filling of perpend joints is not likely to contribute appreciably to the strength and in a German Test four walls constructed of perforated bricks were loaded axially two walls having perpend joints filled and two without.

The two walls without perpend joints failed at a load of 10% less than the two walls with perpend joints filled. The first crack in the two walls without perpend joints appeared at a load of 30% less than in the two walls with perpend joints filled. It might be expected that the correct bedding of frogged bricks and the avoidance of furrowed bed joints would be more important than with wire cut bricks because of the
possibility of edge loading with deeply frogged bricks. Some tests on walls with bricks laid frog down\textsuperscript{25} revealed a reduction in strength of approximately 30% when compared with walls laid with the frog uppermost. This reduction in strength might also be expected to apply to walls with bricks laid on furrowed beds.

Considerable reductions in brickwork strength have been observed when the bricklayer persists in tapping brickwork 4 or 5 courses down from his working course,\textsuperscript{26} ten to 15 minutes after laying to align and plumb. This results in a complete breakdown of bond between the mortar and the brick and may sometimes introduce an uneven bedding and possibly eccentric loading.

The overall importance and influence on strength of workmanship is acknowledged in American\textsuperscript{2}, Swiss\textsuperscript{20} and New Zealand\textsuperscript{27} Codes where increased design stresses are permitted for supervised construction.
Comparison of sieve analysis of sands for general purpose mortars and schedule of relative diameters

Table 2.1

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>American</th>
<th>British equivalent</th>
<th>American A.S.T.M.</th>
<th>British % weight passing larger sieve</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>No.</td>
<td>Size in mm</td>
<td>No.</td>
<td>Size in mm</td>
</tr>
<tr>
<td>4-8</td>
<td>4</td>
<td>4.76-2.38</td>
<td>3/16</td>
<td>4.76-2.41</td>
</tr>
<tr>
<td>8-16</td>
<td>8</td>
<td>2.38-1.19</td>
<td>7</td>
<td>2.41-1.2</td>
</tr>
<tr>
<td>16-30</td>
<td>16</td>
<td>1.19-0.59</td>
<td>14</td>
<td>1.20-0.6</td>
</tr>
<tr>
<td>30-50</td>
<td>50</td>
<td>0.59-0.3</td>
<td>25</td>
<td>0.60-0.3</td>
</tr>
<tr>
<td>50-100</td>
<td>100</td>
<td>0.3-0.15</td>
<td>52</td>
<td>0.30-0.15</td>
</tr>
<tr>
<td>100-200</td>
<td>100</td>
<td>0.15-0.08</td>
<td>100</td>
<td>0.15-0.08</td>
</tr>
</tbody>
</table>

Table 2.2

Tensile and compressive strength of mortars

<table>
<thead>
<tr>
<th>Mortar mix by volume cement/lime/sand (paste/sand ratio in brackets)</th>
<th>Compressive strength (lb/ins²)</th>
<th>Tensile strength (lb/ins²)</th>
<th>Strength ratio compressive/tensile</th>
</tr>
</thead>
<tbody>
<tr>
<td>1:4:15</td>
<td>363</td>
<td>65</td>
<td>5.6</td>
</tr>
<tr>
<td>1:4:11½</td>
<td>643</td>
<td>83</td>
<td>7.7</td>
</tr>
<tr>
<td>1:1:6</td>
<td>2479</td>
<td>342</td>
<td>7.2</td>
</tr>
<tr>
<td>1:1:4¾</td>
<td>3663</td>
<td>396</td>
<td>9.3</td>
</tr>
<tr>
<td>1:1/4:3¾</td>
<td>5358</td>
<td>512</td>
<td>10.5</td>
</tr>
<tr>
<td>1:3/4:3-3/16</td>
<td>7392</td>
<td>513</td>
<td>14.4</td>
</tr>
</tbody>
</table>
Table 2.3

Influence of bed joint material on the compressive strength of brick couplets

(after Monk\(^2\))

(Compressive strength of brick = 15,936 lb/ins\(^2\))

<table>
<thead>
<tr>
<th>Bed joint material</th>
<th>No. of specimens</th>
<th>Ultimate compressive strength of couplet (lb./ins(^2))</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/32 ins aluminium between ground surfaces</td>
<td>3</td>
<td>15,402</td>
</tr>
<tr>
<td>3/8 ins Celotex</td>
<td>6</td>
<td>14,252</td>
</tr>
<tr>
<td>Ground brick surfaces, no joint material</td>
<td>5</td>
<td>14,234</td>
</tr>
<tr>
<td>0.016 ins galvanised sheet between ground surfaces</td>
<td>3</td>
<td>13,992</td>
</tr>
<tr>
<td>Ground brick surfaces, with wheel bearing grease</td>
<td>5</td>
<td>13,922</td>
</tr>
<tr>
<td>Plaster of Paris</td>
<td>5</td>
<td>12,400</td>
</tr>
<tr>
<td>Neat Cement Paste (7 days)</td>
<td>5</td>
<td>11,193</td>
</tr>
<tr>
<td>cement-sand (1:0:2 - 1/4) with wire mesh (7 days)</td>
<td>5</td>
<td>10,967</td>
</tr>
<tr>
<td>Polyester Plastic Filler (2 days)</td>
<td>2</td>
<td>10,584</td>
</tr>
<tr>
<td>cement-sand (1:0:2 - 1/4) (7 days)</td>
<td>2</td>
<td>10,310</td>
</tr>
<tr>
<td>Epoxy Cement-sand (1:3) (2 days)</td>
<td>2</td>
<td>10,285</td>
</tr>
<tr>
<td>Gypsum cement (6 days)</td>
<td>3</td>
<td>10,142</td>
</tr>
<tr>
<td><strong>cement-lime-sand (1:½:4(\frac{1}{2})) (28 days)</strong></td>
<td>5</td>
<td>9,845</td>
</tr>
<tr>
<td>sodium Silicate with 10% talc (28 days)</td>
<td>5</td>
<td>9,559</td>
</tr>
<tr>
<td>sodium silicate with talc (1:3) (6 days)</td>
<td>2</td>
<td>9,434</td>
</tr>
<tr>
<td>dry sand- contained with masking type</td>
<td>3</td>
<td>9,430</td>
</tr>
<tr>
<td>cement-sand (1:0:3) (7 days)</td>
<td>5</td>
<td>9,366</td>
</tr>
<tr>
<td>cement-lime (1½:4:0) (7 days)</td>
<td>5</td>
<td>9,274</td>
</tr>
<tr>
<td>cement-lime-sand (1:3:4(\frac{1}{2})) (28 days)</td>
<td>5</td>
<td>8,762</td>
</tr>
<tr>
<td>cement-lime-sand (1:2:4(\frac{1}{2})) with wire mesh (7 days)</td>
<td>3</td>
<td>8,686</td>
</tr>
<tr>
<td>cement-lime-sand (1:½:4(\frac{1}{2})) (7 days)</td>
<td>3</td>
<td>7,948</td>
</tr>
<tr>
<td><strong>cement-lime-sand (1:½:4(\frac{1}{2})) (28 days)</strong></td>
<td>5</td>
<td>7,757</td>
</tr>
<tr>
<td>cement-sand (1:0:3) (9 days)</td>
<td>3</td>
<td>6,507</td>
</tr>
<tr>
<td>cement-sand (1:½:4(\frac{1}{2})) (28 days)</td>
<td>5</td>
<td>6,355</td>
</tr>
</tbody>
</table>
INFLUENCE ON BRICKWORK STRENGTH OF MORTAR AND BRICK STRENGTH (AFTER THOMAS)
INFLUENCE ON BRICKWORK STRENGTH OF BRICK SUCTION (AFTER HALLER)
Influence on brickwork strength of mortar joint thickness (after SCPFRF)
SLENDERNESS RATIO $\frac{h}{c}$

- 12 CM. WALL
- 25 CM. WALL

1 KG/SQ.CM. = 14.22 LB/SQ.INS.

INFLUENCE ON BRICKWORK STRENGTH OF SLENDERNESS RATIO AND ECCENTRIC LOADING FOR SINGLE LEAF AND BONDED WALLS (AFTER HALLER)
Influence on brickwork strength of eccentric loading (after Davey & Thomas) 4 1/2" wall tests
Comparison between British and Swiss Design Codes for S.R. and Eccentric Loading

Figure 2.6

- C.P.III
- Reduced off-thickness
- Swiss Norm 113 high quality brickwork
- Swiss Norm 113 normal quality brickwork

\( m = \text{eccentricity ratio} = \frac{e}{l} \)
SLENDERNESS REDUCTION FACTORS FOR WALLS STIFFENED BY INTERSECTING WALLS (AFTER HALLER)

fig 2.7
INFLUENCE ON WALL STRENGTH OF MORTAR AND BRICK STRENGTH (AFTER HALLER)
INFLUENCE OF WORKMANSHIP ON STRENGTH OF BRICK WALLS (AFTER MONK)
SAND GRADING EXPRESSED IN TERMS OF MEDIAN SIZE AND SORTING COEFFICIENT (AFTER BESSEY)
RELATIONSHIP BETWEEN STRESS RATIO AND ECCENTRICITY
ASSUMING A LINEAR STRESS STRAIN RELATIONSHIP.

fig 2:11
CHAPTER 3.

EXPERIMENTAL INVESTIGATION - 200 TON FRAME

CRUSHING TESTS ON STOREY HEIGHT WALLS 4½ INS THICK

3.1 General

The object of the work presented in this chapter was to study in detail the effects of eccentric loading, walls off plumb, workmanship, brick strength, mortar strength and joint thickness on the strength of storey height wall panels 4½ ins thick loaded between 4 ins thick concrete slabs.

The tests were intended to extend earlier tests and also to provide data that would supplement the results of tests on 10½ ins thick cavity walls described in Chapter 5, by providing information on the strength of single leaf walls.

3.2 Loading frame.

Figure 3.1 (see Appendix - section 11.2, Fig. 5) shows the frame and test wall. The lower 4 ins thick reinforced concrete slab is 3 ft. wide by 17 ft. long and is supported at the centre on a 9 ins brick wall three courses high, and at the ends by the loading frame.

The test walls were each 3 ft. wide, 8 ft. 2 ins high clear between slabs and of 4½ ins nominal thickness. The loading beam was seated on a single course of brickwork constructed on top of the upper slab and a ½ ins plywood bedding on top of a ¼ ins thick rubber packing was placed immediately beneath the loading beam.

The loading equipment included two loading jacks each of 100 tons capacity of the Tangye Hydraulic Detached Ship Type having simple packed rams. The ram dia. was 7 ins and the maximum ram travel was 6 ins.

The two jacks were bolted to the underside of the crosshead beam and because the rams travel was downwards, the rams were fitted with return
springs.

The basic pump unit consists of a horizontal tank mounted pump unit with a maximum output of 50 cubic ins per minute and a maximum continuous pressure of 6000 lb/in² and intermittent pressure of 9000 lb/in². The tank capacity is 560 cubic ins and the pump unit was manufactured and supplied by Messrs. Epco Ltd.

The load applied from each ram was measured by two 100 t load cells which were made in the work shop prior to commencing these tests. Each was made up by electrical resistance strain gauges bonded to the steel column of the load cell and connected to form two arms of the wheatstone bridge.

For the first 3 wall tests the remaining two arms of the bridge were provided by a Brüel and Kjaer strain gauge box, and the electrical signal from the bridge when under load was read visually by noting the needle deflection.

For later tests a switch box was made up to include two standard resistances to form the remaining two arms of the bridge and the electrical signal was read on the digital voltmeter described in Appendix 1.

It was observed that the bridge was temperature sensitive, the signal under zero load varying with the ambient temperature. The signal range over a 100 t loading range was constant however, and the temperature correction easily made. The calibration curve is given in Fig. 3.2.

3.3 Materials

3.31 Bricks

Bricks of four different crushing strengths were used for the wall tests, and each was nominally 9 ins long by 4½ ins wide. The two batches
of pressed double frogged bricks were $2\frac{7}{8}$ ins deep and the perforated wire cut bricks and the single frogged bricks were $2\frac{5}{8}$ ins deep.

A summary of the brick properties is given in Table 3.1.

**3.32 Sand**

Three building sands were obtained locally and used for the brickwork construction and ordered to conform with Table 1 of B.S.1200. Sieve analysis revealed an excess of fines in all three sands - the greatest excess being with sand No. 3. The sands were stored in the open and for the construction of nine walls it was dried before mixing. For the remaining 7 walls wet sand was used.

The results of the sieve analysis are given in Table 3.2 together with a schedule of walls for which the sands were used.

**3.33 Lime**

For the 1:1:6 mortar mix a class A hydrated lime (Hydralime) in accordance with B.S.890 was used.

**3.34 Cement**

A rapid hardening Portland cement (Ferrocrete) was used for all mortar to give early mortar strengths.

**3.35 Mortar**

Each batch of mortar was mixed by hand. Mortar proportions were made up by volume and their weights recorded.

The sand used for the mortar in nine of the wall tests was dried and for the remainder it was wet.

The nominal volume mixes gave therefore a richer mix when the wet sand was used, because of sand bulking. For each mix the bricklayer was allowed to add sufficient water to give optimum workability. From each mortar mix, 4 ins cubes were made by compacting in two 2 ins layers with a
Kango Hammer. After 24 hours the cubes were removed from the mould and stored in water until tested.

The results of the mortar cube crushing tests are given in Table 3.3.

3.4 Experimental procedure

3.41 Tests on walls - general

Each of the walls was constructed within the loading frame shown in Fig. 3.1 by a professional bricklayer and both upper and lower R.C. slabs were in position prior to constructing the test wall. To facilitate this the upper slab was raised $\frac{1}{2}$ ins at the centre. Each test wall was constructed in the normal manner and a bed of mortar approximately $\frac{5}{8}$ ins thick was placed on top of the wall. The upper slab was then lowered $\frac{1}{2}$ ins onto the fresh bed of mortar which reduced from $\frac{5}{8}$ ins to $\frac{3}{8}$ ins in thickness. After curing in air, the walls were tested by applying the load through a loading beam seated on a single course of brickwork on the upper slab.

The bricklayer was instructed to construct the walls plumb and true to line and level with all joints completely filled with mortar.

A storey height course rod was supplied to the bricklayer, and $\frac{3}{8}$ ins nominal thickness mortar joints adopted for all test walls except wall 3 which had joints of slightly less than $\frac{3}{16}$ ins thickness.

A check on the plumbness of wall 1 and 2 revealed they were up to $\frac{1}{4}$ ins out of plumb and for all subsequent walls a timber jig consisting of vertical 4 ins x 2 ins timbers was used to ensure accurate alignment.

Walls 2 to 10 inclusive were similar in materials and construction and walls 2 to 7 were tested under nominally axial loading.

Walls 8 and 9 were reinforced horizontally in every second mortar joint with B.R.C. Fabric and loaded axially.
Walls 10 and 11 were constructed plumb but $\frac{3}{4}$ ins off centre to the line of loading.

Walls 12, 13 and 14 were constructed centrally at their base and $\frac{3}{4}$ ins off plumb, so that the line of load application was $\frac{3}{4}$ ins eccentric at the top of the wall.

During several of the tests the single course of brickwork above the upper slab failed simultaneously, and in two tests it failed before the wall and had to be rebuilt and cured before the wall test could be completed.

The upper slab cracked whilst carrying out several tests and the faulty section was broken out and fresh concrete cast. This arrangement was found to be completely satisfactory and preferable to replacing the complete slab.

The load was recorded for all the wall tests and for each, excluding 12, 13 and 15, two vertical extensometers were attached to both wall faces and strain readings measured over a gauge length of 72 ins at various loads increments.

Demec studs were fixed to one face for wall 1 and to two faces for walls 3 and 4 and vertical and horizontal strains measured over a gauge length of 8 ins.

The positions of the extensometers are shown in Fig. 3.3.

A summary of results of the wall tests is given in Tables 3.4 and 3.5.

3.42 Tests on walls - Description of tests - Walls 1 to 16 inclusive

Wall 1

Wall 1 was built by bricklayer no. 1, seconded from a local building site who was instructed to build the wall plumb and true to line and level with all joints completely filled.
The workmanship was in fact quite poor and the first 8 courses sloped up \( \frac{1}{4} \) ins towards the centre from both sides.

The top three courses, where the bricklayer was working at forehead level were noticeably \( \frac{1}{4} \) ins out of plumb. Certain perpend joints were not completely filled and workmanship was not of a very high standard.

The bricks had a high total water absorption (28.75% by weight - 24 hours absorption test) and after laying the first 4 courses with dry bricks it was decided that subsequent bricks be wetted before laying.

After 38 days curing the test load was applied gradually over a period of approximately 1\( \frac{1}{2} \) hours and the first crack was heard when the load was 50\( ^t \). The loading was increased and sudden failure took place by vertical splitting followed by crushing of the brickwork near the centre of the wall. Spalling of the top 5 courses was observed after vertical splitting as can be seen in Figs. 3.4 and 3.5.

The stress/strain curve is given in Fig. 3.6 and Demec readings shown graphically in Figs. 3.7 and 3.8.

Wall 2

Wall 2 was built by bricklayer no. 2 also seconded from a local building site to similar instructions as wall 1. Upon lowering the upper slab onto the fresh bed of mortar it was observed that the upper mortar in three or four horizontal joints squeezed out and were consequently less than the nominal \( \frac{3}{8} \) ins thickness. The wall was found to be \( \frac{1}{2} \) ins out of plumb for half its width.

A preliminary loading to 92\( ^t \) over a period of approximately 1\( \frac{1}{2} \) hours was carried out two days before the final test to destruction. During the final test the first crack was heard at a load of 103.5 tons. The load was then held steady whilst instruments were removed. The load was
accidentally allowed to increase to $120^t$ and spalling of the upper brickwork courses began. The load was immediately reduced to $75^t$ until all instruments had been dismantled. The loading was again increased and at a load of $103.5^t$ a loud noise was heard and a vertical crack appeared through the centre of the wall. Spalling of the top courses and to the one half of the wall $\frac{1}{2}$ ins out of plumb took place simultaneously - Figs. 3.9 and 3.10.

The stress/strain curve is given in Fig. 3.11.

Wall 3

Wall 3 was built by bricklayer no. 1 to the standard instructions. A timber jig consisting of two 4 ins by 2 ins vertical timbers was used for the construction of this and all subsequent $4\frac{1}{2}$ ins thick walls, to ensure accurate alignment and plumb. The average joint thickness was 0.16 ins. The load was applied gradually over $1\frac{3}{4}$ hours the load being held steady at intermediate stages whilst readings were taken.

Failure occurred suddenly at 114.5 tons by the formation of 2 vertical cracks in the top half of the wall and considerable spalling of the top four courses on both faces. There were no indications of approaching failure such as slight spalling at the top of the wall or visible (or audible) hair cracks appearing. After failure the load was re-applied and almost complete disintegration of the top four courses occurred. The upper face of the top course of bricks, in contact with the under side of the upper slab, remained intact forming a "wedge" or inverted Vee 2 ins - 3 ins deep, as shown in Fig. 3.12.

The stress/strain curve is given in Fig. 3.13 and demec readings in Figs. 3.14 and 3.15.
Wall 4

Wall 4 was built by bricklayer no. 3 to the standard instructions. The load was applied gradually over 1 1/2 hours and at a load of 135 t an oil pipe connection failed and the load was reduced to zero. Four hours later, after repair, the load was increased to 150 t over 1/4 hour. At this load faint cracking sounds were heard and after a further minute sudden failure occurred.

Failure was by diagonal shear at 45° accompanied by crushing and spalling local to the shear plane at about 1/3 of the wall height. Figs. 3.16 and 3.17.

The stress/strain curve is given in Fig. 3.18 and Demec readings in Figs. 3.19 and 3.20.

Wall 5

Wall 5 construction was similar to wall 4 above. The rate of loading was greater than earlier tests and completed in 1/2 hour.

Failure at 157 t was sudden and a single vertical crack occurred at a 1/3 point extending for the upper 2/3 of the wall and dividing into two 45° shear cracks at the lower end. Local crushing was observed near the centre of the wall at the intersection of the vertical and shear cracks and also at the lower end of the shear cracks. The failure was similar to that shown in Fig. 3.21 obtained from an earlier series of tests.

The stress/strain curve is given in Fig. 3.22.

Wall 6

Wall 6 construction was similar to wall 4 above. Initial loading to 93 tons was carried out in 1/2 hour and the load then released.

Two hours later loading was recommenced and failure took place
after 8 minutes from the beginning of loading at a load of 150 tons. Failure occurred suddenly by what appeared to be buckling of the wall.

An inspection of the wall debris after failure revealed that some spalling and local crushing had occurred at about mid height and also that there was a poor bond between the brick and mortar. There were no signs of spalling or crushing of the top courses and failure may have been similar to that for wall 4 or alternatively by local crushing and spalling in a horizontal bond of brickwork at about mid height.

The stress/strain curve is given in Fig. 3.23.

Wall 7

Wall 7 construction was similar to wall 4 above. The wall was loaded initially to 22.5 tons over 20 minutes and the load released.

Two minutes later the loading was recommenced and at a load of 140 t a vertical crack appeared approximately 1 ft. in from one edge and extending from the top of the wall to about mid wall height. The load was slowly increased and at 168 t the crack increased noticeably in size; at 172 t sudden failure occurred by:

a) The upper one foot width of brickwork sheared across the 1 ft. width at mid height - Fig. 3.24.

b) Local spalling and splitting within the 4½ ins width in the top three courses.

The total time for the second loading was 1 hour.

The stress/strain curve is given in Fig. 3.25.

Wall 8

Wall 8 construction was similar to wall 4 above except that alternate
horizontal mortar joints were reinforced with B.R.C. Fabric. The two reinforcing wires - approximately $\frac{1}{8}$ ins in diameter - were $3\frac{1}{2}$ ins apart and when built into the joints were $\frac{1}{4}$ ins from the wall face. The wall was $\frac{1}{4}$ ins out of plumb.

The load was increased steadily and at $100^t$ several vertical hair cracks were observed on one elevation only in the top 4 courses passing through the bricks, and, surprisingly, not on the line of the perpend joints.

Loading was continued and at $125^t$ the hair cracks were visible on the reverse elevation.

As the load increased, further cracking was heard and at $149.5^t$ sudden failure occurred, the wall collapsing completely. The total time for loading was 1 hour.

The centre $\frac{1}{3}$ of the wall was thrown further away. It appeared from re-assembling the numbered courses of brickwork that excessive local crushing had occurred about four courses from the bottom on the same side as the collapse and the $\frac{1}{4}$ ins wall inclination.

The remainder of the wall appeared free from any vertical splitting, crushing or spalling apart from the hair cracks in the top 4 courses.

Failure may have been similar to that for wall 4 or alternatively by local crushing and spalling in a horizontal band of brickwork at about $1/3$ height.

The stress/strain curve is given in Fig. 3.26.

Wall 9

Wall 9 construction was similar to wall 8 except that the cross centres of the two reinforcing wires was reduced from $3\frac{1}{2}$ ins to $2\frac{1}{2}$ ins.

After loading to 150 tons the single course of brickwork above the
slab crushed.

The wall was examined and found to be undamaged and after rebuilding the single course of brickwork the wall was loaded to 188 tons over \( \frac{1}{2} \) hour when an oil connection failed.

Two hours later, after repair loading was carried out over \( \frac{1}{3} \) hour up to the 200 ton capacity of the loading frame.

To avoid overloading of the pipe connections the test was abandoned.

The stress/strain curve for the 1st loading is given in Fig. 3.27.

**Wall 10**

Wall 10 was similar to wall 4 above except that the wall was built \( \frac{3}{4} \) ins off the centre of load application and plumb. The load was increased steadily and at 103 tons local crushing of the top course end brick commenced.

Loading was continued to 132 tons when cracking of the brickwork was heard. The load was held steady for \( \frac{1}{2} \) minute when sudden failure occurred by extensive spalling of the top four courses on the compression face only and further crushing of the top course end bricks right hand on compression face in Fig. 3.28.

A vertical crack was observed in the top left of wall, again on the compression face only, approximately 18 ins long - Fig. 3.28. An inspection of the wall revealed that only the top six courses of brickwork were damaged. The total time for loading was \( \frac{1}{2} \) hour.

The stress/strain curve is given in Fig. 3.29.

**Wall 11**

Wall 11 was similar to wall 10 above. The load was increased steadily and at a load of 117 tons failure took place by total collapse of the wall. There were no signs of local crushing of the top courses. Some
vertical splitting and spalling was observed in the bottom three courses - spalling was on the normal tension face indicating a reversal of stress and hence some restraint at the slab/wall junction. The upper slab was seen to be cracked on a line with the compression edge of the test wall.

A very poor bond between the brick and mortar was observed. The total time for loading was $\frac{1}{2}$ hour.

The stress/strain curve is given in Fig. 3.30.

Wall 12

Wall 12 construction was similar to wall 4 except that the wall construction was $\frac{3}{4}$ ins off plumb. The wall was central at its base and the top $\frac{3}{4}$ ins eccentric to the line of load application above the top slab.

The load was increased steadily until failure at 720 tons after 35 minutes.

Failure occurred suddenly with the appearance of a diagonal shear crack and associated spalling on both faces at about $\frac{2}{3}$ height, together with vertical splitting. (Figs. 3.31 and 3.32)

Wall 13

Wall 13 construction was similar to wall 12. The load was increased steadily until failure at 131 tons after 35 minutes.

Failure occurred suddenly with excessive spalling and crushing at the top of the wall. - Fig. 3.33.

Wall 14

Wall 14 construction was similar to wall 12, but of 3710 lb/ins$^2$ bricks. The load was increased steadily until failure at 81.5 tons after 30 minutes.

Failure occurred suddenly by shearing within the $\frac{4}{8}$ ins brick width
over a depth of 4 or 5 courses for the full 3 ft wall width – Fig. 3.34. Spalling and local crushing in the area of the shear failure was also observed – Fig. 3.35.

The stress/strain curve is given in Fig. 3.36.

Wall 15

Wall 15 construction was similar to wall 4. The load was increased steadily until failure at 70 tons after 25 minutes. The first cracks were seen (and heard) at a load of 70 tons and comprised two vertical cracks extending for the lower 1/3 of the wall. The loading was increased and cracking continued without the noise associated with cracking in tests 1 to 13. (because of weaker bricks).

At a load of 70 tons the wall failed by local crushing in the lower 1/3 of the wall, extensive crushing taking place at either end of the wall on a shear plane – Fig. 3.37.

Wall 16

Wall 16 construction was similar to Wall 4. The load was increased steadily until failure by vertical splitting and crushing at 65° after 30 minutes.

The stress/strain curve is given in Fig. 3.38.

3.43 Tests on brickwork cubes

Several brickwork cubes were constructed along side each test wall using the same batch of bricks and mortar as for the test walls. The cubes were nominally 9 ins x 9 ins x 3 and also 4 courses high; each was built with a thin mortar bed at the bottom and none at the top and the upper most frogs were filled flush. The bricklayer was instructed to fill all joints.
The cubes were air-cured and subjected to an axial compression test on the same day as the wall test.

A sheet of $\frac{1}{6}$ ins thick plywood was placed on the top and bottom of the cubes and they were loaded at a rate of 2000 lbs/in$^2$/minute. A further series of cubes were constructed having mortar joint thicknesses of $\frac{1}{4}$ ins to 1 ins. No special jig was used for the construction and the workmanship was typical of that found in a site control brickwork cube.

### 3.5 Results

A summary of the results of the 16 wall tests together with a comparison with the permissible design stresses based on the code of practice (C.P.111 1964) is given in Tables 3.4 and 3.5.

A summary of the permissible design stresses for axial and eccentric loading is given in Table 3.6.

The average crushing strengths of the brickwork cubes associated with the wall tests together with a comparison with the wall strengths are given in Tables 3.7 and 3.8.

The results of tests on brickwork cubes to investigate the influence upon strength of brickwork of joint thickness and age are given in Table 3.9 and the strength ratios given in Table 3.10. The relevant 4 ins mortar cube crushing strengths are given in Table 3.11. The stress/strain curves and Demec readings are noted in Section 3.42 with the relevant wall.

### 3.6 Calculations

The permissible design stresses included in Tables 3.4 and 3.5 for axial and eccentric loading have been calculated in accordance with the code of practice (C.P.111 1964). The code does not differentiate
between cases where an eccentric load is applied at the top of the wall only and when applied at both top and bottom of the wall. For the purposes of calculating the reduction factors for combined slenderness ratio and eccentric load, the two cases have been considered equal. So that the walls $\frac{3}{4}$ ins off plumb are considered similar to walls plumb but $\frac{3}{4}$ ins eccentric as far as the calculation of permissible design stresses are concerned.

For areas of brickwork having a cross-sectional area less than 500 sq. ins an additional reduction factor is applied.

The code is at present being redrafted and the new code will required a reduction for small areas of only 250 sq. ins and less.

The permissible design stresses were calculated on the following basis and are summarised in Table 3.6.

**Slenderness ratio**

\[
\text{Slenderness ratio} = \frac{\text{effective height}}{\text{effective thickness}} = \frac{\frac{3}{4} \times 8.167 \times 12}{\frac{4}{3}} = 16.5
\]

Reduction factor for axial loading (from Table 4 of C.P.111) = 0.56

Additional reduction factor for small area =

\[
\frac{4.125 \times 35}{1000} + 0.75 = 0.894
\]

Combined reduction factor for axial loading and small area =

\[
0.56 \times 0.894 = 0.50
\]

Eccentric load of $\frac{3}{4}$ ins is approximately $t/6$ where $t =$ wall thickness of $4.125$ ins. Reduction factor for $t/6$ eccentric loading (from Table 4 of C.P.111) = 0.495.

Combined reduction factor for eccentric loading and area =

\[
0.495 \times 0.894 = 0.442.
\]

The basic compressive stresses given in Table 3.6 are obtained by interpolation from Table 3 of C.P.111. The permissible design stress
for axial loading is obtained by multiplying the basic compressive stress by the reduction factor 0.5.

For the brick strength of 4825 lb/ins$^2$ the design stress for axial loading would be $264 \times 0.5 = 132$ lb/ins$^2$.

The maximum permissible design stress for $\frac{3}{4}$ in eccentric loading (i.e. edge stress) is obtained by multiplying the basic compressive stress by the reduction factor 0.442 and increasing this by 25%.

For the brick strength of 4825 lb/ins$^2$ the design maximum edge stress for $\frac{t}{6}$ eccentric loading would be $264 \times 0.442 \times \frac{125}{100} = 146$ lb/ins$^2$.

It can be seen from Table 3.12 that for $\frac{t}{6}$ eccentricity the minimum stress is zero. The average design stress therefore is half the maximum stress and for the brick strength of 4825 lb./ins$^2$ is $146 \times 0.5 = 73$ lb/ins$^2$.

In fact however, the bending moment caused by the eccentric load will be distributed to the wall and floor slabs in proportion to their stiffnesses as discussed in section 2.62.

The relative stiffnesses of the floor slabs and wall, assuming the slabs to be pinned at one end and the wall to be fixed at both ends are given in Table 3.14.

The appropriate reduction factor for combined slenderness ratio and eccentric loading (from Table 4 of C.P.111) for an eccentricity of 0.211 (when the ratio of E concrete to E brickwork is 2) x $\frac{t}{6}$ and S.R. of 16.5 is 0.55 ($e = \frac{t}{28.5}$). Allowing for the small area reduction this becomes $0.55 \times 0.894 = 0.491$.

The maximum design edge stresses calculated for $\frac{t}{6}$ and $\frac{t}{28.5}$ eccentricity and the equivalent average stresses are shown in Table 3.6,
assuming a linear stress/strain distribution across the wall section.

3.7 Discussion of results

3.7.1 Brick strength

The strength of the brickwork increased with increase in brick strength although not proportionately, with the exception of wall 1. As described in section 3.4.2 wall 1 was badly constructed and this probably influenced the low strength and low brickwork/brick ratio of 0.17. For the walls having a brick crushing strength of 6235 lb/ins² the brickwork/brick ratio for axial loading ranged between 0.26 for wall 2 and 0.47 for wall 7. These cover a wider range than earlier tests where ratios of 0.29 to 0.37 were obtained for similar tests.

Various empirical formulae have been developed to relate brick strength to strength of brickwork as described in section 2.7 and most of these show the strength of brickwork to be proportional to the square root of the brick strength; so that when the brick strength is doubled the brickwork strength might be increased by approximately 40%.

However, these are only likely to give useful results when numerical constants are introduced for different types of brick.

For example, deeply frogged bricks may preclude an even bedding and the transfer of vertical load may be at the edges only. Also when deep frogs are completely filled with mortar in a wall subjected to vertical loading there may be additional lateral thrusts from the mortar acting on the sides of the frog introducing additional stresses.

3.7.2 Mortar strength

A comparison between axially loaded walls 2 to 7 in Table 3.4 indicate only a slight increase in strength of brickwork with increase
in strength of mortar.

Earlier tests\textsuperscript{18} indicated that there was little gain in brickwork strength for mortar strengths over 1,000 lb/ins\textsuperscript{2} crushing strength and the lesser influence of mortar strength in the higher strength range is indicated by the convergence of the mortar strength curve shown in Fig. 2.1.

The empirical formulae described in section 2.7 consider brickwork strength to be proportional to either the cube root or the fourth root of the mortar strength so that when the mortar strength is doubled the brickwork strength might be increased by only 19\% to 26\%.

However, the transverse strain of the mortar causing transverse splitting of the brickwork at failure will be influenced by Young's modulus and Poisson's ratio and the effects of these two properties on brickwork strength is not yet fully understood.

Considerable care in proportioning and mixing the mortar materials was taken by the bricklayer and the large variations in mortar cube strength seen in Tables 3.3 and 3.11 were surprising. It is possible that aged cement was used for certain tests.

3.703 Joint thickness

All the test walls had \(\frac{3}{8}\) ins thick mortar joints except wall 3 which had \(\frac{3}{16}\) ins thick mortar joints. There was no significant difference in the strength of wall 3 compared with similar walls 2 and 4 having \(\frac{3}{8}\) ins joints and variations in workmanship and materials may have overshadowed any differences due to changes in joint thickness.

Earlier tests\textsuperscript{18} on brick piers and wallettes indicated that there was no reduction in brickwork strength for mortar joint thicknesses ranging between \(\frac{3}{8}\) ins and \(\frac{3}{4}\) ins.
The tests on brickwork cubes and piers described in section 3.43 (see also Tables 3.9 and 3.10 and Fig. 2.5) indicate a loss in strength of 12% for a joint thickness of \( \frac{3}{4} \) ins when compared with \( \frac{3}{8} \) ins joints.

American tests 19 on wallettes six bricks high indicate a loss in strength of 46% for similar joint thicknesses and these values are compared in Table 3.10.

3.704 Effect of reinforcement

Alternate horizontal bed joints in walls 8 and 9 were reinforced with B.R.C. Fabric made up of 2 steel wires approximately \( \frac{1}{8} \) ins dia and \( \frac{3}{8} \) ins apart. The \( \frac{3}{8} \) ins cross centres of the wires gave very little cover when placed in the mortar joint (approximately \( \frac{1}{2} \) ins) and for wall 9, the wires were closed to \( \frac{3}{4} \) c/cs.

Wall 8 failed at a load of 149.5 tons and showed no significant increase in strength over the average load of 153.5 tons for the similar but unreinforced walls 4 to 7 inclusive built by the same bricklayer.

The strength of wall 9 was greater than the 200 ton capacity of the testing frame and consequently was not tested to destruction. It was tested at 148 days and much of the increased strength over that of wall 8 may be due to the age at test. Tests on the accompanying brickwork cubes indicate an increase in strength of 32% at 148 days over that at 8 days and support this view. The results are inconclusive and further tests are required.

The size of the frog may influence the efficiency of the reinforcement as discussed in section 3.701.

3.705 Effect of eccentricity

The mean strength of walls 10 and 11 constructed plumb but \( \frac{3}{4} \) ins
eccentric to the applied load was 125.5 tons compared with 153.5 tons for the similar but axially loaded walls 4 to 7 inclusive. This represents a reduction in strength of 18% and compares reasonably well with earlier tests where the strength of a 1 ins eccentrically loaded wall was 17% weaker than a similar but axially loaded wall.

The mean strength of walls 12 and 13 constructed 3/4 ins off plumb was 121 1/2 tons after reducing the strength of wall 13 from 131 tons to 123 tons to give the equivalent wall strength for 6235 lb/ins$^2$ bricks in accordance with Fig. 2.1. This represents a reduction in strength of 21% when compared with the mean value of 153.5 tons for axially loaded walls 4 to 7 inclusive.

If the "equivalent" eccentricity is taken as 3/4 ins (t/6) for walls 10 and 11 and walls 12 and 13 the theoretical reduction in strength (C.P.111) compared with similar but axially loaded walls is 46.5% (From Fig. 2.6).

If the slab/wall junction is considered fully fixed and the bending moment distributed to the slabs and wall in proportion to their stiffnesses the "equivalent" eccentricity resulting in a strength reduction of approximately 20% is $t/12$ ($m = 1/2$) (from Fig. 2.6) and the ratio of Young's modulus for concrete to brickwork is about 0.75 to 1. (see Table 3.14).

The mean value for the mortar crushing strength used for walls 12 and 13 was 730 lb/ins$^2$ and for walls 2 to 7 inclusive 1800 lb/ins$^2$. The influence of this on brickwork strength would be in the ratio 1:1.35 when proportional to the cube root and 1:1.25 when proportional to the fourth root of mortar strength.
From fig. 2.1 the brickwork strength ratio is 1:1.14 for a 6235 lb/ins² brick. However, it is not altogether certain that the mortar cube properties relate directly to those in the mortar joint and also it is not clear from the wall tests that the mortar strength relationship is as shown in Fig. 2.1.

For example wall 6 having a mortar strength of 865 lb/ins² was stronger than walls 2, 3, 4 constructed of considerably stronger mortar.

The rate of loading however, may have influenced these results.

Wall 14 built 3/4 ins off plumb was stronger than similar but axially loaded walls 15 and 16. This unexpected result may have been due to a small horizontal crack passing through the full 4 ins thickness of concrete in the upper slab about mid span on the w.s elevation for tests 15 and 16.

3.706 Slenderness ratio

The nominal slenderness ratio for all the wall tests was 16.5 based on the nominal thickness of 4 1/8 ins and 17.8 when based on the actual thickness of 4.125 ins.

The earlier tests described in section 2.6 indicate that for this range of slenderness ratio there is no appreciable reduction in strength over stiffer walls. Slenderness is discussed in sections 2.6 6.3 and 6.4.

3.707 Rate of loading

It is known that the rate of loading of concrete specimens will influence the ultimate strength and it might be expected that brickwork will also be influenced by this.

The rate of loading of the brickwork cubes and piers was maintained
at 2,000 lb/ins$^2$ per minute as for concrete cubes and as agreed between 28 consulting engineers for site control testing.

Simms gives an overall rate of loading of 20 to 30 lb/ins$^2$ per minute for 9 ins wall tests and the rate of loading at V.S.Z.S. (Switzerland) was reported as 2 to 3 kg/cm$^2$/min (28 - 43 lb/ins$^2$/min). At E.M.P.A. (Switzerland) the rate of loading was reported as 30 tonne/min for a wall 15 cm x 75 cm (380 lb/ins$^2$/min).

The rates of loading for the wall tests described in this thesis are given in brackets in column 'h' of Tables 3.4 and 3.5.

Comparing similar walls 2 to 7 it can be seen that walls 2, 3 and 4 have lower strengths than walls 5, 6 and 7 and were loaded over a longer period of time.

Walls 2 and 3 were built by bricklayers 1 and 2 and are discussed also under workmanship. Wall 4 however, was built by bricklayer 3 as were walls 5, 6 and 7 and there is a noticeable reduction in strength for wall 4, loaded over the longest period.

3.708 Workmanship

Comparing walls 2, 3 and 4 built by three different bricklayers it can be seen that wall 3 having 3/16 ins thick bed joints has the lowest strength.

From sections 2.4 and 3.703 it follows that a reduction in joint thickness will result in an increase in brickwork strength, and other factors being equal, wall 3 would be expected to have a greater strength than walls 2 and 4. The duration of loading is similar for each of the walls, and the reduction in strength would appear to be due to workmanship; bricklayer 1 building weaker walls than bricklayers 2 and 3.

The 15 tons difference in ultimate loads between walls 2 and 4 could
be due to differences in eccentric loading resulting from walls being built slightly off plumb or off centre.

3.709 Load factors

For the axially loaded walls the load factor, based on the Code of Practice, C.P.111, 1964 varied between 6.3 and 12.3.

A judicious load factor for brickwork construction might be between 4 and 6.

3.710 Mode of failure

For axial loading, several modes of failure were observed. For example, wall 3 failed by vertical splitting and local crushing at the top courses. Wall 1 failure was similar but also exhibited crushing at mid height.

Wall 4 failed by shear at 45° and wall 5 failed by a central vertical crack and 2 shear cracks at 45° in lower part of the wall. The eccentric walls normally failed by crushing in the top 3 or 4 courses on the compression face and sometimes accompanied by splitting within the 4\(\frac{1}{8}\) ins brick thickness in the top 5 or 6 courses.

3.711 Demec strain readings

The demec strain readings for walls 1, 3 and 4 are shown graphically in Figs. 3.7, 3.8, 3.14, 3.15, 3.19 and 3.20.

For wall 1 certain studs were positioned on the mortar joint and the results shown in Fig. 3.7 have been modified in Fig. 3.8 to remain in keeping with results for walls 3 and 4.

There appeared to be no clear strain pattern on the wall face. Comparing the horizontal strains wall 1 (Fig. 3.8) showed an increase in strain at \(\frac{3}{4}\) wall height where as wall 3 (Fig. 3.14 and 3.15) showed an increase in strain at the extreme top and bottom of the wall. The mean
values for wall 4 however indicate an even strain throughout the wall except for a local increase at the bottom.

Comparing the vertical strains wall 1 (Fig. 3.8) showed a slight increase from top to bottom and for wall 4 (Fig. 3.19 and 3.20) the increase was greater (over 30%).

Wall 3 (Fig. 3.14 and 3.15) however, indicated an increase in strain at the top and bottom of the wall.

It would appear from these results that there is no consistent strain distribution throughout the wall face, and this has been observed in similar tests carried out abroad.

3.712 Young's modulus

It can be seen from the stress/strain curves that the secant modulus decreases with load.

For the walls constructed in 1:3 cement/sand mortar the secant modulus ranged between 2.7 and 3.5 x 10^6 lb/ins^2 at a stress of 155 lb/ins^2 and 1.5 to 2.3 x 10^6 lb/ins^2 at a stress of 1240 lb/ins^2. (Brick strength 6,000 lb/ins^2)

Walls 5 and 9 (Figs. 3.22 and 3.27) were noticeably stiffer at the higher stresses. This may be due in part to the high mortar strength of 2450 lb/ins^2 for wall 5 and the joint reinforcement in wall 9, although if mortar strength alone has some influence it might be expected that walls 2 (Fig. 3.11) and 7 (Fig. 3.25) having mortar strengths of 2015 and 2335 lb/ins^2 respectively might be similar to wall 5. In fact, they are not and the age at test may also have some influence since wall 5 was tested at 78 days compared with 46 and 18 days for wall 2 and 7 respectively.

For wall 1, constructed in a 1:1:6 cement/lime/sand mortar and walls 14 and 16 constructed in a 1:3 cement/sand
mortar the secant modulus ranged between 0.9 and $1.6 \times 10^6$ lb/ins$^2$ at a stress of 155 lb/ins$^2$ and 0.5 and $0.9 \times 10^6$ lb/ins$^2$ at a stress of 620 lb/ins$^2$. (Brick strength less than 5,000 lb/ins$^2$)

3.7.3 Strength/age relationship

From Table 3.9 it can be seen that the increase in brickwork cube strength from 7 to 28 days for series B cubes was 113 and 112% for the 3 course and 4 course high cubes respectively.

A summary of the strength/age relationship for the brickwork cube tests is given in Table 3.13 and for the series B and C cubes the increase in brickwork strength with age is approximately proportional to the increase in the cube root of the mortar cube strength.

For the cubes tested with walls 2, 9 and 10 the relationship appears to lie between the square and cube root of the mortar cube strength.

There would seem however, inconsistencies in the results since the mean strength ratios of each joint thickness and cube size for series B given in Table 3.9 show a decrease in strength with age for the $\frac{1}{4}$ ins joint 3 course high cubes.

There is also a decrease for the $\frac{1}{4}$, $\frac{3}{8}$ and $\frac{5}{8}$ ins joints.

It may be that the 12 to 13% increase in strength with age is well within differences in strength due to variations in workmanship and firm conclusions cannot be drawn.

3.8 Conclusions

1. The typical mode of failure by transverse splitting indicates that the tensile strength of the brick and also the properties of the horizontal mortar joints such as Young's modulus and Poisson's ratio
1. Maybe of primary importance in determining the strength of brickwork.

2. There appeared to be no clear pattern of horizontal and vertical strains over the wall face and a more detailed study is required. Uneven bedding of the top slab may have a considerable influence on this.

3. The secant modulus decreased considerably with increase in load and for the 6235 lb/ins\(^2\) brick in 1:3 cement/sand mortar values ranged between 2.7 and 3.5 \( \times \) \( 10^6 \) lb/ins\(^2\) at a stress of 155 lb/ins\(^2\) and 1.5 and 2.3 \( \times \) \( 10^6 \) lb/ins\(^2\) at a stress of 1240 lb/ins\(^2\).

4. The effect of brick strength on the strength of brickwork was generally in agreement with the work of the Building Research Station.

The effect of mortar strength appeared to be less significant than that shown in the work of the Building Research Station and the strength of brickwork may be proportional to only the cube or fourth root of the mortar cube crushing strength.

5. The effects of horizontal reinforcement in the bed joints is not fully understood. The type of reinforcement and size of brick frog may considerably influence the strength of reinforced brickwork.

6. Tests on brickwork cubes and piers indicate that the strength of brickwork may be increased by 15% when the joint thickness is decreased from \( \frac{3}{4} \) ins to \( \frac{1}{4} \) ins. A reduction in strength of 23% was observed when the joint thickness was increased from \( \frac{3}{8} \) ins to 1 ins. American tests on six brick high wallets indicate that the reduction in brickwork strength may be greater.
7. The reduction in strength of walls resulting from eccentric loading when loaded between reinforced concrete floor slabs is considerably less than when loaded between knife edges. A reasonable design basis for walls eccentrically loaded between floor slabs would be to consider the joint as fully fixed and to distribute the bending moment due to the eccentric load in proportion to the slab and wall stiffnesses.

8. The code of practice C.P.111 (1964) limit of 18 for the slenderness ratio of load bearing walls appears to be conservative for walls restrained by concrete slabs.

9. The rate of loading of test walls appears to influence the ultimate strength and walls loaded over \( 1\frac{1}{2} \) hours failed at loads approximately 20% less than walls loaded over \( \frac{1}{2} \) hour.

10. The load factors for the axially loaded wall tests varied between 6.3 and 12.3. A mean value of 5 or 6 would be adequate for design.

11. The ratio of \( E \) concrete to \( E \) brickwork appeared to be 0.75 to 1 when calculating the theoretical stiffnesses of members. The low value for the concrete slabs may be due to the appearance of small cracks across the 3ft width of the upper concrete slab.
### Table 3.1

**Brick properties**

<table>
<thead>
<tr>
<th>Type of brick</th>
<th>Crushing strength B.S.1257 and 1391 (lb/ins²)</th>
<th>Strength range (lb/ins²)</th>
<th>Water absorption 24 hour (by wt.)</th>
<th>Wall no.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pressed double frog (batch 1) 2⅝ ins deep</td>
<td>6655</td>
<td>2660</td>
<td>5.8</td>
<td>2 to 12 inclusive</td>
</tr>
<tr>
<td>Pressed double frog (batch 2) 2⅝ ins deep</td>
<td>6150</td>
<td>2950</td>
<td>5.9</td>
<td>excluding 9</td>
</tr>
<tr>
<td>Pressed double frog (batch 2) 2⅝ ins deep</td>
<td>6400</td>
<td>2470</td>
<td>4.5</td>
<td></td>
</tr>
<tr>
<td>Perforated wire cut. 2⅝ ins deep</td>
<td>5735</td>
<td>1700</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Perforated wire cut. 2⅝ ins deep</td>
<td>mean 6235</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Single frog 2⅝ ins deep</td>
<td>7110</td>
<td>3640</td>
<td>6.1</td>
<td>9, 13 and series of brickwork cube tests.</td>
</tr>
<tr>
<td>Single frog 2⅝ ins deep</td>
<td>6625</td>
<td>2110</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Single frog 2⅝ ins deep</td>
<td>mean 6855</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Single frog 2⅝ ins deep</td>
<td>4825</td>
<td>1280</td>
<td>28.7</td>
<td>1</td>
</tr>
<tr>
<td>Single frog 2⅝ ins deep</td>
<td>3710</td>
<td>1350</td>
<td></td>
<td>14 to 16 inclusive</td>
</tr>
</tbody>
</table>

### Table 3.2

**Sand Sieve Analysis - Percentage by weight passing B.S. Sieves**

<table>
<thead>
<tr>
<th>B.S. Sieve no.</th>
<th>Sand No. 1</th>
<th>Sand No. 2</th>
<th>Sand No. 3</th>
<th>B.S. 1200 Table 1.</th>
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</thead>
<tbody>
<tr>
<td>7</td>
<td>99.5</td>
<td>98.7</td>
<td>97</td>
<td>90 - 100</td>
</tr>
<tr>
<td>14</td>
<td>94</td>
<td>97.5</td>
<td>96</td>
<td>70 - 100</td>
</tr>
<tr>
<td>25</td>
<td>81.8</td>
<td>93.2</td>
<td>94</td>
<td>40 - 100</td>
</tr>
<tr>
<td>52</td>
<td>57.2</td>
<td>47.5</td>
<td>86.5</td>
<td>5 - 70</td>
</tr>
<tr>
<td>100</td>
<td>21.6</td>
<td>13.4</td>
<td>27.1</td>
<td>0 - 15</td>
</tr>
<tr>
<td>Wall</td>
<td>1 to 11 incl.</td>
<td>12.13</td>
<td>15.16</td>
<td>14</td>
</tr>
</tbody>
</table>
Table 3.3

Crushing strength of 4 ins mortar cubes (related to wall tests 1 to 16 inclusive)

Cubes vibrated in 2 ins layers with Kango Hammer and Tank cured. 1:3 cement/sand mix by volume.
Except wall 1, which was 1:1:6 cement/lime/sand by volume.
* Age at test in days. Mean strengths shown in brackets.

<table>
<thead>
<tr>
<th>Wall no.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
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<td></td>
<td></td>
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</tr>
<tr>
<td>Crushing</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>strength</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(lb/ins²)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>38*</td>
<td>645(660)</td>
<td>1400</td>
<td>1360</td>
<td>1740</td>
<td>2660</td>
</tr>
<tr>
<td>2</td>
<td>1295</td>
<td>1225(1305)</td>
<td>1850</td>
<td>1510</td>
<td>2100</td>
<td>2450</td>
</tr>
<tr>
<td>3</td>
<td>710</td>
<td>66* 475(595)</td>
<td>1890</td>
<td>1660</td>
<td>2450</td>
<td>78* 2590(2450)</td>
</tr>
<tr>
<td>4</td>
<td>45* 2210(2015)</td>
<td>1820</td>
<td>1410</td>
<td>1395</td>
<td>3080</td>
<td>2390</td>
</tr>
<tr>
<td>5</td>
<td>645</td>
<td>466* 700(675)</td>
<td>1640</td>
<td>1435</td>
<td>66* 475(595)</td>
<td>2140</td>
</tr>
<tr>
<td>6</td>
<td>418* 1360(1585)</td>
<td>1435</td>
<td>1260</td>
<td>1640</td>
<td>3180</td>
<td>2590(2450)</td>
</tr>
<tr>
<td>7</td>
<td>18* 1490(1540)</td>
<td>3180</td>
<td>2590(2450)</td>
<td>2390</td>
<td>29*3030(3000)</td>
<td>2390</td>
</tr>
<tr>
<td>8</td>
<td>5*</td>
<td>895(630)</td>
<td>600</td>
<td>8* 610(770)</td>
<td>610(770)</td>
<td>8* 77</td>
</tr>
<tr>
<td>9</td>
<td>3180</td>
<td>3020(3005)</td>
<td>835(765)</td>
<td>26* 2380(1855)</td>
<td>26* 2380(1855)</td>
<td>148* 3020(3005)</td>
</tr>
<tr>
<td>10</td>
<td>1260</td>
<td>3180</td>
<td>1830(1400)</td>
<td>420</td>
<td>2030</td>
<td>2030</td>
</tr>
<tr>
<td>11</td>
<td>1640</td>
<td>8* 610(770)</td>
<td>610(770)</td>
<td>3180</td>
<td>2590(2450)</td>
<td>3080</td>
</tr>
<tr>
<td>12</td>
<td>7* 1060(805)</td>
<td>600</td>
<td>600</td>
<td>18* 1010(940)</td>
<td>18* 1010(940)</td>
<td>148* 3020(3005)</td>
</tr>
<tr>
<td>13</td>
<td>19* 1060(805)</td>
<td>600</td>
<td>600</td>
<td>18* 1010(940)</td>
<td>18* 1010(940)</td>
<td>148* 3020(3005)</td>
</tr>
<tr>
<td>14</td>
<td>19* 1060(805)</td>
<td>600</td>
<td>600</td>
<td>18* 1010(940)</td>
<td>18* 1010(940)</td>
<td>148* 3020(3005)</td>
</tr>
<tr>
<td>15</td>
<td>19* 1060(805)</td>
<td>600</td>
<td>600</td>
<td>18* 1010(940)</td>
<td>18* 1010(940)</td>
<td>148* 3020(3005)</td>
</tr>
<tr>
<td>16</td>
<td>19* 1060(805)</td>
<td>600</td>
<td>600</td>
<td>18* 1010(940)</td>
<td>18* 1010(940)</td>
<td>148* 3020(3005)</td>
</tr>
</tbody>
</table>
Table 3.4

Results of wall tests 1 to 8 inclusive

*1 First figure in brackets denotes bricklayer. Second figure in brackets denotes sand
W denotes wet sand.

*2 Value in brackets denotes average rate of loading in lb/ins²/min.

*3 Value in brackets denotes eccentricity calculated from strain readings (using Fig. 2.11.)

<table>
<thead>
<tr>
<th>Wall*1 no.</th>
<th>Brick strength</th>
<th>Mortar strength</th>
<th>Ultimate load</th>
<th>Average compressive stress (lb/in²)</th>
<th>Permissible design stress (C.P.111, 1964) (lb/in²)</th>
<th>Load factor</th>
<th>Duration of loading (minutes)*2</th>
<th>Strength ratio bwk/brick</th>
<th>Loading*3 Age (days)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 (1/1)</td>
<td>4825</td>
<td>660</td>
<td>53.6</td>
<td>835</td>
<td>132</td>
<td>6.3</td>
<td>90 (9)</td>
<td>0.17</td>
<td>Axial (t/22)</td>
<td>38</td>
</tr>
<tr>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1:1:6 mortar</td>
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</tr>
<tr>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>cement/lime/sand</td>
<td></td>
</tr>
<tr>
<td>2 (2/1)</td>
<td>6235</td>
<td>2015</td>
<td>120</td>
<td>1630</td>
<td>217</td>
<td>7.5</td>
<td>90 to 92 tons (12)</td>
<td>0.26</td>
<td>Axial (t/17)</td>
<td>46</td>
</tr>
<tr>
<td>3 (1/1)</td>
<td>6235</td>
<td>1585</td>
<td>114.5</td>
<td>1790</td>
<td>217</td>
<td>8.2</td>
<td>105 to 114.5 tons (17)</td>
<td>0.29</td>
<td>Axial (t/55)</td>
<td>18</td>
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<td></td>
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<td></td>
<td>joint thickness 3/16 ins.</td>
<td></td>
</tr>
<tr>
<td>4 (3/1)</td>
<td>6235</td>
<td>1540</td>
<td>135</td>
<td>2110</td>
<td>217</td>
<td>9.7</td>
<td>90 to 135 tons (23)</td>
<td>0.34</td>
<td>Axial (t/40)</td>
<td>18</td>
</tr>
<tr>
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<td></td>
</tr>
<tr>
<td>5 (3/1)</td>
<td>6235</td>
<td>2450</td>
<td>157</td>
<td>2450</td>
<td>217</td>
<td>11.3</td>
<td>30 (82)</td>
<td>0.39</td>
<td>Axial (t/62)</td>
<td>78</td>
</tr>
<tr>
<td>6 (3/1/a)</td>
<td>6235</td>
<td>865</td>
<td>150</td>
<td>2330</td>
<td>217</td>
<td>10.7</td>
<td>30 to 90 tons (48)</td>
<td>0.37</td>
<td>Axial (t/67)</td>
<td>2</td>
</tr>
<tr>
<td>7 (3/1/w)</td>
<td>6235</td>
<td>2335</td>
<td>172</td>
<td>2670</td>
<td>217</td>
<td>12.3</td>
<td>20 to 22.5 tons (17)</td>
<td>0.43</td>
<td>Axial (t/24)</td>
<td>18</td>
</tr>
<tr>
<td>8 (3/1)</td>
<td>6235</td>
<td>700</td>
<td>149.5</td>
<td>2320</td>
<td>217</td>
<td>10.7</td>
<td>60 (39)</td>
<td>0.37</td>
<td>Axial (t/29)</td>
<td>7</td>
</tr>
</tbody>
</table>
Table 3.5

Results of wall tests 9 to 16 inclusive

<table>
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<tr>
<th>a</th>
<th>b</th>
<th>c</th>
<th>d</th>
<th>e</th>
<th>f</th>
<th>g</th>
<th>h</th>
<th>i</th>
<th>j</th>
<th>k</th>
<th>l</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall*1</td>
<td>Brick strength</td>
<td>Mortar strength</td>
<td>Ultimate load</td>
<td>Average compressive stress</td>
<td>Permissible average design stress (C.P. 111, 1964)</td>
<td>Load factor</td>
<td>Duration of loading</td>
<td>Strength ratio bwk/brick</td>
<td>Loading*3</td>
<td>Age</td>
<td>Remarks</td>
</tr>
<tr>
<td>no.</td>
<td>(lb/ins²)</td>
<td>(lb/ins²)</td>
<td>(tons)</td>
<td>(lb/in²)</td>
<td>(lb/in²)</td>
<td>(minutes)*2</td>
<td>(days)</td>
<td>(ton/brick)</td>
<td>(days)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>6855</td>
<td>1400</td>
<td>&gt;200</td>
<td>&gt;3100</td>
<td>235</td>
<td>&gt;14.3</td>
<td>30 to 150 tons (39)</td>
<td>0.45</td>
<td>Axial</td>
<td>148</td>
<td>Reinforced every second course</td>
</tr>
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<td>(3/1)</td>
<td>3070</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>6235</td>
<td>1100</td>
<td>132</td>
<td>2060</td>
<td>120</td>
<td>17.2</td>
<td>30 tons (69)</td>
<td>0.33</td>
<td>eccentric 3/4&quot; (t/5.5 - t/7.8)</td>
<td>6</td>
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</tr>
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<td>(3/1/4)</td>
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<td></td>
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<tr>
<td>11</td>
<td>6235</td>
<td>630</td>
<td>117</td>
<td>1825</td>
<td>120</td>
<td>15.2</td>
<td>0.29</td>
<td>eccentric 3/4&quot; (t/5.5 - t/3.6)</td>
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<td>(3/1/4)</td>
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<tr>
<td>12</td>
<td>6235</td>
<td>765</td>
<td>120</td>
<td>1630</td>
<td>120</td>
<td>13.6</td>
<td>0.26</td>
<td>3/4&quot; off plumb</td>
<td>7</td>
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<tr>
<td>(3/2)</td>
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</tr>
<tr>
<td>13</td>
<td>6855</td>
<td>695</td>
<td>131</td>
<td>2035</td>
<td>129</td>
<td>15.7</td>
<td>0.30</td>
<td>3/4&quot; off plumb</td>
<td>8</td>
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</tr>
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<td>(3/2)</td>
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<td></td>
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</tr>
<tr>
<td>14</td>
<td>3710</td>
<td>805</td>
<td>81.5</td>
<td>1265</td>
<td>77</td>
<td>16.4</td>
<td>0.34</td>
<td>3/4&quot; off plumb</td>
<td>19</td>
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</tr>
<tr>
<td>(3/3/4)</td>
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</tr>
<tr>
<td>15</td>
<td>3710</td>
<td>770</td>
<td>70</td>
<td>1085</td>
<td>140</td>
<td>7.7</td>
<td>0.29</td>
<td>axial</td>
<td>7</td>
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<td>(3/2)</td>
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</tr>
<tr>
<td>16</td>
<td>3710</td>
<td>940</td>
<td>67</td>
<td>1040</td>
<td>140</td>
<td>7.4</td>
<td>0.28</td>
<td>axial</td>
<td>13</td>
<td>mortar at 17 days</td>
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<td>(3/2/4)</td>
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</tr>
</tbody>
</table>

(*, **, *** see footnotes to table 3.4)
Table 3.6

Summary of permissible design stresses used in wall test results

(Tables 3.4 and 3.5)

<table>
<thead>
<tr>
<th>Brick strength (lb/ins²)</th>
<th>mortar mix cement/lime/sand</th>
<th>Basic compressive stress (lb/ins²)</th>
<th>Permissible design stress lb/ins²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Axial load (R.F. = t/6)</td>
</tr>
<tr>
<td>4825</td>
<td>1:1:6</td>
<td>264</td>
<td>132</td>
</tr>
<tr>
<td>6235</td>
<td>1:0:3</td>
<td>435</td>
<td>217</td>
</tr>
<tr>
<td>6855</td>
<td>1:0:3</td>
<td>470</td>
<td>235</td>
</tr>
<tr>
<td>3710</td>
<td>1:0:3</td>
<td>280</td>
<td>140</td>
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</table>

*1 average stress calculated assuming ratio of stresses (and strains) is 1.53 as obtained from Fig. 2.11.

These are slightly greater than for axial loading due to the 25% increase and the axial loading stresses should be used. i.e. No reduction in load for small eccentricity.
Table 3.7
Strength of brickwork cubes associated with wall tests 1 to 8 inclusive

<table>
<thead>
<tr>
<th>Wall no.</th>
<th>Brick strength (lb/ins²)</th>
<th>Mortar strength (lb/ins²)</th>
<th>Wall strength (lb/ins²)</th>
<th>cube strength No. courses high</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3</td>
<td>4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>4825</td>
<td>660</td>
<td>835</td>
<td>1460</td>
<td>775</td>
<td>80.6</td>
<td>90.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>835 (1035)</td>
<td>1025</td>
<td>970 (925)</td>
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</tr>
<tr>
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<td></td>
<td>2360 (2265)</td>
<td>2170 (2005)</td>
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<td>3100 (3255)</td>
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<td>1980 (2025)</td>
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<td>1950</td>
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<td></td>
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<td>2360 (2265)</td>
<td>2170 (2005)</td>
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<td>6235</td>
<td>2335</td>
<td>2670</td>
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<td>2200 (2410)</td>
<td>1980 (2025)</td>
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<td>8</td>
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<td>2080</td>
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<td>2020</td>
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<td>2510 (2460)</td>
<td>1980 (2025)</td>
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<table>
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<tr>
<th>Strength ratios</th>
<th>wall/cube(3)</th>
<th>wall/cube(4)</th>
<th>cube (3)/cube (4)</th>
</tr>
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<tbody>
<tr>
<td>Age (days)</td>
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<td>8</td>
<td>46</td>
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<tr>
<td>18</td>
<td>18</td>
<td>18</td>
<td>78</td>
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<td>2</td>
<td>2</td>
</tr>
<tr>
<td>18</td>
<td>18</td>
<td>18</td>
<td>18</td>
</tr>
<tr>
<td>18</td>
<td>18</td>
<td>18</td>
<td>18</td>
</tr>
<tr>
<td>78</td>
<td>78</td>
<td>78</td>
<td>78</td>
</tr>
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<td>18</td>
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</tr>
<tr>
<td>7</td>
<td>7</td>
<td>7</td>
<td>7</td>
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</tbody>
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Table 3.8
Strength of brickwork cubes associated with wall tests 9 to 16 inclusive

<table>
<thead>
<tr>
<th>wall no.</th>
<th>brick strength (lb/ins²)</th>
<th>mortar strength (lb/ins²)</th>
<th>wall strength (lb/ins²)</th>
<th>cube strength</th>
<th>Strength ratios</th>
<th>Age (days)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>No. courses high</td>
<td>wall cube (3)</td>
<td>wall cube (4)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3</td>
<td>4</td>
<td>2970</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3070</td>
<td>&gt;3100</td>
<td>3910</td>
</tr>
<tr>
<td>9</td>
<td>6855</td>
<td>1400</td>
<td>--</td>
<td>2790</td>
<td>2380</td>
<td>2230(2330)</td>
</tr>
<tr>
<td>10</td>
<td>6235</td>
<td>1100</td>
<td>--</td>
<td>3230</td>
<td>3080</td>
<td>3080(2965)</td>
</tr>
<tr>
<td>11</td>
<td>6235</td>
<td>630</td>
<td>1825</td>
<td>2640</td>
<td>2640</td>
<td>2330(1965)</td>
</tr>
<tr>
<td>12</td>
<td>6235</td>
<td>765</td>
<td>1630</td>
<td>2540</td>
<td>2580(2595)</td>
<td>2510</td>
</tr>
<tr>
<td>13</td>
<td>6855</td>
<td>695</td>
<td>2035</td>
<td>2570</td>
<td>2580(2595)</td>
<td>1605</td>
</tr>
<tr>
<td>14</td>
<td>3710</td>
<td>805</td>
<td>1265</td>
<td>1780</td>
<td>1540</td>
<td>1380(1565)</td>
</tr>
<tr>
<td>15</td>
<td>3710</td>
<td>770</td>
<td>1085</td>
<td>1780</td>
<td>1505</td>
<td>1470</td>
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<td>16</td>
<td>3710</td>
<td>940</td>
<td>1040</td>
<td>1780</td>
<td>1540</td>
<td>1470</td>
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Table 3.9

Strength of brickwork cubes - influence of joint thickness and age upon strength

* strength of cube in tons.

<table>
<thead>
<tr>
<th>Series</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>B</th>
<th>C</th>
<th>Strength ratio % Series B</th>
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</thead>
<tbody>
<tr>
<td>Age at test (days)</td>
<td>7</td>
<td>7</td>
<td>14</td>
<td>28</td>
<td>28</td>
<td>28/7</td>
</tr>
<tr>
<td>Height of cube in brick courses</td>
<td>3   4</td>
<td>3   4</td>
<td>3   4</td>
<td>3   4</td>
<td>3   4</td>
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</tr>
<tr>
<td>Joint thickness (ins)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1/4</td>
<td>103*</td>
<td>96</td>
<td>97</td>
<td>85</td>
<td>89</td>
<td>102</td>
</tr>
<tr>
<td>3/8</td>
<td>95</td>
<td>85</td>
<td>74</td>
<td>71</td>
<td>91</td>
<td>96</td>
</tr>
<tr>
<td>1/2</td>
<td>70</td>
<td>80</td>
<td>74</td>
<td>80</td>
<td>61</td>
<td>80</td>
</tr>
<tr>
<td>5/8</td>
<td>73</td>
<td>80</td>
<td>61</td>
<td>63</td>
<td>80</td>
<td>75</td>
</tr>
<tr>
<td>3/4</td>
<td>69</td>
<td>75</td>
<td>56</td>
<td>77</td>
<td>71</td>
<td>86</td>
</tr>
<tr>
<td>7/8</td>
<td>59</td>
<td>59</td>
<td>46</td>
<td>46</td>
<td>79</td>
<td>68</td>
</tr>
<tr>
<td>1&quot;</td>
<td>60</td>
<td>66</td>
<td>53</td>
<td>59</td>
<td>75</td>
<td>69</td>
</tr>
<tr>
<td>Mean strength ratio % cube(3) cube(4)</td>
<td>102.2</td>
<td>119</td>
<td>97.6</td>
<td>120</td>
<td>116</td>
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</tr>
<tr>
<td>Total load per series</td>
<td>529</td>
<td>535</td>
<td>535</td>
<td>604</td>
<td>571</td>
<td>113</td>
</tr>
<tr>
<td>Mean increase in strength with age. Series B at 7 days taken as 100%</td>
<td>100</td>
<td>100</td>
<td>131</td>
<td>113</td>
<td>107</td>
<td>117</td>
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Table 3.10
Strength ratios of brickwork cubes and piers
having different mortar joint thicknesses.
* strength with \( \frac{3}{8} \) ins joint taken as unity

<table>
<thead>
<tr>
<th>Joint thickness (ins)</th>
<th>Strength of ratio</th>
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<tr>
<td></td>
<td>Brickwork cubes and piers</td>
<td>American tests</td>
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<tr>
<td>( \frac{1}{4} )</td>
<td>1.15</td>
<td>1.12</td>
</tr>
<tr>
<td>( \frac{3}{8} )</td>
<td>1.00*</td>
<td>1.00</td>
</tr>
<tr>
<td>( \frac{1}{2} )</td>
<td>0.99</td>
<td>0.84</td>
</tr>
<tr>
<td>( \frac{5}{8} )</td>
<td>0.95</td>
<td>0.69</td>
</tr>
<tr>
<td>( \frac{3}{4} )</td>
<td>0.88</td>
<td>0.54</td>
</tr>
<tr>
<td>( \frac{7}{8} )</td>
<td>0.79</td>
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<tr>
<td>1</td>
<td>0.77</td>
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Table 3.11
Crushing strength of 4 ins mortar cubes
(related to brickwork cube and pier tests)

<table>
<thead>
<tr>
<th>Age at test (days)</th>
<th>7</th>
<th>14</th>
<th>28</th>
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</thead>
<tbody>
<tr>
<td>Series</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>8.1</td>
<td>10.9</td>
<td>11.3(10.1)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(141.5 lb/in^2)</td>
</tr>
<tr>
<td>B</td>
<td>8.25</td>
<td>8.49</td>
<td>11.76</td>
</tr>
<tr>
<td></td>
<td>11.3</td>
<td>11.65</td>
<td>15.45</td>
</tr>
<tr>
<td></td>
<td>13.75(10.7)</td>
<td>20*(15.8)</td>
<td>(2210 lb/in^2)</td>
</tr>
<tr>
<td></td>
<td>(1500 lb/in^2)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>11.5</td>
<td>12.75</td>
<td>16.7</td>
</tr>
<tr>
<td></td>
<td>14.31</td>
<td>17.61</td>
<td>18 (17.4)</td>
</tr>
<tr>
<td></td>
<td>15.96 (13.7)</td>
<td>18 (17.4)</td>
<td>(2435 lb/in^2)</td>
</tr>
<tr>
<td></td>
<td>(1920 lb/in^2)</td>
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</table>

Load in tons, mean load in tons and stress in lb/ins^2 in brackets.
Table 3.12
Relationship between eccentricity and stress block

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<th>Eccentricity</th>
<th>0</th>
<th>t/12</th>
<th>t/6</th>
<th>t/4</th>
<th>t/3</th>
<th>t/2.4</th>
<th>t/2</th>
</tr>
</thead>
<tbody>
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<td></td>
<td>m = 0</td>
<td>m = 1/2</td>
<td>m = 1</td>
<td>m = 1 1/2</td>
<td>m = 2</td>
<td>m = 2 1/2</td>
<td>m = 3</td>
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<tr>
<td>Stress</td>
<td></td>
<td></td>
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</tr>
<tr>
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<td>for calculation of average stresses</td>
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<td>t</td>
<td>t</td>
<td>0.75t</td>
<td>0.5t</td>
<td>0.25t</td>
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<tr>
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<td>2 x average</td>
<td>2 x average</td>
<td>2 x average</td>
<td>20 lb/in²</td>
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<tr>
<td>Eccentricity load reduction factors*</td>
<td>1,000</td>
<td>0.833</td>
<td>0.625</td>
<td>0.469</td>
<td>0.312</td>
<td>0.15t</td>
<td>0.050**</td>
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</tbody>
</table>

* these values are the theoretical load reduction factors for eccentric loadings only, assuming a linear stress/strain relationship across the wall section NOT including slenderness effects, but allowing an increase in edge stress of up to 25% over the permissible uniform stress provided such increase is due solely to the eccentric loading.

** Tensile stress of 10 lb/ins² allowed.

* Tensile stress of 10 lb/ins² allowed.

** Tensile stress of 10 lb/ins² allowed.
Table 3.13

Increase in brickwork cube strength with age

<table>
<thead>
<tr>
<th>Test</th>
<th>Strength ratio % with age</th>
<th>Mortar strength lb/ins$^2$</th>
<th>4 ins cube</th>
<th>square root</th>
<th>cube root</th>
<th>fourth root</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>Height of cube in brick courses</td>
<td>Age (days)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>4</td>
<td>6</td>
<td>8</td>
<td>46</td>
<td>1305</td>
</tr>
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<td>Table 3.7 wall 2</td>
<td>136.5</td>
<td>115</td>
<td>135</td>
<td>131.5</td>
<td>6</td>
<td>1400</td>
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<td>Table 3.8 wall 9</td>
<td>135</td>
<td>131.5</td>
<td>117</td>
<td>127.5</td>
<td>6</td>
<td>1100</td>
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<td>Table 3.8 wall 10</td>
<td>117</td>
<td>127.5</td>
<td>113</td>
<td>112</td>
<td>7</td>
<td>1500</td>
</tr>
<tr>
<td>Table 3.9 series B</td>
<td>113</td>
<td>112</td>
<td>107</td>
<td>90</td>
<td>14</td>
<td>1920</td>
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Table 3.14

Slab and wall stiffnesses for different E values

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<th>I/L</th>
<th>Ratio of Young's modulus</th>
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<tr>
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<td>E (Concrete)</td>
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<tr>
<td>slab</td>
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</tr>
<tr>
<td>slab</td>
<td>( 4.5 \times \frac{3}{8.5} )</td>
</tr>
<tr>
<td>Total stiffness at junction</td>
<td>20.55</td>
</tr>
<tr>
<td>Proportion of total bending moment transferred to wall</td>
<td>0.416</td>
</tr>
<tr>
<td>Equivalent eccentricity assuming eccentricity for total bending moment equals t/6</td>
<td>t/14.5</td>
</tr>
</tbody>
</table>
POSITION OF
EXTENSOMETERS

fig 3.3

\[ E \times 10^6 \sec^{-1} \]

fig 3.6
shortening \( \times 10^{-2} \) gauge length 7/8

wall 4
FIG. 3.4 - Wall 1 W.S. elevation at failure

FIG. 3.5 - Wall 1 C.P. top of wall at failure
strain $1.01 \times 10^{-4}$ in/ins

HORIZONTAL

W.S. ELEVATION

demec strain readings, gauge length $8'$ wall
strain \[ 1.01 \times 10^{-4} \text{ ins/ins} \]

HORIZONTAL

VERTICAL

W.S. ELEVATION

fig 3.8
demec strain readings gauge length 8'
wall 1
FIG. 3.9 - Wall 2 C.P. elevation at failure

FIG. 3.10 - Wall 2 W.S. elevation at failure
FIG. 3.12 - Wall 3 W.S. top of wall at failure
fig 3.13  shortening $\times 10^2$ gauge length $\frac{1}{8}$
strain $1 \times 10^{-4}$ ins/ins

HORIZONTAL

VERTICAL

C.P. ELEVATION

in 3:14 demec strain readings gauge length 8" wall 3
strain \(1.01 \times 10^{-4}\) ins/ins.

HORIZONTAL

W.S. ELEVATION

VERTICAL

fig 3.15 demec strain readings gauge length 8' wall 3
FIG. 3.16 - Wall 4 C.P. elevation at failure

FIG. 3.17 - Wall 4 W.S. centre of wall at failure
fig 3.18  shortening $\times 10^{-2}$, gauge length $\frac{7}{2}$"
strain $1 \times 10^{-4}$ ins/ins

HORIZONTAL

VERTICAL

C.P. ELEVATION
strain $1 \cdot 01 \times 10^{-4}$ ins/ins

HORIZONTAL

W.S. ELEVATION

VERTICAL

fig 3.20 demec strain readings gauge length 8'

wall 4
FIG. 3.21 - Wall 5. Failure of earlier wall test similar to wall 5.
fig 3.22 shortening x 10^{-2} gauge length 72' wall 5
Fig 3.23  
shortening $\times 10^2$  
gauge length 72'  

wall 6
FIG 3.24. - Wall 7 W.S. elevation at failure
fig 3.25  shortening x 10^2 gage length ft

LOAD TONS

E x 10^6 secant
fig 3-26  shortening $\times 10^{-2}$, gauge length $\frac{1}{2}$
Fig. 3.27. Shortening x 10^2 versus load, gauge length 72 in.
FIG. 3.28 - Wall 10 W.S. top of wall at failure
Figure 3.29: Shortening x 10^-2 vs. wall length, wall 10
fig 3.30 shortening $\times 10^{-2}$ gauge length $\frac{72}{8}$
FIG. 3.31 - Wall 12 W.S. elevation at failure

FIG. 3.32 - Wall 12 C.P. centre of wall at failure
FIG. 3.33 - Wall 13 C.P. top of wall at failure
Shortening $\times 10^{-2}$ vs. load tons

Gauge length $\frac{7}{2}''$
FIG. 3.37 - Wall 15 W.S. bottom of wall at failure
4.1 Introduction

In Switzerland and elsewhere in Europe the traditional external wall construction for weather protection and thermal insulation in normal low rise and high-rise domestic building is a solid brick wall rendered on the outside face.

In Great Britain the cavity wall has been the norm for over 2 decades because of its superior qualities over solid brickwork for weather protection and thermal insulation. Its use however, involves structural uncertainties such as the distribution of load and bending moment at the floor slab/wall junction. Research to date on cavity walls is limited to a series of five wall tests which were primarily concerned with the performance of the wall ties.

Designers have, of necessity, made general assumptions concerning the design of cavity walls and the growing advances in the application of cavity wall load-bearing brickwork to multi-storey building necessitate that more detailed information related to the structural behaviour at floor slab/cavity wall junction be made available.

4.2 Fixity at floor slab/wall junction

As far as is known, no tests to investigate the degree of fixity at floor slab/cavity wall junctions have been carried out.

Sahlin has carried out preliminary tests on six solid storey height walls shown in Fig. 4.1. The walls were tested between knife edge bearings at their lower end and ball and socket joints at the upper end. Walls 2, 3 and 4 were loaded eccentrically by means of a load distributing beam, and the eccentricity immediately below the
concrete slab was intended to be 5 cm. It appears that the actual eccentricity at failure was measured as the horizontal distance between the vertical line of the applied load P and the centre line of the wall. The eccentricities together with a summary of the test data are given in Table 4.1. It can be seen that the C.P. 111 theoretical reduction in load for eccentric loading is insufficient and that the Swiss Norm 113 recommendations are more accurate.

The stress/strain relationship is given in Fig. 4.2.

The deflection of the wall and the angle of rotation of the concrete slab were measured by means of scales attached to the wall and slab, using a theodolite, all measurements being made by reference to a vertical plane passing through the theodolite.

The angular rotations at the slab/wall junction for the wall, slab and joint are shown graphically in Figs. 4.3, 4.4 and 4.5 for increase in moment and the rotation of the joint shown in Fig. 4.5 is the difference between the slab and wall rotations and represents the relaxation at the joint. It can be seen that there is an approximately linear relationship between moment and rotation. The rotations at maximum moment were 7.5, 12 and 4.5 x 10^-3 radians for the wall, slab and joint respectively. The high relaxation of 4.5 x 10^-3 radians for the slab representing a 60% increase above the wall rotation is due chiefly to the low mortar strengths of 120 lb/in^2 for walls, 1, 2, 3, 5 and 6 and 225 lb/in^2 for wall 4.

Comparing walls 2 and 4 of similar construction and loading except that the mortar strengths are 120 and 225 lb/ins^2 respectively
it can be seen from Figs. 4.3, 4.4 and 4.5 that the slab, wall and joint rotations are similar. This is a surprising result since the mortar strength of 225 lb/ins$^2$ for wall 4 is almost double that for wall 2. Although these mortar strengths are based on tests on cylinders and may not represent exactly the equivalent cube strength they should be sufficiently accurate (to say $\pm 20\%$) for a general comparison.

Sahlin's calculated values for wall rotation, given in Fig. 4.3 show a significant difference for the three eccentrically loaded walls and indicate that mortar strength is of importance.

In this country a mortar strength of at least 600 lb/ins$^2$ would be used for load-bearing construction and it would be expected that this increase in mortar strength would reduce the slab relaxation and also the wall, slab and joint rotations generally.

It was concluded from these preliminary tests that initial cracking takes place above the edge of the concrete slab and this confirms Shalin's theoretical studies concerning the effect produced by the restraining moment on the horizontal stresses in brickwork. Before crack formation above the concrete slab, the connection between the wall and the slab is reliable. At crack development the relaxation - or the difference in angle of rotation between the wall and the slab - increases considerably. The restraining moment on the concrete slab at failure was 39,800 lb ins/ft. and the slab should therefore be provided with top reinforcement at the wall.

A second series followed the preliminary tests and these were designed to represent in principle the floor slab/external solid wall detail met in normal Swedish Building. Certain details were
simplified and the main tests were made on narrow slabs spanning in one direction, as shown in Fig. 4.6 and 4.7.

The load acting on the wall situated above the slab was applied to give a known constant eccentricity $e_u$.

The wall was suspended by means of pendulums comprising vertical steel ties hinged at the top so that the horizontal forces at the foot of the wall could be accurately measured by recording the strain in the horizontal tie bars. The observed values of the horizontal force were used for calculating the eccentricity $e_1$ of the compressive force acting on the wall on a level with the bottom surface of the slab. The ties also laterally restrained the wall. Lateral support to the walls was also provided at slab level. A single 50 ton jack situated on the top of each wall simulated load due to storeys above and the jacks were hinged at both ends in order to determine the line of action of the force. A single jack was used to apply central floor loading to the slab and the ratio of wall load/slab load was maintained constant for individual tests because one pump operated all three jacks.

A summary of the test results is given in Table 4.2 and it can be seen that Sahlin's eccentricities $e_1$ calculated from the horizontal force in the tie bar are about double those calculated from the maximum mean and edge stresses at failure. The following general conclusions were drawn from the tests. The eccentricity of loading decreased as the loading increased and decreased rapidly in the initial portions of the curves, becoming more or less constant at higher loads.

The deformations of the slab and wall remained similar up to the
instant when the wall cracked above the slab. The angle of rotation of the slab then became greater than that of the wall, because at this stage the wall acted as two separate slender walls above the slab. That half of the wall which was supported on the slab followed the displacement of the slab, whereas the other half of the wall rotated through an angle that was smaller (Fig. 4.8).

The bending moment in the floor slab at the supports was almost 50% of the centre span moment.

Failure of the type B specimens normally commenced at between 25% and 50% of the ultimate load by vertical cracks opening above the edge of the slab.

In some cases, the eccentricity of the load was greatly decreased at maximum load and this decrease is reflected in the low values of the ultimate edge stress shown in Table 4.2. In these cases, the value of the edge stress directly preceding the maximum load has also been calculated because this is useful in assessing the causes of failure.

Failure of walls built in 1:5 lime/sand mortar was normally by compression in the brickwork whereas failure of walls built in 2:1:15 lime/cement/sand appears to be due to the ultimate angle of rotation being exceeded.

No Type B walls failed immediately below the slab. All walls underwent final failure at a low level on the internal face of the wall; even in the case where the loading eccentricity was greatest at the slab.
4.3 Vertical load

4.31 Distribution of bending moment and eccentric load

A series of eight walls have been tested to investigate the efficiency of light metal wall ties. Each wall was approximately 9 ft. high and 4 ft 6 ins wide and constructed of Fletton bricks.

Details of the wall construction, loading and performance are given in Table 4.3 and the lateral deflection under eccentric loads shown graphically in Figs. 4.9 and 4.10. Butterfly wire ties spaced 3 ft apart horizontally and 18 ins apart vertically were used for the construction of all the cavity walls.

Walls 1 to 5 were built in 1 to 3 cement/sand mortar and ties for walls 3, 4 and 5 were 9 S.W.G. steel wire. Walls 6, 7 and 8 were built in a 1:1:6 cement/lime sand mortar and ties for walls 7 and 8 were 12 S.W.G. hard drawn copper wire. The method of testing the walls is not clear but it is likely that the axially loaded walls were capped with concrete blocks and steel channels and tested between flat ends as shown in Fig. 2.5. For applying eccentric load a 1 ins sq. bar was most likely used as for the earlier tests shown in Fig. 2.5.

It can be seen from Table 4.3 that the 1 ins eccentrically loaded wall 2 is only 20% weaker than the similar but axially loaded wall 1. This is a surprisingly small reduction in strength for 1 ins eccentricity since other tests shown graphically in Fig. 2.5 show a reduction in strength of 77% for 1 ins eccentricity. The mortar strength for the tests summarised in Table 4.3 is not given by it is likely to be very similar to that used for the tests shown in Fig. 2.5 since the ultimate
axial loads of 128 tons and 138.5 tons for the single leaf walls are similar.

It can be seen from Figs. 4.9 and 4.10 that the wire ties act efficiently both in tension and compression and within the normal range of design loading the ties ensure that the two leaves of the cavity wall take up the same arc of bending. Any bending moments will therefore be shared by both leaves in proportion to their stiffnesses.

The bending moment may be due to eccentric loading on one or both leaves of the cavity wall through off central loading from above or from the floor slab support moment being distributed to both leaves in proportion to their stiffnesses. The loaded leaf will support vertical load and moment and the unloaded leaf moment only. The typical stress block for this is given in Fig. 3 of section 11.2 and it can be seen that tension develops in the unloaded leaf. This will normally result in failure of the unloaded leaf in bending before that of the loaded leaf and whilst a surprising result at first sight this is quite logical since tensile stresses are developed. The unloaded leaf of a cavity wall stiffens the loaded leaf without any appreciable gain in strength. It does, however, absorb part of the moment due to the eccentric load and hence reduces the stresses due to bending in the loaded leaf.

It can be seen from Table 4.3 that the axially loaded single leaf wall 1 failed at a higher load (128 tons) than the cavity wall 3 axially loaded on one leaf (104 tons). This is a surprising result and it may be that a repeat test series might shown the reverse.
The cavity walls 4 and 5 eccentrically loaded on one leaf were approximately 20% weaker than a similar wall 3 axially loaded on one leaf. In each case failure was by tension in the unloaded leaf.

For walls 6, 7 and 8 a weaker mortar was used (1:1:6 cement/lime/sand) and there was a noticeable reduction in strength of 64% when compared with walls 2, 4 and 5 in a 1:3 cement/sand mortar.

However, the tests on walls 2, 3 and 6 represent tests on single walls only and further tests are required before firm conclusions may be drawn.

For example the strength of walls 7 and 8 differ by 30% and the higher strength of 37 tons was for wall 7 with the ties in compression where the lateral movement of the unloaded leaf was rather less than that of the loaded leaf. Failure normally occurred in the unloaded leaf and even though the lateral movements were reduced (probably by bending of the wall ties) it might be expected that wall 7 would have failed at a lower load than the single leaf wall 6.

This view is confirmed by comparing wall 2 with walls 4 and 5. In fact wall 6 failed at 31 tons, 16% less than wall 7.

4.32 Distribution of load

The assessment of load distribution onto the two leaves of a cavity wall when the floor slab or edge beam bears on the two leaves is complex. The stiffness of the floor slab will play an important part as also will the fixity at the slab/wall junction. For example, if a stiff slab (7 ins) bears fully onto a 10⅓ ins cavity wall from a relatively short span (12 ft) the load might well be assumed to be equally distributed between the two leaves when of similar construction. If the slab were reduced in thickness to 4 ins and
the span increased to say 13ft 6 ins it might be expected that the inner leaf would support the greater part of the load when the two leaves are of similar construction. Indeed for such a case designers often assume that 2/3rds of the load is supported by the inner leaf and 1/3rd by the outer leaf.

The relative stiffnesses of the two leaves will also influence the load on each.

For example, if the two equal leaves were supporting the 7 ins slab spanning 12 ft and the outer leaf was of a strong facing brick — say 10,000 lb/ins² brick in 1:1:6 cement/lime/sand mortar — and the inner leaf was of common brickwork — say 4,000 lb/ins² bricks in 1:1:6 mortar — the relative E values might well be $4 \times 10^6$ and $1 \times 10^6$ lb/ins² respectively. The stronger leaf might then support four times the load on the weaker leaf. Relaxation at the slab/wall junction might complicate the assessment as might also any differential movements between the two leaves discussed under section 4.5.

4.4 Lateral loading

The tests described under section 4.3 show that when a cavity wall is constructed with wall ties conforming to B.S.1243 the ties are sufficiently strong in both tension and compression to ensure that both leaves of the cavity wall take up the same arc of bending, and hence the stiffness of the cavity wall will be the sum of the stiffness of the two leaves. Any bending moments will be shared by the two leaves in proportion to their stiffnesses.

4.5 Dimensional stability

In cavity wall construction the outer leaf is subjected to the changing weather conditions whilst the inner leaf will normally be
maintained at room temperature.

The probable temperature range of the outer leaf may be 60°F ± 40°F and for the inner leaf 60°F ± 20°F. There is consequently differential thermal movement between the two leaves which could result in loosened wall ties and/or excessive movements at roof coping level.

To overcome this the code of practice C.P.111 1964 recommends that the outer leaf be broken by an expansion joint or restrained by the floor slab or edge beam at least every 30 ft vertically - this corresponds to every 3rd floor. Where an expansion joint is provided in the outer leaf then it cannot be load-bearing and each storey above the expansion joint level needs to be supported on the inner leaf. This is clearly inefficient and uneconomic and a more suitable alternative is to restrain the outer leaf by the floor slab or edge beam and assume it to be load-bearing.

The increase in stress for the outer leaf due to differential thermal expansion with the inner leaf is however, an unknown and usually ignored.

Moisture expansion of brickwork in this country is very rare and can be eliminated by storing the bricks on site for three weeks before building with them. However, when such precautions are omitted additional stresses might be built up in the outer leaf.

In the few buildings in Switzerland where cavity brickwork is used the outer leaf is non-load-bearing apart from its own weight for the full height of the building, which may be as high as 16 storeys. In this case the outer leaf is supported at floor levels only by flexible ties. Provision is then made at windows and parapet level for the outer leaf to move differentially.
Table 4.1

Summary of results of Sahlin's preliminary wall tests

<table>
<thead>
<tr>
<th>Wall</th>
<th>max. load (tons)</th>
<th>mortar strength (lb/ins²)</th>
<th>Eccentricity</th>
<th>average stress (lb/ins²)</th>
<th>max. edge stress (lb/ins²)</th>
<th>strain at max. load (lb/ins² x 10⁶)</th>
<th>E. at max. load (lb/ins² x 10⁶)</th>
<th>Load ratio - eccentric/axial</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>53.4</td>
<td>120</td>
<td>t/250</td>
<td>533</td>
<td>--</td>
<td>5.19</td>
<td>0.1025</td>
<td>--</td>
</tr>
<tr>
<td>2</td>
<td>18.9</td>
<td>120</td>
<td>t/5.00</td>
<td>188</td>
<td>418</td>
<td>1.31</td>
<td>0.1440</td>
<td>0.354/0.5=0.556</td>
</tr>
<tr>
<td>3</td>
<td>15.5</td>
<td>120</td>
<td>t/4.64</td>
<td>154</td>
<td>364</td>
<td>1.67</td>
<td>0.0925</td>
<td>0.29/0.9=0.362</td>
</tr>
<tr>
<td>4</td>
<td>16.4</td>
<td>225</td>
<td>t/4.9</td>
<td>164</td>
<td>370</td>
<td>0.35</td>
<td>0.466</td>
<td>0.307/0.9=0.334</td>
</tr>
<tr>
<td>5</td>
<td>53.4</td>
<td>120</td>
<td>Axial</td>
<td>533</td>
<td>--</td>
<td>6.30</td>
<td>0.084</td>
<td>--</td>
</tr>
<tr>
<td>6</td>
<td>53.4</td>
<td>120</td>
<td>&quot;</td>
<td>533</td>
<td>--</td>
<td>4.79</td>
<td>0.111</td>
<td>--</td>
</tr>
</tbody>
</table>

Brick strength 3780 lb/ins²

Slenderness ratio (excluding wall 3) = 2070/250 = 8.3.

Slenderness ratio (wall 3) = 2605/250 = 10.4

wall cross-section 9.85 ins by 22.8 ins (225 ins²)

* using curves for normal quality brickwork in Fig. 2.6.
Table 4.2

Summary of results of Sahlin's main wall tests

Brick strength 3780 lb/ins²  
wall cross-section 9.85 ins by 19.7 ins (194 ins²)

\[ P \] = compressive force acting on vertical member immediately below a floor slab.

\[ P_1 \] = compressive force acting on vertical member immediately above a floor slab.

\[ e_1 \] = eccentricity of the compressive force acting on a vertical member immediately below a floor slab.

\[ e_u \] = eccentricity of the compressive force acting on a vertical member immediately above a floor slab.

<table>
<thead>
<tr>
<th>Test no. and type</th>
<th>mortar strength (lb/ins)</th>
<th>( P_{u}/P_1 )</th>
<th>Eccentricity</th>
<th>stresses (lb/ins²)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>( e_u )</td>
<td>( e_1 )</td>
<td>Sahlin</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(ins)</td>
<td>(ins)</td>
<td>( e_1 )</td>
</tr>
<tr>
<td>1 B</td>
<td>208</td>
<td>1/1.23</td>
<td>1.58</td>
<td>t/624</td>
</tr>
<tr>
<td>1 B</td>
<td>208</td>
<td>1/1.23</td>
<td>0.92</td>
<td>t/10.7</td>
</tr>
<tr>
<td>2 B</td>
<td>114</td>
<td>1/1.23</td>
<td>0.79</td>
<td>t/12.5</td>
</tr>
<tr>
<td>2 B</td>
<td>114</td>
<td>1/1.23</td>
<td>1.49</td>
<td>t/6.61</td>
</tr>
<tr>
<td>3 B</td>
<td>189</td>
<td>1/1.16</td>
<td>1.89</td>
<td>t/5.2</td>
</tr>
<tr>
<td>3 B</td>
<td>189</td>
<td>1/1.16</td>
<td>0.84</td>
<td>t/11.7</td>
</tr>
<tr>
<td>4 B</td>
<td>136</td>
<td>1/1.16</td>
<td>0.55</td>
<td>t/17.9</td>
</tr>
<tr>
<td>4 B</td>
<td>136</td>
<td>1/1.16</td>
<td>-0.27</td>
<td>t/36.5</td>
</tr>
<tr>
<td>5 A</td>
<td>323</td>
<td>0/1</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>5 A</td>
<td>323</td>
<td>0/1</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>6 A</td>
<td>228</td>
<td>0/1</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>6 A</td>
<td>228</td>
<td>0/1</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>7 B</td>
<td>125</td>
<td>1/1.16</td>
<td>1.18</td>
<td>t/8.35</td>
</tr>
<tr>
<td>7 B</td>
<td>125</td>
<td>1/1.16</td>
<td>0.42</td>
<td>t/23.5</td>
</tr>
<tr>
<td>8 B</td>
<td>270</td>
<td>1/1.16</td>
<td>1.97</td>
<td>t/5</td>
</tr>
<tr>
<td>8 B</td>
<td>270</td>
<td>1/1.16</td>
<td>0.96</td>
<td>t/10.3</td>
</tr>
</tbody>
</table>

* Using Fig. 2.11.
Table 4.3

Summary of results of cavity wall tests (after Davey and Thomas)

<table>
<thead>
<tr>
<th>Wall no.</th>
<th>Mortar mix cement/lime/sand</th>
<th>Failing load</th>
<th>Type of loading</th>
<th>type of wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>tons</td>
<td>lb/ins²</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>1.0.3 Butterfly 9. S.W.G. steel wire</td>
<td>128</td>
<td>1250</td>
<td>concentric</td>
</tr>
<tr>
<td>2</td>
<td>1.0.3</td>
<td>91</td>
<td>890*</td>
<td>1&quot; eccentric</td>
</tr>
<tr>
<td>3</td>
<td>1.0.3</td>
<td>104</td>
<td>1015</td>
<td>concentric (one leaf)</td>
</tr>
<tr>
<td>4</td>
<td>1.0.3</td>
<td>86</td>
<td>840</td>
<td>1 ins eccentric on one leaf (ties in compression)</td>
</tr>
<tr>
<td>5</td>
<td>1.0.3</td>
<td>83</td>
<td>810</td>
<td>1 ins eccentric on one leaf (ties in tension)</td>
</tr>
<tr>
<td>6</td>
<td>1.1.6 Butterfly 12 S.W.G. hard drawn copper</td>
<td>31</td>
<td>303</td>
<td>1 ins eccentric</td>
</tr>
<tr>
<td>7</td>
<td>1.1.6</td>
<td>37</td>
<td>362</td>
<td>1 ins eccentric or one leaf (compression)</td>
</tr>
<tr>
<td>8</td>
<td>1.1.6</td>
<td>26</td>
<td>254</td>
<td>1 ins eccentric on one leaf (tension)</td>
</tr>
</tbody>
</table>

* mean stress
fig 4.1 SAHLIN'S PRELIMINARY TESTS — WALL SPECIMENS

dimensions in mm.
100 mm = 3.94 ins.
Fig 4.2 Sahlin's Preliminary Tests

- Load vs. Strain (\(10^3\))
- Stress vs. Strain (\(10^3\))

- Wall Rotation vs. Stress
- Wall Rotation vs. Load

1 \(\text{Mpa} = 0.984 \text{ tons}\)
1 \(\text{kp/cm}^2 = 14.22 \text{ lb/ins}^2\)
1 \(\text{Mpm/m} = 26,400 \text{ lb.ins/ft}\)

Fig 4.3
Sahlins Preliminary Tests

WALL SPLITTING ABOVE
TOP SURFACE OF
CONCRETE SLAB
**SPECIMEN TYPE A**

**SPECIMEN TYPE B**

All dimensions in mm.

100 mm. = 3.94 ins

SAHLINS' MAIN TESTS
FIG. 4.7  Sahlin's Main Tests - testing equipment
LATERAL DEFLECTION OF CAVITY WALLS
WITH STEEL WIRE TIES

fig 4.9

LATERAL DEFLECTION OF CAVITY WALLS
WITH COPPER WIRE TIES

fig 4.10
CHAPTER 5
EXPERIMENTAL INVESTIGATION - 600 TON FRAME

5.1 Introduction

The object of this work was to investigate the structural behaviour of storey height cavity walls 10½ ins thick subjected to vertical load, with particular reference to the distribution of load and bending moment to the two leaves of the wall at the floor slab/wall junction.

A loading frame of 600 tons capacity was designed to test wall panels 8 ft 2 ins high by 4 ft 6 ins wide which are within the limits of 8 ft to 9 ft high by 4 ft to 6 ft wide specified in C.P.111, 1964 for test panels. The wall panels were loaded between reinforced concrete slabs so as to simulate more closely the end conditions of the walls in an actual building. The design, fabrication and erection of the test frame was a considerable task and it has been possible to test only three preliminary walls, and these to ensure that the frame and loading equipment were operating satisfactorily.

5.2 Loading frame

The design and operation of the 600 ton loading frame is described in detail in Appendix 1. For the cavity wall tests the superstructure was erected on a single grillage unit and the walls loaded between 6 ins, thick reinforced concrete floor slabs as shown in Figs. 8.2 and 8.4.

5.3 Materials

5.31 Bricks

Pressed single frogged bricks 2½ ins deep were used for all three
wall tests and properties are given in Chapter 3, Table 3.1.

5.32 Sand

Building sand no. 2 was used for wall 1 and sand no. 3 for the remaining two wall tests. The sieve analysis is given in Chapter 3 Table 3.2.

5.33 Lime

For the 1:1:6 mortar mix a class A hydrated lime (Hydralime) in accordance with B.S.890 was used.

5.34 Cement

A rapid hardening Portland cement (ferrocrete) was used for the construction of all three walls.

5.35 Mortar

Each batch of mortar was mixed by hand. Mortar proportions were made up by volume and their weights recorded. Wet sand was used for all three walls and the nominal volume mixes gave therefore a richer mix than for a similar volume mix using dry sand because of sand bulking. For each mix the bricklayer was allowed to add sufficient water to give optimum workability.

From each mix, 4 ins cubes were made by compacting in two 2 ins layers with a Kango hammer. After 24 hours the cubes were removed from the mould and stored in water until tested. The results of mortar cube crushing tests are given in Table 5.1.

5.4 Experimental procedure

5.41 Tests on walls - general

Each of the walls was constructed within the loading frame shown in Figs. 8.2 and 8.4 by a professional bricklayer and both the upper and
lower 6 ins thick reinforced concrete slabs were cast in position prior to constructing the test wall. To facilitate this the upper slab was raised \( \frac{1}{2} \) ins at the test wall end. Each test wall was constructed in the normal manner, the centre line of the wall being marked on the sides of each slab, and a bed of mortar approximately \( \frac{5}{8} \) ins thick placed on top of the wall. The upper slab was then lowered \( \frac{1}{2} \) ins onto the fresh bed of mortar which reduced from \( \frac{5}{8} \) ins to \( \frac{3}{8} \) ins in thickness. After curing, the walls were tested by applying the load through a loading beam seated on a 9 ins thick single course of brickwork (brick on edge) on the upper slab.

The bricklayer was instructed to construct the walls plumb and true to line and level with all joints completely filled with mortar. A storey height course rod was supplied to the bricklayer and a nominal \( \frac{3}{8} \) ins mortar joint adopted for all the test walls.

Each wall comprised 33 course of 2\( \frac{5}{8} \) ins thick bricks in the 8 ft 2 ins clear height between floor slabs.

Each of the walls was axially loaded and the first wall was of 9 ins solid construction. The remaining two walls were of 10\( \frac{1}{2} \) ins cavity construction.

5.42 Tests on walls - description of tests

Wall 1

Wall 1 was 9 ins solid thickness and built by bricklayer 3 who was instructed to build the wall plumb and true to line and level with all joints completely filled. The brickwork was constructed in English bond that is 3 stretcher courses to every header course.

The wall was loaded several times up to 60 tons each loading cycle.
taking about \( \frac{1}{2} \) an hour, and all the pipe connections checked for leakage.

The following day the load was taken up to 93.5 tons over a period of \( \frac{1}{2} \) hour and released. During this loading fine vertical cracks were observed at 56 tons, the cracks passing through the 9 ins wall thickness and generally passing through the perpend joints. Loading was recommenced and at 108 tons slight spalling occurred on both wall faces in the top course of brickwork. Loading was continued and at a load of 150 tons failure occurred by vertical splitting through the wall face and also through the centre of the 9 ins wall thickness. (Fig. 5.1 and 5.2).

The stress/strain curve is given in Fig. 5.3.

Wall 2

Wall 2 was of 11 ins cavity construction and built to similar instructions as wall 1.

The brickwork bond was stretcher and the two leaves were tied with galvanised strip metal wall ties at 3 ft centres horizontally and 1 ft 6 ins centres vertically and staggered. (Fig. 5.4).

Loading was increased steadily to 153 tons over a period of \( \frac{3}{4} \) hour. The load was held steady at this load for about 1 minute after which faint sounds of cracking were heard.

Local crushing then took place on the cavity face of the inner leaf about 7 to 8 courses down from the top. Spalling also occurred on the outside face of the outer leaf in the top 4 - 5 courses. Loading was continued to magnify failure cracking and vertical cracking appeared in both leaves. Cracking was more severe on the inner leaf. (Figs. 5.5 and 5.6)
The stress/strain curve is given in Fig. 5.4.

**Wall 3**

Wall 3 was of 10\(\frac{1}{2}\) ins cavity construction and construction was similar to wall 2. Loading was increased steadily to 169.5 tons over a period of 45 mins.

Two vertical cracks appeared first at the top right of the inner leaf under jack no. 3. Loading was continued, although maximum load was already reached. The cracks increased in size and spalling took place in the top centre of 3 courses of the outer leaf. (Figs. 5.7 and 5.8). The stress/strain curve is given in Fig. 5.9.

**5.5 Results**

A summary of the 3 wall tests and a comparison with the permissible design stresses based on the Code of Practice C.P.111, 1964 is given in Table 5.2.

The average crushing strength of the brickwork cubes associated with the wall tests together with a comparison with the wall strengths is given in Table 5.3. The results of crushing strength tests on the 4 ins mortar cubes is given in Table 5.1.

The stress/strain curves are given in Figs. 5.3, 5.4 and 5.9.

**5.6 Calculations**

The permissible design stresses given in Table 5.2 have been calculated in accordance with the Code of Practice C.P.111 (1964), as follows:

For 10\(\frac{1}{2}\) ins cavity wall - slenderness ratios:

\[
\frac{3}{4} \times \frac{8.167 \times 12}{\frac{2}{3} \left(\frac{4\frac{1}{2}}{2} + 4\frac{1}{2}\right)} = 12.25
\]

Reduction factor for axial loading (from Table 4 of C.P.111) = 0.75.
The basic permissible compressive stress for a brick strength of 3710 lb/ins² and 1:3 cement/sand mortar mix is 285 lb/ins² (by interpolation of Table 3 in C.P.111).

The maximum permissible compressive design stress is the product of the basic permissible stress and the reduction factor = 285 x 0.75 = 213 lb/ins².

For the 9 ins solid wall:

slenderness ratio = $\frac{\frac{3}{4} \times 8.167 \times 12}{9} = 8.15$

reduction factor (assuming axial loading) = 0.92

basic permissible stress (1:1:6 mortar) = 220 lb/ins²

permissible compressive design stress = 220 x 0.92 = 202 lb/ins²

The relative stiffnesses of the floor slabs and wall, assuming the slab to be fixed at the loading end and pinned at the other, and the test wall to be fixed at both ends are:

slab = $\frac{2 \times 1 \times 6^3 \times \frac{3}{4}}{12} = 27$

wall = $\frac{1 \times 1 \times 4.125^3 \times 2}{8.2} = 17.1$

total stiffness at junction = 27 + 17.1 = 44.1

bending moment transferred to slab = $\frac{27}{44.1} = 0.612$

bending moment transferred to wall = $\frac{17.1}{44.1} = 0.388$

5.7 Discussion of results

5.7.1 Testing frame

The operation and performance of the test frame for the three preliminary tests was generally satisfactory although some difficulty was experienced in holding the load steady at loads exceeding 100 tons.
The visual load measuring arrangement using the digital voltmeter is accurate to the nearest 5 and this represents a load of 0.59 tons on each of the three rams making a total possible error of 1.77 tons over the possible loading range of 600 tons. This represents an error of less than 0.3% and is quite satisfactory. When steadying the load however, it was difficult to ascertain when the load was slowly falling or increasing because the load was noted visually on the digital voltmeter at intervals of 1.77 tons. This difficulty was overcome by watching one of the dial gauges forming the extensometers and using the needle movement as a loading indicator.

The dial gauges were out of line of vision of the loading operator and the help of a technician was required when steadying the load. It is hoped to arrange a mirror system for future tests so that the loading operator can observe at least one dial gauge without assistance.

The fine control valves were coarser than desired at high loads and one of these will be replaced for subsequent wall tests.

The three loading rams operated satisfactorily except for ram no. 3 which leaked a small amount of oil for loads up to 20 tons total. Above this load there was no leakage.

5.72 Mortar strength

The 4 ins mortar cube strengths are shown in Table 5.1 and it can be seen that the cube strengths for all three walls showed a considerable variation.

The usual control was carried out by the bricklayer and the reasons for the wide variations in strength are not understood. The cement used,
however, was at least 8 weeks' old and this may have contributed towards the low strengths and perhaps the variations in strength.

Wall 2 having a mortar strength half of that for wall 3 failed at a load 10% less.

5.73 Wall strength

The average compressive stress at failure of 725 lb/ins$^2$ for the 9 ins solid wall 1 given in Table 5.2 compares well with walls 4 and 5, which were tested by a brick manufacturer. The stress at failure of walls 4 and 5 was 925 lb/ins$^2$ and 890 lb/ins$^2$ respectively. These might be expected to be higher than for wall 1 because they were constructed of slightly stronger bricks and mortar. Walls 4 and 5 were also 9 ins thick but only 3ft 2 ins wide by approximately 4 ft 6 ins high.

The strength of 10 1/2 ins cavity walls 2 and 3 was 770 lb/ins$^2$ and 855 lb/ins$^2$ respectively, and these were noticeably weaker than the 4 1/2 ins walls 15 and 16 (1085 lb/ins$^2$ and 1040 lb/ins$^2$ respectively) detailed in Chapter 3 table 3.5. Walls 15 and 16 were, however, constructed of a stronger mortar (770 lb/ins$^2$ and 940 lb/ins$^2$ respectively) than walls 2 and 3 (335 lb/ins$^2$ and 670 lb/ins$^2$ respectively) and this most likely accounts for much of the difference in strength. The points discussed in section 5.74 however, also influence the wall strengths.

5.74 Distribution of load and bending moment

It can be seen from Fig. 5.3 which shows the stress/strain curve for wall 1 that the strains on opposite wall faces are very nearly equal and that therefore the wall was axially loaded.
For walls 2 and 3 however, (Figs. 5.4 and 5.9) there was a considerable difference between the strains on opposite wall faces indicating that the loading was eccentric.

This could be due to several factors, including:

a) Wall built off plumb
b) upper slab not bedded evenly on top of the two leaves.
c) one leaf constructed slightly higher than the other.

In fact, none of these factors was observed although either factor b) or c) or a combination of the two may have been present. It would appear significant that the decrease in apparent eccentricity with increase in load shown graphically in Fig. 5.10 was similar for both walls 2 and 3.

The eccentricities shown in Fig. 5.10 have been deduced from the ratio of strains on opposite wall faces assuming a linear stress strain relationship and assuming that any bending is shared equally by both leaves. The calculations are summarised graphically in Fig. 2.11.

This eccentricity is unlikely to have been caused by the upper 6 ins thick R.C. floor slab spanning from one side since it had no influence on wall 1. The maximum compressive strains were in any case in the outer face of the outer leaf and it would be expected that any moment transferred from the floor slab to the wall would in fact result in reduced stresses in the outer face of the outer leaf and increased stress on the outside face of the inner leaf. The observed strains were the reverse of this.

The eccentricity may have been due to an uneven bedding at the upper slab seating or due to one leaf being slightly higher than the other so that each leaf supported vertical loading only, but not
equally, due to the leaves being of unequal height. If this is so the proportion of the total load supported by each leaf would be as shown in Fig. 5.10.

5.75 Instrumentation

A more accurate picture of the load and bending moment distribution in cavity walls requires strain measurements on the cavity face of each leaf within the 2 ins cavity. This would be difficult using normal dial gauges and long bracket mountings would be required so that the dial gauges may be fitted outside the cavity.

Small inductance gauges can be obtained that could easily be set up within the 2 ins cavity and these would be well suited to strain measurements within the cavity.

The rotation of the floor slab at the slab/wall junction was not measured for the axially loaded walls 1, 2 and 3. The rotation should, however, be measured on subsequent wall tests using an inclinometer or a theodolite.

5.76 Secant modulus

The secant modulus was found to decrease with increase in load although the reduction was not so great as for the 4 1/2 ins thick wall tests 14, 15 and 16 described in Chapter 3. For the 10 1/2 ins cavity walls 2 and 3 the secant modulus was approximately \( 1 \times 10^6 \text{ lb/ins}^2 \) at a stress of 100 lb/ins\(^2\) and approximately 0.53 lb/ins\(^2\) at a stress of 500 lb/ins\(^2\). These values are similar to manufacturers tests on walls of similar materials.

For the 9 ins solid wall 1, the rate of strain surprisingly decreased with increase in load and the secant modulus was approximately
$1.1 \times 10^6 \text{lb/ins}^2$ at a stress of $100 \text{ lb/ins}^2$ and approximately $0.45 \text{ lb/ins}^2$
at a stress of $500 \text{ lb/ins}^2$.

This is contrary to all earlier tests and is not fully understood. It might be due to the "bedding" or "closing up" of the bricks if the wall had been poorly constructed, but this would be expected to have taken place during the initial loading.

5.77 Load factors

For the 9 ins solid axially loaded wall 1 the load factor based on the Code of Practice C.P.111 (1964) was 3.6.

For the 10½ ins cavity axially loaded walls 2 and 3 the load factors were 3.62 and 4.02 respectively.

The strength of the mortars for each of the walls was weaker than expected (although perhaps typical of a construction site!) and the load factors would increase a little with an increase in mortar strength.

5.78 Mode of failure

In all three wall tests failure was initiated by vertical hair cracks without the sudden noisy failure observed in the 4½ ins thick wall tests of 6,000 - 6,600 lb/ins$^2$ bricks. As loading increased spalling and local crushing took place and the vertical cracks increased in size.

5.79 Brickwork cubes.

The results summarised in Table 5.3 show that the strength ratio of the cavity walls/brickwork cubes ranged between 50% and 60%.
5.8 Conclusions

The results of 3 preliminary wall tests indicate that:

1. The 600 ton testing frame is operating satisfactorily.

2. The mode of failure was by vertical splitting followed by spalling and local crushing as in the 4\(\frac{1}{2}\) ins thick wall tests.

3. The secant modulus for cavity walls 2 and 3 was approximately \(1 \times 10^6\) lb/ins\(^2\) at a stress of 100 lb/ins\(^2\) and approximately 0.53 lb/ins\(^2\) at a stress of 500 lb/ins\(^2\).

4. The load factors of the 9 ins solid and 10\(\frac{1}{2}\) ins cavity wall were less than half those for single leaf walls tested in the 200 ton frame.
Table 5.1

Crushing strength of 4 ins mortar cubes (lb/ins$^2$)

All mortar cubes vibrated in two 2 ins layers with a Kango Hammer and then tank cured.

<table>
<thead>
<tr>
<th>Wall</th>
<th>1</th>
<th>2</th>
<th>3</th>
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</thead>
<tbody>
<tr>
<td>Nom. vol. mix.</td>
<td>(1.1.6)</td>
<td>(1.3)</td>
<td>(1.3.3)</td>
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<tr>
<td>Crush. strength</td>
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<tr>
<td>(lb/ins$^2$)</td>
<td></td>
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<tr>
<td>320</td>
<td>140</td>
<td>670</td>
<td></td>
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<tr>
<td>715</td>
<td>365</td>
<td>520</td>
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<tr>
<td>490</td>
<td>105</td>
<td>565</td>
<td></td>
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<td>335</td>
<td>280</td>
<td>485</td>
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<td>825</td>
<td>610</td>
<td>760</td>
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<td>170</td>
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<td>750</td>
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<td>390</td>
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<tr>
<td>Mean</td>
<td>495</td>
<td>335</td>
<td>670</td>
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<tr>
<td>wall no.</td>
<td>brick strength</td>
<td>mortar strength</td>
<td>ultimate load</td>
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<tr>
<td>1 (9&quot; solid)</td>
<td>3710</td>
<td>495 (1.1.6)</td>
<td>150</td>
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<tr>
<td>2 (10½ ins cavity)</td>
<td>3710</td>
<td>335 (1.3)</td>
<td>153</td>
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<tr>
<td>3 (10½ ins cavity)</td>
<td>3710</td>
<td>670</td>
<td>169.5</td>
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<tr>
<td>4 (9 ins. solid) manufactures test</td>
<td>4260</td>
<td>1140 (1.1.6)</td>
<td>--</td>
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<tr>
<td>5 (9 ins solid) manufactures test</td>
<td>4780</td>
<td>1095 (1.1.6)</td>
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<tr>
<td>wall no.</td>
<td>brick strength (lb/ins²)</td>
<td>mortar strength (lb/ins²)</td>
<td>wall strength (lb/ins²)</td>
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</table>
FIG. 5.1 - Wall 1 - 9 ins solid vertical splitting of wall

FIG. 5.2 - Wall 1 - 9 ins solid cracking and spalling in outside 2.
Fig 5.3  shortening \times 10^{-2} = \frac{\text{gauge length}}{72'}
Fig 5.4 shortening $x 10^{-2}$ gauge length $\frac{7}{2}''$
FIG. 5.5 - Wall 2 10\(\frac{1}{2}\) ins cavity - inside face elevation.

FIG. 5.6 - Wall 2 10\(\frac{1}{2}\) ins cavity - outside face, top half of wall
FIG. 5.7 - Wall 3 10\(\frac{1}{2}\) ins cavity, cracking and spalling in top outside face.

FIG. 5.8 - Wall 3 10\(\frac{3}{4}\) ins cavity, shearing within 4\(\frac{3}{8}\) ins brick thickness
fig 5.9  shortening $\times 10^{-2}$  gauge length $\frac{3}{4}''$

wall 3
Fig 5.10. Eccentricity assuming linear stress/strain and using Fig 3.11.
6.1 Introduction

Information on the strength of bricks and brickwork has accumulated during the last 35 years but because of the inherent variables associated with this construction for example, variations in workmanship, brick to mortar bond and in brick and mortar strength, and fixity of wall/slab junctions, few attempts have been made to rationalize design techniques on a theoretical basis.

Predictions of the influence on the strength of brickwork of slenderness and axial, eccentric and concentrated loading have generally been based on a statistical assessment of test data and empirical formulae for wall strengths have been developed as discussed in section 2.7. The more recent theoretical approaches involve simplifying assumptions and are discussed in this chapter.

6.2 Failure by vertical splitting

The mechanism of failure of brickwork subjected to axial compression is normally by vertical splitting due to horizontal tension in the brickwork. This is caused by the transverse strain of the mortar under load which is usually greater than that of the brick. The tensile strength and Poisson's ratio of the brick and mortar are therefore of importance.

Haller\textsuperscript{10} has put forward a simple formula based on elastic theory for the transverse tensile stress in the brickwork in terms of Young's modulus and Poisson ratio of the two materials.

Sinha\textsuperscript{31} has extended this to include for shear stresses in the brickwork and considers the three basic types of brickwork failure by
a) vertical splitting due to horizontal tension when the ratio of tensile/compressive strength of brick is less than about 0.5.

or b) by shear in the brick for tensile/compressive strength ratios of brick above 0.5.

or c) theoretically by crushing of the brick when the tensile and shear strengths of the brick are very high.

The results of his experimental work on 1/6th scale wallettes and piers 2, 3 and 4 courses high were in good agreement with the theoretical prediction of the splitting load when Poisson's ratio for mortar and brickwork were taken as 0.15 and 0.1 respectively.

6.3 Slenderness ratio-axial loading

Normal brickwork construction has little resistance to tension and therefore the lateral stability of walls requires investigation. Several empiricial column formulae are in use relating slenderness ratio with critical load and include the straight line, the parabolic, the Gordon-Rankine and the Perry Robertson relationships.

Southwell used a method of determining the critical load on pin ended struts by plotting experimental values for load and load/deflection ratios. Angervo adopted the Engesser curve for axial loading and Monk the Euler-Engesser theory. The latter appears to provide the "best fit" for the limited American, British and Swiss data available and each series of tests revealed a double curvature relationship between critical load and slenderness.

Adopting a similar notation to Monk the Euler formulae is as follows,

\[ \frac{P}{a} = \frac{mN^2E}{(h/r)^2} \]  \hspace{1cm} (6.1)

or \[ \frac{h}{t} = \frac{A}{\sqrt{\frac{P}{m}}} \] \hspace{1cm} (6.2)
where $A = \sqrt{\frac{m E_i}{12}}$

- $a$ = cross-sectional area, ins\(^2\)
- $b, c, k, n$ = experimental constants
- $\epsilon$ = unit strain value, ins/ins.
- $E_i$ = initial modulus of elasticity
- $E_r$ = reduced modulus of elasticity
- $E_t$ = tangent modulus of elasticity
- $f_m^* = \frac{P}{a} = \text{ultimate wall strength, lb/ins}^2$
- $f_{mn}^* = \text{unit stress value, lb/ins}^2$
- $h = \text{wall height, ins.}$
- $m = \text{boundary condition factor}$
- $M = 4$, fixed ends
- $2$, hinged-fixed
- $1$, hinged
- $\frac{1}{4}$, cantilever
- $P = \text{axial load, lb.}$
- $r = t/\sqrt{12}$ = radius of gyration
- $t = \text{wall thickness, ins.}$

It is assumed for the Euler formulae that the material behaves elastically throughout its entire strain history although this is a false assumption as can be seen from the non-linearity of the stress/strain curves given in Chapters 3 and 5. The non-linear shape of the stress/strain curve can be approximated by the addition of a linear relation corresponding to the initial Modulus $E_i$ and an "n" degree parabola whose constants will depend on the actual shape of the stress/strain curve (see Fig. 6.1). This relation may be expressed as follows:

$$\epsilon = b f_m^* + c (f_m^*)^n$$  \hspace{1cm} (6.3)
Differentiating this function gives the following form to the tangent modulus.

\[ E_t = \frac{E_i}{1 + c (f_m^+)^{n-1}} \] (6.4)

By introducing the tangent modulus function (6.4) into the Euler formulae (6.2) the Engesser modification\(^{36}\) is obtained as follows.

\[ h/t = \frac{A}{\sqrt{f_m^* + c(f_m^*)^n}} \] (6.5)

In this case the key modification lies in the power term.

The Euler Engesser formula may now be applied to sets of wall data to determine the curve of "best fit". This may be carried out either by passing the curve through three points, and from three simultaneous equations determine the constants A, C and b, or alternatively, compute A which is a function of the known initial tangent modulus \(E_i\) and pass the curve through two points to determine C and b.

During column bending the compressive stress on one side increases, while it decreases on the other. This results in two separate stress distributions related to the moduli associated with and without stress reversal. It may be assumed that the modulus during stress reversal is parallel to the initial modulus \(E_i\), and von Karman\(^{37}\) further modified the Euler-Engesser theory to take this into account, by introducing the reduced tangent modulus \((E_r)\) in place of \(E_i\) or \(E_t\) in the classical Euler equation.

\[ E_r = \frac{4 E_i E_t}{(\sqrt{E_i} + \sqrt{E_t})^2} \] (6.6)

This refinement did not improve significantly the "goodness of fit" in Monk's tests\(^{34}\) and the complication of using equation 6.6 does not
appear justified.

Shaney's "Column Paradox" concludes that the actual critical stress lies somewhere between the initial tangent and the reduced tangent method. A comparison of the two methods applied to Monk's tests are shown graphically in Fig. 6.2 where a second degree polynomial was adopted to solve simultaneous equations.

6.4 Slenderness ratio - eccentric loading

6.41 Stress distribution

Haller 1949 has suggested that for eccentric loads the stress distribution over the cross-section should be assumed to be similar in shape to the stress/strain diagram for the wall under axial loading and from this he evolved a graphical, analytical method for determining the buckling (critical) load of brickwork walls.

Angervo 1954 has devised a similar design method but expresses the observed stress/strain diagrams analytically.

Sahlin 1959 and Monk 1965 have assumed a linear stress/strain relationship in their theoretical studies.

6.42 Wall theory

Eccentric loads may be due to several causes including non-concentric vertical loads, bending due to unsymmetrical floor slab spans which may be expressed as an equivalent eccentricity "e" and also to lateral wind loading.

Two limiting cases exist. The first in the upper most storeys where compressive stresses are low and hence where the equivalent eccentricity "e" due to unsymmetrical floor slab spans and loading is
greatest. In this case failure may be in flexure when "e" is high. The second is in the bottom storey where the compressive stresses are high and hence where "e" is lowest. In this case failure may be in compression at one face of the wall.

In the general case of the floor slab/wall junction it is necessary to compute the angle of rotation of the walls as a function of a load which is applied so that its eccentricities are opposite in direction at the top and bottom ends of the wall. This calculation is complicated because the wall has little or no tensile strength and two different methods of calculating the angles of rotation have been put forward and reviewed by Sahlin 21 1959.

The first by Kazinczy 40 (1933-34) and Nylander 41 (1944) calculates the angle of rotation of the walls neglecting the additional deflections which result in changes in the effective cross-sectional area. In the second, developed by Angervo and Putkonen 42 (1957), the additional deflections are taken into account in calculating the angles of rotation at the top and bottom of the wall and equations given for the three different cases of loading as follows, when the line of action of the compressive force lies:

(i) outside the core boundary at the top end of the wall

(ii) within, or coincides with the core boundary, but is situated outside the core boundary in some section of the wall or its imaginary extension.

(iii) within the core boundary.

Sahlin compared the two methods and observed close agreement between
them for calculations of the angle of rotation when the load is relatively much lower than the buckling load for the case of loading in question.

If the additional deflections are considered in calculating the angle of rotation but assuming the point of inflection is fixed at the distance

\[
\frac{m_1}{m_1 + m} \cdot h
\]

from the end having a moment of \(m_1\) then the agreement is closer. where \(m_1\) and \(m = 6e/t\) = eccentricity ratios at top and bottom of wall and \(h = \) wall height.

Sahlin 21 studied the validity of the above assumption that the point of inflection is fixed for several cases of loading and concluded that this was sufficiently accurate bearing in mind the considerably simplified procedure.

Monk 35 1965 adopts a similar but simplified method assuming equal bending moments at the top and bottom of the wall based on the secant formulae and the Euler-Engesser Column Theory. He assumes for mathematical convenience that when a cracked section exists the reduced width \(t'\) occurring at the point of maximum moment, extends for the full wall height. This assumption is on the safe side since the reduced width does not extend for the full wall height and in fact the wall column is a tapered haunched beam having a greater strength than that assumed (Fig. 6.3)

6.5 Design methods

The British C.P.111 (1964) method adopts a double curvature line
for reduction in strength with increase in slenderness ratio and this is in agreement with findings elsewhere. 34,42

The C.P.111 (1964) reduction factors for eccentric loading assume equal eccentricities at the top and bottom of the wall and the permissible load using these stress reduction factors may be calculated on the following basis 4.

\[
\frac{P_e}{P_a} = \frac{5}{4} \left( \frac{1}{1 + \frac{6e}{t}} \right) F_s \quad (6.7)
\]

(for \( \frac{t}{24} \) \( m = \frac{1}{4} \) \( \leq e \leq \frac{t}{6} \) \( m = 1 \))

\[
\frac{P_e}{P_a} = \frac{15}{16} \left( 1 - \frac{2e}{t} \right) F_s \quad (6.8)
\]

(for \( \frac{t}{6} \) \( m = 1 \) \( \leq e \leq \frac{t}{2} \) \( m = e \) )

where \( P_e \) = Permissible design load for eccentrically loaded wall - equal eccentricities top and bottom of wall.

\( P_a \) = Permissible design load for axially loaded wall.

\( e \) = eccentricity of loading

\( t \) = wall thickness

\( F_s \) = reduction factor for slenderness - eccentric load reduction factor for slenderness - axial load (from C.P.111 Table 4)

\( m \) = \( \frac{6e}{t} \).

These results are summarised graphically and compared with the Swiss Norm 113 requirements in Fig. 2.6. It can be seen that there is
a closer agreement between the British and Swiss codes when the slenderness ratio is calculated on the basis of the reduced wall thickness for \( e < t/6 \). However, no guidance is given as to the method of calculation of the eccentricity.

In Switzerland the building is treated as a frame and support moments due to floor and wind loadings are calculated and distributed to the wall and slab members in proportion to their stiffnesses. Also due account is taken of cases where the moments are unequal or on opposite sides at the top and bottom of the wall.

When eccentricities of loading at the top and bottom of the wall are unequal or are on opposite sides the following procedure is used in Switzerland:

\[
\frac{P_e}{P_a} = \frac{1}{2} \left( \frac{e \cdot P_c}{a \cdot P_c} \right) \left( 1 + \frac{m_1}{m} \right) + (1 - 0.1m) \left( 1 - \frac{m_1}{m} \right) \quad (6.9)
\]

where \( \frac{P_e}{P_a} \) = Permissible design load for eccentrically loaded wall - equal or unequal eccentricities at top and bottom of wall.

\( P_a \) = Permissible design load for axially loaded wall.

\( e \cdot P_c \) = Permissible mean compressive stress for eccentrically loaded wall.

\( a \cdot P_c \) = Permissible mean compressive stress for axially loaded wall.

\( m_1 \) and \( m \) = 6e/t = eccentricity ratios at top and bottom of wall.
negative(-) when eccentricities at top and bottom of wall on opposite sides.

Now since mean compressive stresses are used in equation (6.9) above the mean stress ratio

\[
\frac{e^P_C}{a^P_C} = \frac{P_e}{P_a}
\]  

(6.10)

and equation (6.9) may be rewritten

\[
\frac{P_e}{P_a} = \frac{1}{2} \left( \frac{P_e}{P_a} \left(1 + \frac{m_1}{m}\right) + \left(1 - 0.1m\right) \left(1 - \frac{m_1}{m}\right) \right)
\]  

(6.11)

The use of equations (6.9) and (6.11) are time consuming when using Table 4 of C.P.111 (1964) because it gives reduction factors based on stress only. However the ratio \( \frac{P_e}{P_a} \) may easily be obtained by reference to Fig. 2.6 which shows graphically the effects of eccentric loading and slenderness ratio on wall strength.

Angervo\(^{33}\) (1954) has prepared a series of five design charts giving the variation in the ratio of critical stress/axial failing stress with change in slenderness ratio for a range of eccentricities. Each chart has been calculated and plotted for a particular slope of stress/strain curve and for walls subjected to eccentric loading this has been adopted for the shape of the stress block.

Sahlin has extended Angervo's work to take account of the floor slab stiffness and the interaction at the slab/wall junction assuming a linear stress distribution and has summarised much of the calculation work graphically.
6.6 Conclusions

1. The vertical splitting of brickwork under axial loading can be predicted theoretically.

2. The Euler-Engesser curve appears to represent the reduction in brickwork strength with increase in slenderness.

3. For theoretical design the point of inflection may be assumed fixed at a distance \( \frac{h}{m_1 + m} \) from the m_1 end where \( m_1 \leq m \).

4. For theoretical design the stress distribution may be assumed linear.

5. For practical design the building may be considered as a frame and support moments distributed at slab/floor junctions in proportion to their stiffnesses assuming full fixity at the junction.

6. For practical design the Swiss Norm 113 method would appear to be the most rational used in conjunction with Fig. 2.6.

\[
\frac{\bar{P}}{P_a} = \frac{1}{2} \left( \frac{P_e}{P_a} \left( 1 + \frac{m_1}{m} \right) + (1 - 0.1m) \left( 1 - \frac{m_1}{m} \right) \right) \quad (6.11)
\]

7. Sahlin's theory could be expanded to practical design curves and might be preferable to the Swiss Norm 113.
TANGENT MODULUS AND SECANT MODULUS THEORY (AFTER MONK)
Fig. 6.2 Comparison of tangent and reduced tangent modulus curves (after Morey).

\[
\frac{h}{t} = \sqrt{f_m^s + \left(\frac{f_m^s}{1950}\right)^{12.1}} + \sqrt{f_m^s}
\]

Initial tangent modulus curve

\[
\frac{h}{t} = \frac{3000}{\sqrt{f_m^s + \left(\frac{f_m^s}{1950}\right)^{12.1}}}
\]

or approximately

Reduced tangent modulus curve

\[
\frac{h}{t} = \frac{6000}{\sqrt{f_m^s + \left(\frac{f_m^s}{1950}\right)^{13.2}} + \sqrt{f_m^s}}
\]

Mean and range of test data.
PROGRESSIVE DEVELOPMENT
OF CRACKED SECTIONS
IN WALL COLUMN LOADING (AFTER MONK)

fig 6.3
CHAPTER 7

CONCLUSIONS

7.1 General

7.11

The typical mode of failure by transverse splitting indicates that the tensile strength of the brick and also the properties of the mortar joints such as Young's modulus and Poisson's ratio may be of primary importance in determining the strength of brickwork. Poisson's ratio of the mortar may increase with increase in stress, at a rate which might well be influenced by the degree of sand compaction in the mortar. This is controlled by the sand grading and the present gradings allowed in B.S.1200 include many sands which give far from optimum compaction.

7.12

Information is required on the direct tensile strength of bricks, across the $4\frac{1}{8}$ ins x $2\frac{5}{8}$ ins section. The simple transverse test is not considered suitable for general use because many bricks contain one or two frogs and many others are perforated; the transverse test might not therefore indicate the true direct tensile strength.

7.13

There appeared to be no clear pattern of horizontal and vertical strains over the wall face and it is likely that even with good quality workmanship slight variations in the mortar and slab bedding occur giving rise to unpredictable strain patterns.

7.14

A very general relationship for the strength of brickwork indicates that it is proportional to the cube or fourth root of the
mortar strength for a given brick strength and the square root of brick strength for a given mortar strength.

7.15

The secant modulus decreased with increase in load and for the single leaf walls constructed of bricks having a crushing strength greater than 6000 lb/ins\(^2\) in 1:3 cement/sand mortar values ranged between 2.7 and 3.5 \(\times 10^6\) at a stress of 155 lb/ins\(^2\) and 1.5 and 2.3 \(\times 10^6\) lb/ins\(^2\) at a stress of 1240 lb/ins\(^2\). For the bricks having a crushing strength of 3710 lb/ins\(^2\) the secant modulus ranged between 0.9 and 1.6 \(\times 10^6\) lb/ins\(^2\) at a stress of 155 lb/ins\(^2\) and 0.5 and 0.9 \(\times 10^6\) lb/ins\(^2\) at a stress of 620 lb/ins\(^2\).

The ratio of Young's moduli at 0.7 and 0.1 of the ultimate load ranged between 0.44 and 0.82 for the 6,000 lb/ins\(^2\) bricks and between 0.5 and 0.73 for the 3,710 lb/ins\(^2\) bricks.

7.16

The strength of brickwork appeared to decrease in direct proportion to increase in the mortar joint thickness. The results of tests on brickwork cubes and piers indicate a decrease in brickwork strength of 23% when the joint thickness was increased from \(\frac{3}{8}\) ins to 1 ins. An increase of 15% was observed when the joint thickness was reduced from \(\frac{3}{8}\) ins to \(\frac{1}{4}\) ins. American tests on six brick high wallettes indicate reductions in strength approximately double those observed in the cube and pier tests.

7.17

The load factors for the single leaf wall tests based on the Code of Practice C.P.111:1964 ranged between 6.3 and 12.3. A mean
value of 4 to 6 would be adequate for design. The load factors for the cavity and 9 ins solid walls ranged between 3.6 and 4.02. These are considerably lower than the single leaf walls constructed of similar materials and confirm Haller’s findings concerning the greater strength of single leaf walls.

7.18

The rate of loading of the test wall appears to influence the ultimate wall strength and walls loaded over a period of $1\frac{1}{2}$ hours failed at loads approximately 20% less than walls loaded over $\frac{1}{2}$ hour. This is probably due to creep in the mortar.

7.19

For walls loaded between reinforced concrete floor slabs the reduction in strength of eccentrically loaded walls compared with those axially loaded was considerably less than when loaded between knife edges.

It would appear that the bending moment due to the eccentric load is distributed to the wall and floor slabs in proportion to their stiffnesses.

7.2 Wall design with reference to C.P.111:1964

7.21

The Code of Practice C.P.111, 1964 reduction factors for eccentrically loaded walls are less than the Swiss Norm 113 and would appear to require modification to bring them in line with the Swiss requirements summarised in Fig. 2.6 which are based on the results of over 1,700 wall tests. The C.P.111, 1964 reduction factors for eccentricity are based chiefly on pier tests and these do not appear to represent accurately
the results of tests on walls as can be seen from the comparison for Sahlin's tests in Table 4.1.

7.22

Load reduction factors for eccentric loading based on the mean brickwork stress are preferable to the present reduction factors for the maximum permissible edge stress for convenience in calculation.

7.23

The method of determining the loading eccentricity is not given in C.P.111, 1964 and no allowance is made for the case where the loading eccentricity differs at the top and bottom of the wall.

A reasonable design basis would be to consider the wall/floor slab junction as fully fixed in the lower storeys, when the vertical compressive stress is greater than say 100 lb/ins$^2$, and to calculate any bending moments due to out of balance floor spans and loadings or walls off centre or off plumb and distribute to the slabs and walls in proportion to their stiffnesses.

7.24

The Swiss Norm 113 method of design for eccentrically loaded walls described in section 6.5 allows for different eccentricities at the top and bottom of the wall and would appear to be a useful design method when used in conjunction with Fig. 2.6.

7.25

For walls constructed of weak materials (say brick strength 4,000 lb/ins$^2$ or less and mortar strength of 1,000 lb/ins$^2$ or less) the effects of slenderness and eccentric loading are more severe than for wall constructed of stronger materials.
This effect should be accommodated in C.P.111 by specifying a reduced load factor for slenderness and eccentricity for the stronger materials.

7.26

Single leaf walls are noticeably stronger than bonded walls other factors such as brick and mortar strength, slenderness and workmanship being equal and this should also be accommodated in C.P.111 by allowing say a 10% increase in permissible design stresses for single leaf walls.

7.27

The extensive tests carried out in Switzerland indicate that the C.P.111 limit of 18 for slenderness ratio is unrealistic and that a limit of at least 24 would be more practicable.

7.28

The double curvature Euler-Engesser relationship for reduction in strength with increase in slenderness ratio appears to hold good for brickwork walls, 34, 42.

7.29

The C.P.111 makes no allowance in calculating the slenderness ratio for walls stiffened on all four sides and Haller's recommendations described in section 2.63 appear to be quite reasonable and worthy of adoption.
8.1 Introduction

The structural testing frame described in this thesis has been designed to accommodate the testing of many different types of structures and structural elements and hence flexibility has been a primary consideration in the design.

The frame superstructure is based on four steel grillage units each 9 ft square which are interchangeable on all four sides. The steel grillage was adopted in preference to the more commonly used reinforced concrete "strong floor" with anchor bolts at approximately 3 ft c/cs because of the former's greater flexibility for a given cost outlay.

For example, the four steel grillages may be used separately or coupled together to form an 18 ft square test bed. Alternatively, they may be connected in line to give a test bed area of 9 ft by 36 ft. The units may also be coupled together to form a Tee or Ell suitable for testing wall junctions.

The superstructure to be erected on the grillage base units has been selected as a set of standard steel columns and beams capable of assembly in various forms to accommodate a wide variety of structural elements for testing.

The assembly described in detail in this thesis is for testing 9 ins and $10\frac{1}{2}$ ins thick storey height brickwork walls 4 ft 6 ins long restrained by two 6 ins thick R.C. slabs. An alternative assembly for testing floor slabs, shell roofs and similar structures up to 14 ft sq. and capable
of applying vertical loads of up to 1,700 lb/ft$^2$ is also described. Almost any kind of superstructure, however, could be erected on the grillage units to suit a particular testing problem.

8.2 Description of Frame

8.21 Grillage units

The frame is based on four identical steel grillage units each 9 ft by 9 ft on plan, made up of 27$^{1/16}$ ins by 10 ins by 102 lb/ft. High Yield Stress Universal beams welded together at 3 ft cross centres. The units are interchangeable (Fig. 8.1) and each connection between the grillage units is capable of transmitting simultaneously a bending moment of 139.5 ft tons through the flange connections and a vertical shear of 35.7 tons through the web connections.

Each flange splice comprises 12 no. 1 ins dia. H.R.H. bolts (6 bolts either side of break) and two $\frac{1}{2}$ ins thick H.Y.S. plates.

Each of the nine welded joints formed by the intersecting universal beams of the grillage unit is stiffened by $\frac{3}{8}$ ins mild steel plates so as to safely support a 100 ton vertical load.

Four screw jacks are provided under each grillage unit for levelling.

8.22 Superstructure- arrangement for wall test

The initial design assembly of the superstructure comprises three portal frames, each column of which is 10$^{1/8}$ ins by 10$^{3/16}$ ins by 72 lb/ft. High Yield Stress Universal column, capable of transmitting 100 tons axial tension to the grillage units via. 2 ins thick base plates and 4 no high tensile type 'X' anchor bolts 1$\frac{1}{8}$ ins dia. The three portals when spaced 3 ft apart centre to centre may be erected together on one 9 ft square grid. The clear working height from the top of the grillage units to the underside of the cross head beam is 15 ft 9 ins and the
clear working width is 5 ft 1\frac{1}{2} ins between column flanges. The cross head beams are 27\1/16 ins by 10 ins by 102 lb/ft. High Yield Stress Universal Beams and the shear connection to the flanges of the columns, consisting of 20 no. 1 ins dia bolts, is designed for the maximum working shear load of 100 tons.

The frame is stiffened laterally by vertical diagonal bracing, consisting of double angles 3\frac{1}{2} ins by 2\frac{1}{2} ins by \frac{5}{16} ins within the two outer portal frames and horizontal diagonal bracing above the cross head beam consisting of single 3 ins by 3 ins x \frac{3}{8} ins angles.

The completed test wall arrangement is shown in Fig. 8.3 and dimensions given in Fig. 8.4.

The 600 ton loading capacity enables 9 ins solid and 10\frac{1}{2} ins cavity walls to be tested to destruction.

The clear storey height is 8 ft 2 ins as for the smaller 200 ton wall testing frame and lateral restraint to the test wall is provided by two 6 ins thick reinforced concrete slabs spanning 12 ft between centres of the test wall and supporting wall. This arrangement represents the restraint to walls found in a normal load-bearing brickwork building.

8.23 Superstructure arrangement for slab and shell test

For tests on slabs, shells and similar structures the grillage units can be bolted together in square array. Four of the six columns used for the portal frames described under section 8.22 can be erected to form two larger portals of 15 ft span placed 15 ft apart centre to centre. The cross head beams are again 27\1/16 ins by 10 ins by 102 lb/ft and can be erected at various heights above the grillage units ranging between approximately 2 ft and 16 ft, in increments of 1 ft 10\frac{1}{2} ins.

Two similar members span between the cross head beams and the
final arrangement of cross girders is capable of sustaining a vertical load (up or down) equivalent to 1,700 lb/ft\(^2\) on a structure up to 14 ft square (Fig. 8.2). Diagonal angle bracing and horizontal ties are provided between columns to provide lateral stability of the superstructure.

8.24 Loading Beam - wall tests only

The loading beam is a welded box girder 8 ft long built up from a 24 ins by 12 ins by 100 lb/ft universal beam.

The maximum shear force of 200 tons is accommodated by two additional \(\frac{1}{2}\) ins thick web plates and four \(\frac{1}{2}\) ins thick web stiffeners at each of the three loading points.

Three sets of \(\frac{3}{8}\) ins thick guide plates are provided with ball races to travel smoothly along the lined inside flanges of the portal columns.

Two chain wrenches are attached between the top of the loading beam and the underside of the cross head beam to facilitate handling of the loading beam. (see Fig. 8.8)

8.3 Loading equipment

8.31 Hydraulic jacks

Each of the three loading jacks are of 200 tons nominal capacity and of the Tangye Hydraulic Detached Ship type having simple packed rams. The ram diameter is 10 ins and the maximum ram travel is 6 ins.

One jack is bolted to the underside of the cross head beam of each of the portals in a central position. The ram travel is downwards under hydraulic oil pressure and four return springs have been fitted so that the rams return to their former position when the oil pressure is released. (see Fig. 8.8)
8.32 Pumping equipment

The basic pump unit consists of one mono-radial multiple delivery pump having three independent capacities of 55 cubic inches per minute when running at a speed of 940 r.p.m., and suitable for a maximum working pressure of 6,000 lb/ins$^2$. It was manufactured by Messrs. Andrew Frazer, and supplied by Messrs Tangyes Ltd.,

The 3 horse power Squirrel Cage, Drip Proof Motor is arranged on a common base plate with the pump unit and operates on a supply of 400/440 volts, 3 phase, 50 cycles.

8.33 Control valves.

The overhead supply tank has a nominal capacity of 25 gallons (6,920 cub. ins.) The main control valve consists of three manually operated, three positional, three-way control valves manifolded together, one valve supplying each jack.

The "three-way" control offers

a) Supply inlet to ram - ram applying load
or b) Supply inlet to exhaust - load held steady
or c) Supply inlet and ram to exhaust - unloading.

Three shut off valves have been provided so that one or more jacks may be by-passed if required for certain tests and a further three shut off valves provided so that a single pressure gauge may be connected to all or individual jack pipe lines.

A fine control valve connects each of the three pipe lines with exhaust.

A pressure relief valve is fitted to the main supply to each jack, and may be adjusted to accommodate pressures up to 6,000 lb/ins$^2$. For the wall tests they were set at 5,500 lb/ins$^2$. 
Pipework is generally 3/8 ins nominal bore steel piping with Ermeto connections. The layout of the pipework is given in Fig. 8.5 and a general view of the control console and electrical equipment is given in Fig. 8.7.

8.4 Load measuring equipment

8.4.1 Load cells

Three load cells are fitted for the wall testing frame, one load cell being placed centrally below each 200 ton jack to accurately record the load applied to the wall.

The three load cells are of the column type H.D. 110 supplied by Messrs. Davey and United, each of 200 ton nominal capacity and having an overload capacity of at least 50% on the nominal maximum.

They consist basically of electrical resistance strain gauges, bonded to the steel column of the load cell and connected as a temperature compensated Wheatstone Bridge which is balanced for zero load.

The position of the strain gauges on the steel column are such that when a load is applied to the cell, the bridge network becomes unbalanced and an electrical signal, proportional to the applied load is produced. A critical feature in this type of load cell is the height to diameter ratio which should be made greater than 12 to achieve the maximum accuracy. However, the capacity of the load cell to withstand side loads without plastic distortion is then considerably reduced, and the Davey and United design is a compromise based on experience.

The load cells have self aligning caps to ensure concentric loading and the upper surface, which comes in contact with the 200 ton ram, is a ferrobostos disc dressed with a molybdenum disulphide based
grease. The load cell is seated on a 1\(\frac{1}{2}\) ins thick steel plate with machined surfaces, which in turn is seated on the loading beam.

Each load cell has been calibrated in the laboratory using a 250 ton Dennison compression Testing Machine and all were found to be satisfactory. A typical calibration curve is given in Fig. 8.9.

The specification for the load cells type H.D.110 is quoted below

<table>
<thead>
<tr>
<th>Specification</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nominal Maximum Capacity:</td>
<td>200 ton</td>
</tr>
<tr>
<td>Overload Capacity</td>
<td>at least 50% on nominal maximum</td>
</tr>
<tr>
<td>Normal Bridge Supply</td>
<td>10 volts A.C. or D.C. (10.165 volts D.C. actual)</td>
</tr>
<tr>
<td>Bridge Output</td>
<td>16 millivolts (\pm) 0.5% for an applied load of 200 tons and with 10 volts excitation.</td>
</tr>
<tr>
<td>Bridge Output at Zero Load</td>
<td>The &quot;Electrical Zero&quot; will be less than 10% of full load output.</td>
</tr>
<tr>
<td>Temperature Co-efficient of the Electrical Zero</td>
<td>Will be 0.01% (or better) of full load output per degree centigrade change in temperature.</td>
</tr>
<tr>
<td>Impedance</td>
<td>The Nominal impedance of the load cell bridge is 400 ohms.</td>
</tr>
</tbody>
</table>

8.42 Pressure gauge

An 8 ins diameter pressure gauge for measuring pressures up to 10,000 lb/ins\(^2\) has been included in the pipe circuit so as to provide a check, based on oil pressure, on the load cell readings.

Shut off valves have been included in the circuit so that the pressure gauge reading may represent the oil pressure to individual or a combination of jacks.

8.43 Power Supply unit

The low bridge output of 16 millivolts at 200 tons from the load cells necessitated a very stable voltage supply in order to eliminate any errors due to variation in the mains voltage. In addition, variations
in output voltage due to ripple, noise and also temperature had to be eliminated.

The power supply unit selected was of 10 volts D.C. nominal output, (10.165 volts actual) and obtained from Coutant Electronics Ltd.

It was observed that during the first 50 hours of operation the output voltage dropped from 10.180 volts D.C. to 10.165 volts D.C.

This is normal during the early life of this type of power supply unit and it is expected that the voltage will now remain steady at 10.165 volts D.C. for several thousand hours of operation.

A brief specification of the unit is given below:

<table>
<thead>
<tr>
<th>Specification</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stabilisation ratio</td>
<td>(± 7% mains changes)</td>
</tr>
<tr>
<td></td>
<td>(% change in R.M.S. input voltage)</td>
</tr>
<tr>
<td></td>
<td>(% change in D.C. output voltage)</td>
</tr>
<tr>
<td></td>
<td>....5,000:1</td>
</tr>
<tr>
<td>Output impedance</td>
<td>at D.C. .... less than 0.002 ohms</td>
</tr>
<tr>
<td></td>
<td>at 100 Kc/s .. &quot; &quot; 0.05 &quot;</td>
</tr>
<tr>
<td></td>
<td>at 500 Kc/s .. &quot; &quot; 0.02 &quot;</td>
</tr>
<tr>
<td>Ripple and noise</td>
<td>............ &quot; &quot; 200μ V &quot;</td>
</tr>
<tr>
<td>Output voltage</td>
<td>............ pre-set at 10.180 volts D.C. (10.165 volts after 50 hours operation.)</td>
</tr>
<tr>
<td>Input voltage (45-65 c/s.)</td>
<td>............ 100-125 and 200-250 (by tap changing)</td>
</tr>
<tr>
<td>Temperature co-efficient</td>
<td>............ better than 0.02% per °C.</td>
</tr>
<tr>
<td>Maximum ambient temperature</td>
<td>............ 45°C</td>
</tr>
</tbody>
</table>

8.44 Digital voltmeter

The digital voltmeter was made by Digital Measurements Ltd., and the specification is given below:

Model                                      | D.M. 2001 Mk2 |
Voltage input ranges                       | ± .19995 to ± 1999.5 |
Accurary  

± 0.05% of reading or 5 least significant digits, whichever is the greater (final number reads 0 or 5 only).

Calibration  

Built-in, unsaturated standard cell

Total conversion time  

20 μsec.

Overall dimensions  

19½ ins x 8 5/16 ins x 14½ ins

Weight  

48 lb. (21.8 kg.)

Power supply  

100-125 and 200-250 V, 50 c/s

Ambient temperature  

+10°C to +40°C

The digital voltmeter is used for recording the bridge output from the load cell and is capable of reading to an accuracy of 50 micro volts. This corresponds to a load of approximately 0.53 tons on each cell.

8.45 Electrical switch controls

The 10.165 supply voltage is fed into the switch control box direct from the power supply unit. It may then be switched to any of the three load cells, or to the digital voltmeter for periodic checks on the supply voltage. When the supply voltage is fed direct to a load cell, the appropriate bridge output is recorded on the digital voltmeter.

During a loading test the bridge output from each of the three load cells is read at each loading stage, and the actual load determined from the calibration curves.

The electrical circuit is given in Fig. 8.6.

8.5 Operating Procedure

For a wall test the electrical controls are connected up as shown in Fig. 8.6 and 8.7 and allowed 5 minutes to "warm up" after switching on.

The three fine control valves are closed and the remaining six
independent valves shown on the control console in Fig. 8.7 are turned to the "open" position.

The electric motor is switched on and the "three way" main control valve set to the loading position - "supply inlet to ram". Within 3 - 5 minutes the 1\(\frac{1}{2}\) - 3 ins ram travel is taken up and loading commences. This is observed immediately on the digital voltmeter and at this stage one fine control valve is released a few turns.

It will be seen from the pipe layout in Fig. 8.5 that since all valves excluding those for fine control are open the oil pressure is equal throughout the pipe work and hence the rate of loading can now be controlled by the one partly open fine control valve.

The electrical output from each of the three load cells may be switched in turn to the digital voltmeter, and these readings used to interpret the load on each jack by reference to the calibration curves in Fig. 8.9.

During a preliminary loading test each ram was found to be equally loaded within the equivalent \(\pm 0.5\) ton accuracy of the digital voltmeter.

**8.6 Structural calculations.**

**8.61 General**

The design of the structural steelwork is in accordance with B.S.449 (1959 and amendments)\(^{44,45, 46}\).

**Permissible stresses**

**Mild steel (M.S.)**

(to B.S.15. 1961)

close tolerance bolts

Bending = 10.5 tons/ins\(^2\)
Shear = 6 tons/ins\(^2\)

**High Yield Stress Steel (H.Y.S.)**

(to B.S.968. 1962)

close tolerance bolts (B.S.548)

Bending = 14.5 tons/ins\(^2\)
Shear = 8.5 tons/ins\(^2\)
Single shear = 9.0 tons/ins\(^2\)
Black bolts

Type "X" bolts = 8.0 x $\frac{60}{15}$ tension

Single shear = 7.0 tons/ins$^2$

= 32.0 tons/ins$^2$

**NOTATION**

B. = length of stiff bearing plus additional length given by dispersion at 45° plus thickness of flange plates.

B.M. = Bending Moment

D. = Overall depth of beam

d = clear depth of web between root fillets

$f_{bc}$ = compressive stress due to bending

$f_{bt}$ = tensile stress due to bending

$f_t$ = calculated axial tensile stress

H.Y.S. = High Yield Stress

I = Moment of Inertia

L = Span

M = Mild steel

$P_c$ = allowable compressive stress in axially loaded structure.

$P_t$ = allowable axial tensile stress

$P_{bt}$ = appropriate allowable tensile stress in bending.

U.B. = Universal Beam

U.C. = Universal Column

U.D.L. = Uniformly Distributed Load

W = Load

Z = Section modulus

8.62 Grillage units

8.621 case (i)

Portal frames erected on one unit for wall tests. (see Fig. 8.10 and 8.11). Assume 9" thick brick wall 9 ft high along beam "E" and allow 6 ins R.C. slab top and bottom spanning onto wall. The 200 ton test
loads will be applied over each intersection AE, BE and CE. The reaction points through the portal legs will coincide with intersections on lines D and F. (see Fig. 8.10 and 8.11).

wt. of 9 ins wall 4 ft 6 ins wide = 4.5 x 9 x 120/2240 = 1.82t

wt. of 6 ins thick R.C. slabs = 2 x 4.5 x 6 x 72/2240 = 1.94t 3.76t

wt. of beam = 9 x 102/2240 = 0.41t

B.M. = \[
\frac{200 \times 6}{4} + \frac{3.76 \times 9}{4} + \frac{0.41 \times 9}{8}
\]

= 300 + 8.45 + 0.46

= 308.91 ft. tons.

Min. Z required = \[
\frac{308.91 \times 12}{14.5}
\]

= 213 ins\(^3\)

Use 27\(\frac{1}{16}\) ins x 10 ins x 102 lb/ft. High Yield Stress

(Z = 266.3 ins\(^3\))

Deflection

\[f_{bc} = \frac{308.91 \times 12}{266.3} = 13.9t/\text{in}^2\]

\[I = 3604.1 \text{ in}^4\]

deflection on 6ft span due to 200 ton = \[
\frac{WL^3}{48EI} = \frac{200 \times 6^3 \times 12^3}{48 \times 13,000 \times 3604.1} = 0.0332 \text{ ins.}
\]

deflection on 6 ft span due to

a) wt. of wall and R.C. slabs.

and b) self weight of beam

\[
= \frac{3.76 \times 0.0332}{200} + \frac{0.41 \times 6^3 \times 12^3}{384 \times 13,000 \times 3604.1}
\]

= 0.000625 + 0.000085

= 0.00071 ins.

Total deflection = 0.0332 + 0.00071

= Say 0.034 ins
8.622 Case (ii)

Units in square array for slab tests (see Fig. 8.1b)

In designing the connections between the grillage units cost was a consideration and a shear strength of 35.7 ton per connection accommodating a total slab or shell load (dead + superimposed load) of approximately 1,700 lb/ft$^2$ over an area 14 ft square was considered suitable.

A bending moment of 139.5 ft tons may be transmitted simultaneously through the flange connections.

**Shear connections in web** (see Fig. 8.12)

Use six 1" dia. H.Y.S. black bolts each side of break.

Use two $\frac{1}{2}$ ins thick H.Y.S. web plates $6\frac{1}{2}$ ins x 1 ft $7\frac{1}{2}$ ins safe shear value at 7 tons/ins$^2$ = 11 tons (double shear)

Safe bearing value at $12\frac{1}{2}$ tons/ins$^2$ (0.518 ins enclosed) = 6.47 tons

Let safe total load for bearing = W

Direct bearing per bolt = $\frac{W}{6} = 0.1667 W$

Bearing due to moment for bolt A = $\frac{W \times 1.5 \times 7.5}{157.5} = 0.0714W$

where $157.5 = 2(1.5^2 + 4.5^2 + 7.5^2)$

Resultant = Total bearing on bolt A = 0.1813 W.

Now max. load on bolt A = 6.47 tons

Therefore, 0.1813 W = 6.47

and hence $W = \frac{6.47}{0.1813} = 35.7$ tons = Total safe load

**Bending moment connection on flange**

Allow six 1 ins dia. H.Y.S. black bolts either side of break.

Safe shear value at 7 tons/in$^2$ = 11 tons (double shear)

Safe bearing value at $12\frac{1}{2}$ tons/ins$^2$ (0.827 ins enclosed) = 10.32 tons

Max. tension per flange = 6 x 10.32 tons = 62 tons.

depth = 2 ft $3\frac{1}{16}$ ins.
Resistance moment = 62 tons x 2.25 = 139.5 ft tons.
Max. U.D.L. on beam spanning 15 ft for slab and shell tests = W

\[ B.M. = \frac{W L}{8} \therefore W = \frac{8 \times B.M.}{L} \]

Max. U.D.L. on edge beam = \( \frac{8 \times 139.5}{15} \) 74.5 tons

= approximately 5 tons/ft run.

For slab and shell tests, 9 ins brickwork walls will normally be built up off the edge beam and so add to its stiffness.

8.63 Superstructure

8.631 Portal frames erected on one unit for wall tests

Max. vertical uplift per portal leg equals 100 t.

Use 10¼ ins x 10³⁄16 ins x 72 lb/ft. Universal Columns (U.C.)

Section modulus = 80.1 ins³ Area = 21.18 ins²

Less holes, 4 @ 15⁄16 in dia. \( \frac{3.04}{18.14} \) ins²

Axial tension = \( \frac{100}{18.14} \) = 5.52 t/ins²

\[ f_{bt} (ecc) = \frac{100 \times 5.25}{80.1} = 6.55 t/ins² \]

Use H.Y.S. steel.

\[ p_t = 13.5 t/ins² \]

\[ p_{bt} = 14.5 t/ins² \]

\[ \frac{f_t}{p_t} + \frac{f_{bt}}{p_{bt}} = \frac{5.52}{13.5} + \frac{6.55}{14.5} = 0.41 + 0.452 = 0.86 \]

0.K. less than 1.

Bolts to grillage unit

Use 4 no. 1¼ ins dia. high tensile bolts for portal leg.

Yield stress = 60 t/ins²

Permissible stress in tension = \( 8.0 \times \frac{60}{15} \) = 32 t/ins²
Area at bottom of thread = 0.9639 ins²

Safe load per bolt in tension = 0.9639 x 32 = 30.84 tons.

Load per bolt = \( \frac{100}{4} = 25 \) tons.

**Anchor plates** (see Fig. 8.13)

1\( \frac{1}{2} \) ins dia. bolts. 25 tons each. Width of flange 10 ins

C/cs of bolts 11\( \frac{1}{2} \) ins; c/cs of bolts on diagonal 1 ft 7 ins.

Length of bearing say 6 ins. Assume edge of flanges suitably stiffened.

Then B.M. = \( 25 \times 1.5 \) ins = 37.5 tons ins.

Use 2 ins thick plate

\[
\frac{bd^2}{6} = \frac{6 \times 2 \times 2}{6} = 4 \text{ ins}^3
\]

\[
f_{bc} = \frac{37.5}{4} = 9.36 \text{ t/ins}^2
\]

(permisible \( f_{bc} = 10.5 \text{ t/ins}^2 \))

**Cross beam to portal legs**

Span (clear between stanchion flanges) = 5 ft 2 ins

Central load = jack reaction = 200 t

Bending moment = \( \frac{WL}{4} = \frac{200 \times 5.17}{4} = 258 \text{ ft tons.} \)

Use H.Y.S. steel Section modulus required = \( \frac{258 \times 12}{14.5} = 214 \text{ ins}^3 \)

Use 27 ins x 10 ins x 102 lb/ft run (\( E = 266.3 \text{ ins}^3 \))

\[
f_{bc} = \frac{B.M.}{Z} = \frac{258 \times 12}{266.3} = 11.6 \text{ tons/ins}^2 \text{ (permisible} = 14.5 \text{ tons/ins}^2 \)

**Web Buckling** (B.S. 449. cl. 28A.p.48)

Slenderness ratio = \( \frac{d}{t} \sqrt{3} = \frac{24.04}{0.518} \times 1.732 = 80.4 \)

\[
P_c = 8.11 \text{ tons/ins}^2
\]

\[
t = 0.518 \text{ ins}
\]

\[
B = D + 2tp + lb. = 27 \text{ ins} + 0 + \text{say} 12 \text{ ins} = 39 \text{ ins.}
\]
Max. load = $p_c \times t \times B = 8.11^t \times 0.518 \times 39 = 164$ tons.

Provide two sets web stiffeners $\frac{3}{4}$ ins thick 12 ins apart.

**Web crushing**

Length of bearing = say 12 ins + $(27 - 24.04) \cot 30^\circ$

= $12 + 5.2 = 17.2$ ins

Max. permissible crushing load = $17.2 \times 17$ ton/ins$^2 = 292$ tons.

(O.K. Max. applied load = 200$^t$)

**Shear connection**

Max. shear load = 100$^t$

Provide 20 no. 1 ins dia. H.Y.S. steel close tolerance bolts at 7.07 tons (single shear) each = 141.4 tons.

8.632 Portal frames erected on four units for slab and shell tests

Max. bending moment = 139.5 ft. tons (section 8.622)

Max. shear = 35.7 tons (section 8.622)

Max. U.D.L. on beam = $\frac{8 \times 139.5}{15} = 74.4$ tons

Load per corner stanchion = $74.4/2 = 37.2$ tons.

Equivalent U.D.L. for slab tests, over an area 14 ft by 14 ft.

= $\frac{4 \times 37.2 \times 2240}{14 \times 14} = 1,700$ lb/ft$^2$

**Cross beams (See Fig. 8.2)**

Bending moment = $5 \times 37.2 = 186$ ft. tons

Z required = $186 \times 12./10.5 = 206$ ins$^3$

Provide 27 ins x 10 ins x 102 lb/ft. run U.B. mild steel

(Z = 266.3 ins$^3$)

Deflection = $\frac{23WL^3}{648 EI} = \frac{23 \times 37.2 \times 15^3 \times 12^3}{648 \times 13,000 \times 3694.1} = 0.164$ ins

check: $L/360 = \frac{15 \times 12}{360} = 0.5$ ins.

**Internal beam (see Fig. 8.2)**

Bending moment = $\frac{WL}{8} = \frac{2 \times 37.2 \times 15}{8} = 139.5$ ft tons.
Use 27\(\frac{1}{16}\) ins x 10 ins x 102 lb/ft. run U.B. mild steel

**Shear connections**

Max. shear = 37.2 tons.

Use 12 no. 1 ins dia. H.Y.S. steel close tolerance bolts at 7.07 tons (single shear) each = 84.8 tons.

8.64 **Loading beam** (see Fig. 8.14)

**Shear**

Use Universal beam 24 ins x 12 ins x 100 lb/ft. run (web 0.468 ins.)

Shear value = 67.39

Add \(\frac{1}{2}\) ins thick plate each side of web 24 x 0.4 x 6.0 \(t/\text{ins}^2\) = 72 \(\frac{72}{211.39}\) tons

**Bearing** (at ends)

Provide \(\frac{1}{2}\) ins thick web stiffeners 9 ins apart.

Additional \(\frac{1}{2}\) ins thick plates stiffened by \(\frac{3}{8}\) ins thick guide plate.

Beam component = 14.93

Stiff bearing (assume 12 ins.) = 12 x 5.62 = \(67.5\) \(82.43\)

2 plates \(\frac{1}{2}\) ins thick \(\frac{0.5}{0.44} \times 82.43 = 93.5\)

\(93.5\) \(269.45\) tons O.K.

8.65 **Lateral stability**

In the past, very little thought has been given to the possibility of buckling of a compression testing machine when the main members are in tension.

Chilver has shown that buckling of the frame should be investigated and also that the stability of a compression machine may be more difficult to ensure than that of a tension machine.
For the test frame discussed in this thesis with encastré ties, Chilvers paper is applicable as follows:

"Equilibrium of the unstable form is ensured if the lateral component of the force $P$ in the specimen balances the "shear" resistance of the machine. If the connecting ties of the machine are similar in all respects and are disposed symmetrically about a vertical centre line, then the lateral component of $P$ is resisted equally by the connecting ties. On applying the simple theory of bending to the deflected form of a tie it is easily shown that the critical value of $P$ is given by the root of the equation".

$$\frac{L_s}{L_t} = 1 - \frac{\tan h \phi}{\phi}$$

in which

$$\phi^2 = \frac{PL_t^2}{4nEI}$$

Morice has put forward a solution based on strain energy and this agrees closely with the above for ratios of $L_s/L_t$ up to 0.6.

where $L_t =$ unsupported length of tension members = 15.75 ft. (see Fig. 8.15)

$L_s =$ length of compression specimen between knife edges = 8.2 ft. (or between centres of ball ends)

$P =$ Total load in testing machine = 600 tons.

$n =$ number of similar tension members = 6

$EI =$ flexural stiffness of tension members = 13,000 x 420.7 tons, ins$^2$

$$\phi^2 = \frac{600 \times 15.75 \times 15.75 \times 144}{4 \times 6 \times 13,000 \times 420.7} = 0.163$$

$$\phi = 0.404$$
By reference to Fig. 8.16 it is seen that the frame is stable.

However, eccentric loading will be applied to certain test specimens and stiffening angle bracing has been provided.

## Costs

### Steelwork

Steelwork, including four grillage units, six stanchions and associated cross beams, splices, bracing and loading beam £3,300

### Pumping equipment

Pump, motor, control and relief valves, pipework 400

### Loading equipment

Three 200 t Tangye ship jacks and three Davey United load cells 670

### Load measuring equipment

Digital voltmeter, power supply, electrical connections 550

Levelling jacks, guide rollers etc. 80

**Approximate Total** £5,000

The approximate total cost of £5,000 is for the supply of equipment only. It does not include for erection and setting up costs.

### Acknowledgements

The steelwork work was fabricated by Messrs. Redpath Brown & Co. Ltd., Edinburgh.

The pumping equipment was supplied by Messrs. Tangye Ltd.
alternative grillage unit assemblies  fig 8.1

superstructure for slab and shell tests  fig 8.2
FIG. 8.3 - General view of test frame as erected for wall tests
FIG. 8.7 - General view of control console and electrical equipment

FIG. 8.8 - View of load cells and rams
Loadcell calibration curve

D.V.M. reading vs. load per loadcell (tons)

- Loadcell No. 1
- Loadcell No. 2
- Loadcell No. 3

0 to 200 tons
0 to 1800 volts x 10^5
12. 1\(^{\text{st}}\) dia. H.R.H. bolts
2\(^{\text{nd}}\) H.Y.S. flange plates.

600\(T\) total

schematic arrangement of compression machine with encastre ties and ball ended specimen

critical load of compression machine with encastre ties and ball-ended specimen
CHAPTER 9
APPENDIX 2
A COMPARISON OF COMPRESSION TESTING MACHINES
AND TEST SPECIMENS

9.1 Introduction

Considerable variations in the results of crushing strength tests of concrete cube specimens have in the past been attributed to variations in the strength of the specimens themselves. Recent investigations have shown that considerable variations can exist between testing machines, even when regularly calibrated and maintained to the British Standard B.S. 1610. This section describes a limited study to consider the variations in three testing machines.

9.2 Review of previous work

9.2.1 Stability and lateral stiffness

Chilver (1955) has shown that a simple compression machine may be highly unstable if the length of a ball ended compression specimen is short in relation to the length of the connecting ties.

Brick, brickwork and cube specimens are normally very stiff in relation to the testing machine and are therefore not likely to be influenced by induced lateral forces.

9.2.2 Spherical seating and lubrication

The present British Standard B.S. 1881 (1952) states "one of the platens (preferably the one that normally will bear on the upper surface of the cube) shall be fitted with a ball seating in the form of a portion of a sphere, the centre of which coincides with the central point of the face of the platen. The moveable portion of the spherically seated compression platen shall be held on the spherical
can be rotated freely and tilted through small angles in any direction'.

The behaviour of any spherical seating will depend on its moment of resistance - that is the product of its radius and the normal force and co-efficient of friction at the interface. The co-efficient of friction is dependent on the area and type of contact, surface finish and type of lubricant used.

The normal function of the ball seating is to allow the machine platen to bear evenly on the specimen before loading, even though opposite faces of the specimen are not geometrically parallel. After the initial loading is applied the seating should normally lock during subsequent loading.

Atherton 52 (1965) found a marked improvement in performance of a 200 t compression testing machine when the angle of movement of the top platen head was reduced. A further improvement was observed, by way of increased mean crushing strengths and lower variation, when all traces of the polar grease used to lubricate the ball seat were removed and the dry ball seat faces roughened with emery paper to increase the co-efficient of friction between them.

The results of tests on 36 sand lime bricks from a single batch for the three machine states are given in Table 9.1.

Erntroy 49 (1963) came to similar conclusions and found that the use of a polar grease resulted in mean crushing strengths about 10% lower and having a 50% greater variability than when petroleum jelly was used to lubricate the ball seat.

It is of interested that the draft B.S.1881 51 issued for comment in November 1965 no longer recommends lubrication of the ball seat and states "The spherical surfaces of the ball seating may be coated thinly
with a preservative to prevent corrosion but shall not be lubricated in such a way as to enable movement to occur under load. Lubricants designed to maintain a continuous film under high bearing pressures should not be used in ball seatings of testing machines.

9.23 Platens

The British Standard 1881 51 (1952) calls for a testing machine to have platens "whose bearing surfaces when new, shall not depart from a plane by more than 0.005" at any point and they shall be maintained within a permissible variation limit of 0.002 ins."

Non-uniform contact between the ends of the specimen and the machine platens could result in slight variations in cube strength. 50, 53, 54

However, any small divergencies from planeness in the platens may have an effect on the accuracy of the load measuring instruments used for calibration, and might account for variations of up to 5% from the true load. 54, 55

9.24 Specimen alignment

The effect on strength of misalignment has been studied and in one series of tests on 4 ins concrete cubes 50 where the spherical seating was able to tilt during loading a reduction in strength of about 15% was observed for a misalignment of $\frac{1}{4}$ ins.

A spherical seating which locked showed an insignificant difference with the same misalignment. A further series of tests on similar specimens showed reductions in strength of up to 10% for $\frac{1}{4}$ ins misalignment.

9.25 Operator technique

Data on the effects of operator technique on strength is limited. It is clear however, that in the standard compression test, where the
operator places the specimen in the testing machine, loads the specimen by some prescribed method and records the load there is likely to be some variation between operators.

The accuracy in alignment is important as also is the rate of loading, and considerable care must be taken by the operator if "true" readings are to be obtained.

9.3 Individual brick crushing tests in three different machines

9.3.1 Testing machines

Three different testing machines were used and each was situated at a different research establishment. Each was regularly maintained and calibrated in accordance with the manufacturers instructions, and each was fitted with a ball seat to the upper platen.

Machine A was a 200 ton Macklow-Smith hydraulic press and the ball-seat lubricant was Shell Tellus 72.

Machine B was a 500 ton Avery with the ball seat lubricated with mineral oil.

Machine C was a 250 ton Dennison with the ball-seat lubricated with a polar grease for series 1. For series 2 the ball-seat was dry.

9.3.2 Materials

For series 1 tests two batches of pressed double frogged clay bricks were delivered to establishment C. Three representative samples of 12 bricks were then selected from each batch and transported for preparation and testing.

The two frogs in each brick were filled with mortar in accordance with B.S. 1257 56 and B.S.3921 57 (1965) at the appropriate establishment and then stored in water until tested.
For series 2 tests a batch of single frogged sand lime bricks was used and representative samples selected for testing.

The bricks were tested dry between $\frac{1}{8}$ ins thick 3 ply sheets with the frog uppermost and unfilled.

**9.33 Experimental procedure**

The series 1 tests were carried out in the appropriate testing machine in accordance with B.S. 1257 56 and B.S. 3921 57 by the local operator so that the operator technique was included in the machine variation under observation.

The series 2 tests were carried out in machine C only. One set of bricks was crushed with the ball-seat lubricated and the other crushed after the ball-seat had been cleaned of all grease.

**9.34 Results**

The results of the series 1 tests are given in Table 9.2 and those for series 2 in Table 9.3.

**9.35 Discussion of results.**

In the series 1 tests the relative crushing strength for tests in machines A, B and C, taking A as 100% were 100%, 140.5% and 112% respectively for batch 1 bricks and 100%, 125% and 107% respectively for batch 2 bricks.

Machine B recorded considerably higher loads than machine A and C, despite the fact that all three machines are regularly maintained and calibrated.

It was discovered that the manufacturer maintaining machine C did in fact use a polar grease (Molybdenum disulphide) which is known to be one possible cause of low readings. (see section 9.22)
bricks was tested with the ball seat greased and a second batch tested with the ball seat dry.

The results shown in Table 9.3 are surprising since they reveal a 12\% reduction in strength when the ball seat was dry. If the 12\% difference in strength is accepted as lying within the normal range of experimental error and it is accepted that the two samples are of nominally the same strength then the results are still surprising since the use of a polar grease usually results in lower crushing strengths.

9.4 Conclusions

The results show considerable variation between testing machines can occur even when the machines are regularly maintained and calibrated. The reasons for the differences are not fully understood.

The use of polar grease for the ball seat may have influenced the low results for series 1 tests but this was not confirmed by the series 2 tests.

The planeness of the platens may have influenced the calibration of the machines and misalignment of the test specimens in machines A and C may have contributed towards the low results in series 1.

The true causes however are still unknown and a more extensive investigation of the three machines is required.

Atherton's tests $^{52}$ indicate a significant increase (9\%) in crushing strength when the top platen head movement is restrained.
Table 9.1

Results of crushing tests on 36 sand lime bricks

(after Atherton\textsuperscript{52})

<table>
<thead>
<tr>
<th></th>
<th>Unmodified machine with greased ball-seat</th>
<th>Restrained machine with greased ball-seat</th>
<th>degreased ball-seat</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean strength (lb/ins(^2))</td>
<td>5600</td>
<td>6100</td>
<td>6750</td>
</tr>
<tr>
<td>Standard deviation (lb/ins(^2))</td>
<td>770</td>
<td>300</td>
<td>290</td>
</tr>
<tr>
<td>Co-efficient of variation (%)</td>
<td>13(\frac{1}{2})</td>
<td>5</td>
<td>4</td>
</tr>
</tbody>
</table>
Table 9.2
Series 1 results of crushing strength tests on two batches of bricks testing in three different machines (lb/ins²)

<table>
<thead>
<tr>
<th>Machine sample</th>
<th>Batch No.1</th>
<th>Batch No.2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
<td>B</td>
</tr>
<tr>
<td>1</td>
<td>6010</td>
<td>8100</td>
</tr>
<tr>
<td>2</td>
<td>5660</td>
<td>9350</td>
</tr>
<tr>
<td>3</td>
<td>5710</td>
<td>7850</td>
</tr>
<tr>
<td>4</td>
<td>5270</td>
<td>8850</td>
</tr>
<tr>
<td>5</td>
<td>5930</td>
<td>7550</td>
</tr>
<tr>
<td>6</td>
<td>6190</td>
<td>8550</td>
</tr>
<tr>
<td>7</td>
<td>4710</td>
<td>7500</td>
</tr>
<tr>
<td>8</td>
<td>5100</td>
<td>7250</td>
</tr>
<tr>
<td>9</td>
<td>5360</td>
<td>7150</td>
</tr>
<tr>
<td>10</td>
<td>6120</td>
<td>8200</td>
</tr>
<tr>
<td>11</td>
<td>6350</td>
<td>8450</td>
</tr>
<tr>
<td>12</td>
<td>6410</td>
<td>7950</td>
</tr>
</tbody>
</table>

Mean (lb/ins²) 5735  8050  6400  6625  8300  7100


Standard deviation lb/ins² 414  634  652  608  1308  851

Relative crushing strength A = 100% 100  140.5  112  100  125  107
Table 9.3

Series 2. Results of crushing strength tests on one batch of bricks in Machine C with ball-seating lubricated and dry. (lb/ins²)

<p>| sample | Ball seat |         |         |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th>lubricated</th>
<th>dry</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td>4560</td>
<td>3320</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>4290</td>
<td>3780</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td>4430</td>
<td>3640</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>3970</td>
<td>3970</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>4430</td>
<td>3710</td>
</tr>
<tr>
<td>6</td>
<td></td>
<td>4290</td>
<td>4100</td>
</tr>
<tr>
<td>7</td>
<td></td>
<td>4100</td>
<td>4160</td>
</tr>
<tr>
<td>8</td>
<td></td>
<td>4160</td>
<td>3770</td>
</tr>
<tr>
<td>9</td>
<td></td>
<td>4430</td>
<td>3640</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td>4820</td>
<td>3910</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>lubricated</th>
<th>dry</th>
</tr>
</thead>
<tbody>
<tr>
<td>mean (lb/ins²)</td>
<td>4350</td>
<td>3800</td>
</tr>
<tr>
<td>range (lb/ins²)</td>
<td>3970 - 4820 = 850</td>
<td>3320 - 4160 = 840</td>
</tr>
<tr>
<td>standard</td>
<td>276</td>
<td>237</td>
</tr>
<tr>
<td>deviation</td>
<td>(lb/ins²)</td>
<td></td>
</tr>
</tbody>
</table>
CHAPTER 10

APPENDIX 3

BRICKWORK AND MORTAR CUBE CRUSHING TESTS IN TWO DIFFERENT MACHINES

10.1 Introduction

There is considerable interest in the use of brickwork cubes as site control specimens on building contracts where the strength of brickwork is of importance and some attempts have been made to investigate the effect on the cube strength of brick strength, mortar strength, joint thickness $18, 19, 58$ and cube capping$^{58}$. Little is known of the influence on cube strength of changes in workmanship, testing machine and operator, and it has been shown in Chapter 9 and elsewhere that for 4 ins and 6 ins concrete specimens considerable differences in crushing strength results may arise when two or more machines are used.

This describes a preliminary study of the foregoing on seven series of brickwork and mortar cubes made on a building site and in a brickworks laboratory.

10.2 Testing machines

Two different testing machines were used and each was regularly maintained and calibrated in accordance with the manufacturers instructions. Each was fitted with a ball seat to the upper platen and the upper platen and spherical head remained unclamped for testing both brickwork and mortar cubes.

Machine D was a 200 ton Avery compression testing machine housed at a local building college. Machine E was a 200 ton Amsler compression
machine housed at a brick manufacturers laboratory. When testing 4 ins mortar cubes in Machine E the possibility of unsymmetrical load was considered greater than for large specimens, and a ball joint comprised of a 2 ins dia. steel ball between 1½ ins thick plates, was therefore between the top surface of the mortar cube and the top platen of the testing machine.

10.3 Materials

10.31 Bricks

The bricks for series 1 to 5 inclusive were obtained from a building site near to D. These had previously been obtained from manufacturer E. All the bricks were single frogged semi-dry pressed.

Instructions were given that all bricks should be damp when laid and for series 5, 6 and 7 laid in the laboratory the bricks were dipped in water for 5 seconds and allowed to drain for one hour. The moisture content of the bricks as laid, expressed as a percentage of the dry weight was 2.6% for series 5 and 5.3% for series 6 and 7. For series 1 to 4 inclusive the bricks were sprayed on site before laying and no measurements were taken.

10.32 Sand

For series 1 to 5 inclusive the sand was obtained from the building site D and the sieve analysis was similar to that for sands 1 and 2 given in Table 10.1.

For series 6 and 7 a Leighton Buzzard sand was used having the sieve analysis shown for sand 3 in Table 10.1.

10.33 Lime

A class A hydrated lime in accordance with B.S.890 was used.
For series 1 to 5 inclusive the lime was obtained from site and for series 6 and 7 the lime was supplied by the brick manufacturer.

10.34 Cement

An ordinary Portland cement was used. For series 1 to 5 inclusive the cement was obtained from site and for series 6 and 7 the cement was supplied by the brick manufacturer.

10.35 Mortar

The mortar was a 1:1:6 cement/lime/sand mix by volume. The sand was mixed wet and no allowance made for bulking.

For series 5 to 7 inclusive the mortar was mixed in a Creteangle multi-flow paddle mixer of two cubic feet capacity. For series 1 to 4 inclusive the mortar was mixed in a larger site mixer.

For each mix the operator was allowed to add sufficient water to give optimum workability and for series 5 the water/cement ratio was 1.33 including an allowance for the 6.5% measured moisture content of the sand. For series 6 and 7 the water/cement ratio was 1.51 including an allowance for the 4.1% measured moisture content of the sand.

The 4 ins mortar cubes were made by hand compaction in two layers.

10.4 Experimental procedure

The scope of the test series is outlined in Table 10.2 The materials source and place of construction D was a construction site comprising nine blocks of flats in load-bearing brickwork construction.

The materials were selected from the normal site stock and were representative of the materials used in the site brickwork.

The instructions for the construction of the 9 ins brickwork cubes given to the site bricklayer were the same as those for constructing the 9 ins brickwork cubes used for the control of site
construction. The cubes were built off a firm level base to 3 courses in height with the two horizontal joints \( \frac{3}{8} \) ins thick. There was no joint at the top or the bottom. All bed, cross and perpend joints were completely filled with mortar and the uppermost frogs trowelled flush.

The brickwork cubes for series 1 to 4 inclusive were constructed and cured for either 7 or 28 days in the open during the months of May to August.

The brickwork cubes for series 5 to 7 inclusive were constructed and cured in air in the laboratory by a second bricklayer working to similar instructions as for series 1 to 4 except that series 7 brickwork cubes were capped top and bottom with a mortar bedding approximately \( \frac{3}{16} \)th ins thick.

This was achieved by laying a mortar bed on a machine finished flat steel plate and building the 9 ins brickwork cube from this bed. The following day the top surface was bedded by inverting the cubes onto fresh mortar beds placed on a flat steel plate.

The top cap, cast a day later than the remainder of the brickwork, was made of a 1:3 cement/sand mortar.

The mortar cubes tested in machine D were cured in water until tested, after removal from their moulds. They had normally been removed from the tank at least one hour before test and sometimes 15 hours. They were therefore dry or almost dry when tested. For series 1 to 4 inclusive the mortar cubes tested in machine E were cured in water after removal from their moulds for 1 to 2 days. They were then cured in air. Series 6 and 7 were cured in water.

The testing instructions given to the machine operators D and E were as follows.
"Mortar cubes - Compression tests should be made between smooth plane steel plates, without end packing and a load should be applied axially at the rate of approximately 2,000 lb/ins² per minute. One compression plate of the testing machine should be provided with a ball seating in the form of a portion of a sphere, the centre of which coincides with the central point of the face of the plate. Test specimens should be placed in the machine in such a manner that the load is applied to the sides of the specimens as cast.

Brickwork cubes - as for mortar cubes but tested between ½ ins thick sheets of plywood. The tops of cubes should be trowelled flush but not capped (except series 7) and the cube should be placed in the machine with the top as cast, uppermost.

The rate of loading should be 2,000 lb/ins² per minute".

It was observed that certain cubes rocked on their base when positioned for testing because one of the bottom bricks was not level. This was most likely due to the bricklayer tapping the side of the bottom brick after placing the next course, to line the two widths plus a joint of the bottom course with the brick length of the next.

10.5 Results

The results of crushing tests on the brickwork and mortar cubes are summarised in Table 10.2.

10.6 Discussion of results

10.6.1 Comparison of testing machines and operators

In series 1 to 4 half of the total number of specimens in each series were tested in machine D and the remaining half in machine E.

The brickwork and mortar cube crushing strengths are compared in Table 10.3
and it can be seen that there is a very close agreement indeed between the mean strengths obtained from the two machines.

The difference in brickwork cube strengths of 19% and 30% for two comparisons in series 3 and 4 were based on single tests whereas the remaining values given are means of two or three tests.

The mean values have been calculated from all the cube results and it can be seen that for the brickwork cube strengths there is a difference of only 5% between the two machines, D recording the greater strength.

When comparing the mean values for mortar cube strength there is a difference of 6.5%, D recording the lower strength. It might be expected that these differences lie within the normal experimental limits and that the calibration of the two machines is in agreement.

It can be seen that for series 1 and 2 there are differences of 16% and 21% between tests D and E respectively. The lower strengths of the D tests is most likely due to differences in curing. The D mortar cubes were cured in water until tested and this would consequently prevent the carbonation of the lime in the mortar taking place. The E cubes were cured in water for only one or two days before transportation and then cured in air. This would result in higher cube strengths for E. The differences for series 3 and 4 are less obvious.

10.62 Workmanship

A summary of the mean strengths of brickwork and mortar cubes is given in Table 10.4 there can be seen a very close agreement between the brickwork cubes made at D and E at both 7 and 28 days.

For the mortar cubes there is also close agreement between the 7 day strengths. There is, however, a considerable difference in the 28 day strengths. This is due to the high mortar strengths obtained in Series 4. These are double the normal strengths for 1:1:6 mortar cubes and are
most likely due to a richer mortar mix. The influence on the brickwork cube strength is less marked because the mortar strength influence on brickwork strength is proportional to only the third or fourth root as discussed in Chapter 2.

10.63 Materials

The bricks used for series 5 and 6 were all from the same works the only difference between them was that batch D were first delivered to the building site and batch E selected at the works. The crushing strength of bricks D and E would be expected to be similar.

The sand E was coarser than sand D as can be seen from Table 10.1.

The mortar strengths were 840 lb/ins\(^2\) and 595 lb/ins\(^2\) at 7 days and 1160 lb/ins\(^2\) and 1070 lb/ins\(^2\) at 28 days for materials D and E respectively, indicating no large differences between any of the materials so far as strength is concerned.

It can be seen from Table 10.4 however, that brickwork cubes constructed of materials D are 17% and 13% less than those of materials E at 7 days and 28 days respectively.

This may be due to the differences in sand grading and possibly sand material which, whilst not significantly influencing the mortar cube strength might have affected other properties such as shrinkage, Poisson's ratio and modulus of elasticity.

10.64 Capping

The brickwork cubes in series 7 were capped top and bottom with a 3/16th ins bed of mortar.

When compared with similar but uncapped cubes in series 6 there appeared to be no significant difference in strength. The cube strengths for capped and uncapped were 1880 lb/ins\(^2\) and 1980 lb/ins\(^2\) at 7 days.
and 2160 lb/ins\(^2\) and 2180 lb/ins\(^2\) at 28 days respectively and are summarised in Table 10.4.

The first crack normally appeared at a higher load for the capped cubes and this is probably due to the uncapped cubes not being perfectly flat on the bedding surfaces.

10.65 Increase in strength with age

The increase in strength with age was noted in series 5, 6 and 7 where the increase in brickwork cube strengths from 7 days to 28 days was 20%, 15% and 10% respectively. The corresponding increase in the mortar cube strengths were 38%, 80% and 80% respectively. These results are summarised in Tables 10.2 and 10.4.

10.7 Conclusions

1. There was good agreement between the crushing test results obtained in the two testing machines.

2. There was good agreement between the crushing strengths of brickwork cubes made of similar materials by two difference bricklayers despite some differences in the strength of certain mortar cubes.

3. There was a 17% and 13% difference in strength at 7 and 28 days respectively for brickwork cubes constructed of difference materials by one bricklayer. The mortar cube strengths from both materials were similar and the strength difference may be due to differences of sand composition and grading.

4. There appeared to be no difference in crushing strength between capped and uncapped brickwork cubes.

5. The increase in brickwork cube strength from 7 days to 28 days ranged between 10% and 20% and for mortar cubes between 38% and 80%.
Table 10.1
Sand sieve analysis - percentage by weight
passing B.S. sieves

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Table 10.2
Results of brickwork and mortar cube crushing tests

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w. cured in water  a. cured in air
Table 10.3
Comparison of brickwork and mortar cube strengths at 7 and 28 days

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<td>1585</td>
<td>1780</td>
<td>1750</td>
<td>1975</td>
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<td>1630</td>
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<td>1350</td>
<td>1830</td>
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"* mean based on all results".
Table 10.4
Comparison of the variables, bricklayer, materials
and capping on cube strength
(see also Table 10.2)

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<th>Mean cube strength</th>
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The basis of the following paper is some notes prepared originally by F. D. Entwisle (Building Surveyor for the City of Sheffield in the Department of the City Engineer and Surveyor and member of the drafting committee for the British Standard Code of Practice, C.P.111) and submitted to the Bureau for comment. Discussion and examination prompted their expansion to the present paper which is put forward as a preliminary guide on the design of non-loadbearing brickwork to resist wind forces.

It will be appreciated that there are many structural situations which are not covered by the present data. The Bureau will be glad to hear from designers about any specific problem and look forward to receiving comments on Technical Note No. 6, which is in no sense claimed to be comprehensive.

Wind Forces on Non-Loadbearing Brickwork Panels

by

R. E. BRADSHAW, A.M.I.C.E., A.M.I.Struct.E.,
and

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Introduction

Over the last few years, very severe damage has been caused to buildings by gales—particularly in Yorkshire in February 1962—and there is now a growing interest in the stability of external infill panels subjected to wind loading.

This note discusses the problems associated with lateral loading on wall panels of this type built of brickwork, and puts forward an approximate method of design for safe panel sizes and thicknesses to resist given wind loading. It is not concerned with the lower storeys of brickwork which form the supporting structure as well as the wall panel, since these walls are subject to vertical compression and hence are less likely to develop critical tensile stresses. However, the uppermost two storeys of such buildings should be investigated.

The limited research carried out to date on lateral loading of wall panels shows that, provided the panel is adequately supported at the edges, failure will usually be by bond at the brick-mortar interface, although tension failure of a well bonded panel may take place in the brick itself or in the body of the mortar when weak materials are used.

There are three main types of bond failure at the brick-mortar interface:

(i) When bending is in the vertical direction, the horizontal joint will open (Fig. 1), i.e. tensile bond failure.

(ii) When bending is in the horizontal direction the bricks may slide across mortar joint (Fig. 2), i.e. shear bond failure.

(iii) When bending is in the horizontal direction a well bonded panel may fail as shown in Fig. 3, the tension crack passing through the brick and perpend joint.

Among the factors (Ref. 1, 2, 3) influencing bond strength are:

- the initial water content and water retentivity of the mortar;
- type of mortar (cement/sand, cement/lime/sand, cement/sand with plasticizer, etc.) and cement content;
- type of brick (solid, perforated, frogged);
- thickness of mortar bed;
- workmanship.

The tensile bond strength is markedly reduced by the use of bricks of high suction rate or mortars weaker than 1:1:6 cement/lime/sand. Typical test results for tensile bond strength of mortar to solid bricks range between 10 lbf/in² and 80 lbf/in² for varying strength mortars, assuming good workmanship, correctly moistened bricks and mortar of reasonable consistency. Values as low as 4 lbf/in² and even 2 lbf/in² have been recorded when using a brick of high suction and a dry mortar. The shear bond strength may be four times as great as the tensile bond strength; during a series of tests on small panels of brickwork using cement/lime/sand mortars (Ref. 2), the modulus of rupture was a maximum of 220 lbf/in² with a failure as illustrated in Fig. 3.

It will usually be impracticable to carry out all the tests related to bond strength. Where a brick-to-mortar bond strength of 10 lbf/in² or more is required, it is recommended that samples of the bond illustrated in
Design of Panels

The following notes and graphs are put forward as an approximate method for determining safe panel sizes and wall thicknesses to resist wind loading. They are not in any way intended as an accurate stress analysis, and it is proposed to improve and modify them as more data on bond strength and on the resistance of panels to lateral loading become available.

When large infill panels are used it will often be more economical to reinforce the brickwork to span horizontally and/or vertically rather than to increase the wall thickness.

Reinforced brickwork cannot however be considered in this note, but may be referred to elsewhere (Ref. 13).

There are broadly three panel support conditions:

(a) When the panel is supported top and bottom, the sides being free, i.e. door or large window openings to either side. In this condition the wall will tend to span horizontally between discontinuous supports and failure will normally be by tensile bond (Fig. 1). The Bending Moment will be equal to approximately \( W L \) where 'L' is the vertical distance between restraints.

(b) When the panel is supported on all four sides. In this condition the wall will tend to span in two directions and failure may be by tensile bond (Fig. 1), shear bond (Fig. 2) or tension in the brick and perpend joint (Fig. 3), depending on the panel dimensions.

The maximum Bending Moment will depend upon the panel dimensions and the support conditions. Reference should be made to Tables 2 and 3.

(c) When the panel is supported at the sides—i.e. by return walls, brick piers or brick, steel or R.C. columns—and free at the top. In this condition the upper part of the wall panel will tend to span horizontally between return walls, piers or columns. The lower part will tend to cantilever from the base and failure may be by tensile bond (Fig. 4), shear bond (Fig. 2) or tension in the brick (Fig. 3) depending on the panel dimensions.

There is a growing conviction that it is wise to leave a movement joint at the top of a panel to accommodate the differential thermal and moisture movements of the frame and infill.

The designer may wish to ignore the two-way span effect due to support from three sides, since the maximum Bending Moment is usually high. In place of this he may provide a ring beam or horizontal reinforcement at the top of the wall to give top support, and consider as case (b).

Alternatively, the two-way span effect of the panel supported on three sides may be ignored and the panel considered as spanning horizontally between piers or return walls. In this case the panel may be designed for a maximum shear bond stress of 20 lbf/in\(^2\) and where the panel spans horizontally between discontinuous supports the Bending Moment may be taken as WL/8. Where the panel spans horizontally between continuous supports the Bending Moment may be taken as WL/12.

'I' in both cases is the horizontal distance between supports. (Note that where panels are supported on four sides 'L' is taken as the least panel dimension).

A check should be made on the stability of supporting piers when these are used.

For the conditions (a) and (b) it is assumed that the maximum tensile bond stress does not exceed 10 lbf/in\(^2\).

At mid-storey height the direct stress due to self-weight alone will be approximately equal to 5 lbf/in\(^2\)* and the

Resistance Moments given in Table 1 have been calculated assuming a total tensile resistance to lateral loading of 5 lbf/in\(^2\) (10 lbf/in\(^2\) \(\pm\) 5 lbf/in\(^2\)). This agrees with the revised C.P.111 (Ref. 5) as follows:

'Tensile stresses in brickwork or blockwork.

* In general, no reliance should be placed on the tensile strength of brickwork or blockwork in the calculations. The designer should assume that part of the section will be intractive and the remainder will carry compressive stress only.

* No tension should be relied upon at a damp-proof course or where water is present at the back of a wall.

* In some types of wall, tensile stresses in bending may be taken into account at the discretion of the designer. In such cases the walls should be built with bricks or blocks prepared before laying according to C.P.111.101 "Brickwork".

* For mortar not weaker than a 1 : 1 : 6 cement/lime/sand mix or its equivalent, the permissible tensile stress in bending should not exceed 10 lbf/in\(^2\), when the direction of this stress is at right angles to the bed joints, and should not exceed 20 lbf/in\(^2\) when the direction of tensile stress is at right angles to the perpend joints. The higher value should not be used where the crushing strength of the brick or block is less than 1,500 lbf/in\(^2\).

The New Zealand Standard (Ref. 4) permits tensile stresses of 5 lbf/in\(^2\) for work constructed without continuous inspection and 10 lbf/in\(^2\) for work constructed with continuous inspection.

When these stresses are due solely to wind and/or earthquake disturbances, they may be increased by one-third to 6.7 lbf/in\(^2\) and 13.3 lbf/in\(^2\) respectively.

These values are low when compared with some published figures and if site tests are carried out they may be modified accordingly, adopting a suitable load factor in accordance with the code of practice.
The section moduli for varying wall thicknesses are also given in Table 1. In certain instances, the tension stress values and the resistance moments may be increased at the designer's discretion, and examples of special situations are given below:

(a) When loads from floors or roof are supported by the wall, so increasing the direct stress.

(b) When the panel height is greater than 15 feet, the direct stress due to self weight of brickwork will be more than the 5 lb/in² allowed.

(c) When perforated bricks are used, the tensile and shear bond strengths, mortar to brick, may well be greater than for solid bricks, and a permissible stress greater than the 10 lb/in² allowed may be justified. Tests are needed to ascertain this.

(d) When steel reinforcement is incorporated in the brickwork.

(e) When the brickwork is prestressed or poststressed, using horizontal or vertical rods or wires.

**SHEAR:** In determining the shear force along the perimeter of the panel, a wind force equivalent to 1.5 p should be taken (Ref. 7).

The strength of the various support conditions shown in Table 3 should be checked to ensure that they safely support the shear forces due to wind.

**SLENDERNESS RATIO (S.R.):** The maximum length for 'least panel dimension' given in Table 1 for S.R. of 18, 24 and 30 are based on:

(a) Effective height or length equal to the lesser of actual height or length of panel;

(b) Effective height equal to three-quarters of the actual height of panel. According to the new Code, C.P.111, (Ref. 5) the factor of 0.75 may be adopted when:

(i) the floor slabs restraining the wall are of reinforced concrete construction and bearing a minimum of 4" on to the wall;

(ii) when the floors restraining the wall are of timber spanning on to the wall and where metal anchors are used (Ref. 5 and 6).

It is not intended to apply where the horizontal distance between restraints is used to ascertain the S.R., although for conditions such as F, G, H and L shown in Table 3 there is justification for doing so.

A limiting S.R. of 18 is normal for walls supported top and bottom only.

A limiting S.R. of 24 is reasonable for panels supported on four sides, and where the panel is approximately square (ratio a/b not more than 1.25) a higher limit of 30 could be adopted.

Recent research on slender walls at Edinburgh University suggests that S.R. may have less effect on the strength of walls than the reduction factors of C.P.111 imply (Ref. 5).

The tensile strength of 10 lb/in² is a permissible stress and no reduction for S.R. is required.

**Wind Loading**

The basic wind pressures shown on graphs 1 and 2, ranging between 10 and 30 lb/ft², are those values for p given in Table 3 of C.P.3, Chapter V (1952) Loading. (Ref. 7).

For plotting the graphs however, the calculations have taken into account the 0.7 reduction factor allowed for wall panels with normal openings (Ref. 7). The reduction is not applicable to buildings with both a ratio of height (to eaves level) to width of building less than one half and a pitch of roof less than 30°, nor for the design of individual panels. For such buildings or panels, the walls should be sufficiently strong to resist a total pressure outwards or inwards of 0.8 p (Ref. 7).

**Calculations**

The section moduli and resistance moments for the seven wall thicknesses considered are shown in Table 1. The five Bending Moments given in Table 2

\[
\begin{align*}
WL & \quad WL & \quad WL & \quad WL & \quad WL \\
10 & \quad 12 & \quad 15 & \quad 18 & \quad 24
\end{align*}
\]

are approximate, and for rectangular panels supported on four sides with provision for torsion at corners.

Two methods of panel edge support have been considered, viz, continuous and discontinuous. Some of the conditions providing such support are given in Table 3.

Where the maximum Bending Moment is at the continuous support, this value is given in Table 2.

Graph No. 1 shows the variation in Bending Moment with least panel dimension and the curves have been plotted for Bending Moments ranging between WL and 10, 20 and

Fig. 2. Failure of poorly-bonded panel, built with high suction bricks and mortar of moderate water retentivity.

Fig. 3. Failure of well-bonded panel, built with medium suction bricks and mortar of moderate water retentivity.

Photographs above reproduced by permission of H.M. Stationery Office. Crown copyright reserved.
Fig. 4. Author's suggested site control test for tensile bond strength of mortar to brick.

4a shows the brick pier as constructed approximately 2' 6" high (ten bricks).

4b shows the brick pier supported over a clear span of 2' 3" after seven days curing, and under load.

4c shows the brick pier after failure.

Basis of rough calculation

(i) Self weight only

Brick pier turned on side after seven days curing and supported over a span of 2' 3". Assuming a deadweight of brickwork equal to 45 lb/ft², the tensile stress developed based on elastic theory will be approximately 10 lb/in².

Bending Moment = \( \frac{WL}{8} \) = \( \frac{9}{12} \times 45 \times 2.25^2 \times \frac{12}{8} \) = 255 lb.in.

Section Modulus = \( \frac{bd^2}{6} \) = \( \frac{9}{12} \times 12 \times 4.125^2 \) \( \frac{12}{6} \) = 25.5 in³.

Stress = \( \frac{M}{Z} \) = \( \frac{255}{25.5} \) = 10 lb/in².

(ii) Pier under load

This site test is put forward primarily to compare the site variables such as moisture content of brick and mortar properties influencing tensile bond.

The greater the load supported by the pier then the greater the tensile bond strength mortar to brick.

The tensile bond stress due to the applied load may be calculated in a similar manner to that for self weight, using the appropriate bending moment. For the load applied uniformly over the length of the pier the Bending Moment will be \( WL \). For the loading shown in Fig. 4b the Bending Moment will be approximately \( WL \).

Typical Calculation for Bending Moment =

\( \frac{WL}{24} \) and \( p = 10 \) lb/ft².

L = least panel dimension (height or length),

W = wind load = wind pressure \( p \) (10 to 30 lb/ft²) multiplied by wind reduction factor 0.7 and multiplied by L.

When \( L = 7 \) feet and \( p = 10 \) lb/ft² (i.e. \( W = 10 \times 0.7 \times 7 \)).

Bending Moment = \( \frac{WL}{24} \) = \( \frac{10 \times 0.7 \times 7^2 \times 12}{24} \) = 172 lb.in./ft.

Typical Calculation for Bending Moment =

\( WL \) and \( p = 30 \) lb/ft². When \( L = 12 \) ft. and \( p = 30 \) lb/ft² (i.e. \( W = 30 \times 0.7 \times 12 \)). Bending Moment = \( \frac{WL}{15} \) = \( \frac{30 \times 0.7 \times 12^2 \times 12}{15} \) = 2,420 lb.in./ft.

By selection of the appropriate Bending Moment from the above calculations, the tensile bond can be calculated.

(a) the wall thickness, given the panel dimensions and wind pressure;
(b) the least panel dimension* given the wall thickness and the wind pressure;
(c) the maximum wind pressure, given the wall thickness and the panel dimensions.

Example 1

Single-storey factory building

Height of panel to eaves, 15 ft, with concrete ring beam at eaves level. Distance between restraints is 20 ft (see Table 3, condition J—discontinuous support at sides).

Wind pressure \( p = 12 \) lb/ft² (exposure D)

\( a = \frac{20}{b} = 1.33 \) where \( a \) and \( b \) are the panel dimensions—\( b \) less than \( a \).

From Table 2, with one support continuous and three

* When the least panel dimension is found, a check on the length/panel thickness and wind pressure shall be required.
discontinuous, the appropriate Bending Moment is

\[
\text{WL} = \frac{10}{12}.
\]

Referring to Graph No. 2 and assuming a Bending Moment of \(\frac{10}{12}\) for a wind pressure of 12 lb/ft² a 10\(\frac{1}{2}\)"—11" cavity wall is not sufficiently strong. Any of the stronger wall thicknesses to the right would be suitable. Architectural considerations and other factors will determine which is the best.

**Example 2**

12-storey block of flats (10\(\frac{1}{2}\)"—11" cavity wall)

Storey height 8' 6". Distance between restraint is 10' 6" (see Table 3, condition G—continuous support at sides);

\[
\frac{a}{b} = \frac{12.5}{8.5} = 1.48 = \text{say, 1.5.}
\]

From Table 2 with four sides continuous the appropriate Bending Moment is approximately WL. Referring to Graph No. 2, the maximum wind pressure for a least panel dimension of 8' 6" is approximately 25 lb/ft², equivalent to Exposure D. (Ref. 7).

**REFERENCES**


6: Department of Health for Scotland. Technical Memorandum No. 1, Revised 1960, "Slender wall construction for houses".


### Table 3

#### SUPPORT CONDITIONS FOR INFILL PANELS

<table>
<thead>
<tr>
<th>Position</th>
<th>Support Condition</th>
<th>Notes</th>
</tr>
</thead>
</table>
| Roof Level     | Discontinuous     | *(a) In situ or precast R.C. slabs bearing 4" minimum on to the brickwork.  
(b) In situ or precast R.C. slabs not bearing on the wall should be tied by metal anchors at intervals of not more than 6' as detail A and Ref. 5.  
(c) Timber anchored to wall using metal anchors of minimum cross-section 1/4" x 1/4" securely fastened to the joists and provided with split and upset ends or other approved means for building into the wall. The anchors should be provided at intervals of not more than 6' in buildings of one or two storeys and not more than 4' for all storeys in other buildings. For details see Ref. 5 and 6.  
If a light roof construction is adopted, take precautions to prevent roof lifting due to wind suction. |
| Intermediate Floors | Continuous     | *(a) When wall continuous past edge of floor—as details A and B. For detail A provide anchors as noted for R.C., precast concrete and timber at roof level.  
(b) When in situ, R.C. cast on top of wall (see details C and D).  
(c) When precast R.C. bearing on wall.  
(d) When brickwork constructed after R.C. framing and floors have been cast (see details C and D). |
| Ground Floor   | Continuous        | When brickwork below ground level retains fill on one side, stiffening piers may be required from Foundation to Ground Level.          |
| Sides          | Continuous        | *(a) When brickwork fully bonded to return and intersecting walls. (See details E and F).  
(b) When brickwork is continuous past R.C. columns or steel stanchions (see details G, H and I).  
(c) When the brickwork not continuous past support (see detail J). |
|                | Discontinuous     |                                                                                                                                          |

![Diagram](image1.png)

The details A to J inclusive are included to show some of the structural implications. They are not in any way intended to be comprehensive and other factors will require consideration.

For example, in detail A, if the floor slab were to deflect or the wall to expand due to thermal conditions the metal anchors would tend to lift and crack the floor screed.

In details I and J a vertical D.P.C. and protection for the ties—wall to column—may be required.

*For notes on details A to J see Table 3.*
GRAPH No.1
Showing increase in Bending Moment with respect to panel dimension for various Bending Moments and wind pressure p.

PANEL DIMENSION
IN FEET
For panels supported on four sides the least panel dimension is given. For panels supported at the top and bottom of the wall and designed for a Bending Moment of approximately 10 the panel height is given.
Set of Graphs showing least panel dimensions for varying Wind Loading and Bending Moment—based on a total tensile resistance to bending of 15 lbf/in² (10 lbf/in² tensile bond and 5 lbf/in² allowance for self-weight of brickwork)

For selection of appropriate Bending Moment refer to Table 2.

Limiting slenderness ratios. For explanation see Table 1.

For panels supported on four sides the least panel dimension is given.

PANEL DIMENSION IN FEET

For panels supported at the top and bottom of the wall and designed for a Bending Moment of approximately \( \frac{10}{3} \), the panel height is given.
The Brick in Slender Crosswall Construction

(REVISIRED)
The following note has been prepared by a member of the staff of the Clay Products Technical Bureau. In view of the highly economical nature of the design, he is aware that many readers may wish to comment on it, and the Bureau therefore invites correspondence.

The Brick in Slender Crosswall Construction

By R. E. BRADSHAW

Introduction

IT IS GENERALLY ACCEPTED that brick crosswall construction offers advantages for certain types of building, principally domestic building, in which the plan arrangement repeats on each floor. Chief among these advantages is economy, particularly where the crosswalls, which are usually 9" thick nominally, are regularly spaced throughout the scheme. These intervals may vary from ten to twenty feet, without varying the total cost of the structure by more than ±10 per cent, and it is possible with such structural arrangements to build to a height of 100 ft. or more cheaper than in steel or reinforced concrete.

The 9" thick crosswall is usually chosen because it not only adequately satisfies the function of structural support, but also gives an adequate standard of insulation against airborne sound and a more than adequate standard of fire resistance. These higher standards of sound insulation* and fire resistance are not required on all buildings, although the buildings may lend themselves to brick crosswall construction in other aspects. In certain cases, a half brick (4¾") wall is suitable as a means of support but is rarely considered because it appears so slender, yet there is no sound structural reason for this apparent prejudice. In fact, two such buildings (both curiously enough in Wales) are envisaged at the time of writing, one of six storeys, a students' hostel at Bangor, and the other a four-storey nurses' home at Glangwili. It is hoped to describe these in some detail in later issues of this series but, in the meanwhile, the following notes have been prepared on the design of a hypothetical six-storey 4¾" crosswall structure, in the belief that this highly economical type of building will prove of wide interest to many.†

General Description

A simple plan and section of the proposed hypothetical structure are set out in Figs. 1 and 2. It will be seen that the spacing of the 4¾" crosswalls is 9' 6" centre to centre (a figure chosen for its roundness rather than its relation to the sizes of bricks), i.e., 9' 6" internal finished room width. The long external walls of the front and back elevations are 11" cavity brick construction and are non-loadbearing. The flank walls are of the same construction but carry their full share of the load from the end floor slabs.

It will be agreed that adequate stability is obtained in the longitudinal direction by the stiffening effect

*According to C.P. 3, Chapter III (1966) "Sound Insulation and Noise Reduction," a wall thickness of 4½" of brickwork and plaster gives adequate sound insulation (45 db) between rooms, and between rooms and corridors in hotels, hostels and buildings of that kind (Page 37, C.P. 3, Chapter III).

†If the construction of a project of this type is envisaged in the L.C.C. area, it may be advisable to contact the Special Structures authorities.

Readers may be interested to know that research work on 4½" thick brick crosswalls commenced at Liverpool University is being extended at Edinburgh University by the Structural Ceramics Research Unit under the direction of Professor Hendry.
of both the external 11" front and rear walls and the 43" corridor walls, but it is appreciated that many may regard the external walls as not always architecturally desirable and that they might often be replaced by panels of lightweight construction. The crosswalls are then insufficiently stiffened solely by the corridor walls, and overall longitudinal stability must be obtained in some other way by, say, staircase and lift towers. The simpler structure was chosen here for the purposes of demonstration, and for simplicity the staircase and lift tower have been omitted.

Choice of floor construction, between in situ (either hollow tile or reinforced concrete) and precast varieties, must be guided by many considerations. Comparative costs, weights and speed of erection are examples, but in 43" crosswall construction there is the additional factor of the amount of end bearing of slabs, which influences, in turn, the bending moment to be considered when designing the wall*. Precast units present difficulties where the wall supports units on either side, because then there is insufficient bearing, but with in situ slabs there is no problem.

**Design Data**

Spans—floor and flat roof ... ... 9' 6"
Centre to centre of crosswalls
Storey height ... ... 8' 6"
Floor finish to floor finish.

**Roof load**

<table>
<thead>
<tr>
<th>Type of Load</th>
<th>Load (lb./ft.²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>O/w 3 layers felt on 2&quot; Stramit and timber</td>
<td>10</td>
</tr>
<tr>
<td>Live load (no access)</td>
<td>15</td>
</tr>
</tbody>
</table>

**Floor loading**

<table>
<thead>
<tr>
<th>Type of Floor</th>
<th>Load (lb./ft.²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>O/w 43&quot; hollow tile floor</td>
<td>36</td>
</tr>
<tr>
<td>Finishes</td>
<td>18</td>
</tr>
<tr>
<td>Plaster, etc.</td>
<td>6</td>
</tr>
<tr>
<td>Live load</td>
<td>60</td>
</tr>
</tbody>
</table>

* A lightweight roof may not always be advisable, and in this example, provision should be made to tie down the roof construction to the external and cross walls, using strip metal anchors at, say, 4' 6" centres.

**Calculations**

**Typical 43" Thick Crosswall and explanation of Table 1**

The critical section considered for each storey height is immediately above the floor slab level, and details of loading and stresses are set out in Table 1.

The slenderness ratio of unity (given in Table 3, page 18, C.P. 111 (1964)) should be multiplied by the factor 0.54.

Column 'g' of Table 1 shows the equivalent minimum strengths of brickwork required, assuming a slenderness ratio of unity. These were obtained by dividing the direct stresses (shown in column 'f', Table 1) by the reduction factor 0.54.

The required strength of composite brickwork can now be read direct from Table 3 of C.P. 111 (1964) page 18.

**11" Cavity Flank Walls**

Assuming the external walls to be constructed in 11" cavity brickwork, the roof and floor loads will be carried on the inner 43" leaf of the 11" cavity flank walls.

The deflection of the hollow tile floors spanning 9' 6" centre to centre of crosswalls would be very small (less than 1/8"), and consequently the eccentricity of load application on the inner 43" leaf of the flank walls would be negligible.

However, since the Code of Practice C.P. 111 offers no guidance on calculation when cavity walls are eccentrically loaded, it is proposed to assume a nominal eccentricity of 1" on the inner leaf to illustrate the procedure.
### Table 1

<table>
<thead>
<tr>
<th>Column</th>
<th>Loading lb.</th>
<th>Load lbf./ft. run per floor</th>
<th>Reduction of imposed load C.P. 3, Chap. V (1952) Table 2</th>
<th>Equivalent Minimum strength of brickwork required (Slenderness ratio = unity) lbf./in.²</th>
<th>Recommended brick and mortar strength</th>
<th>Strength of recommended brickwork (Slenderness ratio = unity) lbf./in.²</th>
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</thead>
<tbody>
<tr>
<td>Line</td>
<td></td>
<td></td>
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<td></td>
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</tr>
<tr>
<td>1</td>
<td>5th</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Floor</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>Roof:</td>
<td>dead load</td>
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<td></td>
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<td></td>
</tr>
<tr>
<td>super</td>
<td>15 lb. x 9.5</td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>o/w 4(\frac{1}{2}) brick</td>
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<tr>
<td></td>
<td><strong>558</strong></td>
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<td><strong>19.5</strong></td>
<td><strong>19</strong></td>
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<tr>
<td>2</td>
<td>4th</td>
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<td></td>
<td></td>
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<td></td>
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<tr>
<td>Floor</td>
<td>dead load</td>
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<td>super</td>
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</tr>
<tr>
<td>Floor</td>
<td>dead load</td>
<td>60 lb. x 9.5</td>
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</tr>
<tr>
<td>super</td>
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<td>Foundation†</td>
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<tr>
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<td></td>
<td></td>
<td></td>
<td><strong>146</strong></td>
<td><strong>146</strong></td>
</tr>
</tbody>
</table>

*Reaction of super load on cross-wall will be increased by 25% when adjacent spans only loaded.

†Effective height ground floor to foundation = say 3' 0".

Slenderness ratio = \(\frac{3 \times 12}{4\frac{1}{2}} = 8\)

Reduction factor = \(0.92\)

\[\text{Equivalent minimum strength} = \frac{146}{0.92} = 154\]

**Stresses shown in column "f" divided by reduction factor 0.54.**
18" apart vertically, and staggered (C.P. 111, (1964), Table 2, p. 15). These centres of ties have been shown to be adequate provided that the ties are in accordance with B.S. 1243 "Metal Wall Ties."

For multi-storey buildings, however, strip metal ties are preferable where wind pressures have to be considered and it should be remembered that ties tend to deteriorate with time, also that some weakening of the bond between the tie and the mortar in which it is embedded may result from continuous differential thermal and shrinkage movements. It is suggested in B.R.S. Digest 74 and C.P. iii (1964) clause 308f, p. 15 that the dimensional stability of the outer leaf should be assisted by projecting the floor slab through the wall at least at every third storey (See Detail A, Fig. 2). This limits possible thermal movement. However, this is not always aesthetically desirable, and there are other alternatives which might be investigated such as a free outer leaf using slotted or flexible wall ties which allow relative movements between the two leaves. Alternatively, differential movements between the two leaves may be prevented by the provision of very stiff wall ties.

11" Cavity Flank Walls and explanation of Table 2

In this design the critical section for bending stresses is taken immediately below the floor slab or roof. It could be argued that the critical section for bending stresses due to the application of eccentric load will occur immediately above the floor slab, since the direct stress will be less, but the author is of the opinion that in the latter case the bending moment may also be less, since the slab will most likely have deflected slightly under its own weight before the upper wall and its slab are completed. In this case the moment in the upper wall will be due solely to the deflection of the slab under superimposed loading.

In the case of precast concrete units this will always be so.

\[
\text{Slenderness ratio} = \frac{\text{Effective height}}{\text{Effective thickness}}
\]

(C.P. 111 (1964), clause 102, p. 10, 12 and 13).

For cavity wall

\[
\text{Effective thickness} = \frac{3}{8} \times \text{the sum of the actual thicknesses of the two leaves.}
\]

(C.P. 111 (1964), clause 307, p. 13.)

\[
= \frac{3}{8} (11" - 2") = 6".
\]

Therefore S.R. = \[
= \frac{12 \times 5 \times 12}{6} = 127.5.
\]

Reduction factor (by interpolation) = 0.73

(C.P. 111 (1964), Table 4, p. 19.)

Using this reduction factor, the equivalent minimum strengths of brickwork required (to resist the direct stresses), assuming a slenderness ratio of unity, are shown in Column "d" of Table 2.

Providing that the bending stresses set out below and given in Table 2, when combined with the direct stresses, do not exceed the maximum permissible stress for the section under consideration by more than a specified amount, it is assumed that the bending stresses, acting at the sections shown in Fig. 3, are satisfactory (C.P. 111 (1964), clause 315d, p. 19).

The critical location for bending stresses in the outer leaf of the cavity wall, due to the application of eccentric load on the inner leaf, will be at roof and 5th floor levels, since the tensile bending stresses developed will not be relieved to any extent by the compressive stress from the relatively small roof and floor loading, whereas on lower floors the direct stress will obviously be greater.

The load/ft. run on the wall at roof level is 120 lb./ft. (Table 2, Column "b").

The bending moment resulting from the assumed eccentricity of 1" = 120 x 1 = 120 lb.in. See figure 3 and discussion.

The section modulus \( Z \) of each 4\( \frac{1}{2} \)" leaf

\[
Z = \frac{bd^3}{6} = \frac{12 \times 4.5^3}{6} = 40.5 \text{ in}^3.
\]

Now, according to Davey and Thomas, any small defects, such as, will be allowed.
Table 2

<table>
<thead>
<tr>
<th>Column</th>
<th>Floor level considered</th>
<th>Loading lbf./ft. run of wall</th>
<th>Total loads lbf./ft. run on inner leaf of wall</th>
<th>Direct stress per 4(\frac{1}{2}) leaf</th>
<th>Equivalent minimum strength of brickwork required based on direct stress only (Slenderness ratio = (\frac{120}{60}) lbf./in., unity)</th>
<th>Bending moment/lb. in.</th>
<th>Bending moment/leaf/BM = W \times \frac{M}{2} lbf. in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Line 1</td>
<td>Roof</td>
<td></td>
<td></td>
<td>120</td>
<td>0</td>
<td>3(\frac{1}{2})</td>
<td>120</td>
</tr>
<tr>
<td>Line 2</td>
<td>5th Floor</td>
<td></td>
<td></td>
<td>915</td>
<td>6</td>
<td>23(\frac{1}{2})</td>
<td>475</td>
</tr>
<tr>
<td>Line 3</td>
<td>4th Floor</td>
<td></td>
<td></td>
<td>1710</td>
<td>12</td>
<td>44(\frac{1}{2})</td>
<td>475</td>
</tr>
<tr>
<td>Line 4</td>
<td>3rd Floor</td>
<td></td>
<td></td>
<td>2505</td>
<td>19</td>
<td>64(\frac{1}{2})</td>
<td>475</td>
</tr>
<tr>
<td>Line 5</td>
<td>2nd Floor</td>
<td></td>
<td></td>
<td>3300</td>
<td>25</td>
<td>84(\frac{1}{2})</td>
<td>475</td>
</tr>
<tr>
<td>Line 6</td>
<td>1st Floor</td>
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<td></td>
<td>4095</td>
<td>31</td>
<td>104(\frac{1}{2})</td>
<td>475</td>
</tr>
<tr>
<td>Line 7</td>
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<td></td>
<td></td>
<td>4890</td>
<td>38</td>
<td>91(\frac{1}{2})</td>
<td>475</td>
</tr>
</tbody>
</table>

* Effective height ground floor to foundation = 3' 0".
\(3 \times 12\)

| Slenderness ratio = \(\frac{120}{60}\) = 6
| Reduction factor = 1
| \(\therefore\) Equivalent minimum strength = \(\frac{91}{1}\) = 91

Equally by each leaf, assuming effective ties, so that

the bending moment on each leaf = \(\frac{120}{2}\) = 60 lbf. in.

and the bending stresses in each leaf = \(\frac{M}{Z}\) = \(\frac{60}{40.5}\) = \(\pm 1.5\) lbf./in.².

These stresses are negligible, particularly when the roof loading is taken into account.

Similarly the bending moment at 5th floor level on each leaf = 238 lbf./in. and the maximum bending stresses = \(\pm 6\) lbf./in.². These stresses are also negligible when the direct stress due to roof and self weight is considered.

Wind Loading

(a) General Stability

According to Chapter V of C.P. 3*, there would be no need to check this structure for overturning due to wind forces, because the height is not more than twice the least depth (Case 1, Page 27, C.P. 3, Chapter V), but it is necessary to investigate the increase of compressive stress on the leeward side of the structure.

Allow for Exposure C (v = 63 m.p.h.) (C.P. 3, Chapter V, 1952)*. Height of building = 6 \times 8.5' = 51 ft.
(b) Wind loading on Crosswalls

Assume all additional pressures due to wind taken on 4½” crosswalls.

Total wind load/crosswall = 51’ × 9.5’ × 15 lbf./ft.² = 7,240 lb.

Total wind moment/crosswall = 7,240 × 51’
= 185,000 lbf. ft.

Moment of inertia of crosswall = I = \( \frac{bd^3}{12} \)

Allow each crosswall to act as two vertical cantilever returns 4’ 4½” wide acting as compression and tension flanges.

I (crosswall 33’ O/A) = \( 2 \left( \frac{4\frac{1}{2} \times 14^3}{12 \times 12} \right) = 172 \text{ ft.}^4 \)

I (4½” flanges) = \( 2 \left( \frac{4\left(4\frac{1}{2}^3 - 13^3\frac{1}{2}\right)}{12} \right) = 280 \text{ ft.}^4 \)

Total Moment of Inertia of one crosswall = 432 \text{ ft.}^4

Total Z = \( \frac{I}{y} = \frac{452 \times 2}{14} \)

f = \( M = \frac{185,000}{64.6} = 2,869 \text{ lbf./ft.}^2 = 20 \text{ lbf./in.}^2 \)

The second moment of area of small flanges = 15 ft.‘
will result in a maximum compressive stress of 
\[(124+20) = 144 \text{lbf./in.}^2\]. Minimum strength of 
brickwork required (for slenderness Ratio=unity) 
\[= 144 \times \frac{1}{54} = 267 \text{lbf./in.}^2\].

The Code of Practice C.P. 111 (1964) allows for an 
increase in permissible stresses of up to 25 per cent, 
provided that such increase is due solely to eccentric 
loads and/or lateral forces (Clause 315d, page 19). 
In this instance the maximum permissible stress, 
allowing for the 25 per cent increase in stress, 
provided the increase is due solely to eccentricity of 
loading and/or lateral forces is 
\[\frac{125}{100} \times 270 = 338 \text{lbf./in.}^2\].

However, the actual stress of 267 lbf./in.\(^2\) is even 
within the permissible value of 270 lbf./in.\(^2\) for axial 
loading, and therefore the stress distribution is 
satisfactory. **From Table 1, Column "i" line 6.

(c) Wind loading on External Wall Panels

Typical panel, 9’ 6” wide × 8’ 6” high 
p=15 lbf./ft.\(^2\). (This value may be reduced to 0.7 
p clause 9, page 15. C.P. 3 Chapter V. Loading, but 
it is proposed to ignore the reduction for this example.)

B.M. ≈ say \[\frac{WL}{20}\]

This value is approximately equal to that given 
for two-way span slabs in C.P. 114\(^7\), assuming that 
the panels are continuous on at least two edges. 
Obviously the sizes and positions of window openings 
may have some effect and each case should be 
considered on its merits. The design of external infill 
brick wall panels subjected to wind loading has been 
discussed in detail elsewhere\(^12\).

\[B.M. = \frac{WL}{20} = 15 \times \frac{8.5^2 \times 12}{20} \]

\[= 50 \text{lbf. in.}\]

\[B.M./\text{leaf}=325 \text{lbf. in.}\]

\[\gamma = \frac{M}{Z} = \frac{325}{40.5} = \pm 8 \text{lbf./in.}^2\]

This bending stress will be a maximum midway 
between storey heights and should be added 
 algebraically to the direct stresses in the walls. 
Consider section midway between 5th floor and 
roof for the outer leaf of cavity wall.

Direct stress due to o/w 
brickwork only (assuming no roof load) \[= \frac{40 \text{lbf.} \times 4}{41} \text{lbf./in.}^2\]

\[= 3 \text{lbf./in.}^2\]

Bending stress \[\pm 8 \text{lbf./in.}^2\]

Combined stresses \[\pm 5 \text{lbf./in.}^2\] Tension 
or \[\pm 11 \text{lbf./in.}^2\] Compression.

It is extremely likely that at least some of the roof 
loading will be carried by the outer leaf, and in all 
probability the maximum theoretical tensile stress 
of 5 lbf./in.\(^2\) would not exist at all. It is considered 
therefore that this condition is satisfactory.

However, it has been suggested elsewhere\(^8\) that a 
tensile stress of up to 30 lbf./in.\(^2\), but never exceeding 
ten per cent of the calculated compressive stress, could 
be allowed in brickwork. There are arguments for 
and against this approach.

Although the tensile strength of a mortar is 
approximately equal to 10 per cent of its compressive 
strength, the critical value, when considering the 
allowable tensile stress in brickwork, is the tensile 
strength of the bond between the mortar and the 
brick. This has been investigated in America where 
a standard test, for measuring the tensile bond 
strength of mortar to brick, consists of setting two 
bricks together and then pulling the top brick away 
by means of a simple lever mechanism. From these 
tests it has been shown that the tensile bond strength 
between mortar and brick varies with the absorption 
and porosity of the brick, as well as with the type 
of mortar used.

The wetting of the bricks prior to laying was 
found to increase the tensile bond strength,\(^8\) and the 
values obtained during one series of tests\(^2\) varied 
between 18 lbf./in.\(^2\) for a dry brick and 61 lbf./in.\(^2\) 
for a wetted brick of medium porosity. These values 
are of the same order as those obtained in this country 
for solid bricks.

From these results it would appear that a 
permissible tensile stress of up to 90 lbf./in.\(^2\) might be a 

\[\text{FIG. 4}\]

\[\text{Stress distribution when}\]
tensile stress equal to 
1/10th. the compressive stress.
little ambitious, when using some types of solid brick. However, provided that the tensile stress is limited to a value not exceeding one tenth of the calculated compressive stress, the stress distribution would, in the author's opinion, be quite satisfactory even if the tensile mortar bond should fail, for under no circumstances would the tension crack extend more than approximately $D/11$ into the wall, $D$ being the wall thickness, i.e., approximately $\frac{1}{2}$" for $4\frac{1}{2}"$ wall, $\frac{3}{4}$" for $9"$ wall (Fig. 4). The revised Code of Practice C.P. 111 (1964) Table 4, page 19, extends this method for eccentricities of loading up to $D/2$, where $D$ is the wall thickness. Assuming no tensile strength is developed, there will be an infinitely high stress concentration at the extreme edge for an eccentricity of $D/2$, and hence to eliminate this, some tensile resistance in the brickwork must be provided.

For eccentricities of loading ranging between $D/6$ and $D/2$ it is usually economical to ignore the tensile strength of the brickwork and calculate stresses on the reduced wall area. In such cases the slenderness ratio should be calculated on the reduced wall thickness.

An explanatory note dealing with brickwork subjected to eccentric loading is being prepared. The use of perforated bricks will provide a greater area of contact surface between the brick and the mortar, as well as some degree of mechanical key, and some types of perforated brick could show an appreciable increase in the tensile bond strength over that of solid bricks.

**Appendix**

In view of the considerable interest shown in the first printing of this Technical Note, the following expanded comments are submitted by the author in reply to questions.

**Question**

The use of $4\frac{1}{2}"$ brick loadbearing walls for multi-storey building allows very little in the way of safety factor for bad workmanship; walls built out of plumb, use of sub-standard materials, etc. Is this really a practical proposition?

**Reply**

Tests to failure on $4\frac{1}{2}"$ thick storey-height walls constructed of $5,000$ lbf./in.$^2$ and $7,500$ lbf./in.$^2$ bricks in $1:1:6$ and $1:4:6$ cement/lime/sand mortars and loaded between $4"$ R.C. slabs indicate a load factor ranging between $11$ and $25$ when designed to the old Code of Practice, C.P. 111 (1948) and between $6:4$ and $18$ when designed to the Revised Code, C.P. 111 (1964). (See Fig. 5).

Failure of the $4\frac{1}{2}"$ walls was not by buckling as might be expected but by parallel vertical cracks through the brickwork at intervals along the length of the panel. It is believed that this was due to the horizontal strain of the mortar under load. (Fig. 6).

The horizontal strain of the mortar was greater than that of the brick and hence a horizontal tension was induced in the brickwork. The horizontal tensile strain causing failure, decreases as the mortar strength increases and as the joint thickness decreases. It is also less when the mortar joint is horizontally reinforced.

Recent work in Switzerland where high strength mortars, equivalent to $1:2$ cement/sand mortar, are $1"$ for multi-storey loadbearing brickwork, supports this view.

It should be remembered that a strong mortar is not always the best and aspects of construction such as dimensional stability—thermal and moisture movement of materials need to be considered.

The research being carried out under the direction...
structural use of 4¼" loadbearing walls on the lines of this note, is reasonable. It can achieve considerable economy and in a design similar to that illustrated but only four storeys high, the estimated saving when compared with a similar structure having a reinforced concrete or structural steelwork frame was approximately 30 per cent of the total cost of the building.

Such structures should be designed by a structural engineer experienced in loadbearing brickwork construction and particular consideration given to local stressing at beam bearings and to overall stability. Site supervision should ensure that walls are constructed true to line and level and are plumb. All vertical and horizontal joints between the bricks should be completely filled with mortar.

The problem of determining the strength of brickwork, as opposed to the strength of mortar cubes and individual bricks is being investigated. Crushing tests on 9" nominal cubes of brickwork have been carried out and it is hoped that they will form a standard site control test for ascertaining consistency in the strength of the brickwork actually built. This will detect any undue variation in mortar or brick strength, as well as in workmanship. (Fig. 7).

The 9" brickwork cube has also been adopted for control on several large loadbearing brickwork projects and it is hoped that the brickwork cube strengths may eventually be related to brick, mortar and brickwork wall strengths. Preliminary results have been published elsewhere.

For site control of tensile and shear bond strength mortar to brick, other shapes of test specimen will be required.

The following letter was received from Mr. W. G. Curtin, a consulting engineer of Liverpool.

November 7th, 1962.

I was most interested in your excellent note "The Brick in Slender Crosswall Construction", particularly so since I am the consultant for the 6 storey 4¼" crosswall structure mentioned.

The irrational prejudice against slender wall stems, I think, from the traditional use of a 9" wall. In a school where the loading was ten times that of a house, there was some opposition to the scheme. The objection proved completely unfounded, and the school has been up for several years now and shows no sign of distress. The reaction to the proposed 4¼" wall 6 storeys high for the student hostel at Bangor is even more violent, and I am sure will prove to be equally unfounded.

The walls have been designed in accordance with the draft requirements of C.P. 111 (1964) and checked in a research programme carried out at the Department of Building Science at the University of Liverpool, under the supervision of Professor Hendry and myself. It was thought that the Code was ultra-conservative, and this has been borne out by the experiments carried out so far. It would appear that the load factor for the walls is in the region of 20 based on C.P. 111 (1949).

However, it must be borne in mind that though brick is a traditional material the traditional method of concrete made by a competent contractor properly supervised on a structural frame is quite a different material from the concrete "sloshed" down by a house-holder making a garden path, so should the brickwork for a frame be of a much higher standard than that used by jerry builders for houses. There is a pressing need for a standard specification for structural brickwork. There is a need for competent workmanship adequately supervised, as in all structural work, to ensure accurate dimensions and quality control of strength.

A further problem arising in the use of slender walls is the provision of holes and chases for services. It would be advisable to trim round holes with mesh reinforcement laid in the mortar. It would be better practice to leave brickwork out rather than to knock a hole after the wall has been built. Holes should also be staggered to prevent undue stress concentrations. The problem of chases is more difficult. A ¾" chase in a 4¼" wall produces a grave risk of the wall splitting...
cut into such walls until the results of further research are available.

May I take this opportunity of congratulating you on the high standard of the information produced by your Bureau which is always interesting and valuable.

Yours faithfully,
(signed) W. G. CURTIN

Question
The design of the external 11" cavity wall assumes an eccentricity of 1". When assuming full fixity at the junction, I calculate an eccentricity of approximately 6".

Is not the value of 1" rather low?

Reply
As stated in the text, the value of 1" was arbitrarily chosen to illustrate the general procedure.

The adoption of a design assumption of full fixity at the external wall and slab junctions would, in the writer's opinion, be unrealistic and lead to over-designed wall sections! However, research into the distribution of Bending Moment and load at cavity wall/floor junctions is being carried out by the writer at the Structural Ceramics Research Unit, Edinburgh University, under the direction of Professor A. W. Hendry.

The Code of Practice C.P. III (1964) gives little guidance on the assessment of Bending Moments transferred to walls at wall/slab junctions and a preliminary guide is being prepared pending the fruition of research which could well take several years.

There are, however, several factors influencing the degree of fixity and the following should be considered:

(i) The length of bearing of the slab or beam. (A 6" reinforced concrete slab built 4½" into a 9" brick wall will not be so rigidly fixed as a similar slab built 9" into the 9" brick wall, other features being constant).

(ii) When an in situ reinforced concrete slab is constructed and designed to bear onto a brick wall and the shuttering is struck before the next lift of walling above is constructed, the slab will deflect under its own weight and form a virtual hinge at the bearing. A similar situation is met when precast floor units are employed. If, however, the next wall and slab are constructed before striking the shutters, there will be some fixity at the junction.

(iii) A cement/lime/sand mortar rather than a straight cement/sand mortar is generally more able to absorb local high stressing by readjustments in the mortar, so that the degree of fixity of the slab/wall junction when the brickwork is constructed in a 1:1:6 cement/lime/sand mortar would be less than when the brickwork is constructed in a 1:4:3 cement/lime/sand mortar.

(iv) In general terms, as the load on the brickwork increases, so the degree of fixity of the slab/wall junction increases, other factors being equal.

This is helpful in design as in the uppermost storeys of a loadbearing brickwork building, any Bending Moments transferred to the wall would be most likely to develop tensile stresses because the direct compressive stresses are low.

However, the degree of fixity of the slab/
wall junction, and hence the Bending Moment actually transferred to the wall, is less than at the lower storeys, and therefore less likely to be critical.

Question 3
(a) When calculating the Moment of Inertia of the crosswalls resisting the wind loading, should not the wall be taken as two separately acting vertical cantilevers?
(b) Also, would not the floor slabs act as deep horizontal girders and distribute the wind moment to all walls in proportion to their stiffnesses? This would mean that most of the wind moment would be carried by the stiffer end walls.

Reply
(a) Moment of Inertia of Crosswalls

According to basic theory, the two parts of the wall will act as separate vertical cantilevers and the Moment of Inertia should be calculated assuming two walls each of 14' depth and this is the basis of the calculation for "Wind Loading on Crosswalls" on page 7 of this note. The points mentioned in part (b) may need to be considered, however.

The calculation in an earlier printing of this Technical Note in June, 1962, assumes an effective depth of crosswall of 33' for assessing the Moment of Inertia.

Recent research at Edinburgh University indicates that for the type of building described in this note—that is wall and slab structures without framing beams across openings between walls—the walls may well not behave as separate cantilevers. Separate walls in the same plane may partially act together providing a more stiff structure than would appear from designs based on separately acting vertical cantilever theory.

For the purposes of this note however the latter theory is considered and any engineer who may wish to take advantage of the composite action of the walls and slabs in stiffening the structure as a whole for the design of a particular loadbearing brickwork project should carry out a laboratory study of the stiffness of a perspex model similar to the building envisaged.

(b) Floor slabs acting as deep horizontal girders

It is agreed that in the simplified plan example, the floor slabs will act as very stiff, deep girders and distribute the horizontal wind forces to the crosswalls in proportion to their stiffnesses.

Hence the gable walls, being stiffer than the crosswalls, will carry a greater proportion of the wind loading.

However, a stair tower and lift tower will normally be incorporated in the design and these together with the effect discussed under (a) make an accurate analysis of wind stresses impossible without the aid of model studies.

Acknowledgements

The author wishes to express his sincere thanks to Mr. D. Foster, A.R.I.B.A. for his assistance in preparing the text and his many helpful suggestions.

**************References **************


NOTE: For the calculations in this issue a nominal brick dimension of \( \frac{4}{8} \) in. is used. Where stresses are likely to be critical, calculations should be based on the actual dimension, i.e., \( \frac{4}{8} \) in.
6.—Crushing Tests on Storey-height Walls 4\(\frac{1}{2}\) in. Thick

By S. Prasan, A. W. Hendry and R. E. Bradshaw

Structural Ceramics Research Unit, Department of Civil Engineering, The University of Edinburgh

ABSTRACT

The results of tests on 17 walls 4\(\frac{1}{2}\) in. thick loaded between reinforced-concrete slabs are given and the relative importance of eccentricity of loading, bending due to unbalanced loading of floor slabs, movements of floor slabs and horizontally reinforced mortar joints on the strength of the wall is indicated. The results of crushing-strength tests on brickwork cubes with mortar joints up to 1\(\frac{1}{8}\) in. thick are discussed.

1. INTRODUCTION

The object of this work was to investigate the crushing strength of storey-height walls, 4\(\frac{1}{2}\) in. thick, in relation to their use in load-bearing brickwork cross-wall construction.

Most earlier research had been carried out on walls tested between knife edges and it was decided that this series of wall panels would be tested between reinforced concrete slabs so as to simulate more closely the end conditions of walls in an actual building.

Small brickwork specimens were also tested to investigate the effects of mortar joint thickness on strength.

2. EXPERIMENTAL METHOD

2.1 Loading Frame

Figure 1 shows the frame and test wall. The lower 4-in. reinforced concrete slab is 3 ft wide by 17 ft long and is supported at the centre on a 9-in. brick wall three courses high, and at the ends by the loading frame.

The test walls were each 3 ft wide, 8 ft 2 in. high clear between slabs and of 4\(\frac{1}{2}\) in. nominal thickness. The upper 4-in. R.C. slab was also 3 ft wide by 17 ft long, and supported at the centre on the 4\(\frac{1}{2}\)-in. test wall and at the ends by the loading frame.

The loading beam was seated on a single course of brickwork constructed on top of the upper slab, and a \(\frac{3}{8}\)-in. plywood bedding
on top of a $\frac{1}{4}$ in. thick rubber packing was placed immediately beneath the loading beam.

2.2 Materials

2.21 Bricks

Bricks of three different crushing-strengths were used for the wall tests and each was nominally 9 in. $\times$ 4\(\frac{1}{2}\) in. $\times$ 2\(\frac{3}{8}\) in. (Table 1).

2.22 Sand

For all the test walls except No. 1, an ordinary building sand conforming to Table 1 of B.S. 1200 was used. It was stored in the open, and in calculating the water: cement ratio an initial moisture content of 2–5% was allowed for.

2.23 Lime

For the 1:1:6 mortar mixes a class A hydrated lime in accordance with B.S. 890 was used.

2.24 Cement

For Tests 1 and 2, Ordinary Portland cement was used. In subsequent tests a rapid-hardening Portland cement (Ferrocrete) was used to give a mortar strength of over 1000 lb/in\(^2\) within 7 days.
Table 1
Brick Properties

<table>
<thead>
<tr>
<th>Types of brick</th>
<th>Crushing strength B.S. 1257 (lb/in²)</th>
<th>Strength range (lb/in²)</th>
<th>Water absorption 24 h (by wt)</th>
<th>Wall No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soid wire-cut</td>
<td>5640</td>
<td>3960</td>
<td>8·44</td>
<td>1, 2, 3, 4, 5, 6, 10, 14, 15, 16, 17</td>
</tr>
<tr>
<td>Pressed doublerogged</td>
<td>7500</td>
<td>—</td>
<td>5·65</td>
<td>7, 8</td>
</tr>
<tr>
<td>Soid wire-cut</td>
<td>4923</td>
<td>1440</td>
<td>10·88</td>
<td>11, 12, 13</td>
</tr>
</tbody>
</table>

2.25 Plasticizer
For tests 5 and 6 and 9 to 17 inclusive, a liquid plasticizer was used in place of lime (1 pint to 10 gallons of water).

2.26 Mortar Cubes
From each mortar mix, 4-in. cubes were made by hand tamping and were then air-cured; the results of the crushing tests are given in Table 2.

2.3 Experimental Procedure
2.31 Tests on Walls
Each of the walls was constructed within the loading frame by a professional bricklayer and load was applied through a loading beam seated on a single course of brickwork on the upper slab.
Six walls, 1, 2, 4, 5, 7 and 8, were tested to destruction under axial loading and, with the exception of wall 4, failure in each case was by transverse splitting.
It appeared that the mortar-joint properties had some influence on both the mode of failure and the ultimate strength of the brickwork, and four walls, 10a, 11, 12 and 13, were tested under axial loading, having varying amounts of horizontal reinforcement in the mortar joints.
**Table 2**

Crushing Strengths of 4-in. Mortar Cubes
Hand-tamped, Air-cured

<table>
<thead>
<tr>
<th>Wall No.</th>
<th>Nominal volume of mix</th>
<th>W/C ratio by weight</th>
<th>M (Age in days)</th>
<th>M7/M28</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>3</td>
<td>7</td>
</tr>
<tr>
<td><strong>PORTLAND CEMENT</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>1:1:6</td>
<td>1.42</td>
<td>392</td>
<td>609</td>
</tr>
<tr>
<td>2</td>
<td>1:1:6</td>
<td>1.02</td>
<td>442</td>
<td>582</td>
</tr>
<tr>
<td><strong>FERROCRETE</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>1:1:6</td>
<td>1.00</td>
<td>875</td>
<td>980</td>
</tr>
<tr>
<td>4</td>
<td>1:1:6</td>
<td>1.36</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>1:1:6</td>
<td>1.38</td>
<td>560</td>
<td>945*</td>
</tr>
<tr>
<td>8</td>
<td>1:1:6</td>
<td>1.83</td>
<td>385</td>
<td>525</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td><strong>FERROCRETE WITH PLASTICIZER</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>1:0:3</td>
<td>0.50</td>
<td>1166</td>
<td>1715</td>
</tr>
<tr>
<td>6</td>
<td>1:0:3</td>
<td>0.68</td>
<td>945</td>
<td>1178</td>
</tr>
<tr>
<td>9</td>
<td>1:0:3</td>
<td>0.59</td>
<td>1350*</td>
<td>1470</td>
</tr>
<tr>
<td>10</td>
<td>1:0:3</td>
<td>0.65</td>
<td>2240*</td>
<td>5320</td>
</tr>
<tr>
<td>11</td>
<td>1:0:1</td>
<td>0.45</td>
<td>2380*</td>
<td>3315</td>
</tr>
<tr>
<td>12</td>
<td>1:1:2</td>
<td>0.45</td>
<td>980*</td>
<td>1100</td>
</tr>
<tr>
<td>13</td>
<td>1:0:3</td>
<td>0.89</td>
<td>840*</td>
<td>1400</td>
</tr>
<tr>
<td>14</td>
<td>1:0:3</td>
<td>0.62</td>
<td>945*</td>
<td>1050</td>
</tr>
<tr>
<td>15</td>
<td>1:0:3</td>
<td>0.89</td>
<td>945*</td>
<td>1350</td>
</tr>
<tr>
<td>16</td>
<td>1:0:3</td>
<td>0.63</td>
<td>975*</td>
<td>1445</td>
</tr>
</tbody>
</table>

M7 and M28: mortar strength at 7 and 28 days respectively.

*Indicates day of test.

Two walls, 3 and 6, were tested under axial loading with alternate spans of the reinforced concrete slabs having superimposed loads of 30 lb/ft² and 80 lb/ft². These two tests formed a preliminary study of the influence of bending moments at wall/slab junctions upon the strength of the wall.

The effects on strength of vertical chases in brick walls were considered of importance and a chase 3 ft 6 in. \( \times \frac{3}{4} \) in. \( \times \frac{1}{2} \) in. deep was cut in one side of wall No. 3 subjected to 30 lb/ft² superimposed loading.

Two walls, 14 and 15, were constructed \( \frac{1}{2} \) in. and 1 in. respectively off centre to the loading beam and the single course of brickwork above the upper slab, to determine the effects of eccentric loading on the strength of the wall.
The final two walls, 16 and 17, together with wall 10b, which was not tested to destruction under axial loading, were subjected to sways of \( \frac{3}{4} \) in., \( \frac{5}{6} \) in. and \( \frac{1}{2} \) in. respectively, prior to loading. This was carried out by constructing the walls plumb and true to line in the normal manner, and, after curing, the upper horizontal slab was jacked laterally a distance equal to the sway, and the ends were grouted up to prevent further movement.

The upper slab bears on the wall and hence the lateral movement of the slab and the top of the wall are identical.

![Figure 2](image)

**Figure 2.**—Typical failure of axially loaded walls by transverse splitting. Note cracking does not pass through 3 ft 6 in. \( \times \frac{3}{4} \) in. \( \times \frac{1}{2} \) in. vertical chase at top left of wall.

### 2.32 Tests on Brick Piers and Wallettes

While selected test walls were being constructed, piers 8\( \frac{7}{8} \) in. \( \times \) 8\( \frac{7}{8} \) in. \( \times 4 \) courses high and wallettes 8\( \frac{5}{8} \) in. \( \times 4\frac{5}{8} \) in. \( \times 4 \) courses high were built from the same batch of bricks and mortar as the test walls.

For each of the selected walls one pier and one wallette was built with mortar-joint thickness of \( \frac{3}{8} \) in., \( \frac{3}{4} \) in. and 1\( \frac{3}{8} \) in., making a total of six specimens alongside each test wall.
The piers and wallettes were air-cured and subjected to an axial compression test on the same day as the wall test; each was built with a mortar joint at the bottom and none at the top, and the uppermost frogs were filled flush.
A sheet of $\frac{1}{8}$ in. thick plywood was placed on the top and bottom of the specimens and they were loaded between two steel plates $\frac{5}{8}$ in. thick (see Figure 3).

3. RESULTS

A summary of the results of 17 wall tests is given in Table 3, and the load factors based on the Code of Practice C.P. 111 (1948) and also the revised version, C.P. 111 (1964), have been calculated and are given in column $g$ of Table 3.

The average crushing-strengths of 4-in. mortar cubes made by hand tamping and air-cured are given in Table 2 for ages of 3, 7, 14 and 28 days.

The crushing strengths of the brickwork piers and wallettes together with a comparison with the wall strength are given in Table 4.

![Figure 3. — Typical failure of pier by transverse splitting. Joint thickness, 1\(\frac{1}{8}\) in.](image)
4. DISCUSSION OF RESULTS

4.1 Wall Tests

4.1.1 Brick Strength

The strength of the brickwork increased with the strength of the brick, though not proportionally. For axially loaded walls the ratio of (Ultimate stress in wall / Crushing strength of brick) varied between 0.29, for wall 8, and 0.36, for wall 2.

The typical failure was by transverse splitting of the brickwork, which indicates that the tensile strength of the individual bricks may influence the strength of brickwork.

4.1.2 Mortar Strength

A comparison of tests 2 and 5 suggests that, for brickwork built with a brick of 5640 lb/in$^2$ crushing strength, the use of a mortar of 2000 lb/in$^2$ crushing strength gives little gain in strength of brickwork over a mortar of 1000 lb/in$^2$ crushing strength.

FIGURE 4.—Vertical splitting and crushing of bricks at top right and centre left. Note thick bed joints.
## Table 3
Summary of Test Results and Calculations

<table>
<thead>
<tr>
<th>a</th>
<th>b</th>
<th>c</th>
<th>d</th>
<th>e</th>
<th>f</th>
<th>g</th>
<th>h</th>
<th>i</th>
<th>j</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Brick strength (lb/in²)</td>
<td>Mortar strength (lb/in²)</td>
<td>Ultimate load (tons)</td>
<td>Average compressive stress (lb/in²)</td>
<td>Permissible design stress C.P. 111 (lb/in²)</td>
<td>Load factor</td>
<td>Strength ratio Brickwork/Brick</td>
<td>Loading</td>
<td>Mode of failure</td>
<td>Remarks</td>
</tr>
<tr>
<td>Wall No.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>5640</td>
<td>1166</td>
<td>1.0</td>
<td>60</td>
<td>915</td>
<td>85</td>
<td>141</td>
<td>10.8</td>
<td>6.5</td>
<td>0.16</td>
</tr>
<tr>
<td>2</td>
<td>5640</td>
<td>945</td>
<td>1.0</td>
<td>134</td>
<td>2040</td>
<td>85</td>
<td>141</td>
<td>24.1</td>
<td>14.5</td>
<td>0.36</td>
</tr>
<tr>
<td>4</td>
<td>5640</td>
<td>80</td>
<td>1.0</td>
<td>44</td>
<td>660</td>
<td>85</td>
<td>141</td>
<td>7.8</td>
<td>4.7</td>
<td>0.12</td>
</tr>
<tr>
<td>5</td>
<td>5640</td>
<td>1960</td>
<td>1.0</td>
<td>137</td>
<td>2080</td>
<td>119</td>
<td>199</td>
<td>17.5</td>
<td>10.4</td>
<td>0.37</td>
</tr>
<tr>
<td>7</td>
<td>7500</td>
<td>945</td>
<td>1.0</td>
<td>170</td>
<td>2580</td>
<td>105</td>
<td>180</td>
<td>24.6</td>
<td>18.3</td>
<td>0.34</td>
</tr>
<tr>
<td>8</td>
<td>7500</td>
<td>630</td>
<td>1.0</td>
<td>142</td>
<td>2150</td>
<td>105</td>
<td>180</td>
<td>20.5</td>
<td>11.9</td>
<td>0.29</td>
</tr>
<tr>
<td>3</td>
<td>5640</td>
<td>1201</td>
<td>1.0</td>
<td>145</td>
<td>2205</td>
<td>85</td>
<td>141</td>
<td>26.0</td>
<td>15.6</td>
<td>0.35</td>
</tr>
<tr>
<td>6</td>
<td>5640</td>
<td>2041</td>
<td>1.0</td>
<td>128</td>
<td>1946</td>
<td>119</td>
<td>199</td>
<td>16.4</td>
<td>13.8</td>
<td>0.34</td>
</tr>
<tr>
<td>10A</td>
<td>5640</td>
<td>1350</td>
<td>1.0</td>
<td>&gt;204</td>
<td>&gt;3100</td>
<td>119</td>
<td>199</td>
<td>&gt;26</td>
<td>&gt;15.6</td>
<td>&gt;0.55</td>
</tr>
<tr>
<td>12</td>
<td>4923</td>
<td>2380</td>
<td>1.0</td>
<td>192</td>
<td>2910</td>
<td>105</td>
<td>178</td>
<td>27.7</td>
<td>16.3</td>
<td>0.59</td>
</tr>
<tr>
<td>11</td>
<td>4923</td>
<td>2240</td>
<td>1.0</td>
<td>141</td>
<td>2138</td>
<td>105</td>
<td>178</td>
<td>20.3</td>
<td>12.0</td>
<td>0.43</td>
</tr>
<tr>
<td>13</td>
<td>4923</td>
<td>980</td>
<td>1.0</td>
<td>104</td>
<td>1580</td>
<td>105</td>
<td>178</td>
<td>15</td>
<td>8.9</td>
<td>0.32</td>
</tr>
</tbody>
</table>

Remarks:
- ½-in. joints, bad workmanship
- Very weak mortar
- Vertical chase 3 ft 6 in. x ½ in. x ½ in. deep
- Reinforced 1 in 1
- Reinforced 1 in 3
- Reinforced 1 in 4
- Reinforced 1 in 5
<table>
<thead>
<tr>
<th>Wall No.</th>
<th>Brick strength (lb/in²)</th>
<th>Mortar strength (lb/in²)</th>
<th>Ultimate load (tons)</th>
<th>Average compressive stress (lb/in²)</th>
<th>Permissible design stress C.P. 111 (lb/in²)</th>
<th>Load factor</th>
<th>Strength ratio Brick-work/Brick</th>
<th>Loading</th>
<th>Mode of failure</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>14</td>
<td>5640</td>
<td>840 (1:0:3)</td>
<td>140</td>
<td>2128</td>
<td>145 max. § 249 max. § 24:6 14:7 0:38</td>
<td>Ecc. ½ in.</td>
<td>Crushing at top before splitting</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>5640</td>
<td>945 (1:0:3)</td>
<td>116</td>
<td>1700 ave.</td>
<td>145 max. § 249 max. § 31:6 18:2 0:31</td>
<td>Ecc. 1 in.</td>
<td>Sway* 2/3 in.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>5640</td>
<td>945 (1:0:3)</td>
<td>100</td>
<td>1520 ave.</td>
<td>145 max. § 249 max. § 21:4 12:8 0:27</td>
<td>Sway* 2/3 in.</td>
<td>Crushing at top before splitting</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>17</td>
<td>5640</td>
<td>975 (1:0:3)</td>
<td>124</td>
<td>1886 ave.</td>
<td>145 max. § 249 max. § 24:5 14:1 0:33</td>
<td>Sway* 2/3 in.</td>
<td>Crushing at top before splitting</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10B</td>
<td>5640</td>
<td>1350 (1:0:3)</td>
<td>204</td>
<td>3100 ave.</td>
<td>145 max. § 249 max. § 36 21:4 0:55</td>
<td>Sway* 2/3 in. re-loaded</td>
<td>Reinforced 1 in 1</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

§Maximum permissible edge stresses, and include the 25% increase for bending stresses allowed by C.P. 111 1948 and 1964.
†The average values have been calculated by dividing the total safe load by the wall area. The total safe load has been determined by assuming that the maximum edge stress is the value marked § and that the wall has no tensile resistance.
*The calculations for permissible stresses for walls subjected to sway are based on an eccentric load only equal to the sway.
### Table 4
Strengths of Walls, Piers and Wallettes (lb/in²)

<table>
<thead>
<tr>
<th>Wall No.</th>
<th>Brick strength (lb/in²)</th>
<th>Mortar strength (lb/in²)</th>
<th>Axial* wall strength (lb/in²)</th>
<th>Joint thickness (in.)</th>
<th>Axial strength</th>
<th>Strength ratio (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Pier</td>
<td>Wall/Pier</td>
</tr>
<tr>
<td>5</td>
<td>5640</td>
<td>1960 (1:0:3)</td>
<td>2080</td>
<td>$\frac{3}{8}$</td>
<td>2104</td>
<td>98.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$\frac{3}{4}$</td>
<td>1962</td>
<td>106</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$1\frac{1}{8}$</td>
<td>1905</td>
<td>109</td>
</tr>
<tr>
<td>7</td>
<td>5640</td>
<td>945 (1:1:6)</td>
<td>2580</td>
<td>$\frac{3}{8}$</td>
<td>2559</td>
<td>100.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$\frac{3}{4}$</td>
<td>2445</td>
<td>105.5</td>
</tr>
<tr>
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<td></td>
<td></td>
<td>$1\frac{1}{8}$</td>
<td>1734</td>
<td>148.7</td>
</tr>
<tr>
<td>8</td>
<td>7500</td>
<td>630 (1:1:6)</td>
<td>2150</td>
<td>$\frac{3}{8}$</td>
<td>2274</td>
<td>94.5</td>
</tr>
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<td></td>
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<td>$\frac{3}{4}$</td>
<td>2246</td>
<td>95.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$1\frac{1}{8}$</td>
<td>1592</td>
<td>135</td>
</tr>
</tbody>
</table>

*Thickness of mortar joints for walls, $\frac{3}{8}$ in.
However, the transverse strain of the mortar, causing the transverse splitting of the brickwork at failure, will be influenced by the Young's Modulus and Poisson's Ratio of the mortar. Little is known about these two properties and an investigation to determine comparative values for $E$ and Poisson's Ratio for different mortar mixes is required.

In Test 4 the very weak mortar was used—high-alumina cement with lime—and in practice it would have been condemned, since the cube strength was only 80 lb/in$^2$. However, the ultimate average compressive stress of the brickwork was 660 lb/in$^2$, giving a load factor of 7.8 and 5.5 for the 1948 and 1964 Codes of Practice, C.P. 111, respectively.

4.13 Joint Thickness

All the test walls, except No. 1, were constructed with a $\frac{3}{4}$-in. nominal joint.

The sand for the mortar used in Test 1 was coarse and the minimum joint thickness practicable was $\frac{1}{4}$ in. The mortar mix precluded "good workmanship" and the strength was 50% less than for Tests 2 and 5.

4.14 Effect of Reinforcement

The horizontal bed joints in test walls 9–13 inclusive were reinforced with Bricktor—a woven steel wire mesh $2\frac{1}{2}$ in. wide with five 17-gauge longitudinal tension wires.

When only one bed joint in five was reinforced, there appeared to be no increase in brickwork strength but, as the number of bed joints that were reinforced increased, the strength of the brickwork increased.

For wall No. 10 when each bed joint was reinforced, the load exceeded the 200t nominal capacity of the test frame and the ultimate axial load could not be determined. However, after the load had been released, a sway of $\frac{1}{4}$ in. was induced and the wall was reloaded, and in this case failure was by crushing of the brickwork just below the upper slab at an ultimate load of 204t. This figure represents an increase in brickwork strength of approximately 60% over axially loaded and unreinforced, but otherwise similar, walls (see Table 3).

4.15 Effect of Superloaded Slabs

Alternate slabs were superloaded (Figure 1) to study the effect on the strength of the wall of bending moments introduced into the wall from the slabs.

In Test 3, alternate slabs were loaded to 30 lb/ft$^2$, and there appeared to be little reduction in strength compared with a similar, but
axially loaded, wall 2. In Test 6, in which alternate slabs were loaded to 80 lb/ft², the ultimate load was 6.5% less than for a similar wall 5 axially loaded. This difference lies within the normal scatter and further tests will be made to establish the full effects of superloaded slabs on the strength of walls.

4.16 Effect of Eccentricity

For test 13, with \( \frac{1}{2} \) in. eccentricity, the strength of the wall was not noticeably less than that of similar wall No. 2 axially loaded. Comparing the axially loaded wall 5 with eccentrically loaded wall 14, the latter was surprisingly stronger, even though the mortar strength was less than half that of wall 5.

With 1 in. eccentricity, however (wall 15), the strength was 17% less than that of a similar wall axially loaded.

4.17 Effect of Sway

The strength of wall 16 with \( \frac{3}{4} \) in. sway was 14% less than a similar wall 15 with a 1 in. eccentric load, and 27% less than the axially loaded walls 2 and 5. It would therefore appear that the effect of sway on the strength of a wall is considerably more severe than that of eccentric loading alone.

4.18 Slenderness Ratio

For all the walls tested the slenderness ratio was 17.8 (based on an effective height of three-quarters of the clear height of the wall between reinforced concrete slabs).

The lateral deflections of the test walls indicate that the effective height of the walls was nearer half the clear height, and it appears that the limit for maximum slenderness ratio of 18 recommended in C.P. 111 (1948 and 1964) is conservative when the wall is restrained top and bottom by reinforced concrete slabs.

4.19 Load Factor

For the axially loaded walls, the load factor, based on the Code of Practice C.P. 111 (1964), ranged between 6.5 for wall 1, with \( \frac{5}{8} \) in. mortar joints and bad workmanship, to 15.6 for wall 3, calculated for a slenderness ratio of 18 and a reduction factor of 0.5.

Wall No. 4, with a load factor of 4.7 (C.P. 111, 1964), has been excluded, since the mortar was so weak—80 lb/in²—that it would have been condemned on any load-bearing brickwork project.

For the walls with the bed joints reinforced horizontally the load factors, based on C.P. 111 (1964), ranged between 8.9 for wall 13, reinforced every 5th bed joint, and 16.3 for wall 12, reinforced every 3rd bed joint.
The strength of wall No. 10 under axial loading was greater than the capacity of the test frame and the load factor was greater than 15.6, based on C.P. 111 (1964).

The load factors based on C.P. 111 (1948) and on the revised C.P. 111 (1964) are included in Table 3.

Tests on Brick Piers and Wallettes

As in the axial wall tests, failure was due to transverse splitting, and there was very close agreement between the strength of the walls and the piers with joints \( \frac{3}{8} \) in. thick built of similar bricks and mortar.

With \( \frac{3}{8} \)-in. mortar joints in the piers, the ratio of (Wall strength/Pier strength) for walls 5, 7 and 8 ranged between 94.5% and 100.8%. When the pier joints were increased to \( \frac{3}{4} \) in. thick, the ratio ranged between 95.7% and 100.8%, suggesting that, for mortar joints up to \( \frac{3}{4} \) in. thick, there may be little loss in strength over similar walls with thinner joints. When the pier mortar joints were increased to \( 1\frac{1}{2} \) in. there was a definite loss in strength, and the ratio (Wall strength/Pier strength) ranged between 109% and 148%.

These results (Table 4) are not conclusive, however, since they were for brickwork piers, and relate to only three test walls.

5. CONCLUSIONS

(1) The typical mode of failure by transverse splitting (Figure 2) indicates that the tensile strength of the brick and also the properties of the horizontal mortar joints, such as Young's Modulus and Poisson's Ratio, may be of primary importance in determining the strength of the brickwork.

(2) The effects observed of brick and mortar strength on the strength of brickwork are generally in agreement with the work of the Building Research Station.

(3) The brickwork piers having mortar-joint thickness greater than \( \frac{3}{4} \) in. were weaker than piers having normal mortar joints.

(4) Increases in brickwork strength of over 60% were observed when every bed joint was reinforced horizontally.

Horizontally reinforced brickwork would appear to be advantageous for highly stressed brickwork and under beam bearings.

(5) The effects of superloaded slabs would appear to be insignificant for the size of wall and slab tested. More extensive tests are needed, however, to investigate the effects of larger floor spans.
(6) The reduction in strength of walls resulting from eccentric loading when loaded between reinforced-concrete floor slabs is considerably less than when loaded between knife edges.


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