The work presented in this thesis has been composed by me unless and otherwise stated.
INVESTIGATION OF THE EFFECT OF SLENDERNESS RATIO ON THE COMpressive STRENGTH OF MASONRY WALL PANELS

A THESIS SUBMITTED FOR THE DEGREE OF DOCTOR OF PHILOSOPHY OF THE UNIVERSITY OF EDINBURGH

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DEPARTMENT OF CIVIL ENGINEERING AND BUILDING SCIENCE

NOVEMBER 1975
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ABSTRACT

The height to thickness ratio is the most significant geometric parameter influencing the bearing strength of masonry panels. In spite of this, its effect on masonry strength has not received adequate attention by various investigators. Specifications in design codes (including C.P.111(1970) are based on relatively few test results and are suspected to be conservative. Such test results as are available are conflicting.

The object of the work presented in this thesis is to investigate this problem in detail. The study is based on a series of tests conducted on 105, one third scale model wall panels made up of clay bricks or light weight concrete (Aglite) blocks under axial and eccentric loading with different end conditions. It is intended as a pilot study to determine whether the reduction factor prescribed in the codes are conservative; if they are, then this investigation will permit determination of revised values on the basis of a limited number of full scale tests.

Various theories related to the topic of this study are analysed and test results compared. The test results show agreement with some theories for particular cases. In general, theories are found to be approximate. The causes of disagreement are discussed in detail.

The reduction factors specified in the codes of various countries are compared with the test results. The results indicate scope for further revision of the codes. Also
permissible loads for different slenderness ratios and eccentricities are calculated according to the different codes. On comparison the Canadian, Swiss and draft codes (C.P.111) are found to be less conservative than others considered. The method of choosing effective height, which is closely related to the problem considered, is found to be arbitrary.
### CONVERSION FACTORS

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<tr>
<td>1 in</td>
<td>25.4 mm</td>
</tr>
<tr>
<td>1 in²</td>
<td>645.2 mm²</td>
</tr>
<tr>
<td>1 ft²</td>
<td>0.0929 m²</td>
</tr>
<tr>
<td>1 in³</td>
<td>16.39 x 10⁻⁶ m³</td>
</tr>
<tr>
<td>1 in⁴</td>
<td>0.4162 x 10⁻⁶ m⁴</td>
</tr>
<tr>
<td>1 lb</td>
<td>0.4536 kg</td>
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<tr>
<td>1 lb/ft³</td>
<td>16.02 kg/m³</td>
</tr>
<tr>
<td>1 lbf</td>
<td>4.448 N</td>
</tr>
<tr>
<td>1 ton f</td>
<td>9.964 kN</td>
</tr>
<tr>
<td>1 lbf/ft</td>
<td>14.59 N/m</td>
</tr>
<tr>
<td>1 tonf/ft</td>
<td>32.69 kN/m</td>
</tr>
<tr>
<td>1 p.s.i.</td>
<td>6.895 kPa (kN/m²)</td>
</tr>
<tr>
<td>1 p.s.f.</td>
<td>47.88 Pa (N/m²)</td>
</tr>
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<td>0.1130 N m</td>
</tr>
<tr>
<td>Pound (lbf) ft.</td>
<td>0.3707 N m/m</td>
</tr>
</tbody>
</table>
NOTATION

h - height

t - thickness

b - width of cross-section

e - load eccentricity

m - 6 e/t relative load eccentricity

y - wall lateral deflection

f - deflection of wall axis at mid-height

Y - distance from centroid

n - distance of neutral axis from compressive face of the wall

A - area of cross-section

S - distance between neutral axis and centre of gravity of stress area, $s = g.n$

ε_{e} - elastic strain

ε_{o} - strain at which slope of stress-strain curve is zero

ε - strain

ε_{*} - ultimate strain

ε_{1}, ε_{2} - largest and smallest compressive strains

E - modulus of elasticity

E_{i} - initial modulus of elasticity

R - radius of curvature

σ - compressive stress

F' - area under stress-strain curve

σ_{ult} - ultimate gross masonry stress

P - vertical load

k - $\varepsilon_{*}/\varepsilon_{e}$

q - column fixity parameter

V - distance of the line of action of the compressive force from the centre line of the wall
1.1. **INTRODUCTION**

Masonry has been used as building material since ancient times and many examples of it which date back at least from Roman times can still be seen in Herculaneum. It reached its highest aesthetic expression in Gothic Architecture. The construction of large span domes which were mainly subjected to compressive stresses was based on the intuitive quality of the designer which was an art rather than a subject of technology. There must indeed be some merit in the method of construction which even today permits the construction of buildings that are technically quite remarkable.

In Britain before the industrial revolution the majority of buildings of any size were built of stone masonry and timber. Increase in population that followed industrial revolution was so great that the supply of stone in many areas was quite inadequate for the new buildings being put up. Coal was cheap, railways spread throughout the country and so cheap, mass produced, bricks replaced stone as the principal material for the walls of all but the larger buildings. The use of wooden building (timber) which was very common in the Tudor age was discarded due to the great fire that occurred in London on 2nd September 1666.

In other countries also because of fire hazard and shortage of wood, brick became a principal building material.
There was no other material at that time to replace it economically or otherwise. With the coming of steel and concrete a new dimension in Structural and Architectural design was introduced. Steel and concrete being relatively expensive it became imperative to have economical design methods based on a rational analysis of the forces acting on the structure. This resulted in the decline of masonry as a serious building material and it no longer played the first role in the orchestra of building materials (that role was given over to the steel and concrete).

Due to lack of experimental and theoretical information the design procedures laid down by various masonry codes were conservative in comparison to the sophisticated design methods laid down for steel and concrete. Practising Architects and Engineers thought masonry to be an obsolete material which was used in the past because there was no better alternative available. The role of masonry in larger buildings was thus reduced to that of a non-load bearing material for partition walls, filler walls and as a cladding material.

The tremendous and unforeseen change in architectural attitude strengthened by structural thinking which began to manifest itself towards the early 60's and the breakthrough to a form of design which laid emphasis on expression gave fresh impetus to masonry. A notable result of this breakthrough was that masonry construction began to be based on rational design principles in contrast to the old practice when the design was based on empirical or rule of thumb methods, or the artistic inclinations or intuition of the
designer. These empirical methods required minimum wall thickness, and placed conservative limitations on wall height, height to thickness ratio and lateral support. They also failed to recognise the intrinsic strength of masonry especially when high strength bricks could be used. Because of this improvement in the design method, taller, more slender and economical structures were built. These taller buildings which are structurally more demanding require rigorous methods of design in order to achieve the desired economy and safety. This has made it necessary to study the behaviour of masonry under various types of loads.

Extensive compressive testing of full scale and model scale structures in the last 10 - 15 years have therefore been carried out and some of the factors which influence the compressive strength and the phenomena which accompany compressive failure are now more adequately understood. But there are still many important parameters about which very little is known. For example the effect of slender-ness on compressive strength is a topic of masonry research which has received very little attention, yet it is a fundamental factor in the design of load bearing walls. Because in bearing wall design, ultimate strength of masonry, slenderness of the member and effective eccentricity of the applied loads are the primary consideration.

In multistorey buildings it is essential that thin slender walls should be used in order to reduce the dead weight of the structure and thereby reduce the foundation size and overall cost. Slenderness ratio influences the
load carrying capacity of the wall especially for slender walls subjected to eccentric loading. It is hypothetical to expect any wall to be 100% plumb especially when they are very slender. Because of this shortcoming in workmanship, slenderness effect becomes an important factor in design. All design codes take this into account in specifying permissible stresses. These codes specify the stress or the load reduction factor to take into account the decrease in strength due to slenderness effect. The values of reduction factors in different codes vary and it is suspected that these values are conservative, being based on such information as was available at the time they were prepared. Nevertheless there have been substantial changes in the reduction factors over the years, the reduction factors in the current (1970) C.P.111(46) being about 50% to 126% higher than those given in the 1948(64) version. The difference increases with the increase in slenderness ratio.

Chapter 2 on review will show the amount of experimental information available so far is too small for further revision of the reduction factors to be proposed with confidence. Most of the test results available are on piers with hinged ends which do not represent the conditions prevailing in an actual building. Also available experimental results are conflicting: some show no decrease(3) at all in the strength of walls and some show significant reduction(22) in strength with the increase in slenderness ratio. Theories developed to relate slenderness ratio with the ultimate strength of walls are conservative because of the assumption that masonry is incapable
of withstanding any tensile stresses. If the tensile resistance is taken into consideration then the theory and its solution become complicated. It will no longer present a simple procedure for the designer.

Apart from lack of tensile strength, masonry, being an inelastic non-homogeneous (two-phase) material, is not amenable to theoretical analysis and so the ultimate strength and slenderness ratio relationship should be based on test results. It was therefore felt necessary to carry out a detailed experimental study of this topic to see whether there is scope for further revision of the reduction factors for slenderness.

The investigation to be described was carried out at model scale and is intended as a pilot study: if from the model work it appears that the reduction factors are in fact conservative, it will then be possible to arrive at revised values on the basis of a comparatively limited programme of full scale tests.

Slenderness ratio is defined in various codes as the ratio of the effective height "H" or effective length in the case of walls (if this is less) to the effective thickness "t". But the usual connotation of slenderness ratio is effective height "H" to radius of gyration "r" or thickness "t". In this study slenderness ratio implies ratio of effective height to effective thickness (H/t) only.

1.2. SCOPE OF INVESTIGATION

Chapter 2 reviews the study carried out on various aspects of masonry in general and slenderness effect on the
ultimate strength of walls in particular.

Chapters 3, 4 and 5 describe in detail the experimental technique developed for testing single leaf, bonded brick walls and single leaf light weight concrete (Aglite) block walls at one third model scale. The total number of walls tested was 105, but the results of 93 walls have been considered. Due to experimental error and change in test programme, results of remaining walls are not discussed. The test results are compared with other test results and discussed in detail keeping in view the various parameters influencing their strength.

Chapter 6 is concerned with the theoretical analysis of the strength-slenderness relationship. The test results are compared with various theoretical methods of predicting wall strength and are then discussed keeping in view the limitation involved in experimental technique and the assumptions involved in developing theoretical methods.

In Chapter 7 a critical study of the design provision of various codes pertaining to reduction factor - slenderness ratio relationship is done. The Authors' test results and other similar test results are compared and then discussed in the light of these codes' provision. The latest draft revision of the C.P.111 (1970) code is also studied and its proposals are compared with other test results in order to see the justification for these proposals.

Chapter 8 gives general conclusion of the investigation carried out.

Appendix I:- Gives basis for the eccentricity calculation.
Appendix II:-- Gives details of the instruments used in this investigation.

Appendix III:-- Gives details of pier tests.
CHAPTER 2

REVIEW OF PREVIOUS WORK

2.1. BRICKWORK

This chapter reviews past work carried out on the different aspects of the brick masonry in general and the slenderness effect on the ultimate strength of masonry in particular.

The earliest rational testing of brick masonry started in 1860 at the Watertown Arsenal in U.S.A. Previous to that, work had been mostly confined to testing of individual brick units and mortar units.

Tests were carried out on 173 brick masonry piers to study the effect of different (1) brick strength, (2) mortar strength, (3) slenderness ratio, on the compressive strength of masonry. The results of these tests were not conclusive because only one or two specimens were tested for each factor influencing the strength of the brickwork. However, there was a decrease in ultimate compressive strength of the pier with the increasing slenderness ratio (Fig. 2.1.).

In Britain, R.I.B.A. carried out tests on 59 brick piers built with both cement and lime mortar. The results of these tests (Published 1905) coupled with American test results formed the basis for the design of masonry structure in the U.S.A. in those days.

Because test data available so far were based on a more favourable laboratory condition of the test than may be realised in practice, the National Brick manufacturers Association felt the necessity of more comprehensive...
REDUCTION FACTOR

Vs.

SLENDERNESS RATIO

(AFTER SCHRIF)

Fig. 2.1

REDUCTION FACTOR

Vs.

SLENDERNESS RATIO

[BAYE & THOMAS]

Fig. 2.2

Fig. 2.3
investigation. As such in 1914 a test programme was prepared for a large number of brick piers and the test carried out in collaboration with Bureau of Standards. The results of these tests were reported (2) by Bragg in 1918 along with a brief description of some of the previous test results relevant to their testing programme.

Among the different test results described in the above report, the test results of Prof. Krueger on piers of 11 ins.² cross-section and of varying height are of interest. According to him there is increase in pier strength when mortar strength is increased and brick strength is kept constant, or vice-versa. He also studied the effect of varying height and of non-axial loading on the strength of the pier, with the mortar and brick strength constant. The test results showed that there is decrease in pier strength with the increase in height. Fig. 2.1. and Table 2.1. give results of the test. The one snag in the test was that the Brickwork to Brick strength ratio cannot be compared with other tests because of difference in testing method of individual bricks.

Bragg's own investigation (2) dealt with the effect of brick type, mortar type, bond type, and grade of workmanship on the ultimate strength of piers. His conclusions were not very different from that of previous investigators. According to him primary failure of brickwork is due to tensile splitting of the individual bricks. The ultimate strength of pier is increased by any of the factors, such as (1) by increasing depth of the brick unit, (2) laying wire mesh in all horizontal joints, (3) by having thin mortar
<table>
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<tr>
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<td>10.6</td>
<td>78.8</td>
<td>610</td>
<td>0.26</td>
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</table>
joints of uniform thickness, (4) by having the right combination of brick and mortar units. Also in 1:3 cement sand mix, 25% by volume of cement may be replaced by hydrated lime without appreciably affecting the strength of pier.

Based on these results he obtained a relationship between brickwork strength and compressive and transverse strength of individual units as

\[ P = \frac{z}{\sigma} = Z \sigma_T \]

where \( z \) is constant depending upon grade of mortar. \( \sigma \) and \( \sigma_T \) are compressive and transverse strength of brick piers respectively. The bricks used in the test were selected from four different regions of U.S.A. so as to have geographical representative sample.

In 1921 Faber(3) published a report on an experiment carried out to study stability of thin walls and concluded that under truly axial loading (which is difficult to achieve) slenderness had remarkably little effect on wall strength except when weak mortar such as lime mortar is used. The lime mortar reduces the stiffness of the wall and may induce stability failure.

In the early 1930's large number of tests were carried out on brick piers in Denmark by Svenson.(4) On the basis of these test results he emphasised the important effect the modulus of elasticity \( E \) of brick units have on compressive strength of masonry walls. The variation in \( E \) value of brick units results in localised stress concentration or internal eccentricity which lead to irregular failure pattern over a given cross-section.
Till 1934 there was no standard procedure for testing masonry structures and their units, this resulted in considerable difficulty in analysing various test results available.

Glanville and Barnett\(^{(5)}\) published a report in 1934 in which they attempted to co-relate and compare different available test results and their procedure in order to give maximum possible guidance in predicting the strength of masonry structures built from these units.

Apart from the above-mentioned work, they also dealt with other factors related to masonry strength such as (1) mortar strength, (2) wall geometry, (3) workmanship, (4) bond, (5) stress-strain relationship, (6) mode of failure, (7) effect of variation in the brick strength on the strength of mortar.

They suggested the most suitable form of test on an individual brick unit is by filling the frogs with the mortar in the brick and testing them in flat position between plywood sheets.

Regarding the remaining above-mentioned factors their conclusions were similar to that of the previous investigator. They also pointed out the limitation in predicting masonry strength based on the brick units forming them, because of unavoidable variation in the form and texture etc. of bricks.

Davey and Thomas\(^{(6)}\) also studied different methods of brick test along with other related factors such as (1) mortar strength, (2) brick strength. Interestingly enough they suggested a strong case for using weak mortar
because of its ability in adjusting to differential movements, as the cracking, according to them, in brickwork is rarely due to vertical applied load. A strong mortar if used will result in the development of fine cracks between mortar and brick which may permit passage of water through the masonry apart from giving bad appearance.

They also devoted their attention to the effect of slenderness and eccentricity on compressive strength of wall. According to them the ultimate load $P$ is a function of different parameters such as (1) Eccentricity $e$, (2) width "w", (3) thickness "t" and (4) height "h". For convenience the effect of eccentricity and S.R., in spite of their being interrelated, were dealt with separately. Hence

$$\frac{P_a}{P_o} = f_3 \left( \frac{h}{t} \right) = \alpha$$
$$\frac{P}{P_a} = f_2 \left( \frac{e}{t} \right) = \beta$$

In order to study this effect piers of different dimensions and different mix were tested. The test results are shown in Fig. 2.2. It is seen that load ratio factor $\alpha$ is not only dependent on height but also on eccentricity.

In the case of 9 ins. x 9 ins. piers there is no significant difference in strength below slenderness Ratio 14 and Eccentricity $E \leq \frac{t}{P}/4$ compared to decrease in strength for greater eccentricity and greater height than mentioned above. However 13 1/2 ins. x 13 1/2 ins. square cross-section piers built of similar brick and mortar and having smaller slenderness ratio (12) showed no marked difference in strength as compared to that of 9 ins. x 9 ins. piers.

Regarding the effect of eccentricity there is greater
decrease in load ratio \( p \) for piers having higher slenderness ratio particularly at higher eccentricity.

The experimental load ratio is compared with the theoretical load ratio, which is given by the equation

\[
P/P_a = \left( \frac{1}{1 + 6 e/t_p} \right)
\]

For \( e \geq 0 \) and \( e \leq t_p/6 \)

Also \( P/P_a = \frac{3}{4} (1 - 2 e/t_p) \)

For \( e \geq t_p/6 \) \( e \leq t_p/2 \).

From these results they suggested that excluding 9 ins. x 9 ins. square pier corresponding to slenderness ratio of 18, the theoretical relationship could be used in the design of brick structures.

In the case of slender structures slenderness and eccentricity are interrelated; this results in further reduction in load-carrying capacity of the structure than would be expected for that particular eccentricity. This is attributed to the bending deflection which can be excessively high for very slender structures under high eccentric loading.

Thirty 4\( \frac{1}{2} \) ins. thick brick walls were tested axially and eccentrically with \( \frac{1}{2} \) in. and 1 in. load eccentricity. Except for one wall all walls were tested under flat-ended condition thus providing directional restraints.

The results indicate decrease in strength Fig. 2.3. with the increase in height, but without any definite relationship between them. The reduction factor given in C.P.111(64) (1948) is on the lower side. The slenderness effect on the strength of the walls has been assumed to be of less importance in comparison to that of the pier, because in the
case of the walls due to larger width the tendency is to neutralise the causes responsible for lowering its strength in comparison to that of the pier. C.P.111 (1948) therefore specified taking the effective height of the wall as 3/4 of the actual height of pier.

They also carried out studies on cavity walls and reinforced brickwork.

In spite of large numbers of test results available on brick piers in Britain, no attempt was made towards formulating a more rational basis for the design of brick structures. Davey and Thomas made efforts in this direction in order to reduce the factor of uncertainty involved in deciding the influences of various factors on the masonry strength.

In 1953 F.G. Thomas suggested changes in C.P.111 (1948) after analysing the various test results available with Building Research Station. He studied the effect of brick and mortar strength and suggested that C.P. (64)111 (1948) be modified to permit high working stress for brickwork using cement lime mortar or lime mortar with high strength bricks. He also showed the capability of brickwork to resist lateral forces when built into a steel framework.

The effect of slenderness on the ultimate compressive strength of brickwork was divided into two sections by Thomas. One set of results was for brick piers and the other for brick walls. Piers of 9 ins. and 13½ ins. square cross-section built with different combination of brick and mortars were axially tested with hinged ends up to slenderness ratio of 18. The results show a definite
decrease in strength with the increase in slenderness ratio of the piers. The proportion by which the strength decreases is different for different combinations of brick and mortar (Fig. 2.4.). The results are reported in terms of strength reduction coefficient (Factor) which is the ratio of pier strength of specific slenderness to pier strength of slenderness ratio unity. Weak brick and weak mortar pier combination has greater decrease in strength as compared to strong brick and strong mortar.

The reduction factors given in C.P.111(64) (1948) are lower than the reduction factor for strong brick and strong mortar. In the case of weak brick and weak mortar the reduction factor is lower than the reduction factor of C.P.111(64) (1948) up to slenderness ratio of 12, beyond this reduction factor is higher than that given in C.P.111(64) (1948). The average reduction factors so obtained were suggested by Thomas(3) to be adopted when the code is revised, provided the scatter of results about the mean value is considered in a general load factor. Since the piers were tested between knife edges, the effective height was taken as the actual height of pier thus avoiding any directional restraint present. In actual practice pier ends will always have some restraint provided by the floor slab.

Thomas(3) also dealt with non-axially loaded piers of different brick and mortar strength combinations which showed higher strength reduction factor in comparison to axially loaded piers, because of reduced lateral stiffness of the pier due to cracking, particularly for a tall pier.
AVERAGE REDUCTION FACTOR Vs. SLENDERNESS RATIO (3 C.P. 111 (AFTER THOMAS) (1948))

Fig. 2.4

Fig. 2.5. LOAD FACTOR OF PIER Vs. SLENDERNESS RATIO.

9" x 9" PIERS

<table>
<thead>
<tr>
<th>BRICK STR.</th>
<th>MORTAR STR.</th>
<th>BRICK STR.</th>
<th>1:1:6 STR.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1140 p.s.i.</td>
<td>140 p.s.i.</td>
<td>2600 p.s.i.</td>
<td>670 p.s.i.</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>BRICK STR.</th>
<th>1:0:3 STR.</th>
</tr>
</thead>
<tbody>
<tr>
<td>2600 p.s.i.</td>
<td>3200 p.s.i.</td>
</tr>
<tr>
<td>8700 p.s.i.</td>
<td>2000 p.s.i.</td>
</tr>
</tbody>
</table>

| e=t12 | e=0 | e=t3 | e=t6 | e=t14 |

Fig. 2.6
Tests on a number of 9 ins. and 13½ ins. square section piers conducted at the Building Research Station do not exhibit any significant difference in behaviour despite different combinations of brick and mortar (Fig. 2.5.), and hence Thomas assumed a uniform reduction factor to take care of different brick and mortar effects. Based on this assumption he suggested another strength reduction factor to take into account different eccentricity of load at different slenderness ratio (Fig. 2.6.).

Due to relatively high compressive stress which develops at eccentricity of load greater than t/3 and also because of the small amount of test data available for eccentricity t/2, Thomas limited eccentric loading up to t/3. Reduction factors shown in Fig. 2.6. for \( e = \frac{t}{2} \) are therefore to be used with caution.

The reduction factors applied to axially loaded piers are also applicable to walls under similar load condition. The difference is in the calculation of slenderness ratio, which is 3/4 of that of the pier. As explained earlier, it is because the factors responsible for reducing the strength are less effective in walls in comparison to piers. This does not seem to be the only reason, because in the case of walls tested under flat-ended conditions the end restraints provided by the beam are also considered.

C.F.111 (64) (1948) and Thomas' recommendation about effective height do not seem to be consistent, because walls loaded with hinge ends will have no end restraints and hence the effective height will be greater than 3/4 times the actual height as specified by them. The value of 3/4...
REDUCTION FACTOR
Vs.
SLENDERNESS RATIO
(3)
(THOMAS)

Fig. 2.6
of actual height is arbitrary and should be based on the actual end rotation, lateral deflection and curvature variation along the height. This aspect will be discussed in detail at a later stage.

Tests carried out by Thomas (3) on 4\(\frac{3}{4}\) ins. thick wall built from medium strength brick and 1:1:6 mortar mix, do not show any considerable reduction in strength with the increase in slenderness ratio. The results of 4\(\frac{3}{4}\) ins. thick wall loaded at eccentricity of t/9 and 2t/9 show decrease in strength reduction factor with the increase in height. Thomas compared his load factors (Table 2.2.) with load factors specified by the C.P.111 (1948) and suggested use of values given in code because of greater variation in load factor for higher slenderness ratio.

In Germany, in 1952, comprehensive work (7) concerning influence of factors such as (1) brick strength, (2) mortar strength, (3) bond type, (4) window opening and (5) chases in wall on the load bearing capacity of brick wall was carried out. The results of these tests, though similar to previous work, no doubt assisted in clarifying the effects besides brick and mortar strength which are of significance for wall strength.

Chapman and Slatford (8) in 1957 studied the mechanism of wall and column failure made up of brittle material with no tensile strength under different end conditions such as (i) hinge, (ii) eccentric load, (iii) clamped ends. In their theoretical analysis, which was basically a formulation of a differential equation between load, deflection and eccentricity, they introduced an imperfection constant to
### TABLE 2.2.

**LOAD-FACTORS$^3$**

<table>
<thead>
<tr>
<th>S.R. Based on C.P.111</th>
<th>$e/t = 1/9$</th>
<th>$e/t = 2/9$</th>
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<td></td>
<td>Based on C.P.111</td>
<td>Based on Fig. 2.6</td>
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<td>7.5</td>
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<td>9.1</td>
</tr>
<tr>
<td>22.5</td>
<td>14.1</td>
<td>7.9</td>
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</table>
take care of any defect in the vertical alignment of the wall or column under consideration. Since the material studied was brittle, the shape of this initial imperfection was taken as triangular instead of sinusoidal or parabolic or arc of circle. Haller, Monk, Turkstra have taken the deflection curve as sinusoidal and have found good agreement between the theory and experimental result.

This imperfection constant is neglected when eccentricity of loading is considered in the analysis.

The Davey and Thomas test results were found to be in good agreement with their theoretical analysis. In 1962 when calculated brickwork was still in its infancy state in Britain, Foster did a study of various brick buildings built in Switzerland and critically compared Swiss and British codes with particular reference to strength and slenderness relationship. Based on detailed calculation for a wall of particular slenderness ratio he showed that British code unduly penalises slenderness and to a greater extent the eccentricity. Like Thomas he recommended drastic changes should be made in C.P.111.01 (1946).

By the middle Sixties enough research was done to alter the existing approach of using high safety-factors because of uncertainty in the behaviour of walls under different loading conditions. It was felt necessary that with the improved quality of brick and mortar, brick structures could be designed more economically by utilising the latent excellence properties of brick and mortar.

In 1967 the Texas Conference on masonry structural systems paved the way towards this approach of using more
reasonable safety factors and so avoiding uneconomical and heavy masonry structures.

Grenley et al. tested single leaf walls under uniform vertical compressive and transverse loads in order to study interaction between flexural and compressive stress. These walls were built of three different types of bricks and two different types of mortar (ordinary mortar and high bond mortar). The theoretical interaction curve so developed when compared with their test results showed good agreement.

The use of high bond mortar increased considerably the compressive and flexural strength of the wall over ordinary mortar. These curves enable reasonably accurate prediction of masonry strength in comparison to those curves which only take the negative effects of the interactions.

These interaction curves form the basis of the study done by Yokel et al. to investigate the slenderness effect of concrete-block walls on their compressive strength.

Monk studied the behaviour of walls under eccentric loading by assuming a non-linear stress-strain curve for cracked and uncracked cross-sections with, and without, introducing slenderness effect into the analysis. Three types of eccentricity were considered: (i) load eccentricity; (ii) chance eccentricity, which is due to inaccurate construction of wall and inherent variation of mortar and brick properties; (iii) bending eccentricity due to lateral deflection of the wall. The eccentricity was of different magnitudes and was on different sides of the wall.
column ends.

To facilitate the solution of the cubic formula so derived, Monk suggested a computer program which could facilitate solution of this otherwise complicated formula. He compared his theoretical results with the experimental results of SCPRF for different cases of load eccentricity and end conditions. In all types of loading including for axially loaded walls, a minimum chance eccentricity of t/20 was taken in the analysis. This allows more reasonable comparison of experimental results with the theoretical values. The theory is discussed in detail in Chapter 6.

Risager (10) studied statical behaviour of linear elastic walls without tensile strength (brick walls), erected between floor slabs without sidesway and with equal angular rotation at top and bottom ends. He developed a theoretical method by which for the given values of height, thickness, width, elastic modulus, compressive stress and end rotations, the bearing capacity, eccentricity of compressive force at the wall ends, mode of failure and crack condition could be determined.

Hilsdrof (11) developed a theory to predict the compressive strength of masonry which is a function of strength of brick under uniaxial compression and biaxial tension, uniaxial compressive strength of mortar unit, geometry of the masonry units and non-uniformity coefficient to account for workmanship.

Motteu (12) studied physical and mechanical properties of brickwork units under masonry load using different mortar mix and emphasised the necessity of further research into the
effects of workmanship, masonry, geometry and curing time.

West et al.\(^{(13)}\) reported tests on walls of wire-cut bricks to study the effects of perforations. The relationship between brick cube strength and masonry wall strength was also discussed.

Creep in brickwork was studied by Lenczner\(^{(14,50)}\) who stressed the importance the creep effect has in evaluating a more realistic value of Young's modulus in brickwork theory, obtaining more accurate analysis pertaining to buckling, stability and deformation problems.

Haller\(^{(15)}\) studied the load-bearing capacity of brick masonry under compressive vertical loads and proposed a theory based on stress-strain properties of the masonry. This theory will be dealt with in detail in Chapter 6.

Chen and Atsuta\(^{(23)}\) evolved a general method to analyze strength of walls of different materials, such as steel, unreinforced concrete, brick and concrete blocks. The materials were assumed to be elastic, perfectly plastic and yield stress levels in tension and compression may not necessarily be the same. The method is based on determining equivalent column length by iterative procedure. Based on this equivalent length the curvature distribution is found and then slope and deflections are readily calculated by numerical method. The loadings and the end moments may not necessarily be the same and the end moments may be either due to load eccentricity or due to the rotation of the slab or beam resting on the wall.

A notable feature of this method in comparison to other methods of analysis is the inclusion of small tensile
strength and ductility in the analysis which results in an appreciable effect on the strength of the walls. Comparison of a few of these analytical methods with the experimental results of Yokel et al. showed good agreement.

Turkstra studied\(^{(24)}\) strength of masonry walls under eccentric loading based on parabolic stress-strain relationship of a short wall. Sensitivity of the wall to different shape of stress-strain curve was also studied. Eccentricity considered in the analysis is composed of (1) load eccentricity and (2) accidental eccentricity, which is due to wrong alignment and defective construction of the wall, and is unavoidable.

Similar to Haller's analysis, two cases were considered; one when the neutral axis lies outside wall section, and the other when the axis lies inside the wall section, i.e. the case when the tension cracks occur. The analysis completely ignores the tensile strength of masonry. Theoretical results so obtained were compared with the test results of Haller and Monk which showed reasonable agreement with the theory. This will be discussed in detail at a later stage, in Chapter 6.

Prasan et al.,\(^{(16)}\) investigated the effects of eccentricity, bending due to unbalanced floor moments and horizontal reinforcement in bed joints on the load capacity of walls. The results of test on \(4\frac{3}{4}\) ins. thick wall loaded between reinforced slabs showed less reduction in strength of eccentrically loaded walls in comparison to similarly loaded walls between knife-edges.

Elastic modulus, Poisson's ratio of mortar and tensile
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Prasan et al. (16) investigated the effects of eccentricity, bending due to unbalanced floor moments and horizontal reinforcement in bed joints on the load capacity of walls. The results of test on 4½ ins. thick wall loaded between reinforced slabs showed less reduction in strength of eccentrically loaded walls in comparison to similarly loaded walls between knife-edges.

Elastic modulus, Poisson's ratio of mortar and tensile
strength of brick are of importance in estimating brickwork strength. Use of reinforcement in mortar beds increases brickwork strength. Increase in thickness of mortar joint reduces brickwork strength. The limiting value of S.R. = 18 specified at that time was conservative.

Murthy and Hendry\textsuperscript{(18,65)} initiated model studies related to brickwork structures. They did extensive tests on one third and one sixth scale brickwork and compared their results with the results of full scale tests previously done by Building Research Station, taking into account the effect of mortar strength, slenderness ratio and eccentricity. Slight variation was noticed between the two results. This study paved the way towards model testing in brickwork. Thus avoids heavy expenditure of money and time spent on full scale tests. The model test results could form the pilot programme of the main full scale tests or even their results could be taken as conclusive as has been satisfactorily done in many cases.

Francis, Harmans and Jerems\textsuperscript{(19)} dealt with the mechanism of brickwork failure quantitatively and the effects of joint thickness and number of courses on brickwork strength.\textsuperscript{(20)}

Watstein et al. studied the influence of (1) organic modified high bond mortar, (2) ordinary sand mortar, (3) slenderness ratio and (4) eccentricity of loading have on the load bearing capacity of brickwork and on their elastic modulus. The test results showed:

\begin{enumerate}
\item An increase in compressive strength of prisms built with high bond mortar over conventional mortar. Also increase of the secant modulus of prisms built with high bond mortar over prisms built with ordinary cement-sand mortar.
\end{enumerate}
(2) Ultimate strength of walls loaded at eccentricity of t/3 and t/6 having slenderness ratio of 22.8 and built in high bond mortar showed higher strength over similar walls built in conventional mortar.

(3) There is a decrease in strength of walls and prism with increasing slenderness ratio.

Turnsek and Cacovic\textsuperscript{21} studied the effects of axial, eccentric and horizontal loads on the strength, deformation and failure mechanism of brickwork. They measured the lateral strain responsible for causing tensile splitting of brickwork. Also, based on the regression analysis of the test results of several walls they plotted a generalised stress-strain curve and compared the measured curve with the calculated curve which did not differ much in shape.

SCPI\textsuperscript{22} published several reports dealing with the compressive, transverse and racking strength of masonry walls. Some of the reports also dealt with the slenderness effect on the compressive strength of wall panels. These reports will be discussed later in Chapter 3.

Morton\textsuperscript{51} did a detailed study of static and dynamic lateral resistance of brickwork panels. In this he showed that for a particular precompression, the lateral load capacity of brick wall decreases with the increase in slenderness ratio. He also studied the effect of gas explosion on the strength of brickwork.

Based on Chen and Atsuta method, Mazzolani\textsuperscript{52} studied the strength of walls in the stable and unstable range by means of a step by step simulation method. The stress-strain relationship has been assumed to be elastic-perfectly plastic. The loading process takes into account the end axial loads...
with different end eccentricities. The study was carried out with the help of a computer. The method needs to be verified with the available test results.

2.2. BLOCKWORK

In the past concrete block structures did not attract as much attention as clay brick structures. This resulted in comparatively less study being carried out on block structures. Mostly the results of studies carried out on brick structures were applied to them. The design procedure laid down in the building codes were similar to those used for clay brick structures.

In the following section a brief review of the work carried out on different aspects of blockwork in general and slenderness effect on the compressive strength of blockwork in particular will be dealt with now.

Early testing of block walls started in U.S.A. Prior to 1932 little information was available regarding various factors influencing strength of walls. The only test results available at that time were on pilasters and walls constructed mainly for fire tests. In 1932 Richart et al. (27) conducted extensive tests to study the different factors governing block wall strength. Apart from this the other objectives of their study were:

a) to establish a relationship between physical properties of masonry units, mortars and walls,
b) to study the stability and strength of concrete masonry walls under concentric and eccentric loads.

They tested 69 storey height walls of 6 ft. length built with 16 ins. x 8 ins. x 8 ins. three oval core dense block of compressive strength ranging between 550 p.s.i. to 1570 p.s.i. The mortar which was generally stronger than the units had cylinder crushing strength between 670 p.s.i. and...
2200 p.s.i. The walls were cured for 32 days.

In order to determine whether the small wall panels i.e. wallettes could be taken to be representative samples of storey height walls, they also tested 42 wallettes 32 ins. long and 48 ins. high.

The test results showed that

1. compressive strength of large wall panels varied between 335 p.s.i. and 850 p.s.i. and is dependent on the strength of block rather than on the mortar strength,

2. average ratio of wall to block strength was found to be 0.53 which remained constant,

3. the wallettes strength could be taken to be representative of the actual storey height walls because strength ratio of wall to wallette is 0.91,

4. the compressive strength of 8 ins. wall made with 3-oval core units with face-shell mortar bedding was about 80% of that obtained from similar walls with full mortar bedding. The flexural strength was almost the same for both cases,

5. initial modulus of elasticity varied between 0.3 x 10^6 and 1.17 x 10^6 p.s.i.,

6. the values of modulus of rupture vary from 18 p.s.i. to 50 p.s.i.,

7. the walls in which the eccentric load was applied at the edge of the middle third of the wall thickness deformed very consistently and developed strengths averaging 76% as great as were obtained for axial loading,

8. the factor of safety of walls in axial compression based upon working stress of 70 to 80 p.s.i. for units with an average strength of 700 p.s.i. or more varied from 5.0 to 11.5.
In order to study the influence of mortar and block strength on the walls, Copeland and Timms (28) tested wallettes built with hollow units of strength 320 to 4180 p.s.i. and mortar ranging from 150 p.s.i. to 4800 p.s.i. and found the strength ratio lying between 0.33 and 0.79. The test result showed that for a specific mortar strength, wall strength increases linearly with block strength and that the potential strength of the wall can be obtained by using mortar at least as strong as the blocks.

Kristen and Schulze (29) carried out tests similar to those carried out by Richart et al. (27) on the walls and wallettes made with various types of blocks and found the wall to wallette ratio as 0.81 which is not very different from those found by Richart et al. (27).

In 1939 Whitmore, Stang and Parsons (30) tested single leaf cavity walls under compressive transverse, concentrated, impact and racking loads. In compression tests, failure of single leaf walls was by vertical splitting of different courses and crushing of top four courses. In cavity walls failure occurred in the loading. Failure under lateral loading was by rupture of the bond between the facing and backing at the bed joints. In racking tests, loads were applied near the upper end of each wall specimen to a bearing plate covering both facing and backing. In the case of the walls built with mortar of approximately the same strength as that of the block, the failure was by crushing of blocks in both the faces and diagonal shear along a diagonal path through the blocks. Failure of walls built with weaker mortar than block was by rupture of bond between the mortar
and the block in a stepwise crack.

Based on a number of test results of the walls built of solid and hollow blocks, Hermann(31) in 1943 formulated an expression between blockwork strength, block and mortar strength as

\[ W = K \sqrt[3]{m \cdot s^3} \]

where
- \( W \) = blockwork strength
- \( m \) = mortar strength
- \( s \) = block strength
- \( K \) = block characteristic constant having different values for different mortar and block strength ratio.

Nylander(32) investigated strength of walls built with block and mortar other than the ones he used in his tests before. Three walls from each specimen were tested under axial and non-axial loading applied at top and bottom, in opposite and in same direction. On the basis of these test results he developed a formula in which he related wall strength with mortar and block strength as

\[ W = m + K \sqrt[3]{s^3} \cdot \sqrt[3]{s} \]

where
- \( m \) and \( s \) are mortar and block strength respectively
- \( K \) = characteristic constant of the block.

This formula is valid when the properties of blockwork such as thickness of the joint dimension of the block are constant.

In order to study physical properties of block, bond between mortar and block, strength of masonry and effects of racking and flexure loads Fishburn(33) in 1961 conducted tests on walls of hollow blocks and of composite construction. From these test results he concluded that mortar strength had
little effect on strength of blockwork and the walls with their bed joints parallel to the span were three to four times stronger than similar walls when tested with bed joints normal to the span. Failure always occurred in bond indicating that the tensile strength of mortar was greater than its bond strength to the masonry.

Copeland and Saxer \(^{(34)}\) tested a good number of block couplets in order to study the influence of various variables on the strength of masonry. Tests showed that the type of Portland cement, lime and admixtures in mortars, specimen storage, re-tempering of mortar, all these factors had little effect on the tensile bond strength. However factors such as compressive strength of mortar and its consistency, method of curing, had important effect on the tensile bond strength. Damp curing resulted in tensile bond strength of epoxy joints being higher than the tensile strength of the concrete.

Palmers and Parsons \(^{(35)}\) and other investigators observed that mortar with high compressive strength yielded higher bond strength and also proper curing of joints had a marked effect on the bond strength of the masonry.

In order to determine tensile bond between brick and mortar Keunning \(^{(36)}\) tested crossed brick couplet using tripod method as described in A.S.T.M.E 194-66. He found that due to high magnitude of bending during experiment, the couplet is more highly stressed at the edges of the mortar joint than at its centre, developing larger tensile stresses at the ends and resulting in an early failure.

Hedstrom \(^{(37)}\) studied the compressive and flexural
strength of blockwork laid in nine different patterns and compared the result obtained with that of conventional running bond pattern and concluded that various patterns are satisfactory for load-bearing purposes. He also studied the effect of block and mortar strength on the blockwork strength and like other (previous) investigators he concluded that blockwork strength depends on the block strength and is little affected by mortar strength.

In 1966, B.R.S. \cite{38} conducted axial tests on storey height block walls built with solid and hollow blocks of crushing strength 1560 p.s.i. and 4640 p.s.i. respectively. 1:1:6 cement:lime:sand mortar mix was used and walls were tested after being cured for 28 to 39 days. The failure in the wall was by tensile vertical splitting and crushing of upper portion of the test walls. The wall to block strength ratio varied from 0.52 to 0.78. Based on these test results the modulus of elasticity was related to block gross strength $\mu_b$ by the following formula

$$E = 850 \mu_b \text{ psi}$$

Erntroy and Weeks \cite{39} tested axially loaded walls constructed of 8 ins. x 8 ins. x 16 ins. hollow concrete blocks with plain and reinforced concrete infill. 1:1:3 cement:lime:sand mortar had compressive strength of 5360 p.s.i. and the blocks strength varied between 1370 p.s.i. and 2400 p.s.i. The test result showed that

a) infilling increased wall strength by about half,

b) vertical steel reinforcement increased the wall reinforcement by about 25% but did not have any effect when low strength blocks were used for wall construction,
c) horizontal steel in addition to vertical reinforcement caused a further 10% increase in wall strength.

Because of the limited amount of test data available on the compressive strength of slender concrete masonry walls, Yokel et al. tested a number of block walls under vertical load. The objective of their test was to determine and analyse the effects of wall slenderness and load eccentricity on the strength of the slender concrete walls. They tested 60 reinforced and unreinforced walls of 6 ins. and 8 ins. thickness respectively. The walls were 4 ft. wide, 10 ft., 16 ft. and 20 ft. in height. Blocks used in the walls had dimension 8 ins. x 8 ins. x 16 ins. and 6 ins. x 8 ins. x 16 ins. The mortar strength varied from 700 p.s.i. to 1768 p.s.i. and block strength for 6 ins. thick wall was 2280 p.s.i. and for 8 ins. thick wall was 2213 p.s.i.

Along with these walls they tested 2-block and 3-block high prisms in order to determine the effect of prism height on the prism strength.

Wall panels were tested in a steel frame with an adjustable top cross beam that could be raised or lowered to take care of various wall heights. Loads were applied by means of hydraulic rams attached to the cross beam.

The test set up was so designed as to prevent any rotation at the base of the wall while permitting free rotation at the top. Lateral drift was checked by tying the loading frame to the laboratory wall. Vertical strain was measured by attaching a 2 ins. diameter aluminium tube to the side of the wall. The top of the tube had
pinned connection to the wall and at the lower end it was attached to a guide which kept the tubes in line with the centre line of the wall but allowed the tube to slide downwards as the wall shortened under the load. Fig. 2.7. shows the test set up.

16 ft. and 20 ft. 6 ins. thick reinforced walls failed at 4 to 7 courses from the top. Fig. 2.7. shows the failure loads of these walls. 8 ins. thick unreinforced walls, when loaded axially, failed by tensile vertical splitting and crushing of top and bottom courses.

In the analysis of the test results they assumed a linear stress-strain relationship and the compressive strength in bending of a prism equal to compressive strength of an axially loaded prism. Based on these assumptions and load-moment interaction diagram, they predicted the strength of a slender concrete wall under compressive eccentric and combination of eccentric and transverse loads.

From their test results and analysis they concluded

(1) theoretical interaction curves for the capacity of short concrete masonry walls closely predict axial compressive load capacity and conservatively predict moment capacity,

(2) Flexural compressive strength of masonry increases with increasing strain gradients (increasing load eccentricity),

(3) slender concrete masonry wall capacity can be conservatively predicted by moment magnifier method, when short wall capacity is based on compressive strength of axially loaded prisms,

(4) the capacity of short and slender concrete masonry walls can be predicted with reasonable accuracy when the increase
in flexural compressive strength with increasing strain gradients is taken into account.

In 1972 Read and Clement\(^{(42)}\) published two reports, the first report was on the construction of and proving of a suitable test frame for concrete block walls, in which they tested 11 walls built in 1:1:3 cement:lime:sand mortar mix (having 4 ins. cube strength between 1335 to 5610 p.s.i.) in order to test the suitability of the testing frame for testing masonry walls with maximum dimensions of 8 ft. 10 ins. high and 6 ft. 6 ins. wide. Similar tests carried out at Building Research Station were repeated in this frame so as to compare the performance and reliability of the two machines. Failures in all the walls was by splitting and the tests on two groups of four identical walls showed a coefficient of variation of 6%. Read and Clements' test results were 10% and 25% higher than the Building Research Station results for low strength and high strength walls respectively. The variation in wall strength characteristics were due to the variable character of the material, workmanship and the capping. The strength ratio varied from 0.44 to 1.04, the lowest value being for the wall built in blocks with the highest crushing strength. The modulus of elasticity $E$ ranged from $1.12 \times 10^6$ to $2.33 \times 10^6$ p.s.i. Block strength varied from 1360 p.s.i. to 3755 p.s.i. The lowest value of $E$ was obtained for walls built in lowest strength cellular blocks.

In the second\(^{(43)}\) phase of their investigation they tested 38 walls and 2000 control specimens. Factors such as strength ratio for the wall and the masonry couplets, the
slenderness effect using three different slenderness ratio (by changing the wall thickness instead of height of the wall), stiffness of units and walls and their failure characteristics were studied. Suitability of two block high couplets over one block as control specimen was also investigated. Walls measuring 8 ft. 6 ins. high and 5 ft. 11 ins. wide were built in 1:1:3 mortar mix having high crushing strength. The strength of mortar was assessed by crushing 4 ins. cubes. Solid and hollow concrete blocks of strength 855 p.s.i. to 4365 p.s.i. were used in the wall. The walls and other specimen were built and tested in accordance with B.S. 2028-1968 and C.P.111 Part II - 1970.

For the solid block wall the mortar mix was much higher than the block strength. Since single block couplet was easier to prepare in comparison to the two block couplet they concluded that it is advantageous to use single block couplet as control specimen. The single block has a ratio of height to thickness which is sufficient to prevent the load at failure being very much affected by platen restraint which would be evident on smaller units.

They also suggested large increase in basic permissible stress can be obtained if the designer has sample wall panels tested in axial compression.

Slenderness effects were investigated by altering the thickness of the wall while keeping other dimensions constant. The result showed that there is an increase in strength of wall with the increase in slenderness ratio, which contradicts the test results of several previous investigators and indicates that slenderness ratio is not a critical variable.
By changing the thickness of the block there is corresponding change in the shape factor of the block. The effect of shape factor on the strength of wall has not been studied in detail so far. Read and Clement suggest further study to be carried out on this aspect.

The stiffness of the wall was determined by using an extensometer at each corner to obtain average value of $E$ over a large gauge length and by taking more localised readings across and between mortar joints using 4 ins. gauge length Demec Gauge. Based on these studies they suggested that more research should be carried out to study the measurement of strain in and around mortar joint.

Rostampour studied design aspects of multi-storey buildings in lightweight concrete blocks. He investigated (i) ultimate strength of wall panels under axial compression, (ii) the ultimate shear strength of single storey shear wall structures with openings, and (iii) behaviour of a five-storey shear wall structure subjected to lateral loading. In order to ascertain the suitability of model blocks, he tested a number of full scale and one-third scale storey height panels built with four varieties of lightweight aggregate concrete blocks covering a wide strength range. His test results showed that it is possible to reproduce, with reasonable accuracy, the strength of full scale blockwork by means of model tests. Other relationship and properties such as stress-strain, modulus of elasticity, Poisson's ratio, tensile strength, ultimate load and mechanism of failure were also investigated by him.

He also carried out several racking tests on one-third
scale single storey coupled shear walls connected through slabs. Deflections, strains, ultimate shear strength and failure mechanism under a set range of precompression were studied. The test result showed that blockwork shear walls depending on the amount of applied precompression, exhibit two distinct types of failure, (i) shear failure, a combination of bond and frictional resistance due to precompression, (ii) tensile failure, governed by the maximum tensile strength of blockwork.

He further analysed the structure by finite element method and equivalent frame analogy method and found that both methods give close estimate of stresses and deflection at high precompression.

2.3. SUMMARY

In this section the work of a number of investigators dealing with various aspects of the problem of strength of walls has been considered, such as (1) tensile and compressive strength of masonry units, (2) compressive strength of mortar, (3) eccentricity of loading, (4) bond type, (5) relationship between block and couplet strength and masonry strength. However it is found that not much attention has been paid by any of them towards slenderness effect on the compressive strength of walls. Such test evidence as is available is conflicting. This provides further justification of the objective of the investigation as defined in Chapter 1, (to study this aspect in detail).
3.1 INTRODUCTION

The object of work presented in this chapter was to test one third scale.

1) 2 ins. thick (6 ins. equivalent) lightweight concrete block walls (AgLite) corresponding to CW series.
2) 1.5 ins. thick (4\(\frac{1}{2}\) ins. equivalent) single leaf brick walls corresponding to WM series.
3) 3 ins. thick (9 ins. equivalent) bonded brick wall corresponding to BW series.

These walls were tested axially in the Avery Universal Testing Machine under flat ended conditions.

The model test results reported here can be applied on full scale structures, because for the blockwork Hendry and Rostampour (26) have shown, that the strength of full scale blockwork for a given strength of block and mortar may be reproduced by means of model tests, provided mortar joints are scaled down, sand used for the models to be sieved and coarse fraction discarded.

Similarly for the brickwork Murthy and Hendry (18) have established that the strength of brickwork for a given strength of brick and mortar can be reproduced by means of model tests if one inch mortar cubes are used for the determination of mortar strength.

3.2 TEST PROGRAMME

The test programme consisted of testing walls of different height corresponding to different slenderness ratio. Fig. 3.1. and 3.2. gives detail of the walls and testing programme.
Fig. 3.1
TEST PROGRAMME
OF
BLOCK WALLS

<table>
<thead>
<tr>
<th>WALL NO.</th>
<th>CW 6</th>
<th>CW 12</th>
<th>CW 17</th>
<th>CW 24</th>
<th>CW 29</th>
</tr>
</thead>
<tbody>
<tr>
<td>THICKNESS</td>
<td>A</td>
<td>1.89</td>
<td>2.90</td>
<td>2.90</td>
<td>3.90</td>
</tr>
<tr>
<td>WIDTH B</td>
<td>18.5&quot;</td>
<td>19&quot;</td>
<td>19&quot;</td>
<td>22&quot;</td>
<td>26.7&quot;</td>
</tr>
<tr>
<td>SLENDERNESS RATIO</td>
<td>5.4</td>
<td>10.8</td>
<td>15.5</td>
<td>21.5</td>
<td>26.7</td>
</tr>
<tr>
<td>WALL TYPE</td>
<td>SINGLE LEAF STRETCHER BOND</td>
<td>CW SERIES</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>NO. OF WALLS</td>
<td>5</td>
<td>5</td>
<td>4</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>LOADING</td>
<td>AXIAL FLAT ENDED</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Fig. 3.2
TEST PROGRAMME
OF
BRICK WALLS

<table>
<thead>
<tr>
<th>WALL NO.</th>
<th>WM 6</th>
<th>WM 12</th>
<th>WM 18</th>
<th>WM 25</th>
</tr>
</thead>
<tbody>
<tr>
<td>THICKNESS</td>
<td>A</td>
<td>1.375&quot;</td>
<td>1.44&quot;</td>
<td>1.44&quot;</td>
</tr>
<tr>
<td>WIDTH B</td>
<td>18.5&quot;</td>
<td>19&quot;</td>
<td>19&quot;</td>
<td>19&quot;</td>
</tr>
<tr>
<td>SLENDERNESS RATIO</td>
<td>5.4</td>
<td>10.8</td>
<td>15.2</td>
<td>22.5</td>
</tr>
<tr>
<td>WALL TYPE</td>
<td>SINGLE LEAF STRETCHER BOND</td>
<td>WM SERIES</td>
<td></td>
<td></td>
</tr>
<tr>
<td>NO. OF WALLS</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>LOADING</td>
<td>AXIAL FLAT ENDED</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Fig 3.5 UNIT AND MORTAR STRESSES DUE TO APPLIED AXIAL COMpressive LOAD. (a) REFER PAGE 44.
3.3 MATERIALS

3.3.1 Blocks

Blocks of size 6 ins. x 3 ins. x 2 ins. were cut from the full size light weight blocks of size 18 ins. x 8 ins. x 3 ins. They were cut in a "Clipper" machine using abrasive blades and wet cutting process. The compressive strengths of these blocks were tested in accordance with B.S. 2028-1364-1968. Table 3.1 gives a summary of their properties.

3.3.2 Bricks

One third scale bricks were used. Bricks came in batches so their strength varied. They were tested in accordance with B.S. 3921-1969 (Part 2). Table 3.2 gives a summary of their properties.

3.3.3 Lime

Hydrated lime, Class A was used to conform with B.S. 890.

3.3.4 Cement

Rapid hardening Portland cement (Ferrocrete) was used for all mortars to give early mortar strength.

3.3.5 Sand

3.3.5.1 Block walls

For block walls ordinary building sand sieved to remove coarse particles was used. The grading is shown in Table 3.3.

3.3.5.2 Brick walls

For brick walls dry Leighton Buzzard 25/52 was used. The grading curve is shown in Fig. 3.3.
### TABLE 3.1.
**BLOCK PROPERTIES**

<table>
<thead>
<tr>
<th>Properties</th>
<th>Range</th>
<th>Mean</th>
<th>Standard Deviation</th>
<th>Coeff. Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length (ins.)</td>
<td>5.7 - 5.9</td>
<td>5.84</td>
<td>0.068</td>
<td>1.1%</td>
</tr>
<tr>
<td>Width (ins.)</td>
<td>1.89 - 2.09</td>
<td>1.976</td>
<td>0.1</td>
<td>5.0%</td>
</tr>
<tr>
<td>Height (ins.)</td>
<td>2.93 - 2.86</td>
<td>2.886</td>
<td>0.029</td>
<td>1.0%</td>
</tr>
<tr>
<td>Compressive Strength p.s.i.</td>
<td>1600 - 2281</td>
<td>1887.6</td>
<td>226</td>
<td>12.0%</td>
</tr>
</tbody>
</table>

**NOTE:** Average Block Density = 83.0 lb/cu.ft.

---

### TABLE 3.2.
**BRICK PROPERTIES**

<table>
<thead>
<tr>
<th>Properties</th>
<th>Range</th>
<th>Mean</th>
<th>Standard Deviation</th>
<th>Coeff. Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length (ins.)</td>
<td>2.94 - 3.09</td>
<td>3.005</td>
<td>0.036</td>
<td>1.2%</td>
</tr>
<tr>
<td>Width (ins.)</td>
<td>1.375 - 1.44</td>
<td>1.42</td>
<td>0.022</td>
<td>1.5%</td>
</tr>
<tr>
<td>Height (ins.)</td>
<td>0.91 - 0.97</td>
<td>0.94</td>
<td>0.015</td>
<td>1.6%</td>
</tr>
<tr>
<td>Compressive Strength p.s.i.</td>
<td>3277.14 - 4938.6</td>
<td>4359.4</td>
<td>445</td>
<td>10.1%</td>
</tr>
<tr>
<td>Water Absorption (%)</td>
<td>12.9 - 16.12</td>
<td>13.85</td>
<td>0.86</td>
<td>6.2%</td>
</tr>
</tbody>
</table>

**NOTE:** Average Brick Density = 138.6 lb/cu.ft.

---

### TABLE 3.3.
**SIEVE ANALYSIS OF ORDINARY SAND USED IN CONSTRUCTION OF ONE THIRD SCALE BLOCK WALLS (BS 1200-1955)**

<table>
<thead>
<tr>
<th>BS Sieve No.</th>
<th>% Passing by weight</th>
<th>% retained</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/8</td>
<td>98.5</td>
<td>1.5</td>
</tr>
<tr>
<td>7</td>
<td>98.0</td>
<td>0.5</td>
</tr>
<tr>
<td>14</td>
<td>96.7</td>
<td>1.3</td>
</tr>
<tr>
<td>25</td>
<td>92.9</td>
<td>3.8</td>
</tr>
<tr>
<td>52</td>
<td>63.4</td>
<td>9.5</td>
</tr>
<tr>
<td>100</td>
<td>17.3</td>
<td>46.1</td>
</tr>
</tbody>
</table>

Fineness Modulus = 0.627.
Fig. 3.1 LEIGHTON BUZZARD SAND GRADING CURVE

Fig. 3.4. STRAIN MEASURING ARRANGEMENT
3.3.6 Mortar

For block walls 1:1:3 cement:lime:sand mix was used. Water-cement ratio varied from 1.01 to 1.1. For brick walls 1:3 cement:sand mortar mix was used. Water-cement ratio varied from 0.8 to 0.95.

Each batch of mortar was mixed by hand and proportions were made up by weight. With each wall six one-inch cubes were made by hand compaction in the mould. After 24 hours the cubes were removed from the mould and stored in water for seven days after which they were cured at laboratory temperature before being tested in Instron machine.

3.4. EXPERIMENTAL PROCEDURE

3.4.1 Construction of Walls

3.4.1.1 Block walls

Walls CW6, CW12, CW17, CW24 and CW29 were built in wooden jigs of required size. The thickness of the mortar bed was scaled down to \( \frac{1}{2} \) in. thick. For this purpose guide lines were drawn on the wooden backing of the jigs to mark each course of blockwork. The jigs were then fixed plumb on to steel base channel of size 2 ins. x 4 ins. Mortar for the walls was prepared in small quantity, enough to be used for half an hour. Walls were cured under polythene cover for a minimum of seven days before testing.

3.4.1.2 Brick walls

Nominal 1.5 in. thick (4\( \frac{1}{2} \) ins. equivalent) walls of WM series in stretcher bond were built in a similar manner as the block walls.

Nominal 3 ins. thick (9 ins. equivalent) walls of B1W series were built in English Bond without jigs up to 36 ins.
high (BlW12) as the mason thought he could build to plumb without jigs, which ultimately did not turn out to be so.
54 ins. high walls (BlW18) were built in a jig on a steel plate of size 60 ins. x 20 ins. x \( \frac{3}{4} \) in. Plate 3.1 shows a wall being built in wooden jig.

3.4.2 Testing Method and Measurements

The walls built were taken to Avery Universal Testing Machine either by crane (Plate 3.2) or manually on trolley, depending upon the height of the wall, and were placed so that the centreline of the loading platen was in line with the centreline of the wall. Prior to each test, 2 ins. x 2 ins. x 20 ins. steel beam for block walls (CW series), 1.5 ins. x 2 ins. x 20 ins. steel beam for single leaf brick wall (WM series) and 3 ins. x 5 ins. x 20 ins. steel beam for bonded walls (BlW series) were bedded with 1:1 cement:sand mortar on top of the wall and then levelled in two perpendicular directions by gradually applying the load through the machine platen in order to have even distribution of load over the section of the wall. Walls CW6 and CW12 were not capped in Avery Machine, but were capped and levelled before being placed in the machine by means of hammer and spirit level. The top of the wall was cured for 24 hours under polythene cover before being tested. Plate 3.3. shows wall ready for test.

3.4.2.1 Application of load

The load was applied by means of Avery Universal Machine. For the block walls and single leaf walls the load was initially applied at one ton intervals up to ten tons, and then at two ton intervals up to failure. For
PLATE 3.1. CONSTRUCTION OF WALL.

PLATE 3.2. WALL BEING TRANSPORTED TO M/C.

PLATE 3.3. WALL READY FOR TEST. — WM.
the bonded walls the load was applied at two ton intervals up to about sixteen tons, and then at four ton intervals up to failure. A ¼ in. thick plywood sheet was placed between the top platen of machine and spreader beam in order to take care of any gap left between them.

3.4.2.2. Strain measurement

3.4.2.2.1. Block walls

Vertical strain was measured by fixing Demec studs on each face of the walls CW6, CW12 and CW17, (Fig. 3.4a), and reading the value by means of Demec Gauge of 6 ins, 8 ins, 12 ins and 24 ins. gauge length depending upon the height of the walls. The average of four readings gave the final strain value for each load. For walls CW6 the horizontal strain was measured by the same method as the vertical strain.

For CW24 and CW29 walls, two compressometers of 33 ins. and 48 ins. average gauge length respectively were fixed on each face of the wall and their reading recorded under different loads (Fig. 3.4b). The average of the four compressometer readings gave the value of strain for each load interval.

3.4.2.2.2. Brick walls

Strain was measured in a similar way as for block walls except that in the case of bonded walls of BlW series compressometers of 46 ins. average gauge length were used for 54 ins. high walls (BlW18).

3.4.2.3. Lateral deflection

This was measured by fixing dial gauge of 0.0001 inch sensitivity. One dial gauge was fixed at the topmost
course, other at the mid-height of the wall and the third at 3 ins. above the base of the plate. This arrangement was adopted for walls up to a height of (i) 34½ ins. for block walls, (ii) 27 ins. for single leaf brick walls, and (iii) 36 ins. for the bonded wall. For higher walls these gauges were fixed at a distance of 12 ins. c/c along the height of the wall. Plate 3.3.

3.4.2.4. End rotation

End rotations for some brick walls were measured either by using electrolevel or by using brackets and dial gauges. In the latter case holes were drilled in the topmost and the lowermost courses of the walls. Through these holes brackets were attached by means of nut and threaded rods. Dial gauges of 0.0001 inch sensitivity were then placed at the ends of these brackets. Fig. 3.5. shows the detail of this arrangement.

3.5. RESULTS

A summary of the results of the wall tests and a comparison of reduction factors given in C.P.111-1970 (Amendment Slip No.1) is given in Tables 3.4. and 3.5.

3.6. DISCUSSION

3.6.1. Modes of Failure

3.6.1.1. Block walls

First cracking sound generally occurred at 80-90% of the failure load. General mode of failure was by vertical tensile splitting, crushing, spalling and shear.

In CW6 walls failure was mainly by vertical splitting and crushing of different courses. In some walls diagonal shear cracks were observed. This was due to restraining
## RESULTS FOR BLOCK WALLS

| Wall No. | S.R. | Age of | Mortar | Age of | Ultimate | Ultimate | Average | Reduct- | R.F. E at | Average | Eccen- | Remarks |
|----------|------|--------|--------|--------|----------|----------|---------|--------|---------|---------|--------|--------|---------|
|          |      |        |        |        | Wall load| Stress    | Stress  |ion      | Factor  | E at 4th| Eccentricity |        |         |
| CW6-1    | 5.4  | 35     | 2113.5 | 41     | 25.7     | 1617.1   | 0.93    | 43/69   |         |         |         |        |         |
| CW6-2    | 5.4  | 35     | 2520.45| 45     | 26.0     | 1641.6   | 0.94    | 10/0    |         |         |         |        |         |
| CW6-3    | 5.4  | 63     | 1661.45| 56     | 23.2     | 1443.0   | 1459.1  | 1.0     | 0.93x10^6 | 44/45  |         |         |         |
| CW6-4    | 5.4  | 59     | 1768.6 | 52     | 21.0     | 1306.2   | 0.96    | 27/0    |         |         |         |        |         |
| CW6-5    | 5.4  | 59     | 1511.45| 55     | 20.7     | 1287.54  | 0.87    | 13/63   |         |         |         |        |         |
| CW12-1   | 10.8 | 62     | 2778.6 | 60     | 21.0     | 1321.0   | 0.91    | 11/36   |         |         |         |        |         |
| CW12-2   | 10.8 | 65     | 3919.74| 62     | 21.5     | 1352.8   | 1.08    | 15/2    |         |         |         |        |         |
| CW12-3   | 10.8 | 36     | 3114.54| 69     | 21.9     | 1362.6   | 1276.4  | 0.87    | 1.05x10^6 | 54/42  |         |         |         |
| CW12-4   | 10.8 | 34     | 2798.4 | 25     | 19.4     | 1206.7   | 1.12    | 19/0    |         |         |         |        |         |
| CW12-5   | 10.8 | 64     | 3561.14| 61     | 21.0     | 1154.9   | 0.84    | 12/50   |         |         |         |        |         |
| CW17-1   | 15.5 | 17     | 3463.15| 20     | 17.56    | 1092.23  | 0.78    | 57/0    |         |         |         |        |         |
| CW17-2   | 15.5 | 17     | 3529.7 | 15     | 19.2     | 1194.5   | 0.87    | 9x10^6  | 17/87   |         |         |         |        |         |
| CW17-3   | 15.5 | 30     | 3274.3 | 33     | 17.9     | 1113.2   | 1112.8  | 0.76    | 0.739   | 1.00    |         |         |        |         |
| CW17-4   | 15.5 | 14     | 3190.6 | 12     | 16.9     | 1051.18  | 0.92    | 10/17   |         |         |         |        |         |
| CW24-1   | 21.6 | 73     | 3009.0 | 73     | 18.8     | 1169.8   | 0.8     | 10/17   |         |         |         |        |         |
| CW24-2   | 21.6 | 67     | 3072.8 | 58     | 16.5     | 1026.3   | 1065.6  | 0.73    | 0.57    | 0.97    | 0.84x10^6 | 15/13  |         |
| CW24-3   | 21.6 | 63     | 3022.9 | 61     | 16.1     | 1001.42  | 0.76    | 27/0    |         |         |         |        |         |
| CW29-1   | 26.7 | 34     | 2850   | 21     | 17.15    | 1067.07  | 0.904   | 15/14   | 46/46   |         |         |         |        |         |
| CW29-2   | 26.7 | 30     | 2442.7 | 20     | 19.10    | 1188.02  | 1129.2  | 0.77    | 0.443   | 1.05x10^6 | 25/15  |         |         |
| CW29-3   | 26.7 | 29     | 3234.7 | 23     | 18.2     | 1132.4   | 1.08    | 15/15   |         |         |         |        |         |
## TABLE 3.5
TEST RESULTS FOR BRICK WALLS

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<tr>
<th>Designation</th>
<th>Age Days</th>
<th>Ultimate Load in tons</th>
<th>Ultimate Stress</th>
<th>Average Stress</th>
<th>Reduction Factor</th>
<th>Reduction Factor based on C.P.I. 1970</th>
<th>Modulus of Elasticity</th>
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* Average of three strains

Remarks:
- Strength of brick 20% less than previous bricks and so adjusted.
action of the machine platen as well as due to high strength of mortar in comparison with block strength. Wall CW12 failed mostly by vertical splitting and spalling. There were vertical and wedge shape cracks in the thickness of the wall. Wall CW17 failed mainly by development of tensile crack in the 2nd, 4th, 6th and 10th course from the top. These cracks widened with the increase in load. Except for wall CW17-2 no crushing and spalling was observed during the failure process. In wall CW24-1,2,3 failure was initiated by the development of tensile cracks followed by crushing of blocks either in the top three courses or bottom three courses. Wall CW24-2 buckled just before failure. In wall CW29 failure was initiated by spalling and crushing of the top five-six courses accompanied by vertical tensile splitting. All the walls except CW29-2 had a few vertical tensile splitting cracks. Wall CW29-1 had wedge shape cracks in the thickness of the wall. Plate (3.4) to Plate (3.5) show various modes of failure of the walls described above.

As is well known the various modes of failure described are mainly due to the presence of triaxial state of stress in the block and mortar. The vertical stresses occur due to application of load and the lateral stresses due to different strain values of the two materials. If the strength of the block is higher than the strength of mortar, the mortar will try to expand more than the block, thus expansion of mortar will be resisted by block which will induce tensile stress in the block, and compressive stress in mortar. Tensile splitting failure will occur when induced
PLATE 3.4(a) SPALLING & WEDGE SHAPE CRACK CW 29.

PLATE 3.4(b) SHEAR FAILURE OF CW 6.

PLATE 3.5. CRUSHING, SPALLING & TENSILE SPLITTING CW 29.
stresses exceed the tensile stress of the block. On the other hand if the block strength is less than the mortar strength, the state of stress in block and mortar joints are reversed and instead of mortar the block is in a state of triaxial compression. Once these induced stresses exceed the block strength, shear failure across an inclined plane in the block occurs. Fig.3.6.

Bi-lateral stresses which act along the interface of a strong brick and weak mortar are concentrated sharply at the edges of the interfaces producing high lateral tensile stresses in the brick units in these regions. When these stresses exceed the strength of brick, tensile splitting in the brick along the edges in the direction of the length of brick occur, resulting in spalling failure.

3.6.1.2. Brick walls

3.6.1.2.1. Single leaf walls of WM series

The first hair line crack in these walls appeared at 60-80% of the ultimate load, which enlarged with further increase in load. General mode of failure of the walls was tensile vertical splitting accompanied by crushing and spalling of various courses of brick. Plate 3.6 to 3.9 shows mode of failure of various single leaf walls (WM). Wall WM6-1 failed by local crushing of the top course accompanied by spalling of brick in the 4th course on both faces of the wall. There was also a vertical crack in the thickness of the wall beginning in the 2nd course from the top and extending down to the third course from the bottom. WM6-2 failed by local crushing in the middle 1st course, spalling in the 2nd and 3rd course accompanied by vertical
PLATE 3.6. BENDING OF WALL AT 75% OF ULTIMATE LOAD - WM25.

PLATE 3.7. FAILURE OF WM25.
crack beginning in the 6th course from the top and extending up to the bottommost course. There were also cracks in the thickness of the wall. WM6-3 failed by spalling in the 2nd and 3rd course, vertical splitting in the 6th and 7th course in the face of the wall, and also in the topmost course in the thickness of the wall. WM12-1 failed with the development of cracks in 2nd, 4th, 6th and 7th courses spalling in first to 5th courses on one side of the wall, also spalling of 2nd and 3rd courses on the other side of the wall and crushing of topmost course. In WM12-2 failure was by spalling of the 13th course from the top, vertical splitting in the centre of the wall on both faces, also in the thickness of the wall on both the ends, and finally crushing and spalling of three courses. Wall WM12-3 failed in a way similar to WM12-2, except that there were no vertical cracks in the thickness of wall. Wall WM18-1, WM18-2 and WM18-3 failed with crushing and spalling of top courses accompanied by vertical splitting on both faces of the wall. WM18-2 had vertical splitting in the thickness of the wall as well. In walls of WM25 group no crack was observed in the thickness of the walls. These walls failed by appearance of vertical hair line crack which started to develop in the centre and in the ends of the wall. These cracks generally started in the 3rd or 4th course and extended throughout the wall's height. After development of these cracks, the wall started to buckle until it failed. Turnsek and Cacovic (21) suggest that because of widening and lengthening of these vertical cracks the wall is divided into separate columns, which in turn buckle with the increase in load.
3.6.1.2.2. Bonded walls of BlW series

In this series the first crack appeared at 70-80% of the ultimate load, and failure was sudden accompanied by explosive scatter of brickwork pieces as far as 10 to 16 feet from the machine. Only a few walls failed with comparatively less explosion and could be photographed. The general mode of failure was by appearance of vertical cracks either in the sides or in the thickness of the wall or in both, and crushing of the top few courses. Plate (3.8) to (3.9) show failure of various bonded walls (BlW).

A notable feature was failure of vertical joints starting from mid-height and mid-course, and extending up to the bottommost course, in a stepped down form. This is similar to shear failure pattern, and could be explained due to lack of good bond between the brick and mortar joints which results in slipping of brick in an outward direction due to the presence of tensile stress, and thereby starting this process of failure (Plate 3.9). From the modes of failure described above for both series of wall, it is clear that the failure in all cases is strength failure (except for walls WM25) rather than stability failure as was expected, because none of the walls had very high slenderness ratio so as to induce instability.

3.6.2. Strain readings

Strain readings on both faces of the block and brick walls were not abnormally different from each other, implying fairly uniform distribution of load on the walls. The differences in strain readings were higher in the initial stage of loading, but this reduced with the increase in load.
PLATE 3.9. TENSILE SPLITTING & BOND FAILURE - B1W12.

PLATE 3.8. CRUSHING & TENSILE SPLITTING - B1W12.
PLATE 3.7(a). TENSILE SPLITTING—WM18

PLATE 4.2(b). W/S WALL READY FOR TEST (REFER P.59)

PLATE 3.10. TENSILE SPLITTING—B1W18
This could be due to unevenness of mortar bed which resulted in reduced contact surface between brick and mortar in the unloaded wall, and when the load was initially applied there was concentration of stresses on this small contact surface which became plastic and got crushed, and bricks made contact over a greater mortar area.

Eccentricity was calculated from strain readings. Details of eccentricity calculation are given in the Appendix. This eccentricity could also be due to lack of plumbness of the wall. Figs. (3.7) to (3.11) show stress-strain curves for block walls, Figs. (3.12) to (3.15) for WM walls, and Figs. (3.16) to (3.19) for BlW walls.

From the literature review it is seen that although ultimate strength of walls and modulus of elasticity have been reported by various investigators, comparatively few stress-strain curves of the walls under different stage of loading have been reported. It will be seen in Chapter 6 of Theoretical Analysis, how significant the effect of shape of stress-strain curve is on the bearing strength of the masonry walls. From the figures of the stress-strain curve reported in this thesis, it is seen that for walls of similar dimensions and similar type of loading the stress-strain relationship varies.

3.6.3. **Modulus of Elasticity**

3.6.3.1. **Block walls**

Stress-strain curves for finding E values were replotted on the basis of an average of four strain readings. Modulus of elasticity determined for various walls is given in Table 3.4. As has been shown by earlier investigators, the
Fig. 3.7(a)
COMPRESSIVE STRESS
Vs.
VERTICAL STRAIN

- Symbols represent different walls:
  - CW6-1
  - CW6-2
  - CW6-3
  - CW6-4
  - CW6-5

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Fig. 3.7(b)
LATERAL STRAIN
Vs.
VERTICAL STRAIN

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<td>CW6-2</td>
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Fig. 3.8
COMpressive STRESS
VS.
VERTICAL STRAIN

Fig. 3.9
COMpressive STRESS
VS.
VERTICAL STRAIN
Fig. 3.12.
COMPRESSIVE STRESS
Vs.
VERTICAL STRAIN

VERTICAL STRAIN IN/IN x 10
0 200 400 500 1000 1400 1800 2200 2500 3000

SYMBOL WALL NO.
○ WM6-1
○ WM6-2
△ WM6-3
Fig. 3.13.
COMPRESSIVE STRESS Vs VERTICAL STRAIN

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Fig. 3.14
COMPRRESSIVE STRESS
VS
VERTICAL STRAIN

Fig. 3.15
COMPRRESSIVE STRESS
VS
VERTICAL STRAIN
Fig. 3.16 (a)

COMPRESSIVE STRESS Vs. VERTICAL STRAIN

SYMBOL | WALL NO.
-------|--------
•        | B1W6-1
•        | B1W6-2
Fig. 3.16 (b) - COMPRESSIONAL STRESS Vs. VERTICAL STRAIN

SYMBOL WALL NO.

Δ B1W6-3

VERTICAL STRAIN IN/IN x 10^4
Fig 3.17
COMPRRESSIVE STRESS
VS VERTICAL STRAIN

SYMBOL WALL NO.
□ B_W-1
○ B_W-2
△ B_W-3
● B_W-4

VERTICAL STRAIN IN/IN x 10'^4
0 200 400 600 800 1000 1200 1400
0 200 400 600 800 1000 1200 1400

Fig 3.18
COMPRRESSIVE STRESS
VS VERTICAL STRAIN

SYMBOL WALL NO.
○ B_W12-1
● B_W12-2
△ B_W12-3

VERTICAL STRAIN IN/IN x 10'^4
0 200 400 600 800 1000 1200 1400
0 200 400 600 800 1000 1200 1400
E value is found to decrease with the increase in load. It fluctuated during small range of loading, but afterwards at about 25% to 30% of the ultimate load it started decreasing with the increase in load (Fig. 3.20).

3.6.3.2. Brick walls

Stress-strain curves were plotted in a similar way as for block walls, except that for wall BlW18-3, in which an average of only three strain readings was taken as one of the compressometers did not work during the experiment. Table 3.5. gives the value of E for all the walls.

In WM series, E value fluctuated during the small range of loading, but afterwards at about 25-40% of ultimate load it started decreasing with the increase in load (Fig. 3.21).

In bonded walls of BlW series, this phenomenon of fluctuation in E value was not as dominant as in WM series. After reaching about 16-20% of ultimate load it started decreasing with increase in load, but not in the same proportions as in WM series (Fig. 3.22).

3.6.4. Deflection

Lateral deflection measurement for block walls and brick walls showed that none of the walls deflected to the same extent, therefore no definite pattern of behaviour was obtained. Some of the walls deflected more at the top end as compared to deflection at mid-height. Some walls had deflection curves similar to a sine-curve. Fig. 3.23 to Fig. 3.26 shows deflection curves at various stresses for CW walls. Fig. 3.27 to Fig. 3.30 for WM walls. Fig. 3.31 to Fig. 3.34 for BlW walls. An attempt was made
Fig. 3.21 MODULUS OF ELASTICITY Vs. COMPRESSIVE STRESS (SINGLE LEAF WM WALLS)

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Fig. 3.22 MODULUS OF ELASTICITY Vs. COMPRESSIVE STRESS (BONDED B.W WALLS)

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<td>B1 W6-3</td>
<td>B1 W6-2</td>
</tr>
<tr>
<td>B1 W12-1</td>
<td>B1 W12-2</td>
</tr>
<tr>
<td>B1 W12-3</td>
<td>B1 W12-3</td>
</tr>
</tbody>
</table>

Graphs showing the relationship between modulus of elasticity and compressive stress for both single leaf WM walls and bonded B.W walls.
DEFLECTION IN INCHES

LATERAL DEFLECTION MEASUREMENTS (CW WALLS)

![Graphs showing lateral deflection measurements for CW walls.](image-url)
Fig. 3.28 LATERAL DEFLECTION MEASUREMENTS

Fig. 3.29 LATERAL DEFLECTION MEASUREMENTS

Fig. 3.30 LATERAL DEFLECTION MEASUREMENTS
DEFLECTION IN INCHES x 10^2

Fig. 3.31. LATERAL DEFLECTION MEASUREMENTS.

Fig. 3.32. LATERAL DEFLECTION MEASUREMENTS.
Fig. 3.33. LATERAL DEFLECTION MEASUREMENTS
to stop this lateral movement at the top of the wall by restraining the spreader beam movement but this arrangement did not work out successfully. This irregularity in deflection may be due to (i) variation in workmanship, (ii) variation in plumbness of the wall, and (iii) variation in properties of brick and mortar. Maximum deflections in the wall were insignificant to influence failure of walls except for WM25 walls.

These deflection curves suggest that the effective height is 0.9H (H is actual height), rather than 0.75H as specified in C.P.111-1970. This seems to be justified, bearing in mind that for complete fixity the height coefficient is 0.5, and for the walls in actual buildings which are loaded between floor slab, the height coefficient is 0.75. In flat ended walls loaded between machine platen the fixity will be less than that provided by R.C. slab and so the value of 0.9 seems to be justified. This effective height in general for flat ended walls loaded between machine platens will depend upon the stiffness of that particular machine. Therefore the value of 0.9H, as in this case, may not be the same for walls tested in some other machine. The subject of study of effective height is beyond the scope of this work. A detailed study based on test results is necessary in order to arrive at some definite conclusion. However an attempt to locate point of inflection is described in Chapter 4.

3.6.5. End Rotation

The load-end rotation relationship for the different walls, as shown in Fig. 3.35 to 3.36, show considerable variation. This is because of (i) different bond between
Fig. 3.35 COMPRESSION STRESS Vs. END ROTATION (RADIANS)
mortar joints and bricks, 2) presence of gaps in the mortar, 3) unevenness in the mortar bed, 4) uneven distribution of load, and 5) presence of micro-cracks in the brick. Due to the above-mentioned factors, rotation of masonry in a particular course may not necessarily be the same at various points on the same course length. This trend has been observed while measuring the rotations. Also walls with similar dimensions and similar loading conditions show different end rotations. The measuring gauges have sensitivity of 0.0001 inches and so slight variation in rotation is monitored by these gauges. There are no test results available with which authors values could be compared. The rotation curves in some cases show higher rotation at the top end in comparison to the bottom end. This is expected because of the difference in end fixity provided by the machine platens. Based on extensive tests a representative value of end rotation can be approximately estimated which can then be used in finding the fixing moments in order to calculate the effective height. Similar load-end rotation relationship has been observed for the walls loaded between R.C. slab as described in Chapter 4.

3.6.6. Slenderness Ratio

3.6.6.1. Block walls

As was expected, walls of the CW6 group had maximum failure stress. The stress reduction factor for this wall is taken, as one wall of CW12, CW17, CW24 and CW29 had lower ultimate stress than CW6. There was not much difference between the average failure stress of walls CW17 and CW24. This was unexpected. Wall CW29 had an average failure stress little
higher than CW17 and CW24, which was again unexpected. This is because walls CW24 and CW29 were built in wooden jigs. This resulted in much more plumb walls than walls of CW6, CW12 and CW17 series. Similar test results were obtained for one third scale bonded brick walls of BlW series. Both types of wall were built by the same mason. Fig. 3.37 shows the wall strength.

3.6.6.1.1. Comparison of authors tests with other tests.

As mentioned in the beginning, very few tests have been carried out on blockwork to study this effect. Therefore test results available for brick walls are compared in addition to the few available test results of block walls (Fig. 3.39). The reduction factors of the American test results\(^{(22)}\) for 4 ins. thick full scale brick walls are lower than the reduction factors of CW walls. Reduction factor for 4\(\frac{1}{2}\) ins. thick one-sixth scale brick wall test by Hendry et al.,\(^{(17)}\) are the same up to slenderness ratio of 12.0. Above this value the CW walls have lower values of reduction factor. The results of 4\(\frac{1}{2}\) ins. thick full scale brick wall tests by Thomas\(^{(3)}\) show almost no decrease in strength reduction factor with increase in slenderness ratio. Results of CW walls are on the lower side when compared with Thomas' results.

Yokel et al.,\(^{(40)}\) conducted tests on full scale two core concrete hollow block walls 6 ins. thick reinforced, and 8 ins. thick unreinforced, to study the effect of slenderness. The lowest wall tested corresponded to slenderness ratio of 20.0 for 6 ins. reinforced wall, and 15.0 for 8 ins. unreinforced wall. It is not possible to
Fig. 3.37
ULTIMATE STRESS
VS.
SLENDERNESS RATIO
(GOUL WALLS)

Fig. 3.38
ULTIMATE STRESS
VS.
SLENDERNESS RATIO
( BRICK WALLS

15" (4.12' Equivalent)
THICK WALLS OF WM SERIES

3" ( 1.0' Equivalent) THICK WALLS OF WM SERIES

Fig. 3.39
REDUCTION FACTOR
VS.
SLENDERNESS RATIO
[Comparison of Test Results]
compare his test results with CW walls because of the difficulty involved in calculating reduction factor due to the difference in taking slenderness ratio of the smallest wall, and also due to the difference in end condition. The 6 ins. thick reinforced wall showed consistent decrease in strength with the increase in slenderness ratio. For 8 ins. thick unreinforced wall the strength of wall of slenderness ratio 24.0 was found to be higher than that of wall of slenderness 15. This decrease is due to high strength of mortar used, and also probably due to better workmanship. As expected wall of slenderness ratio 29.0 had lower strength than the strength of the other two walls.

Read and Clements\(^{(43)}\) tested 8.5 ft. and 6 ft. wide walls built of hollow, solid and cellular blocks of different compressive strength. They varied the slenderness ratio by changing the thickness of the wall, and found that reduction factor increases rather than decreases with the increase in slenderness ratio. This does not represent a true picture because by increasing the thickness of the wall the shape factor of individual units have been altered. This will have some effect on the strength of wall.

3.6.6.2. Brick walls

The average ultimate stress of WM12 wall was higher than that of WM6. This could be due to better workmanship and plumb which is confirmed by referring to Table 3.5, Column 13, the eccentricity developed in WM12 walls is smaller than in WM6 walls. Walls WM18 and WM25 had lower ultimate stress than the above walls as was expected. The
bricks used for wall WM25 were 20% less in compressive strength than the bricks of walls WM6, WM12 and WM18. This would result in lower strength reduction factor. On the basis of available relationship between brick strength and brickwork strength\(^\text{(3)}\) the ultimate strength of WM25 walls has been modified. Fig. 3.38 shows the strength of the brick walls.

3.6.6.2.1. Comparison of Author's Test Results with other Tests

Taking reduction factors for the squat wall WM6 as one, the reduction factors for WM12 walls is increased, for WM18 and WM25 it is decreased. (Fig. 3.39).

These tests, when compared with 4 ins. thick full scale wall, American\(^\text{(22)}\) tests had higher reduction factor up to slenderness ratio of 23.0, above this value of 23.0 the reduction factor is lower than the American tests. This could be attributed to difference in workmanship, which becomes important in the case of more slender walls. The reduction factors in American tests have been calculated on the basis of lowest wall of slenderness ratio 4.3 and not 6 as in the author's test. It is assumed that there is no reduction in strength up to slenderness ratio of 6.0. If American test values are shifted from slenderness ratio of 4.3 to 6.0, then the difference between the two test results is reduced. While comparing different test results this aspect should be kept in mind. The test results of Hendry et al.,\(^\text{(17)}\) (Fig. 3.39) from model walls of 4½ ins. equivalent thickness had some reduction factor up to slenderness ratio of 18, beyond this value the reduction factor is higher than that of WM walls.
The 3 ins. thick bonded walls of BlW series showed very little decrease in strength. Similar to CW29 walls, BlW18 walls had higher failure strength than walls with lower slenderness ratio. Since these BlW18 walls were 54 ins. high the mason built them in wooden jigs. The workmanship and so the vertical alignment of these walls turned out to be better than that of smaller walls. Referring to Table 3.4, column 12, it is seen that the eccentricity of loading is lower for BlW18 walls. The reduction factors for 8 ins. thick full scale American (22) test walls are lower than that of BlW walls. Similar to 4 ins. thick full scale American (22) test walls, the reduction factor for 8 ins. walls is calculated on the basis of lowest wall of slenderness ratio 3.0.

The various test results discussed here will be compared with the C.P.111-1970 in Chapter 7. It will be seen that the code values have lower reduction factor in comparison to test values.

The end conditions in a testing machine do not accurately represent the conditions of actual walls in buildings. Therefore the realistic method will be to test walls with R.C. slabs at top and bottom of the walls in order to simulate conditions prevailing in actual buildings, as was done by Prasan, Hendry and Bradshaw (16) in their crushing strength tests on 4½ ins. thick single leaf storey height walls. This has been briefly discussed in Chapters 4 and 5. The next Chapter (4) describes tests of walls under these end conditions.
3.7. CONCLUSIONS

(1) The mode of failure for the block walls had been a combination of (1) vertical tensile splitting, (2) shear failure and (3) some buckling for taller walls.

(2) Brick walls had a mode of failure of tensile vertical splitting accompanied by spalling and crushing indicating strength failure. Tall walls of WM25 group had some buckling as well.

(3) As expected, the strength of WM, BLW and CW walls decreased with the increase in slenderness ratio. Because of differences in workmanship, strengths of some of the walls were higher than expected.

(4) The stress-strain curves for the walls are in agreement with the curves of previous work, and the modulus of elasticity decreases with the increase in stress.
CHAPTER 4

AXIAL TESTING OF ONE THIRD SCALE BRICK WALLS LOADED BETWEEN R.C. SLABS

4.1. INTRODUCTION

Up till now the variation in ultimate compressive strength of masonry wall with different slenderness ratio has been studied by testing single leaf and bonded walls without slabs at their top and bottom ends. No test results are available for the walls of different height loaded between slabs at their ends. Prasan et al.\(^{(16)}\) studied the restraining effect of slabs on the strength of walls by testing 4½ ins. thick single leaf full scale storey height walls. They did not vary the height of the wall.

This chapter describes the test carried out on one third scale 1.5 ins. thick (4½ ins. equivalent) brick walls of different height with R.C. slabs at their top and bottom ends so as to simulate the end condition of internal wall prevailing in an actual building. As has been mentioned in an earlier chapter, the model test results reported here can be applied on full scale brickwork structures.

4.2. TEST PROGRAMME

The programme consists of testing thirteen walls of different height. Fig. 4.1. gives details of the walls.

4.3. MATERIALS

4.3.1. Bricks

One third scale bricks were used. They were tested
**Fig. 4.1**

TEST PROGRAMME FOR WS WALLS

**Table:**

<table>
<thead>
<tr>
<th>WALL NO.</th>
<th>WS4</th>
<th>WS6</th>
<th>WS9</th>
<th>WS14</th>
<th>WS19</th>
</tr>
</thead>
<tbody>
<tr>
<td>SLENDERNES RATIO</td>
<td>4.7</td>
<td>6.25</td>
<td>9.4</td>
<td>14</td>
<td>19.8</td>
</tr>
<tr>
<td>WALL TYPE</td>
<td>SINGLE LEAF STRETCHER BOND WS SERIES</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>NO. OF WALLS</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>LOADING</td>
<td>AXIALLY LOADED BETWEEN R.C. SLAB</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Fig. 4.2.** TEST ARRANGEMENT
in accordance with B.S. 3921-1969 (Part 2). Table 4.1 gives a summary of their properties.

4.3.2. Sand

Dry Leighton Buzzard 25/52 was used. The grading curve is shown in Fig. 3.3.

4.3.3. Cement

Rapid hardening Portland Cement (Ferrocrete) was used for all mortars to give early mortar strength.

4.3.4. Mortar

1:3 cement:sand mortar mix was used and proportions were made up by weight. Each batch of mortar was mixed by hand. Cubes were prepared and cured in a similar way to that described in Chapter 3.

4.3.5. Reinforced Concrete Slab Details

4.3.5.1. Proportions of mix by weight was 1 cement:2 sand:3 coarse aggregate

Cement and sand used were similar to those used for wall building. 3 ins. graded coarse aggregate was used for slab mixes.

4.3.5.2. Reinforcement

Nearly 1% reinforcement for positive and negative reinforcement were used. The steel used for the reinforcement was mild steel corresponding to B.S. 785 Part 1, 1967.

4.3.5.3. Dimensions of slab

Length of slab varied from 12 ins. to 72 ins. depending upon height of the wall.

Width of slab was 19 ins.

Thickness of slab was 1.5 ins.
<table>
<thead>
<tr>
<th>Properties</th>
<th>Range</th>
<th>Mean</th>
<th>Standard Deviation</th>
<th>Coefficient of variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length (inches)</td>
<td>2.9 - 3.06</td>
<td>2.96</td>
<td>0.06</td>
<td>2.1%</td>
</tr>
<tr>
<td>Width (inches)</td>
<td>1.40 - 1.47</td>
<td>1.43</td>
<td>0.028</td>
<td>1.8%</td>
</tr>
<tr>
<td>Height (inches)</td>
<td>0.94 - 1</td>
<td>0.96</td>
<td>0.019</td>
<td>2.03%</td>
</tr>
<tr>
<td>Compressive strength p.s.i.</td>
<td>2958 - 5100</td>
<td>3798</td>
<td>836</td>
<td>22%</td>
</tr>
<tr>
<td>Water absorption percentage</td>
<td>11.8 - 13.2</td>
<td>12.45</td>
<td>0.46</td>
<td>3.75%</td>
</tr>
</tbody>
</table>
4.4. EXPERIMENTAL PROCEDURE

4.4.1. Construction of wall

The procedure for building the walls is similar to that described in earlier chapters. All walls were built with wooden jigs. For walls WS4 and WS6 the first four courses of bricks were laid on a steel plate measuring 60 ins. x 30 ins. x 1/2 ins. On top of this four course brickwork 1/2 ins. thick, 1:1 cement:sand mortar was spread. (R.C.) slab was then laid on it, the slab was levelled by means of a spirit level and was gradually pressed so as to reduce the thickness of the mortar from 1/2 ins. to 1/3 ins. Far ends of the slab were made to rest on angle sections on which 1:1 cement:sand mortar was already laid. The main wall was then built in-situ on this slab. The upper slab was laid on the topmost course of the wall in a way similar to that in which the lower slab was laid. On top of the upper slab two courses of brickwork were built on which a spreader beam measuring 2 ins. x 2 ins. x 20 ins. was bedded in 1:1 cement:sand mortar, and levelled in two perpendicular directions by spirit level in order to have even distributions of load over the section of the wall, and hence avoid wall failure by stress concentration. Fig. 4.2. shows the test arrangement. Plate 4.1. shows the sequence of the wall building and slab laying. The top of the wall was cured under polythene cover for a minimum of 24 hours before being tested.

Walls WS9, WS14 and WS18 were built separately and were taken to the rig where they were bedded between the slabs in a way similar to that described earlier for walls WS4 and WS6.
PLATE 4.1. SEQUENCE OF WALL BUILDING AND SLAB LAYING.

PLATE 4.2(a). TEST FRAME (RIG).
4.4.2. **Test Frame (Rig)**

The frame structure is based on one steel grillage unit 6 ft. square. Screw jacks are provided under the grillage unit for levelling. For the superstructure to be erected on the grillage base, a set of steel channels has been selected. The assembly of the superstructure consists of a portal frame, each column of which consists of two 4 ins. x 12 ins. channel sections placed back to back. The clear working height from the top of the grillage units to the underside of the portal cross beam is variable. The working height is obtained by lowering or raising the cross head beam in order to accommodate a wall of the particular height for that particular slenderness ratio. The cross head beam consists of two 4 ins. x 12 ins. channels placed face to face in such a way as to form a box section. The clear working width is 4 ft. 4 ins. between the channels. The portal frame was further strengthened by providing angle bracings for the lateral strength. Plate (4.2).

4.4.3. **Loading Equipment**

4.4.3.1. **Hydraulic Jack**

The loading jack is of 200 ton nominal capacity and of the Tangye Hydraulic Detached Ship type having simple packed rams. The ram diameter is 10 ins. and maximum ram travel is 6 ins. The jack is bolted to the underside of the cross head beam of the portal frame in a central position (Plate 4.2). 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. The oil pressure is
applied through a Losenhausen machine.

4.4.3.2. Load cell

The load cell is of the column type H.D.110 supplied by M/S Davy & United of 200 ton nominal capacity having an overload capacity of at least 50% on the nominal maximum. One load cell of this type is placed under the hydraulic jack. Calibration of the cell was carried out in an Avery Universal Testing machine by using a digital voltmeter. The calibration chart is given in Fig. 4.3.

4.4.3.3. Digital Voltmeter (DVM)

A LM1450 four-window digital voltmeter employing an original digitising technique operating on a voltage/time conversion principle was used.

4.4.4. Application of load

The load was applied by using different equipment, details of which have been discussed before. The load was initially applied at intervals of 1 ton up to 6 tons, and then at 2 ton intervals up to failure. The rate of loading varied between 26 to 47 p.s.i. per minute.

4.4.5. Measurements

4.4.5.1. Strain measurement

Vertical strain was measured by fixing Demec studs on each face of the wall and reading the value by means of a Demec gauge of different gauge length depending upon the height of the walls, as has been described in a previous chapter (3).

4.4.5.1. Lateral deflection

For WS4 and WS6 walls three dial gauges were fixed, one at the topmost course another at mid-height and the third
Fig. 4.3. LOAD CELL CALIBRATION

Fig. 4.4. TEST VALUES WS-6-1
at the bottommost course. For the remaining walls, the
dial gauges were placed at 6 ins. c/c along the height of
the wall. The dial gauges used were capable of measuring
deflection up to 0.0001 in.

4.4.5.2. End rotation

This was measured by fixing brackets to the topmost and
bottommost courses of the wall as described in Chapter 3.

4.4.5.3. Curvature measurement

An attempt was made to measure curvature variation along
the height of the walls WS4 and WS6. This was done by
fixing Demec studs at 2 ins. c/c along the height of the
wall and measuring the strain on both faces of the wall by
means of Demec gauge of 2 ins. gauge lengths. The differ-
ence in strain readings on the two faces will give the
value of curvature. Fig. 4.4. shows the positions of
Demec studs along the height of the wall.

4.5. RESULTS

A summary of the results of the wall tests and a com-
parison of reduction factors with C.P. 111-1970 are given
in Table 4.2.

4.6. DISCUSSION

4.6.1. Modes of failure

Generally the first hairline crack appeared at 50-60% of
the ultimate load and got enlarged with further increase in
load as has been observed in the walls of the previous
chapter. The general mode of failure of walls was tensile
vertical splitting accompanied by crushing and spalling of
various courses of bricks. Plates 4.3 to 4.4 shows
different modes of failure of these walls.
<table>
<thead>
<tr>
<th>Wall No.</th>
<th>Slenderness ratio</th>
<th>Age of mortar strength cube days</th>
<th>Age of walls days</th>
<th>Ultimate load ton</th>
<th>Ultimate compressive stress p.s.i.</th>
<th>Average ultimate compressive stress p.s.i.</th>
<th>Test Reduction factor</th>
<th>Modulus of Elasticity $\frac{1}{2}$ ult. stress x $10^6$ p.s.i.</th>
<th>Eccentricity Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>WS4-1</td>
<td>4.7</td>
<td>31</td>
<td>32</td>
<td>32.4</td>
<td>2653.5</td>
<td>2653.5</td>
<td>1.0</td>
<td>0.5</td>
<td>t/16 $\Rightarrow$ 0</td>
</tr>
<tr>
<td>WS4-2</td>
<td>4.7</td>
<td>25</td>
<td>23</td>
<td>32.4</td>
<td>2653.5</td>
<td>2653.5</td>
<td>1.0</td>
<td>0.5</td>
<td>t/22 $\Rightarrow$ 0</td>
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<tr>
<td>WS6-1</td>
<td>6.25</td>
<td>126</td>
<td>108</td>
<td>30.8</td>
<td>2537.5</td>
<td>2537.5</td>
<td>1.0</td>
<td>0.5</td>
<td>t/64 $\Rightarrow$ 0</td>
</tr>
<tr>
<td>WA6-2</td>
<td>6.25</td>
<td>109</td>
<td>106</td>
<td>35.5</td>
<td>2929.0</td>
<td>2726</td>
<td>0.63</td>
<td>0.63</td>
<td></td>
</tr>
<tr>
<td>WS9-1</td>
<td>9.4</td>
<td>38</td>
<td>37</td>
<td>31.0</td>
<td>2566.5</td>
<td>2566.5</td>
<td>1.0</td>
<td>0.797</td>
<td>t/24 $\Rightarrow$ t/75</td>
</tr>
<tr>
<td>WS9-2</td>
<td>9.4</td>
<td>33</td>
<td>32</td>
<td>37.0</td>
<td>3059.5</td>
<td>2842</td>
<td>0.92</td>
<td>0.783</td>
<td>t/30 $\Rightarrow$ 0</td>
</tr>
<tr>
<td>WS9-3</td>
<td>9.4</td>
<td>16</td>
<td>16</td>
<td>35.5</td>
<td>2929.0</td>
<td>2929.0</td>
<td>0.783</td>
<td>0.783</td>
<td>t/30 $\Rightarrow$ 0</td>
</tr>
<tr>
<td>WS14-1</td>
<td>14.0</td>
<td>43</td>
<td>42</td>
<td>25.1</td>
<td>2073.5</td>
<td>2073.5</td>
<td>0.826</td>
<td>0.826</td>
<td>t/22 $\Rightarrow$ 0</td>
</tr>
<tr>
<td>WS14-2</td>
<td>14.0</td>
<td>43</td>
<td>43</td>
<td>24.1</td>
<td>1986.5</td>
<td>2015.5</td>
<td>0.78</td>
<td>0.812</td>
<td>t/25.4 $\Rightarrow$ t/54</td>
</tr>
<tr>
<td>WS14-3</td>
<td>14.0</td>
<td>70</td>
<td>64</td>
<td>24.1</td>
<td>1986.5</td>
<td>1986.5</td>
<td>0.812</td>
<td>0.812</td>
<td>t/21.4 $\Rightarrow$ t/22.4</td>
</tr>
<tr>
<td>WS19-1</td>
<td>19.8</td>
<td>89</td>
<td>83</td>
<td>19.8</td>
<td>1631.25</td>
<td>1631.25</td>
<td>0.768</td>
<td>0.768</td>
<td>t/27 $\Rightarrow$ 0</td>
</tr>
<tr>
<td>WS19-2</td>
<td>19.8</td>
<td>51</td>
<td>50</td>
<td>22.0</td>
<td>1812.5</td>
<td>1879.2</td>
<td>0.62</td>
<td>0.652</td>
<td>t/31 $\Rightarrow$ 0</td>
</tr>
<tr>
<td>WS19-3</td>
<td>19.8</td>
<td>60</td>
<td>58</td>
<td>26.6</td>
<td>2189.5</td>
<td>2189.5</td>
<td>0.710</td>
<td>0.710</td>
<td>t/33 $\Rightarrow$ 0</td>
</tr>
</tbody>
</table>
PLATE 4.3. FAILURE OF WS 12 WALL.

PLATE 4.4. FAILURE OF WS 18 WALL.
In wall WS4-1 there was spalling of 4 courses below the bottom of the slab. There were vertical cracks in the ends and at the centre of the wall. A portion of wall failed by crushing after the cracks had widened. After the failure, a portion of wall left intact between the slab was still strong.

In wall WS4-2 also there was spalling of four courses below the lower slab. Failure of main wall was due to tensile vertical splitting and crushing. The wall collapsed suddenly with explosive ejection of masonry pieces.

Walls WS6-1 and WS6-2 both failed suddenly in a similar manner. Before complete failure there was a vertical crack in the centre of the wall. For wall WS6-1 this crack started from mid-height and went down to the bottom. In WS6-2 the crack started in the second course from the top and continued till the last but one course. There was crushing of the topmost course as well.

Walls WS9-1, WS9-2, WS9-3 developed hairline cracks at 50% of ultimate load.

In WS9-1 the first hairline crack started developing in the fourth course from the bottom and continued up to the topmost course. There was spalling in the middle course of the wall.

Walls WS9-2 and WS9-3 failed in a similar way to WS9-1, except that in WS9-2 there was complete absence of spalling.

Walls WS14-1 and WS14-2 failed with crushing of topmost and bottommost courses. There was a vertical crack in the bottom four courses of the wall WS13-2 and in the middle course of wall WS14-1.
Wall WS14-3 failed with the development of a crack starting from the topmost course in the centre of the wall, and extending down to mid-height of the wall. Another vertical crack developed adjacent to the previous crack but starting from mid-height and extending down to the bottommost course.

Walls WS19-1 and WS19-2 failed by spalling, tensile vertical splitting and bending. The first hairline crack appeared at 80% of the failure load. The walls bent to a considerable extent after vertical tensile splitting and spalling in the 18th, 20th and 24th course from the top.

4.6.2. Strain readings

Strain readings on both faces of walls were not very different from each other, indicating fairly uniform distribution of load on the walls. As has been mentioned in the previous chapter, in the initial stage of loading difference in strain was observed to be higher, but the difference narrowed down with the increase in load. The reason for this has been explained earlier in Chapter 3. Figs. 4.5. to 4.8. give stress-strain curve.

The difference in strain readings could also be due to lack of plumbness of the test wall, and due to the different position of Demec studs on the opposite faces of the wall.

Eccentricity of loading has been calculated in a way similar to that done for other walls in Chapter 3. The values of eccentricity obtained indicate that the condition of loading is fairly axial. It is impossible to have zero eccentricity under axial load.
Fig. 4.5
COMPRESSIVE STRESS
Vs.
VERTICAL STRAIN

Fig. 4.6.
COMPRESSIVE STRESS
Vs.
VERTICAL STRAIN

<table>
<thead>
<tr>
<th>SYMBOL</th>
<th>WALL NO.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>WS4-1</td>
</tr>
<tr>
<td></td>
<td>WS4-2</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>SYMBOL</th>
<th>WALL NO.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>WS6-1</td>
</tr>
<tr>
<td></td>
<td>WS6-2</td>
</tr>
</tbody>
</table>
Fig. 4.7
COMPRESSIVE STRESS
Vs.
VERTICAL STRAIN

Fig. 4.8
COMPRESSIVE STRESS
Vs.
VERTICAL STRAIN
4.6.3. **Modulus of elasticity**

The value of the modulus of elasticity $E$ was obtained from the stress-strain curve. The value of $E$ fluctuated during the lower range of loading, but afterwards, at about 25-40% of load, it started decreasing with the increase in load. Fig. 4.9. shows stress and modulus of elasticity relationship.

4.6.4. **Lateral deflection**

Similar to walls in Chapter 3, none of the walls deflected to the same extent, therefore no definite pattern of behaviour was obtained. Most of the walls deflected more towards the top than at mid-height. As explained in Chapter 3 above, the irregularity in deflection may be due to variation in (1) workmanship, (2) plumbness of the wall, (3) properties of brick and mortar. Maximum deflections in the walls were insignificant to influence failure of walls except for the WML9 wall. Figs. 4.10 to 4.12 show deflection curve for various walls.

4.6.5. **End rotation**

The angular rotation values obtained are not consistent. Like deflection values none of the walls rotated to the same extent. In the course of loading, some walls changed the direction of rotation as well. Because of these erratic results, no definite conclusion could be drawn. Figs. 4.15 to 4.17 show angular rotation and stress relationship.

4.6.6. **Curvature variation**

An attempt to measure curvature variation along the height of the wall was made in order to locate the point of inflections, so as to get a clear idea of the effective
Fig. 4.9. COMPRESSION STRESS vs. VERTICAL STRAIN

Fig. 4.10. MODULUS OF ELASTICITY vs. COMPRESSION STRESS
Deflection in inches

Fig. 6.11 Lateral deflection measurements

Fig. 6.12 Lateral deflection measurements
Fig. 4.15.
COMPRESSION STRESS
Vs.
END ROTATION

Fig. 4.15.
COMPRESSION STRESS
Vs.
END ROTATION

SYMBOL

WALL NO.

W56-1

W56-2

STAIRLR

WALL NO.

W56-1

W56-3
Fig. 4.16. Compressive Stress vs. End Rotation.
**Fig. 4.18**
ULTIMATE STRESS
Vs.
SLENDERNESS RATIO
(WS WALL)

**Fig. 4.19**
REDUCTION FACTOR
Vs.
SLENDERNESS RATIO

**Fig. 4.17**
COMPRESSIVE STRESS x 10^4
Vs. END ROTATION

Fig. 4.17. COMPRESSIVE STRESS Vs. END ROTATION.

**Fig. 4.18.** ULTIMATE STRESS Vs. SLENDERNESS RATIO
(WS WALL)
height. This attempt also did not succeed. The curves so obtained from strain measurements do not give any clear picture regarding the location of the point of inflection (Fig. 4.4.). The curve is based on the following equations:

\[
\sigma = \frac{P}{A} + \frac{M \lambda}{I} \quad \text{.......... (1)}
\]

\[
E \varepsilon_1 = \frac{P}{A} + \frac{E}{R} \gamma \quad \text{.......... (2)}
\]

\[
E \varepsilon_2 = \frac{P}{A} - \frac{E}{R} \gamma \quad \text{.......... (3)}
\]

Dividing (2) and (3) by \(E\) and subtracting (3) from (2) we get

\[
\varepsilon_1 - \varepsilon_2 = \frac{\lambda}{R} \quad \text{or} \quad \frac{\varepsilon_1 - \varepsilon_2}{\lambda} = \frac{1}{R} \quad \text{.......... (4)}
\]

where \(\sigma\) - stress

\(P\) - load on wall

\(A\) - area of cross-section

\(M\) - moment

\(I\) - moment of inertia

\(\gamma\) - distance from centroid

\(\varepsilon_1\) & \(\varepsilon_2\) - strain on opposite face of wall

\(E\) - modulus of elasticity

\(\frac{1}{R}\) - radius of curvature

Strains \(\varepsilon_1\) and \(\varepsilon_2\) are found along the height of the wall by means of 2 ins. demec gauge. The point along the height of the wall at which equation (4) changes sign will give the point of inflection.

Fig. 4.4. shows what the curvature variation along the height of the wall should be ideally. But because of
the brickwork being two phase material and of variable nature, and also due to the presence of micro-cracks in the bricks, the test results so obtained are not consistent. They show a quite irregular pattern of strain difference along the height of the wall (Fig. 4.4.). Since the curvature is a double differential of the deflection there will be a scatter of points along the height of the wall.

Bradshaw (56) studied the strain variation along the height of a 4½ ins. thick full scale storey height wall by using Demec gauge of 8 ins. gauge length. Strain measurements did not give a regular pattern of behaviour. A similar study was carried out by Jecici and Cacovic (59) (74). Their results are also not consistent.

To the author's knowledge no study of curvature variation along the height of the wall has been carried out before by this method. It will be therefore worthwhile if a detailed study is carried out by doing more tests on walls of different height. The walls chosen for this test by the author were of very small slenderness ratio. The lateral deflection, and also the curvature in these walls, will not be significant. Walls above slenderness ratio of 20 will give a better picture of the curvature variation, because in addition to strength failure they will be having some buckling failure as well.

4.6.7. Slenderness ratio

The average ultimate stress of WS4 wall and WS6 wall was almost the same. This goes well with the code assumption that up to a slenderness ratio of 6 there is no decrease in the strength of the wall. The strength of the wall WS9
instead of decreasing was 6% higher than the walls WS4 and WS6. This increase in strength is due to better distribution of load, and only a little higher rate of loading than WS6 walls. In the case of WS14 walls the strain measurements showed that the load was not as evenly distributed as it had been with other walls, the workmanship of the wall was also comparatively poor. This has resulted in a decrease in the strength of the wall rather greater than expected. There is very little scatter of test results. Only in WS19 walls the strength of WS19-3 is noticeably different from that of other walls (Fig. 4.18).

Unfortunately no test results are available with which a comparison could be made. However the test results are compared with the C.P.111 (Fig. 4.19). The reduction factors of the test walls are higher than the code values. If the correction for the variation in rate of loading and variation in brick strength is made, the test values will have a further increase in reduction factor in comparison to code values. A detailed comparison of C.P.111 with these walls is made in Chapter 7.

There is little doubt that the end fixity provided by the R.C. slab is greater than that provided by the machine platen. The coefficient of friction between the slab and the mortar will be more in comparison to coefficient of friction between steel and mortar. This will result in greater bond in the former case and lesser bond in the latter case. Because of this the lateral restraint provided by the R.C. slab is greater, the strength of WS walls should therefore be higher than the WM walls. But this is not the
case, the strength of WM walls in some cases is found to be little higher than WS walls. The reason for this is the frequently cited factors such as 1) difference in rate of loading between the WM and WS walls, 2) difference in load distribution and thus eccentricity, 3) difference in brick strength of the two walls. If these factors are taken into account the strength of WS walls will be higher than WM walls.

Also it has been shown that there is a significant decrease in wall strength when there is sidesway in the wall. In fact the reduction due to sidesway is greater than the reduction resulting from the same degree of eccentricity of loading. This reduction in strength is due to the fact that sway induces some tensile stresses and subjects the wall to a severe type of eccentric loading (Fig. 4.20). These stresses can be approximately estimated by the following equation:

\[ f_t = \frac{M_{AB}}{z} \]

where \( z \) is the section modulus. Since the brickwork is weak in tension the ultimate strength of wall will decrease.

It has been observed while doing the test that WS walls had more sidesway because of difficulty involved in capping the slab than flat ended walls of WM series. Because of these reasons nearly the same strength for both the WM and WS walls does not seem to be unexpected.
4.6.8. Conclusions

1) The strength of the walls decreases with the increase in slenderness ratio, but not in the same proportion as specified in C.P.111(1970).

2) End fixity provided by the R.C. slab is greater than that provided by the machine platen.

3) Mode of failure of all the walls has been mainly strength failure, except for the wall WM19-3 in which there was buckling involved.

4) The modulus of elasticity decreased with the increase in stress as was expected.

5) Because of variable nature of the brickwork, the angular rotation of the topmost and bottommost course of the wall did not show a regular pattern. Similarly irregular strain patterns along the height of the walls were observed when the curvature variation of the wall was measured.
CHAPTER 5
AXIAL AND ECCENTRIC TESTING OF MODEL WALLS WITH HINGE ENDS

5.1. INTRODUCTION

So far most of the tests carried out by various investigators to study slenderness ratio effects have been confined to either axially loaded walls or axially and eccentrically loaded piers. In practice walls are usually subjected to eccentric loading which may arise either due to the positioning of the floor slab or the beam on the wall, or due to the presence of unequal moment at the slab/wall joint.

The object of the work presented in this chapter was to examine the strength of one third scale, 1.5 ins. thick single leaf brick walls of different slenderness ratios with hinged ends under axial and eccentric loading. The most common type of eccentricity of loading of t/6 and t/3 encountered in practice were adopted. Along with these walls, piers of 9½ ins. height were tested with axial and eccentric loading. The testing procedure and their results are discussed in Appendix III. The test results are compared with the results of other investigators. In practice hinged ends are seldom if ever encountered, and in most cases the restraining action of the slab results in increased wall strength. This is consistent with the work carried out by Prasan et al.\textsuperscript{(16)} The test results are discussed in the light of their (Prasan et al.) work in the latter part of this chapter.
5.2. TEST PROGRAMME

The test programme consisted of testing walls of different height corresponding to different slenderness ratios with different positions of loading in accordance with the required eccentricity. Fig. (5.1) gives details of the walls.

5.3. MATERIALS

5.3.1. Bricks

One third scale model bricks were used. Bricks came in batches so that their strength varied. They were tested in accordance with BS-3921-1969 Part (2). Table 5.1. gives a summary of their properties.

5.3.2. Sand

Dry Leighton Buzzard 25/52 as described in previous chapters was used. The grading curve is shown in Fig. (3.3.).

5.3.3. Cement

Rapid hardening Portland cement (Ferrocrete) was used for walls as in previous tests for all mortars to give early strength.

5.3.4. Mortar

1:3 cement:sand mortar mix was used and proportions were made up by weight. Each batch of mortar was mixed by hand. Cubes were prepared and cured as described in previous chapters.

5.4. EXPERIMENTAL PROCEDURE

5.4.1. Construction of wall

The procedure for building the walls was again as described in earlier chapters. All walls were built with wooden jigs, the only difference this time was that a steel
<table>
<thead>
<tr>
<th>Properties</th>
<th>Range</th>
<th>Mean</th>
<th>Standard deviation</th>
<th>Coefficient variation</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Length (inches)</strong></td>
<td>2.90 - 3.06</td>
<td>2.96</td>
<td>0.06</td>
<td>2.1%</td>
</tr>
<tr>
<td><strong>Width (inches)</strong></td>
<td>1.40 - 1.47</td>
<td>1.43</td>
<td>0.028</td>
<td>1.8%</td>
</tr>
<tr>
<td><strong>Height (inches)</strong></td>
<td>0.94 - 1</td>
<td>0.96</td>
<td>0.019</td>
<td>2.03%</td>
</tr>
<tr>
<td><strong>Compressive Strength (p.s.i.)</strong></td>
<td>3000 - 5100</td>
<td>3,800</td>
<td>836</td>
<td>22%</td>
</tr>
<tr>
<td><strong>Water Absorption (%)</strong></td>
<td>11.8 - 13.2</td>
<td>12.45</td>
<td>0.46</td>
<td>3.75%</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Properties</th>
<th>Range</th>
<th>Mean</th>
<th>S.D.</th>
<th>Coefficient variation</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Length (inches)</strong></td>
<td>3.0 - 3.03</td>
<td>3.01</td>
<td>0.04</td>
<td>1.3%</td>
</tr>
<tr>
<td><strong>Width (inches)</strong></td>
<td>1.44 - 1.48</td>
<td>1.47</td>
<td>0.012</td>
<td>0.8%</td>
</tr>
<tr>
<td><strong>Height (inches)</strong></td>
<td>0.97 - 1.0</td>
<td>0.98</td>
<td>0.03</td>
<td>3.4%</td>
</tr>
<tr>
<td><strong>Compressive Strength (p.s.i.)</strong></td>
<td>4048 - 6730</td>
<td>5530</td>
<td>63.4</td>
<td>1.2%</td>
</tr>
<tr>
<td><strong>Water Absorption (%)</strong></td>
<td>11.6 - 12.9</td>
<td>12.12</td>
<td>0.87</td>
<td>7.2%</td>
</tr>
</tbody>
</table>

<table>
<thead>
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<th>Mean</th>
<th>S.D.</th>
<th>Coefficient variation</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Length (inches)</strong></td>
<td>3.0 - 3.03</td>
<td>3.009</td>
<td>0.014</td>
<td>0.47%</td>
</tr>
<tr>
<td><strong>Width (inches)</strong></td>
<td>1.40 - 1.46</td>
<td>1.43</td>
<td>0.023</td>
<td>1.6%</td>
</tr>
<tr>
<td><strong>Height (inches)</strong></td>
<td>0.94 - 0.97</td>
<td>0.94</td>
<td>0.014</td>
<td>14.5%</td>
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<tr>
<td><strong>Compressive Strength (p.s.i.)</strong></td>
<td>3016 - 4074</td>
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<td>334</td>
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<td><strong>Water Absorption (%)</strong></td>
<td>11.7 - 12.8</td>
<td>12.3</td>
<td>0.4</td>
<td>3.2%</td>
</tr>
</tbody>
</table>
Fig 5.1

TEST PROGRAMME OF BRICK WALLS

<table>
<thead>
<tr>
<th>WALL NO.</th>
<th>W-o-6</th>
<th>W-o-12</th>
<th>W-o-18</th>
<th>W-o-25</th>
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<tbody>
<tr>
<td>W-½-6</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>W-¾-6</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>NO. OF WALLS</td>
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<td>3</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>LOADING W0</td>
<td>AXIAL e = 0 HINGE ENDS</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>W½</td>
<td>ECCENTRIC e = ½ HINGE ENDS</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>W¾</td>
<td>ECCENTRIC e = ¾ HINGE ENDS</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>WALL TYPE</td>
<td>SINGLE LEAF STRETCHER BOND</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>THICKNESS Wo A</td>
<td>1.40”—1.47”</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>W½</td>
<td>1.44”—1.48”</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>W¾</td>
<td>1.40”—1.48”</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>WIDTH B</td>
<td>18.5”—19’</td>
<td></td>
<td></td>
<td></td>
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</table>

<table>
<thead>
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<th>KEY TO SYMBOLS</th>
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<tbody>
<tr>
<td>W-o-6-1</td>
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<tr>
<td>WALL</td>
</tr>
<tr>
<td>ECCENTRICITY</td>
</tr>
<tr>
<td>S.H.</td>
</tr>
<tr>
<td>NO.</td>
</tr>
</tbody>
</table>
plate of \( \frac{1}{8} \) in. thickness was fixed on the wooden jigs. This was done in order to protect the wooden jig from warping due to moist conditions.

5.4.2. Testing method and measurement

For capping of walls two beams were used of size \( 4\frac{1}{2} \) ins. \( \times \) 1 in. \( \times \) 20 ins. with two half rounds of size \( \frac{1}{8} \) in. welded to the centreline of each beam. The bottom beam had four levelling screws at its four corners. At each end of each beam distances equal to half the thickness of the wall were marked on either side of the centreline of the beams. This method was adopted for axially loaded walls of W-0 series in order that the centreline of the wall and the beam should be in line. For walls of W-1/6 series, where the eccentricity of loading is \( t/6 \), distances equal to \( \frac{1}{8} \) in. and 1 in. were marked on either side of the centreline of the beam in order that the centreline of the beam (through which the load is applied) and the centreline of the wall should be \( \frac{1}{4} \) in. \( (t/6) \) apart. Similarly for walls of W-1/3 series, distances equal to \( 7/32 \) in. and \( 1-7/32 \) ins. were marked so that the centreline of the wall and the beam should be \( \frac{1}{2} \) in. \( (t/3) \) apart. These distances were marked with the utmost possible accuracy. Fig. (5.2) shows these arrangements.

The beam with the levelling screws was placed in the Avery machine so that the centreline of the beam coincided with the centreline of the machine, and levelled by means of four levelling screws. The walls built were then taken to the machine either by crane or manually on a trolley depending upon their height. 1:1 mortar mix was prepared and spread on the beam and the walls were placed on the beam
Fig 5.2. LOADING ARRANGEMENT.
so that its edges coincided with the appropriate marks on the beam to give the desired eccentricity of loading. After checking the plumb of the wall, it was left to set with the mortar for two hours. The top beam was placed in a similar way and a small load gradually applied by the machine in order to level the beam. The walls were then allowed to cure for 24 hours before being tested to failure. Plate (5.1) shows a wall before test.

5.4.3. Application of loads

For axially loaded walls of $W-\sigma$ series, the loads were applied by 1 ton increments up to 6 tons, and then at 2 ton intervals up to failure. The average rate of loading varied between 23 p.s.i./minute to 46 p.s.i./minute. For eccentrically loaded walls of $W-1/6$ series a similar procedure was adopted up to slenderness ratio of 18; for the wall of slenderness ratio 25 the loads were applied at $\frac{7}{8}$ ton intervals up to failure. The average rate of loading varied between 12.1 p.s.i./minute and 23.5 p.s.i./minute. For the walls with $t/3$ eccentricity belonging to $W-1/3$ series the load was applied at $\frac{1}{2}$ ton increments up to failure. The average rate of loading varied between 4.8 p.s.i./minute to 17.0 p.s.i./minute. In all the walls the load passed from one stage to another in about one minute, and was then held constant for 3 to 4 minutes in order to take measurements. A plywood sheet of $\frac{1}{8}$ in. thickness was placed between the top platen of the machine and the half round in order to take care of any gap left between them.

5.4.4. Strain and lateral deflection measurements

As described in an earlier chapter (3) the vertical
PLATE 5.1. W-O WALL READY FOR TEST.

PLATE 5.2. FAILURE OF W-8-6 WALL.
strain was measured by Demec gauge and the lateral deflection was recorded by means of dial gauges of 0.0001 in. sensitivity at each load increment, until safety considerations made it necessary to remove the gauges and discontinue the readings.

Before starting a test the walls were slightly loaded in order to prevent them from falling down. The zero deflection readings were therefore not exactly at zero load.

5.5. RESULTS

A summary of results of the wall tests and a comparison of the reduction factors with C.P.111(1970) are given in Table 5.2.

5.6. DISCUSSION

5.6.1. Modes of Failure

5.6.1.1. W-O walls (c = 0)

For the walls of W-O series, the first hairline crack appeared at 50-80% of the ultimate load, and enlarged with further increase in load as has been discussed in previous chapters. Walls W-O-6 had tensile cracks at the two faces of the wall accompanied by spalling of the middle course. They also showed crushing of either the first or the second course from the top. There was absence of crack in the thickness of the wall. Half of the W-O-12-1 wall failed due to vertical tensile splitting, crushing and spalling. The remaining half of the wall was intact with a few small tensile cracks. Plate (5. 3). Wall W-O-12-2 had vertical cracks in the centre and in the sides on both the faces. The topmost course had rotated to a considerable degree, and there was some spalling in the top two courses. Wall WM12-3
<table>
<thead>
<tr>
<th>S.R.</th>
<th>Wall No.</th>
<th>Age of cube days</th>
<th>Age of wall days</th>
<th>e=0 Wall stress p.s.i.</th>
<th>Age of cube</th>
<th>e=t/6</th>
<th>Average wall stress p.s.i.</th>
<th>Cube strength p.s.i.</th>
<th>Mean cube of wall</th>
<th>Mean wall stress p.s.i.</th>
<th>Age of wall days</th>
<th>e=t/3 Cube strength p.s.i.</th>
<th>Age of wall days</th>
<th>Average wall stress p.s.i.</th>
<th>Mean wall stress p.s.i.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>W 6 - 1</td>
<td>16</td>
<td>13</td>
<td>28</td>
<td>28</td>
<td>5060</td>
<td>1417.3</td>
<td>28</td>
<td>38</td>
<td>717</td>
<td>43</td>
<td>4398</td>
<td>38</td>
<td>519</td>
<td>601.5</td>
</tr>
<tr>
<td></td>
<td>W 6 - 2</td>
<td>14</td>
<td>13</td>
<td>28</td>
<td>29</td>
<td>3827</td>
<td>1343.0</td>
<td>39</td>
<td>38</td>
<td>42</td>
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<td>292.5</td>
</tr>
<tr>
<td></td>
<td>W 6 - 3</td>
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<td>15</td>
<td>34</td>
<td>33</td>
<td>4698</td>
<td>1458.5</td>
<td>37</td>
<td>37</td>
<td>37</td>
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<td>4519.2</td>
<td>39</td>
<td>569</td>
<td>292.5</td>
</tr>
<tr>
<td>6</td>
<td>W 12 - 1</td>
<td>18</td>
<td>8</td>
<td>31</td>
<td>26</td>
<td>4407.3</td>
<td>1096.0</td>
<td>42</td>
<td>41</td>
<td>287</td>
<td>38</td>
<td>4378</td>
<td>41</td>
<td>287</td>
<td>292.5</td>
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<tr>
<td></td>
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PLATE 5.4. BENDING OF W012 WALL.

PLATE 5.3. SPALLING, CRUSHING & TENSILE SPLITTING - W012.
had a vertical crack similar to that in WM12-2 on the faces of the wall accompanied by some spalling in the 4th and the 12th course from the top. Wall W-0-18-1 failed with the appearance of tensile cracks at the centre face of the wall. This crack started from the topmost course and continued downwards to the 17th course from the top. A similar crack appeared in the thickness of the wall but continued downwards to the 5th course from the top. There was slight spalling in the two central bricks of the third course from the top. Walls W-0-18-2 and third course from the top for W-0-18-3. These cracks continued downwards to the bottommost course for both the walls.

Walls W-0-18-2 and W-0-18-3 had a considerable amount of bending as well. Walls W-0-25-1, 2 and 3 had a similar type of failure, except that W-0-25-1 had some spalling and crushing and W-0-25-2 had a shear crack line in the bottom of the wall.

5.6.1.2. W-1/6 walls (e = t/6)

In all the walls of W-1/6 series the crack appeared at 70-80% of the failure load. Wall W-1/6-1 and 2 failed due to spalling in the middle and bottom courses, accompanied by tensile vertical cracks and bending. W-1/6-6-2 also had a crack in the thickness of the wall, and there was significant rotation of the topmost course. Wall W-1/6-6-3 had spalling of the top three courses, crushing of the middle course, accompanied by a tensile crack in the thickness of the wall. There was considerable rotation of the topmost course resulting in break of bond in the horizontal joint. The middle brick of the topmost course had a tensile crack at its mid-depth, this was peculiar and could be due to the
presence of excessive micro-cracks in that brick and good bond between mortar joint and brick, which may have induced vertical tensile stress.

Wall W-1/6-12-1, 2 and 3 failed due to the presence of spalling either in the top 3-courses or in the courses at the mid-height accompanied by tensile splitting in the thickness and the face of the wall. At the time of failure the bending of the wall became very noticeable. Wall W-1/6-18-1, 2 and 3 mainly failed due to excessive lateral deflection. W-1/6-18-1 had a number of small tensile vertical cracks spread on both the faces of the wall. There was also some spalling in the central brick of the topmost course. The 1:1 mortar mix for capping of the wall was not of uniform thickness and this may have resulted in poor distribution of load. Wall W-1/6-25-1, 2 and 3 had stability induced failure due to excessive lateral deflection of the wall. There was complete absence of spalling and tensile splitting. All the walls had maximum deflection between 11th and 14th courses from the top, and the failure occurred when the bond between brick and mortar at these joints gave way.

5.6.1.3. W-1/3 walls (e = t/3)

In walls W-1/3-6-1, 2 and 3 a very light cracking sound occurred at 20 to 25% of the ultimate load, and at about 35% of the ultimate load the sound became more clear without any appearance of a crack. All the walls failed due to excessive lateral deflection which generally occurred at mid-height. In W-1/3-12-1, 2 and 3 walls faint cracking sounds followed by clear cracking sounds occurred between 20 to 35% of failure load without any appearance of the crack. There was complete absence of spalling, crushing and vertical tensile splitting
indicating that the failure was due to instability. The maximum horizontal deflection occurred between the 5th and the 8th course from the top. Walls W-1/3-18-1, 2 and 3 and W-1/3-25-1, 2 and 3 had a similar failure pattern in which faint and clear cracking sounds occurred between 20 to 35% of the failure load, without appearance of any crack. In walls W-1/3-18-1, 2 and 3 the maximum horizontal deflection occurred between 10th to 14th course from the top. Walls W-1/3-25-1, 2 and 3 had crack sounds at the loads similar to the above wall. The maximum horizontal deflection in these walls occurred between 14th and 20th course from the top.

In all these walls the failure occurred at the mortar-brick interface due to lack of bond between the mortar and the brick at the time of maximum deflection. Water absorption and the bed face roughness of the bricks are the main factors which greatly influence the bond developed at the interface. Plate (5.5) shows a typical failure of wall at the mortar-brick interface.

5.6.2. Strain readings

For axially loaded walls of W-0 series the strain readings on both the faces of the walls were not very different from each other, implying fairly uniform distribution of load on the walls of this series. As has been discussed in previous chapters the difference in strain readings was higher in the initial stages of the loading, but this reduced with the increase in loading. Fig. (5.3) to (5.6) shows the stress-strain relationship.

Walls of W-1/6 series having eccentricity of loading
PLATE 5.5(a). SPALLING AND BENDING FAILURE W-4-25.

PLATE 5.5(b). BENDING-(BOND FAILURE AT BRICK-MORTAR INTERFACE.) W-3-25.
Fig. 5.3
COMPRESSION STRESS
Vs.
VERTICAL STRAIN

<table>
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<tr>
<td>○</td>
<td>Wo6 - 2</td>
</tr>
<tr>
<td>△</td>
<td>Wo6 - 3</td>
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</tbody>
</table>
Fig. 5.4. COMPRESSIVE STRESS Vs. VERTICAL STRAIN

Fig. 5.5. COMPRESSIVE STRESS Vs. VERTICAL STRAIN
Fig 5.6
COMPRESSIVE STRESS
Vs.
VERTICAL STRAIN

Fig 5.7a)
COMPRESSIVE STRESS
Vs.
VERTICAL STRAIN
e = t/6 (condition when the load is at the kern) had compressive strain on one face and tensile strain on the other. Ideally the strain on the tension face should have been zero, but due to inaccuracy involved in building the wall and applying the desired eccentricity there is some tensile strain (Fig. 5.7 to 5.10). The presence of this strain implies that the eccentricity of loading is greater than t/6, particularly at higher loads when the lateral deflection causes further increase in the eccentricity. The eccentricity calculated from the strain measurements confirm this increase and show that the eccentricity is 25% to 55% more than the intended nominal eccentricity of t/6.

Walls of W-1/3 series having eccentricity of loading e = t/3 (when load is beyond middle third) have tensile as well as compressive strain as was expected. The strain measurements indicate increase in applied eccentricity by 21%. The reasons for this increase are the same as for W-1/6 walls. Fig. (5.11) to (5.14) gives the stress-strain relationship for these walls.

5.6.3. Modulus of elasticity

The stress-strain curve for finding the modulus of elasticity E was plotted on the basis of an average of four strain readings for all the walls. Table (5.2) gives the E value for different walls. Except for walls W-0-6-1, 2, W-0-12-1 and W-0-18-3 all the remaining walls of this series showed a consistent decrease in E values with increase in stress. The E values fluctuated significantly up to stress of 150 p.s.i., beyond that there was a regular decrease in their values with the increase in stress. This fluctuation
Fig. 5(b) COMpressive Stress Vs vertical Strain

Fig. 5(a) COMpressive Stress Vs vertical Strain

SYMBOL WALL NO
Θ W-1-6-7
△ W-1-6-7
Fig. 5.9(b)
COMPRESSIVE STRESS
Vs.
VERTICAL STRAIN

SYMBOL WALL NO
○ W-4-17-2
△ W-4-17-3

Fig. 5.9
COMPRESSIVE STRESS
Vs.
VERTICAL STRAIN

SYMBOL WALL NO
○ W-4-18-1
● W-4-18-2
△ W-4-18-3
Fig. 5.10
COMPRESSIVE STRESS
\( \text{Vs.} \)
VERTICAL STRAIN

Fig. 5.11
COMPRESSIVE STRESS
\( \text{Vs.} \)
VERTICAL STRAIN
Fig. 5.14(a)  
COMPRESSIVE STRESS  
Vs.  
VERTICAL STRAIN

Fig. 5.14(b)  
COMPRESSIVE STRESS  
Vs.  
VERTICAL STRAIN
may be due to the presence of shrinkage cracks in the mortar and unevenness of the mortar beds. Fig. (5.15) shows $E - \sigma$ relationship.

Walls of $W-1/6$ series show less fluctuation in the $E$ values at low stresses. There is a consistent decrease in the $E$ value with the increase in stress. In walls of $W-1/3$ series the modulus of elasticity is found to be low because of the use of low strength bricks. As with the walls of other series, there is fluctuation in $E$ value initially which tapers off at higher stress levels. Fig. (5.16) and (5.17) shows the $E-\sigma$ relationship for eccentrically loaded walls.

5.6.4. Lateral deflection

Unlike walls of $W1, B1W, CW$ and $WS$ series discussed in previous chapters, walls of $W-0$ series showed a definite pattern of deflection. Except for walls $W-0-6-2$ and $W-0-12-2$ all walls had deflection curves similar to sine curves. Some of the walls, particularly $W-0-6-3$ and $W-0-12-3$ showed a certain amount of lateral drift at the top at higher loads. The maximum deflection in all the walls except $W-0-18-2$ and $W-0-18-3$ occurred at mid-height. Apart from the non-homogeneous character of brickwork, the existence of some sidesway due to erroneous vertical alignment of the wall at the time of capping are the possible reasons for this lateral drift. Also there was some play in the top platen of the machine which may have contributed to this effect. Although this seems unlikely because once the load is applied the platen is almost fixed. None of these walls belonging to a particular slenderness ratio had the same amount of deflection. Fig. (5.18) to (5.21) shows the deflection curve for different
Fig. 5.17. MODULUS OF ELASTICITY

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Fig. 5.18. LATERAL DEFLECTION MEASUREMENTS

Fig. 5.19. LATERAL DEFLECTION MEASUREMENTS
Fig. 5.20 LATERAL DEFLECTION MEASUREMENTS

Fig. 5.21 LATERAL DEFLECTION MEASUREMENTS
walls of W-0 series.

Almost all the walls of W-1/6 series had their deflection curve similar to sine curve. Except wall W-1/6-6-1, none of the walls had lateral drift at the top. Most of them had approximately the same amount of deflection for a particular slenderness ratio at a particular load. Walls of W-1/3 series also had their deflection curve similar to sine curve. There was complete absence of lateral drift. The amount by which the walls of particular slenderness ratio deflected was not the same. Figs. (5.22) to (5.29) show deflection curves. The location of the section of maximum deflection was the same in walls of same slenderness ratio for both values of eccentricity. And so the ratio of the distance of that section from the topmost end of the beam to the overall length of the wall was nearly constant for all the walls.

5.6.5. Slenderness ratio and compressive strength

As was expected, the squat walls of W-0, W-1/6 and W-1/3 series having a slenderness ratio of 6 had higher ultimate strength than other walls. Unlike walls of WM, BLW and WS series, none of the walls having slenderness ratios greater than 6 had higher compressive strength. Fig. (5.30) shows ultimate strength - slenderness ratio relationship. According to expectation the wall strength decreased with the increase in slenderness ratio and the eccentricity. The results are reasonably consistent and the scatter is not excessive.
Fig. 5.22. Lateral deflection measurements

Fig. 5.23. Lateral deflection measurements
DEFLECTION IN INCHES $\times 10^4$

**Fig. 5.24** LATERAL DEFLECTION MEASUREMENTS

**Fig. 5.25** LATERAL DEFLECTION MEASUREMENTS
Fig. 5.28. LATERAL DEFLECTION MEASUREMENTS

Fig. 5.29. LATERAL DEFLECTION MEASUREMENTS
Fig. 5.30
ULTIMATE STRESS
Vs.
SLENDERNESS RATIO

- ø M0, e = 0
- ø M1, e = 1/6
- ø M1, e = 1/3

Fig. 5.31
ULTIMATE STRESS
Vs.
ECCENTRICITY RATIO
(M0, W1 & W2)

- SL = 6
- SL = 12
- SL = 18
- SL = 25

Fig. 5.32
MASONRY STRENGTH p.s.i. x 10^6

V.S.
WALL STRENGTH
Vs.
BIRDS SECTOR CAPACITY

- T1 = 6
- T1 = 7
- T1 = 8

Fig. 5.33
MASONRY STRENGTH p.s.i. x 10^6

V.S.
PERCENT AVERAGE
The reduction factors for these walls are compared with the various codes in Chapter (7). The test results of axially loaded walls are higher than the values specified in C.P.111. For eccentricity loaded walls R.F. values of C.P.111 are the same as for W-1/6 walls but higher for W-1/3 walls.

Fig. (5.31) shows wall strength and eccentricity relationship. There is a sharp decrease in the strength of eccentrically loaded walls in comparison to axially loaded walls. This decrease, however, is not so sharp in the test results of other investigators. (55)(20)

The main reasons for this sharp reduction in strength between axially and eccentrically loaded walls are as follows:

1) Lower rate of loading.
2) Defects in vertical alignment.
3) Differences in brick strength.
4) Variation in suction properties of brick.
5) Errors involved in obtaining the desired eccentricity while capping the top and bottom ends of the wall.

1) Lower rate of loading:-

Bradshaw (56) studied the effect of the rate of loading on the ultimate strength of walls and showed that the walls loaded over 1\(\frac{1}{2}\) hours failed at loads approximately 20% less than the walls loaded over \(\frac{1}{2}\) an hour. As mentioned in Section 5.4.3, the rate of loading in the case of axially loaded walls of W-0 series was two times more than W-1/6 walls and four times more than that of W-1/3 walls. This decrease in the rate of loading for the eccentrically loaded
walls was because the time consumed in taking the measurements was the same for both axially and eccentrically loaded walls, whereas the failure load decreased with the increase in eccentricity. This resulted in lower rate of loading for the eccentrically loaded walls and hence lower ultimate strength.

2) Defect in vertical alignment:

Due to the practical limitation of workmanship, the tendency for large lateral deflections to occur is accentuated in the case of eccentrically loaded slender walls. This results in reduced effective area for resisting the applied load and hence reduction in strength.

3) Brick strength:

The strength of bricks used in the walls of W-1/3 series was lower than those used in W-0 and W-1/6 walls. Also the dimensions of the bricks of W-1/3 walls were different from those of W-0 and W-1/6. Thomas tests have shown that walls built with weak brick and weak mortar have greater reduction in strength with increase in slenderness ratio than walls built with strong brick and strong mortar. A similar conclusion has been drawn by Haller. Although the mortar used in W-0 and W-1/6 walls are the same as used in W-1/3 walls, due to lower strength bricks used in W-1/3 walls, there is reduction in strength.

This greater decrease in strength with the increase in slenderness and eccentricity when weaker materials are used may be connected with lower EI value which would result in lower failure load. Swiss Norm takes this effect into account and gives different reduction factors for the different grades of brickwork.
4) **Suction properties of bricks**

Haller's tests have shown that the suction of the bricks when laid can considerably influence the strength of the brickwork especially when dense cement mortars are used (such as 1:3 cement:sand mix) and the wall is eccentrically loaded. Fig. (5.32) shows decrease in strength of wall with the increase in suction. This is because high suction bricks absorb an excess of water from the mortar and hence prevent complete hydration of the cement, and may result in lack of bond between brick and mortar. With cement lime mortar this effect is relatively less pronounced because of the water retaining property of the lime. The test results of W-1/6 and particularly of W-1/3 walls should be studied keeping this aspect in view. The mode of failure of these walls shows failure generally at mid-height due to excessive lateral deflection. This maximum deflection results in break of bond between the mortar and the brick. Absorption tests on the bricks (Table (5.1)) show that these model bricks used in the walls have different suction capacities. In order to reduce their suction capacity, the bricks were soaked in water for a fixed period before being laid. The bricks still had different suction capacities at the time of laying in spite of the constant soaking time, but suction capacity was uniformly reduced.

5) **Errors involved in obtaining desired eccentricity:**

The strain measurements confirm that the eccentricity of loading is greater than the intended eccentricity: This is because the thickness of the wall being small, there is every likelihood of errors cropping up while placing the
loading beam no matter how precisely one attempts to mark and position the beam and the wall.

5.6.6. Comparison of test with other investigators

Unfortunately there are very few test results available from other sources for eccentrically loaded walls of different slenderness ratio. Most of the test results available do not have the same parameters as the authors tests. However the few test results of other investigators which are near enough to the author's tests are compared. Since the brick strength and the mortar strength are different for different tests, the comparison has been made in terms of dimensionless reduction factor.

Fig. (5.33) shows a comparison of test results carried out by SCPI(66), Haller(15) and the author. The smallest wall of SCPI tests had a slenderness ratio of 3.7. There were walls of slenderness ratio 6.6 and 6.7 as well. In calculating the reduction factors the basic stress has been taken as the average of the ultimate stresses of walls of slenderness ratio 3.7, 6.6 and 6.7. In the authors test the lowest wall is of slenderness ratio 6.0. This makes the comparison between the two more reasonable, and the reduction factor is based on the ultimate stress of this wall of slenderness ratio 6. With the exception of walls of slenderness ratio 22.6, the reduction factors of other SCPI test walls are about 10% lower than those of W-0 walls.

The eccentrically loaded walls of W-1/6 series have nearly the same reduction factor as the SCPI test values, whereas W-1/3 walls have a reduction factor which is approximately 25% higher than SCPI tests. For all the three walls
Fig 5.33
COMPARISON OF TEST RESULTS

- W0 e=0
- Wx e=1/6
- Wx e=1/3
- e=0
- e=0/6
- HALLER'S TESTS
- e=1/6
- e=1/3

NOTE: 1. R.F calculated by taking S.R=12.4
   - ALLEN RESULTS.
   - 2. S.R = 9. BASIS for GRAVE & MOTTIE TEST

Fig 5.34
COMPARISON OF TEST RESULTS

- W0 e=0
- Wx e=1/6
- Wx e=1/3
- e=0
- e=0/6
- e=1/3
- e=0
- WATSTEIN & ALLEN
- GRAVE & MOTTIE

SLENDERNESS RATIO
of W-0, W-1/6 and W-1/3 series, the maximum slenderness ratio did not exceed 25.0, whereas the SCPI tests had walls up to slenderness ratio of 46.1. In actual practice single leaf walls do not have slenderness ratios more than 30. Testing walls beyond a slenderness ratio of 30 would therefore seem more a matter of academic than of practical interest.

Haller's test results are about 18% lower than the author's test results for the walls of W-0 series. In the case of walls with eccentricity of loading e = t/6, Haller's test results are lower than the author's results. The difference between the two is less up to slenderness ratio of about 19, beyond which the difference somewhat increases. Haller had a maximum slenderness ratio of 35.0.

Fig. (5.34) shows the results of the test carried out by Watstein and Allen the author, and Grave and Motteu. The lowest wall in Watstein and Allen's tests had a slenderness ratio of 12.4 and so the reduction factor is based on the ultimate strength of this wall. Similarly the lowest wall in the Grave and Motteu test had a slenderness ratio of 9.0. Unlike the author's test, the eccentricity of loading in the Watstein and Allen test was only at the top of the wall and the bottom end of the wall was loaded axially. While comparing the results, these deviations in the test should be kept in view. The reduction factor of Watstein and Allen's walls for all the three types of loadings are higher than the author's test walls. This was expected because of the difference in calculating reduction factor and the load eccentricity. The reduction factor in Grave and Motteu's tests are all higher than the author's
wall of W-O series. In this test the wall having the lowest slenderness ratio of 9.0 was a bonded wall of 11.5 ins. thickness. The remaining walls were single leaf walls of 5.5 ins. thickness. The strength of the wall is found to decrease with the increase in thickness. This reduction in strength becomes more pronounced if it is a bonded wall. This is due to increased percentage of mortar in the wall due to the addition of an internal vertical joint (Collar Joint). Haller's test shows that the strength of an axially loaded 5 ins. single leaf wall can be 48% greater than that for a 10 ins. bonded wall when the slenderness ratio of both are equal to 10. This is the reason for the Motteu's test walls having higher reduction factor.

Differences in rate of loading and method of testing etc. may be responsible for the variation in results of different tests.

Although Grave and Motteu tested a number of walls they changed various parameters such as type of loading, end condition and thickness thus restricting comparison of their remaining test results with those of the author.

Prasan et al. tests on single leaf 4½ ins. thick full scale brick walls eccentrically loaded between R.C. slabs showed that the reduction in strength of walls resulting from eccentric loading when loaded between R.C. slabs is noticeably less than when loaded between knife edges. Most of the tests carried out so far to study slenderness effect on the strength of eccentrically loaded walls have been on walls with hinged ends. This is very different from the
actual conditions encountered in a building. Provision of R.C. slabs simulates more closely the end conditions of walls in an actual building and therefore it is desirable to study the effect of R.C. slabs at the wall ends. In this thesis tests on the walls loaded between R.C. slab with different slenderness ratio under axial load have already been described in Chapter (4).

5.7. CONCLUSIONS

(1) Axially loaded walls of W-0 series failed partly due to tensile splitting and partly due to bending depending on the slenderness ratio. Eccentrically loaded walls of W-1/6 and W-1/3 series failed mainly due to excessive bending indicating stability induced failure.

(2) The eccentricity of loading does not seem to affect the modulus of elasticity of the wall. A similar conclusion has been reached by Wattstein and Allen. The value of the elastic modulus decreased with the increase in stress.

(3) Unlike walls of Wm, BlW, WS and CW series, all the walls of this series (W-0, W-1/6 and W-1/3) showed a regular deflection shape which approximated to a sine curve.

(4) As was expected, there is decreasing trend in the ultimate strength of brick walls with increase in slenderness ratio. Other available test results confirm this trend. The amounts by which the strength reduces is different for tests carried out by different investigators.

(5) Walls tested between R.C. slabs will give more realistic values of ultimate strength and reduction factors as compared with walls with hinged ends. Chapter (4) describes such walls with axial load only and not with eccentric load.
CHAPTER 6
THEORETICAL ANALYSIS

6.1. GENERAL

The objective of this chapter is to analyse various theories related to the topic of this thesis and to compare them with the author's test results in order to verify their validity.

Various theories related to slenderness effect and compressive strength are based on the stress-strain relationship of the masonry. Thus the interaction between the deformation of the units and mortars is included in the analysis. This stress-strain property of the masonry can either be

(i) Actual stress-strain curve obtained from axial loading test results, or
(ii) Assumed stress-strain curve which can be a linear or a non-linear relationship between stress and strain.

6.2. ASSUMPTIONS

1) Cross-sections normal to the axis of wall stay plane after load deformation is applied.
2) Wall deflects in the form of a sine-curve and is given by the equation $y = f \sin \left(\frac{\pi x}{h}\right)$.
3) Masonry has no tensile-resistance.

The deflection curve of the wall could equally be of parabolic shape, or the arc of a circle. The sine curve is chosen because it is easy to handle mathematically. Because of this assumption the wall will have maximum
deflection and curvature at mid-height and hence the load carrying capacity is defined by conditions at mid-height.

In the light of the above assumptions, various theories are discussed in the following sections.

In all these theories two cases are considered:
(i) When load is outside middle third,
(ii) When load is inside middle third.

In addition to these two cases, Angervo\(^{(70)}\)(71) included a case in which the load is within the kern at one end but outside the kern at certain other parts of the wall.

6.3. HALLER'S THEORY\(^{(15)}\)

Unlike other theories which will be discussed later, Haller's theory is based on the actual stress-strain curve obtained from pier tests of slenderness ratio 5. This particular value of slenderness ratio is chosen because at slenderness ratio less than 4.0, the mode of failure of the pier becomes diagonal shear instead of the characteristic mode of failure of vertical tensile splitting perpendicular to the bed joint. The diagonal shear failure results in higher apparent masonry strength.

A masonry wall of height h, thickness t and of unit width subjected to eccentric load is considered. Fig. (6.3.1a).

Referring to Fig. (6.3.1a, b & c), from the geometry and the principle of equilibrium a relationship between a parameter \( n \) expressing distance of neutral axis and other parameters is obtained and is as follows:

\[
 n = \frac{1}{2} \frac{t/2 - e}{1 - g} + \sqrt{\frac{1}{4} \left(\frac{t/2 - e}{1 - g}\right)^2 - \frac{\varepsilon_1 h^2}{(1 - g) \pi^2}} \quad \text{Eq.(6.2.1)}
\]
CASE II NA. OUTSIDE WALL THICKNESS

6.3.1. (d)

CASE II NA. OUTSIDE WALL THICKNESS
The value of \( n \) for a wall of particular slenderness ratio is calculated by choosing a value of strain \( \varepsilon_1 \) and the corresponding value of \( u \) from the stress-strain curve.

The second term \( \frac{\varepsilon_1 h^2}{\pi^2} \) in the above equation becomes large if slenderness ratio is large and \( \varepsilon_1 \) chosen is large. This means that at high slenderness ratio the failure strain should be very small in order that the terms under the square root sign remain positive and hence a real solution is possible.

The compressive stress \( \sigma \) is then calculated from equation

\[
\sigma = \frac{F}{\varepsilon_1} \cdot \frac{n}{t} \hspace{1cm} \text{Eq. (6.2.2)}
\]

In the same way a number of trial values of \( \varepsilon_1 \) are chosen and the corresponding value of \( \sigma \) is calculated. A curve between \( \sigma \) and the corresponding \( \varepsilon_1 \) is plotted. The highest point on the curve gives the bearing capacity of the wall under consideration.

When the load is inside middle third then the stress distributed over the thickness of the wall will be as shown in Fig. (6.3.1d). In this case in addition to solving equation (6.2.1) equation (6.2.3) is to be satisfied by

\[
n = \varepsilon_1 \frac{t}{\varepsilon_1 - \varepsilon_2} \hspace{1cm} \text{Eq. (6.2.3)}
\]

choosing an appropriate value of \( \varepsilon_2 \). Once the \( n \) is obtained the stress is calculated from equation (6.2.2). The ultimate stress or bearing capacity is then calculated as described above.
6.3.1: **Discussion**

The value of $\varepsilon_1$ at which the maximum stress $\sigma$ is obtained is the failure strain of the wall. This value explains the type of failure. If the value of $\varepsilon_1$ is greater than the ultimate strain $\varepsilon_*$ then the failure is a strength failure, indicating that the full strength of the wall has been utilised. This type of failure is common with less slender and concentrically loaded walls.

If $\varepsilon_1$ is less than $\varepsilon_*$, then failure is due to lack of stability indicating that full strength of masonry has not been utilised, and is very common with very slender and eccentrically loaded walls. Fig. (6.3.2.) explains the two modes of failure and the effective regions of the stress-strain curve.

6.4. **TURKSTRA'S THEORY**\(^{(24)(88)}\)

Unlike Haller, Turkstra assumes a non-linear stress-strain curve of parabolic shape given by equation

$$\sigma = E_1\varepsilon \left(1 - \frac{\varepsilon}{2\varepsilon_*}\right)$$

where $\varepsilon$ is $\leq \varepsilon_*$

The degree of non-linearity is defined by a parameter $k$ which varies between 1.0 indicating a linear stress-strain relationship to 2.0 indicating a non-linear relationship (parabolic shape).

Referring to Fig. (6.3.1) a relationship between wall strength and slenderness ratio of a pin ended wall is developed for the two locations of the neutral axis (i) Neutral axis within the thickness of the wall (ii) Neutral axis outside the thickness of the wall.
IN STRENGTH RANGE $\varepsilon = \varepsilon_x$

FAILURE STRAIN $\approx$ ULTIMATE STRAIN $\varepsilon_x$

FAILURE STRESS $\approx$ ULTIMATE STRESS

$\sigma_{\text{max}} = \sigma_{\text{ult}}$ (MASONRY STRENGTH)

BUCKLING RANGE $\varepsilon = \varepsilon_x$

FAILURE STRAIN $\approx$ ULTIMATE STRAIN

$\sigma_{\text{max}} < \sigma_{\text{ult}}$

FAILURE STRESS $\approx$ ULTIMATE STRESS

FAILURE DUE TO EXCESSIVE BENDING OR INSTABILITY

Fig 6.3.2, MODES OF FAILURE
This relationship is shown in Fig. (6.4.1) in terms of a non-dimensional parameter \( \alpha = \frac{h}{t} \sqrt{\sigma_{\text{ult}} E_i} \) (and is completely independent of any strain which the theoretical derivation includes) and relative wall strength \( \frac{P}{t \sigma_{\text{ult}}} \).

6.4.1. Discussion

It can be seen from Fig. (6.4.1) that the effects of stress-strain relationship are opposite for low and intermediate values of \( \alpha \). For a low \( \alpha \) greater non-linearity leads to greater strength while for intermediate values greater non-linearity leads to relatively less strength. For high slenderness ratio, all stress-strain curves yield nearly equal relative capacity.

An important characteristic of all the cases studied is the relatively rapid decrease of strength with height in the neighbourhood of \( \alpha = 1.0 \).

6.5. Monk's Theory (25)

Monk's approach to the study of slenderness effect is similar to that of Turkstra except in the method of calculating the load from the stress-strain diagram. Also, Monk's analysis takes into account the additional eccentricity due to excessive bending of wall or column. With the help of the moment area method a relationship between failure stress and slenderness ratio is developed and is given as

\[
\sigma = \frac{12 E_i q \pi^2}{(q \pi^2 G + 1)(h/t)^3}
\]

Thus the failure stress is dependent upon elastic modulus, slenderness ratio and a geometric function \( G \). The function \( G \) depends on the eccentricity, the lateral deflection
and the end conditions which in turn depend on the values of $q$.

6.6. ANGERVO'S THEORY \((70)(71)\)

The approach in Turkstra's and Haller's theory has been to approximate the deflection curve of the wall by some suitably chosen analytical curve. After this the conditions of equilibrium between external forces and stresses developed at the mid-height of the wall cross-section are satisfied. Angervo formulates and then solves the differential equation by taking secondary deflection into account as well.

Referring to Fig. (6.6.1) the differential equation for the three cases of load eccentricity can be formulated.

i) When load is outside the kern

$$\frac{d^2V}{dx^2} = - \frac{2P}{9Eb(t/2-V)^3}$$

ii) When load is inside the kern

$$\frac{d^3V}{dx^3} = - \frac{P.V}{EI}$$

iii) When load is within the kern at an end of the wall but outside in some parts of the wall

$$\frac{d^2V}{dx^2} = \frac{P.V}{EI}$$

Based on the solution of the above equations Angervo\((69)(70)(71)\) prepared curves for calculating ultimate strength of walls. Sahlin\((69)\) also prepared several diagrams for the walls relating their slenderness ratio and ultimate strain with strength ratio for the different ratios of end eccentricity (Fig. (6.6.1b)).
Fig. 6.6.1(c)
SAHLIN'S CURVE

- $\sigma_{\text{MT}}$: Mean Stress
- $\sigma_{\text{UT}}$: Ultimate Stress
- $\varepsilon_{\text{f}}$: Strain at Failure

---

(a) WALL IN DEFLECTED SHAPE

(b) EFFECTIVE THICKNESS OF WALL

---

Consider the graph with the following axes:
- $x$-axis: $h/\sqrt{E_h}$
- $y$-axis: $\sigma_{\text{MT}} = \Delta \sigma_{\text{MT}}$

Key values for $m = 0.25$: $t/24$, $0.5$, $t/12$, $1.0$, $1.25$, $t/6$, $1.5$, $1.75$, $2.0$, $2.75$, $2.5$, $2.25$, $t/3$.
6.7. COMPARISON OF TEST RESULTS

6.7.1. WM and B1W Walls

Fig. (6.7.1a, b & c) shows comparison between test results and the theories for WM and B1W walls.

Turkstra's and Angervo's theories underestimate the wall strength. The difference between theory and experiment is approximately between 18% and 33%. There is good agreement between Monk's theory and the test results except for the walls B1W18 in which the theory underestimates the strength by 39%. This particular group of walls had higher strength than the walls of lesser slenderness ratio which was contrary to the expected behaviour. Better workmanship and better distribution of load are the reasons for this increase.

6.7.2. WS Walls

Fig. (6.7.1c) and Fig. (6.7.1d) show comparison between the theories and experiment for WS walls. There is fairly good agreement between the two. The difference varies from 5% to 30%. In general theory underestimates the wall strength.

6.7.3. CW Walls

Fig. (6.7.1e)

Light weight concrete block walls are in reasonably good agreement with Turkstra's and Angervo's theories except for the wall CW29. Similar to B1W18 walls, the walls of CW29 group had higher load bearing capacity than walls of lesser slenderness ratio. These walls of greater height were built in wooden jigs; this has resulted in better vertical alignment and hence higher strength. Monk's theory overestimates experimental wall strength. The maximum variation being 35% for CW24 wall. The main
Fig. 6.7.1(a) COMPARISON OF TEST WITH THEORIES FOR WALLS
Fig. 6.7.1(b) COMPARISON OF TEST WITH THEORIES FOR B.I.W. WALLS
Fig. 6.7.1(c) COMPARISON OF TEST WITH MORRIS'S THEORY

- B.W.
- WS
- WM
- CW
Fig. 6.7.1 (d)
COMPARISON OF TEST WITH THEORIES
WS WALLS

Fig. 6.7.1 (e)
COMPARISON OF TEST WITH THEORIES CW WALLS
difference between the theory and the experiment is due to 
the variation in the elastic modulus, as will be explained 
later in Section 6.8.

6.7.4. W-0, W-1/6, W-1/3 Walls

Fig. (6.7.2) compares theoretical and experimental 
results for the walls with hinged ends.

Walls of W-0 series which are tested under axial 
loading show a lower bearing capacity than the theories 
predict at the slenderness ratio of 6 and 12. For 
slenderness ratio of 18 and 25 the difference between the 
theories and experiment is small. Of all the theories, 
Angervo's theory show good agreement with the test results.

For walls of W-1/6 series loaded with eccentricity 
e = t/6 there is good agreement of all the three theories 
except at the slenderness ratio 6 and 25 where the difference 
between the two is high.

There is marked difference between the theories and 
experiment for walls of W-1/3 series having load eccentricity 
of t/3. For such a severe eccentricity, Turkstra's theory 
does not permit walls beyond slenderness ratio of 15.0. 
Monk's and Angervo's theories do not have such limits but 
the wall strength tapers down to almost zero above slenderness ratio of 18.0. In view of the assumptions involved in 
deriving the various equations this wide difference does 
not seem to be unexpected.

6.8. DISCUSSION

In the case of flat ended walls the basis for calculating 
slenderness ratio by Monk is different from that of 
Turkstra and Angervo and so the test results are compared 
separately.
Fig. 6.7.2.
COMPARISON OF TEST WITH THEORIES
FOR HINGED WALLS.

- MONK
- ANGERVO
- TURKSTRA

X e = 0 W - o
X e = t/6 W - ½
X e = t/3 W - ⅓
Turkstra's theory gives higher strength of WM, BlW and CW walls as compared with Angervo's theory. There are three main reasons for this: 1) Difficulty in estimating values used in a non-dimensional parameter $\alpha$, 2) Difference in the shape of the stress-strain curve, 3) Effect of secondary deflections is taken into account in Angervo's theory but neglected in Turkstra's theory.

From the above comparison of the author's test results with theories it is seen that the amount by which the theory and experiment differ from each other is variable. For some walls theory and experiment correlate well, whereas for some other walls the difference may go as high as 53%. There are several reasons for this variation. In the case of Turkstra's theory the important reason in the authors view for the walls of WM, BlW, WS and CW series is the choice of effective height in the calculation of non-dimensional parameter $\alpha$.

As mentioned earlier Turkstra's theory is based on walls with hinged ends and so the relationship between strength ratio $P/t \sigma_{ult}$ and non-dimensional parameter $\alpha$ (Fig. (6.4.1) is based on actual height $h$.

In the case of walls of WM, BlW, CW and WS series there is lateral end restraint provided by the steel platens of the machine (R.C. slab in the case of WS walls). This will result in shifting of hinges along the height of the wall. Therefore the height for WM, BlW and CW walls has been taken as 0.9 $h$. This value of 0.9 $h$ is very approximate and is based on the trends observed in the lateral deflection curves of the walls and the recommendation of Australian
Code of Practice. For WS walls, the height is taken as 0.75 h in the calculation of \( \alpha \) and is based on C.P.111. This value seems to be reasonable since for a wall with complete fixity the height is 0.5 h. This approximation in the choice of effective height \( h \) for calculating \( \alpha \) is one of the main reasons for the variation in theoretical and experimental values.

In the case of Angervo's theory the error involved in estimating the value of the ultimate strain \( \varepsilon_u \) (which is unknown) is the most important factor, contributing to this variation. Because of experimental limitations it has not been possible to measure the ultimate strain and so the value has to be approximated from the available stress-strain curves for squat walls. These curves generally terminate at 60% of the failure load and so the curve is not well defined beyond that point. The relationship between stress and strain near failure load will be different from that at lower load. To extend these curves on the basis of their initial shape is likely to involve errors. From the Fig. (6.6.1b) it is seen that the strength of walls is very sensitive to variation in ultimate strain especially at high slenderness ratio as a small error in estimating \( \varepsilon_u \) will result in a relatively higher error in the ultimate strength.

For the axially loaded wall, the curve for an eccentricity of loading \( t/24 \) (m = 0.25) was used in Fig. (6.6.1b). This is an overestimation of the eccentricity for most of the walls, because from strain measurements Tables (3.4.) (3.5.) and (4.2.) of previous chapters, it is seen that actual eccentricity in most of the walls is less
than assumed by the eccentricity of $t/24$. This assumption will also reduce the theoretical strength of the walls.

6.9. CONCLUSIONS

All the important theories relating wall strength to slenderness ratio discussed above are found to be approximate partly because of the assumption that the masonry has no tensile resistance. If however the tensile resistance of the masonry is taken into account then the theory becomes complicated and it would be cumbersome to calculate the strength. In actual practice, codes give reduction factors to take care of the slenderness effect of the wall. These reduction factors are based on test results in which conditions prevailing in actual buildings are simulated. Hence the presence of tensile resistance along with other related accidental factors are taken into account. Since in the code the basic permissible strength is based on the strength of a particular type of masonry (depending upon unit and mortar strength) the prediction of wall strength by using code recommendations tend to be more realistic than by theoretical analysis. Hence the design of slender masonry walls based on theoretical methods is mainly of academic interest. However they are of value in developing an understanding of the factors controlling the strength of walls.

Of all the theories discussed above, Turkstra's and Angervo's seem to be the ones which are nearest to the test results, and are also easiest to use. Turkstra takes into account the shape of the stress-strain curve by using
a factor $k$. In Angervo's theory, if the ultimate strain can be accurately estimated, then it becomes simple to predict a wall strength which is near enough to the test results.

With the exception of the Swedish Code (62) most of the codes have a similar and simple approach to the design. Chapter (7) discusses the details of the design codes of various countries in the light of British Code C.P.111 (1970).
CHAPTER 7

A STUDY OF THE PROVISION FOR SLENDERNESS EFFECT IN DESIGN CODES

7.1. INTRODUCTION

Studies of design procedure laid down in various codes have been made by various authors(75)(76)(77)(83) including Gross and Dikkers,(78) Macchi(79)(80) and Shelbach.(60) Motteu(81) studied the design procedure of the Belgian code. Bradshaw and Foster(82) studied design methods and their basis as laid down in C.P.111(1948)(64) and (1964). None of these studies concentrated exclusively on reduction factor-slenderness ratio relationship laid down in various codes.

In this chapter an attempt to study reduction factor-slenderness ratio relationship of the various codes has been made with the help of available test data. The reduction factors given in American, Canadian, Australian, Swiss and British codes of practice are compared and discussed. Also the permissible loads for various eccentricities and slenderness ratios are compared. Permissible loads of the various codes have been calculated keeping in view the fact that different codes have different ways of arriving at basic stresses, and also to some extent the codes have different approaches in specifying the reduction factors, and also a different quality of workmanship. With these limitations in mind the design provisions of various codes have been examined and compared.

7.2. U.S.A. Code - SCPT(66)

According to this code the permissible load P, is a
wall depends upon four variables:

1) The eccentricity coefficient $C_e$,

2) The slenderness coefficient $C_s$,

3) The ultimate compressive stress $f_m$, and

4) The gross cross-sectional area $A_g$.

In symbols $P$ allowable $= C_e C_s f_m A_g$,

where

a) $C_e = 1$ when $e \leq t/20$

b) $C_e = \frac{1.3}{1 + 6 \frac{e}{t}} + \frac{1}{2} \left( \frac{e}{t-1/20}(1-e_1/e_2) \right) $ ... Eq. (7.2.1a)

when $e > t/20$ and $e \leq t/6$.

c) $C_e = 1.95 \left( \frac{3}{e} - \frac{e}{t} \right) + \frac{1}{2} \left( \frac{e}{t-1/20}(1-e_1/e_2) \right) ...$ Eq. (7.2.1b)

when $e > t/6$ and $e \leq t/3$.

$C_s$ - The slenderness coefficient is calculated by means of the following formula.

$C_s = 1.2 - \frac{h/t}{300} 5.75 + (1.5 + e_1/e_2)^2 \leq 1.0 ...$ Eq. (7.2.2)

$e_1$ - smaller eccentricity either at top or bottom

$e_2$ - larger eccentricity either at top or bottom.

Unlike the British, Australian and Swiss codes, this code considers end conditions in detail. The slenderness ratio in the code is defined as the ratio of effective height ($h$) to effective thickness ($t$) and should not be greater than $h/t \leq 10(3 - e_1/e_2)$ where $e_1/e_2$ is positive where member is bent in single curvature and negative when member is bent in double or reverse curvature. If $e_1$ or $e_2$ is zero, then $e_1/e_2$ will also be zero.

It will be seen that the maximum permitted slenderness ratio is 20 when the wall is loaded with the equal eccentricity top and bottom on the same side (i.e. absence of reverse curvature). A maximum value of $h/t = 40$ is permitted
when the ends of the loaded wall are fixed or when the wall is loaded with equal eccentricity top and bottom in opposite directions (i.e. presence of reverse curvature).

The various end conditions and eccentricity ratios envisaged in the code are shown in Fig. (7.2.1).

The eccentricity $e_1$ is taken as the smaller virtual eccentricity at the top or bottom support.

The effective height, unlike the British and Australian codes, is not altered because of different end conditions except in the case when the top end of the wall is free and the bottom fixed; for all other cases when the wall is laterally supported at both ends, the effective height is taken as the actual height of the wall. The effect of end conditions is taken care of in the slenderness and eccentricity coefficients which are dependent upon the eccentricity ratio $e_1/e_2$. The eccentricity ratio $e_1/e_2$ in turn depends upon the end conditions of the wall.

The slenderness and eccentricity coefficients proposed in the code are based on the tests carried out on walls under different end conditions. From a study of eccentricity coefficient equations (7.2.1) it is seen that the reduction in the strength of wall not only depends on the magnitude of eccentricity but also on the type of bending, i.e. single or double curvature. Test results (66) on which these equations are based confirm this trend.

The eccentricity coefficient $C_e$ takes care of non-uniform stress distribution for the eccentrically loaded walls so that no increase in the basic stress is allowed; this differs from British and Australian design practice.
Fig. 7.2.1(a) COMPARISON OF TESTS WITH U.S. CODE

HINGED WALL $e_1/e_2 = +1$

<table>
<thead>
<tr>
<th></th>
<th>AUTHOR'S</th>
<th>HINGED WALL $e_1/e_2 = +1$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$W-5$</td>
<td>$e = t/6$</td>
<td></td>
</tr>
<tr>
<td>$W-5$</td>
<td>$e = t/3$</td>
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</tr>
<tr>
<td>SCPI</td>
<td>$e = t/6$</td>
<td></td>
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<tr>
<td>SCPI</td>
<td>$e = t/3$</td>
<td></td>
</tr>
</tbody>
</table>

US CODE $e = t/6$

CP111 $e = t/6$

US CODE $e = t/3$

CP111 $e = t/3$

LOAD REDUCTION FACTOR

END CONDITIONS
7.2.1. Comparison of Test Results with Code Reduction Factors

7.2.1.1. SCPI and the Author's tests in relation to the American and British codes.

The test results for eccentrically loaded hinged walls having eccentricity ratio $e_1/e_2 = \pm 1$ are compared in Fig. (7.2.1). The SCPI test results are higher than the code values. The reduction factor values have been calculated on the basis of prism tests having slenderness ratios between 2.5 and 4. The author's test results for W1/6 walls also have higher values than the code values. For the walls of W1/3 series having eccentricity of loading $e = t/3$ the SCPI test values are more or less in agreement with the code values except for walls of slenderness ratio 14.0, where the test result is lower than the code value. The author's test results are lower than the code values for all slenderness ratios. The reason for this is, as has been mentioned in Chapter (5), that low strength bricks were used, and also the eccentricity of loading was more than the desired eccentricity of $t/3$.

The reduction factors of C.P.111(1970)(46) seem to be nearly the same except at slenderness ratio of 12 or over, where the difference in values is noticeable. The lowest wall used in the British code is of slenderness ratio 6 and in American code is of 5. When this is taken care of the difference between the two is reduced.

Fig. (7.2.2) shows comparison of test results with the code for the axially loaded walls with hinged ends, corresponding to eccentricity ratio of $e_1/e_2 = 0$. The
SCPI\(^{(66)}\) test results and the author's are higher than the code specification. There is a slight difference between the American code and the British code. If a basic slenderness ratio of 6 is taken in the American code then the difference between the two is reduced.

Fig. (7.2.3) shows test results for fixed end walls under axial loading corresponding to eccentricity ratio \(e_1/e_2 = -1\). There is a large scatter of test results for the SCPI test. Except at slenderness ratio of 10 and 12 the other walls have higher values than the code specification. British code C.P.111(1970)\(^{(46)}\) values are lower than the test results. Except for the wall of slenderness ratio 18, the author's test results are also higher than the American code values. There is a difference of about 5% between American and British code, when the slenderness ratio of C.P.111 is multiplied by 4/3 to make it comparable with the American code. In the case of flat ended walls where the fixity is not as complete as in the case of walls with R.C. slabs, the slenderness ratio is divided by 0.9. Reduction factors for these walls are about 10-15% lower than those given in the American code. This difference is understandable considering the relatively lower degree of fixity.

Fig. (7.2.4) shows the reduction factors for the case when top and bottom eccentricity are in opposite faces, thus leading to deflection of wall in double curvature. It will be seen that the values in this case are higher than they would be when top and bottom ends are fixed.

Similarly Fig. (7.2.5) gives the reduction factor when the top end of the wall is hinged and the bottom
Fig. 7.2.4.
COMPARISON OF TEST (SCPI)
WITH US. CODE
REVERSE CURVATURE \( \varepsilon_1/\varepsilon_2 = -1 \)

\[
\begin{align*}
\text{--- } & \varepsilon_1 = 1/20 \\
\text{-- } & \varepsilon_1 = 1/6 \\
\text{-- } & \varepsilon_1 = 1/3 \\
\end{align*}
\]

LOAD REDUCTION FACTOR

SLENDERNESS RATIO

Fig. 7.2.5.
COMPARISON OF TEST (SCPI)
WITH US. CODE
TOP HINGED, BOTTOM FIXED
\( \varepsilon_1/\varepsilon_2 = -1/2 \)

\[
\begin{align*}
\text{○ } & \varepsilon_1 = 1/20 \\
\text{△ } & \varepsilon_1 = 1/6 \\
\text{△ } & \varepsilon_1 = 1/3 \\
\end{align*}
\]

LOAD REDUCTION FACTOR

SLENDERNESS RATIO
end is fixed. The values of reduction factors in this case are less than the values for the case when $e_1/e_2 = -1$ for both types of end conditions.

7.3. CANADIAN CODE (1970)

As in the SCPI code the allowable vertical load on a wall or column is calculated as

$$ P = C_e C_s f_m A_n \quad \text{Eq. (7.3.1)} $$

where $C_e$, $C_s$ and $f_m$ have the same meaning as those in SCPI.

$A_n$ - Net cross-sectional area

$C_e$ - The eccentricity coefficient is

a) $C_e = 1.0$ for $e \leq t/20 \quad \text{Eq. (7.3.2a)}$

b) $C_e = \frac{1}{1 + \frac{6}{t} e/t}$ for $e > t/20$

$$ e \leq t/6 \quad \text{Eq. (7.3.2b)} $$

c) $C_e = \frac{2}{t}(1 - \frac{2e}{t})$ for $e > t/6$

$$ e \leq t/3 \quad \text{Eq. (7.3.2c)} $$

d) $C_e = \frac{1}{6e/t - 1}$ for $e > t/3 \quad \text{Eq. (7.3.2d)}$

$C_s$ - slenderness coefficient

$$ C_s = 1 - C_b (h/t - 5) \quad \text{Eq. (7.3.3a)} $$

where $C_b = 0.003 (e_1/e_2)^2 + 0.012(e_1/e_2) + 0.025 \quad \text{Eq. (7.3.3b)}$

Maximum value of slenderness ratio permitted in the code is

$$ h/t \leq 10 \left[ 3 - (e_1/e_2) \right] \quad \text{Eq. (7.3.4)} $$

This is exactly similar to SCPI code. According to this equation the maximum value of $h/t$ permitted varies between 20 and 40 depending upon the eccentricity ratio $e_1/e_2$. This code is also explicit about position of eccentricity and hence the end condition. The effective height is taken as the actual height.
7.3.1. Comparison of test results with the code

7.3.1.1. Axially loaded walls (CW, WM, BlW, WS)

The test values for BlW series are higher than the code values. Similarly the values for WM and CW series are also higher than the code values. Except for WS18 wall, the remaining walls of WS series have higher reduction factors. The C.P.111(46) value for the axially loaded walls are nearly the same as for Canadian values, Fig. (7.3.1).

7.3.1.2. Hinged walls axially and eccentrically loaded (WO, W1/6, W1/3)

In the case of hinged walls the test values are greater than those specified in Canadian code for the axially and eccentrically loaded wall of e = t/6 eccentricity. For the wall of W1/3 series having eccentricity of loading e = t/3 the test values are nearly the same as given in the Canadian code.

The reduction factor given in C.P.111(46) is similar to the values given in the Canadian code for axially loaded walls. But for hinged walls the C.P.111(46) values are 25% higher than Canadian values, Fig. (7.3.2).

7.4. AUSTRALIAN CODE(63)

According to Australian practice, the allowable vertical compressive load (when eccentricity of load is less than t/24 and the eccentricity is the same top and bottom) is calculated as

\[ P_a = K_a (0.2 F_m') A_g \]  \quad \text{Eq. (7.4.1)}

For the eccentricity of vertical load greater than t/24, the allowable vertical compressive load is given by equation
Fig. 7.3.1.
COMPARISON OF TEST (AUTHOR'S) WITH CANADIAN CODE
CANADIAN $e_i / e_s = -1$

Fig. 7.3.2.
COMPARISON OF TEST WITH CANADIAN CODE

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<thead>
<tr>
<th>Test</th>
<th>Load Reduction Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>CP 111 $e = 0$</td>
<td>1.0</td>
</tr>
<tr>
<td>CANADA $e = 0$</td>
<td>0.8</td>
</tr>
<tr>
<td>CANADA $e = t/6$</td>
<td>0.6</td>
</tr>
<tr>
<td>CP 111 $e = t/6$</td>
<td>0.4</td>
</tr>
<tr>
<td>CP 111 $e = t/3$</td>
<td>0.2</td>
</tr>
<tr>
<td>CANADA $e = t/3$</td>
<td>0.0</td>
</tr>
</tbody>
</table>

SLENDERNESS RATIO
where \(K_a\) and \(K_e\) are the stress reduction factors and are functions of slenderness ratio and eccentricity.

\[ P_e = K_e \left(0.25 \frac{F_m}{A_g}\right) \quad \text{Eq. (7.4.2)} \]

\(F_m\) - Minimum ultimate strength of brickwork
\(A_g\) - Gross cross-sectional area.

Similar to other codes this code also penalises the permissible load for load eccentricity greater than \(t/24\). Unlike SCPI and the Canadian code it is not explicit about the type of eccentricity.

The slenderness ratio is calculated as the ratio of effective height to effective thickness. The effect of end conditions are taken care of while determining the effective height. Hence the code differs from the American and Canadian practice of incorporating the effect of end conditions by specifying different values of slenderness and eccentricity coefficient depending upon the ratio of eccentricities at top and bottom support.

The method of determining effective height is similar to British practice except that it is slightly more elaborate than the British code. The various values of effective height, depending upon the support conditions, are given in Table (7.4.1). Partial rotational restraint is said to be provided in the above table if the masonry is bearing on or supporting for its full thickness a stable concrete element such as footing, floor or roof.

7.4.1. Comparison of tests with Australian code (63)

7.4.1.1. Axially loaded walls (CW, WM, BlW, WS)

The reduction factor for all the axially loaded walls
# TABLE 7.4.1

**EFFECTIVE HEIGHT - AUSTRALIAN CODE**

<table>
<thead>
<tr>
<th>CONDITION OF SUPPORT</th>
<th>EFFECTIVE HEIGHTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1) Adequate lateral support and partial rotational restraint at top and bottom</td>
<td>0.75 H</td>
</tr>
<tr>
<td>2) Adequate lateral support and partial rotational restraint at either top or bottom and lateral restraint at opposite end (bottom or top)</td>
<td>0.85 H</td>
</tr>
<tr>
<td>3) Adequate lateral support at top and bottom</td>
<td>1.0 H</td>
</tr>
</tbody>
</table>
| 4) Adequate lateral support and partial rotational restraint at bottom and no lateral support or rotational restraint at the top | i) Piers 2H  
                  | ii) Wall 1.5 H    |
| 5) Free standing non-load bearing members                                              | 2.0 H             |
of CW, WM, BlW and WS series are higher than those specified in the Australian code. A wall of WS series having slenderness ratio of 14 has the same reduction factor as given in the code. The C.P.111(46) values are 5% to 16% higher than those given in the Australian code. The difference increases with the increase in slenderness ratio. Fig.7.4.1.

7.4.1.2. Hinged walls axially and eccentrically loaded (WO, W1/6, W1/3)

In the case of hinged walls the test values are higher for the axially loaded wall of WO series and eccentrically loaded wall of W1/6 series having eccentricity of loading \( e = t/6 \). But for the wall of W1/3 series having eccentricity of loading \( e = t/3 \), the test values are below the Australian values. Fig.7.4.2.

The reduction factor values given in C.P.111(46) are about 20% higher than the Australian values(63) for the eccentrically loaded walls having eccentricity of loading \( e = t/6 \) and \( t/3 \).

7.5. SWISS CODE(57)

Unlike the other codes discussed so far, the Swiss code is simple to use. The permissible stresses are calculated directly from Table (7.5.1). The code gives different permissible stresses for walls having different thickness and different quality of workmanship. As will be seen from the table, walls with five different thicknesses and two types of workmanship are considered.

The maximum permissible stress is given at slenderness ratio of 5. These stresses are linearly reduced to zero at the limiting slenderness ratio \( A \), Fig. (7.5.1). The
Fig. 7.4.1.  
COMPARISON OF TEST (AUTHOR'S) WITH AUSTRALIAN CODE  
FLAT ENDED WALLS  
- CW WALLS  
- BIW  
- WM  
- WS  
- CP111  
- AUS. CODE  

Fig. 7.4.2.  
COMPARISON OF TEST (AUTHOR'S) WITH AUSTRALIAN CODE  
HINGED END WALLS  
- W0 e = 0  
- W1/6 e = 1/6  
- W1/3 e = 1/3  
- AUS. CODE
TABLE 7.5.1.
PERMISSIBLE STRESS FOR BRICKWORK WITH PORTLAND CEMENT (Swiss Norm S.I.A.No,113) p.s.i.

<table>
<thead>
<tr>
<th>Type of Brickwork</th>
<th>Wall Thickness inches</th>
<th>Compressive stress and abscissae</th>
<th>High Quality Brickwork MBSC</th>
<th>Special Quality MBSC</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$m = 0$  $m = \frac{1}{2}$  $m = 1$  $m = 1\frac{1}{2}$  $m = 2$  $m = 0$  $m = \frac{1}{2}$  $m = 1$  $m = 1\frac{1}{2}$  $m = 2$</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>$e = 0$  $e = t/12$  $e = t/6$  $e = t/4$  $e = t/3$  $e = 0$  $e = t/12$  $e = t/6$  $e = t/4$  $e = t/3$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stretcher Bonded Brickwork</td>
<td>4.8</td>
<td>$\bar{\sigma}_b$</td>
<td>454.4  341  213.0  114.4  14.2  710  568  426  284  142</td>
<td></td>
</tr>
<tr>
<td></td>
<td>A</td>
<td>50  42  32  24  10  55  45  36  28  16</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>$\bar{\sigma}_b$</td>
<td>333.4  298.2  213.0  114.4  14.2  625  511.2  426  284  142</td>
<td></td>
</tr>
<tr>
<td></td>
<td>A</td>
<td>50  42  32  24  10  55  45  36  28  16</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>7.3</td>
<td>$\bar{\sigma}_b$</td>
<td>340.8  270  213.0  114.4  14.2  568  483  426  284  142</td>
<td></td>
</tr>
<tr>
<td>Bonded Bonded Brickwork</td>
<td>A</td>
<td>50  42  32  24  10  55  45  36  28  16</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>.10</td>
<td>$\bar{\sigma}_b$</td>
<td>256  199  156  95  14.2  483  412  355  227.2  85.2</td>
<td></td>
</tr>
<tr>
<td>Bonded Bonded Brickwork</td>
<td>13</td>
<td>A</td>
<td>50  42  32  24  10  55  45  36  28  16</td>
<td></td>
</tr>
</tbody>
</table>
value of limiting slenderness ratio varies from 10 to 55 depending upon the type of loading and the thickness of the wall and nature of workmanship. In the case of eccentrically loaded walls, the load eccentricity is considered to be on the same side of the wall and hence it avoids the presence of reverse curvature.

The effective height for calculating slenderness ratio is taken as the actual height. No effect of end restraints is taken into account.

7.5.1. Comparison of test results with Swiss code

7.5.1.1. Axially loaded walls (CW, WM, BLW, WS)

Axially loaded walls of WS, WM, BLW and CW series all have higher load reduction factor than are given in the Swiss code. In Fig. (7.5.3) the reduction factor values for WM and WS walls are compared with the reduction factor given in the Swiss code for 4.8 ins. thick walls. For the BLW walls the reduction factor is compared with 10 ins. thick walls and for the lightweight concrete block walls of CW series the comparison of reduction factors is made with 6 ins. thick walls. 4.8 ins., 10 ins., and 6 ins. thick walls are the nearest full scale walls comparable to these model walls.

7.5.1.2. Hinged walls axially and eccentrically loaded (WO, W1/6, W1/3). (Fig. 7.5.2.)

Axially loaded, hinged walls of WO series also have a higher reduction factor than those given in the Swiss code. Similarly for the eccentrically loaded walls of W1/6 and W1/3 series, the test results are higher than the code values, particularly for the walls of W1/3 series, where the difference
Fig. 7.5.2. Comparison of test with Swiss code.

- \( W_0, c = 0 \)
- \( W_{\frac{1}{2}}, c = 1/5 \)
- \( W_{\frac{1}{3}}, c = 1/3 \)
- C.P. 111
- Swiss code

Fig. 7.5.3. Comparison of test with Swiss code.

- W.S.
- W.M.
- C.W.
- B.W.
- C.P. 111 (1970)
- 6" thick wall, Swiss
- 10" thick wall, Swiss

Load reduction factor vs. slenderness ratio.
between the test and code values is as high as 90%. The reduction factor given in C.P.111(46) is less than that given in the Swiss code(57) for the walls of W0 series above slenderness ratio of 12.0. For the eccentrically loaded walls of W1/6 series where the eccentricity of loading is e = t/6, the reduction factors given in C.P.111(46) are about 7% higher than those given in the Swiss code.(57) In the case of walls of W1/3 series, the reduction factors given in C.P.111(46) are very much higher than those given in the Swiss code.(57) Also the Swiss code(57) permits maximum slenderness ratio of 16.0 for the walls with the eccentricity of loading e = t/3, whereas C.P.111(46) permits maximum slenderness ratio of 22.0. The load reduction factor for the eccentrically loaded bonded wall having eccentricity as e = t/6 is greater than that given in C.P.111.(46)

7.6. BRITISH CODE C.P.111

According to the British Code C.P.111,(46) the permissible stress in the masonry is calculated as

\[ \sigma = R.F. \times \text{Basic Permissible Stress} \]

As in the Australian code, the reduction-factor (R.F.) takes care of both the slenderness and eccentricity effect.

The first version of C.P.111(64) issued in 1948 calculated reduction factors on the basis of wall of slenderness ratio 1.0. The values were based on the test results of square piers obtained from different sources. Maximum slenderness ratio permitted in the code was 18 when cement-mortar was used, and 12 when lime-mortar was used. The values of reduction factors for different slenderness
ratios for axially loaded walls is shown in Fig. (7.6.1). No separate reduction factors were specified for the walls with eccentric loading. The effective height for calculating slenderness ratio was taken as the actual height.

In 1950, a paper by Davey & Thomas,\(^{(6)}\) as discussed in Chapter (2), suggested the necessity for the modification of the code and in 1953 F.G. Thomas\(^{(3)}\) paper not only gave useful suggestions for the modification, but formed the basis for the revised code issued in 1964.

The 1964 issue of C.P.111\(^{(84)}\) was less conservative than the 1948\(^{(64)}\) issue. The reduction factors were calculated by taking a wall of slenderness ratio 6.0 instead of 1.0 as in the previous issue. Also the values of the reduction factors are about 48\% to 126\% (Av. 70\%) greater than those specified in the 1948 issue, Fig. (7.6.1). The limiting value of the slenderness ratio, however, remained unchanged. The wide difference in the reduction factors between the two issues was due to the difference in choosing the minimum slenderness ratio. If the slenderness ratio-reduction factor relationship line of the 1948 issue is shifted from slenderness ratio 1.0 to 6.0 then the difference between the two is significantly reduced at all slenderness ratios except at the limiting slenderness ratio of 18.0, where the difference remains the same. The reduction factors given in the 1964 code are the average of the values for weak and strong brickwork as reported by Thomas.\(^{(3)}\) Hence for the weak brick and weak mortar the code over-estimates the reduction factor by 20\% for the walls above
Fig. 7.6.1
REDUCTION FACTOR FOR AXIALLY LOADED WALLS

STRENGTH DECREASE
REDDUCTION FACTOR
NORMAL LOADING
ACTUAL LOADING

SLENDERNESS RATIO
0 5 10 15 20 25 27 30

Fig. 7.6.2(a)
STRESS REDUCTION FACTORS FOR SLENDERNESS AND ECCENTRICITY
CP111: PART 2 1970

SLENDERNESS RATIO
0 5 10 15 20 25 27

Fig. 7.6.2(b)
LOAD REDUCTION FACTORS FOR SLENDERNESS AND ECCENTRICITY

SLENDERNESS RATIO
0 5 10 15 20 25 27

e/t = 0 to 1/24

e/t = 1/3

e/t = 1/4

e/t = 1/6
slenderness ratio of 10 and for strong brick and strong mortar it underestimates the reduction factor by 18% above a slenderness ratio of 10. At lower slenderness ratios the difference is not great. The difference in load factor which might arise due to a common reduction factor for both types of brickwork (weak and strong) is reduced (not removed) by choosing appropriate basic permissible stress.

The basic stresses in the 1970\(^{(46)}\) version of the code are SI equivalents of the values specified in 1964 code.\(^{(84)}\) The main changes in this code are from nominal thickness to actual thickness, an increase in the limiting slenderness ratio from 18 to 27 for cement mortar and revision in the reduction factor for different slenderness ratio. The value of 27.0 will correspond to height to radius of gyration ratio of 93.5, which, when compared to the limiting value for the steel columns, is much lower. This is reasonable because of the absence of tensile resistance in masonry walls. These reduction factors are based on the tests carried out on walls and piers. The reduction factor–slenderness ratio relationship for the axially loaded wall is shown in Fig. (7.6.1). The values of new reduction factor are not different from the 1964\(^{(84)}\) code up to slenderness ratio of 10, where the maximum difference between the two was 6%. Above a slenderness ratio of 10, the maximum difference is as high as 35%. The reduction factors for eccentrically loaded walls is also found to be higher. The 1970 code,\(^{(46)}\) like the 1964 code,\(^{(84)}\) is not explicit about the nature of eccentricity.
Fig. (7.6.2) shows stress and load reduction factor for axially and eccentrically loaded walls as given in C.P.111(1970).

The effective height in the calculation of slenderness ratio is taken as follows:-
1) Walls with lateral support top and bottom = $\frac{3}{4}H$.
2) Walls with no top and bottom support = $H$.
3) Walls with fixed bottom and top end free = $1.5H$.

$H$ is the actual height of the wall.

As in other codes, the choice of effective height for the different end conditions of the wall is arbitrary. The value of effective height of $\frac{3}{4}H$ is based on the reasoning that for an ideal fixed wall the effective height is $0.5H$ and for walls in actual building when complete fixity is not possible, the effective height is increased to $0.75H$. The code does not give any justification for choosing this value; it is probably based on intuition.

When this is compared with methods of steel column design, the C.P.111 approach seems to be crude. This is understandable because very little is known from actual tests, either model or full scale, of the real mode of action of walls and floors in a building. Up till now, all versions of C.P.111 relate slenderness effect in terms of stress reduction factors. But the latest draft code, in limit state terms issued in 1973, is in terms of load reduction factors. This is in line with the SCP1 code, the Canadian and the Australian code. The stress reduction factor values of C.P.111(1970) have been converted into load reduction factors after taking into account the increase in
basic stress due to non-uniform stress distribution as a result of eccentric loading. Fig. (7.6.3) shows the load reduction factor (L.R.F.), eccentricity and slenderness ratio relationship. A comparison between L.R.F. of C.P.111 (1970) and the draft code is shown in the Fig. (7.6.4). The values of L.R.F. in the draft code are based on the assumption of a stress block resisting the eccentric load, with an average stress of \( \frac{f_k}{\gamma_m} \). It is seen that the draft code values are about 8% higher between slenderness ratio range of 13 to 22, for the axially loaded walls. In the case of eccentrically loaded walls having eccentricity of loading \( e = \frac{t}{6} \), there is a wide difference between C.P.111 and the draft values. Draft values do not show any decrease in strength up to two-thirds of slenderness ratio range for different eccentricities of loading. For walls with eccentricity of loading \( t/3 \), the draft values are 1.76 times the C.P.111 value at slenderness ratio of 18.0. Codes of other countries do not have such a high L.R.F. at this slenderness ratio. The difference may probably be due to different position of load eccentricity. However in the following section it will be seen that some of the test results justify the draft proposals.

7.6.1. **Comparison of test results with code:-(C.P.111)1970**

7.6.1.1. **Author's test results**

Comparison of author's test results with C.P.111 have been made while comparing codes of other countries in the previous section. However, a general comparison of the author's test results along with other available test results is shown in Fig. (7.6.1.5). The figure shows a wide
Fig. 7.6.3.
LOAD REDUCTION FACTOR
CP111: PART 2: 1970

Fig. 7.6.4.
CP111 (1970) COMPARED TO DRAFT (1973)
FIG. 7.6.1.5.

REDUCTION FACTOR VERSUS SLENDERNESS RATIO

- B1WH=3/4 H/t
- B1WH=0.9H
- Ws RCC SLAB
- W0 e=0
- W/6 e=1/6
- W/3 e=1/3
- CP111 1970
- DRAFT 1973 [OCT]
- WMH=3/4 H/t

△ SCP1 TEST 203mm
FULL SCALE.
■ SCP1 1016mm THICK
□ 114.3mm WALL THOMAS
FULL SCALE
△ 114.3mm MODEL 1/6 th SCALE WALL
HENDRY ET AL
+ WMH=0.9H
scatter of test results for axially loaded walls, variations in testing procedure and material properties, and differences in choosing the height of the wall are factors responsible for this scatter. The author's test results for eccentrically loaded walls are compared with both C.P.111 and draft values. Test results are in agreement with the code recommendations except in a few cases where the test values are higher than the code values.

7.6.2. **Comparison of draft proposals with the test results**

Most of the test results have so far been discussed in relation to C.P.111(1970) values. In order to study the validity of the draft proposals, comparisons are made with other available test results in the following section.

7.6.2.1. Davey & Thomas tests on 9 ins. (nominal) brickwork piers—weak and strong mortar

The lowest pier built in weak mortar being of 12 ins. height will give misleading strength because of the restraint provided by the steel platens and so the load reduction factors are compared (Fig. 7.6.2.6) on the basis of the strength of the piers of slenderness ratio 6.7 and 10.7. The results are generally higher than draft values especially at lower values at slenderness ratio of 10.5 and 15.0. These could be due to defective workmanship and errors involved in obtaining the desired eccentricity.

Load reduction factor values of the piers built with strong mortar, Fig. (7.6.2.7) are in fairly good agreement with the draft recommendations. In this case also the load reduction factors are calculated on the basis of piers of
slenderness ratio 6 and 10. At slenderness ratios of 10 and 14.0, the test results are lower for piers having $t/4$ and $t/3$ eccentricity of loading.

7.6.2.2. SCPI tests\(^{(85)}\) on 8 ins. brickwork piers

In this case the eccentricity of loading is similar to that envisaged in the draft code Table (5)\(^{(68)}\) Section 4.6.2. i.e. the top end is eccentrically loaded and the bottom end axially. The test results for most of the cases are above code specification. It is interesting to note that test results agree with the draft proposals for walls having load eccentricity of $t/3$ at slenderness ratio of 21.0. No reduction in strength with the increase in slenderness ratio up to slenderness ratio of 18.0 for the wall of eccentricity $t/3$ seems strange, but the few available test results justify the draft values, Fig. (7.6.2.8.).

7.6.2.3. SCPI tests on brick walls\(^{(66)}\)

The load reduction factor of the walls with different load eccentricity along with the code proposals are shown in Fig. (7.6.2.9). Each test result shown in the figure is the average of three walls. The load reduction factor for all the cases are found to be higher than the draft values. Here also the test results for the walls with load eccentricity of $t/3$ at the top end are higher than the draft values at all slenderness ratios.

7.6.2.4. Yokel et al.\(^{(41)}\) tests on 6 ins. reinforced and 8 ins. unreinforced walls

Similar to SCPI test results load reduction factor values for Yokel's test results show higher values for axially and eccentrically loaded walls at all slenderness ratios,
Figs. (7.6.2.10) and (7.6.2.11). The test details of Yokel's test have been discussed in Chapter (2).

7.6.3. Comparison of permissible loads of different codes

So far the basis and the provision in the various codes for calculating permissible stress or load after taking due cognisance of slenderness effect have been dealt with. It is seen that these codes have the same approach for calculating the permissible design stresses or load, but give different values of load reduction factor and basic stresses. The method of calculating effective height is also different in different codes. In order to see what the permissible design stresses or loads turn out to be for the different codes, a comparison of these loads is made in Table (7.6.3.1) and Table (7.6.3.2) for hinged and fixed end walls.

While doing the calculations the following assumptions have been made.

1) Brick strength $\rightarrow 4000$ p.s.i. (28N/mm$^2$).

2) Mortar used is one or other of the following mixes

   a) Cement:Lime:Sand $1:\frac{1}{3}:3$.

   b) Cement:Sand $1:3$.

   c) M-type Mortar - as per SCPI and Canadian specification. This is the mix which is the nearest equivalent to the above two mixes.

3) For eccentrically loaded walls the average stress has been calculated.

4) Inspected workmanship.

5) Area of the wall is unity.

6) In draft code the partial safety factor $\gamma_m$ is taken as 2.9.
Fig. 7.6.2.10.
YOKEL TESTS ON 6" REINFORCED WALL (BLOCK)

Fig. 7.6.2.11.
YOKEL TESTS ON 8" PLAIN BLOCK WALL
### TABLE 7.6.3.1.

**DESIGN LOADS (IN POUNDS) FOR HINGED END WALLS**

<table>
<thead>
<tr>
<th>Code</th>
<th>SLENDERNESS RATIO</th>
<th>( e/t )</th>
<th>10</th>
<th>16</th>
<th>( e/t )</th>
<th>20</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stress R.F.</td>
<td>C.P.111(1970)</td>
<td>298</td>
<td>372</td>
<td></td>
<td></td>
<td>0.89</td>
</tr>
<tr>
<td></td>
<td>C.P.111(1970)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>265</td>
</tr>
<tr>
<td>Load R.F.</td>
<td>C.P.111(1970)</td>
<td>298</td>
<td>-</td>
<td></td>
<td></td>
<td>0.89</td>
</tr>
<tr>
<td></td>
<td>Draft C.P.111</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>265</td>
</tr>
<tr>
<td></td>
<td>Sept. 1973</td>
<td>475</td>
<td>-</td>
<td></td>
<td></td>
<td>0.92</td>
</tr>
<tr>
<td>Load R.F.</td>
<td>Australia</td>
<td>330</td>
<td>413</td>
<td></td>
<td></td>
<td>0.88</td>
</tr>
<tr>
<td></td>
<td>U.S.A.</td>
<td>320</td>
<td>320</td>
<td></td>
<td></td>
<td>0.93</td>
</tr>
<tr>
<td></td>
<td>U.S.A.</td>
<td>320</td>
<td>320</td>
<td></td>
<td></td>
<td>298</td>
</tr>
<tr>
<td></td>
<td>Canada</td>
<td>400</td>
<td>512</td>
<td></td>
<td></td>
<td>0.88</td>
</tr>
<tr>
<td></td>
<td>Switzerland</td>
<td>711</td>
<td>-</td>
<td>Special Quality</td>
<td>495</td>
<td>563</td>
</tr>
</tbody>
</table>

**NOTE:** For U.S. and Canadian codes, the load R.F. values correspond to \( e_1/e_2 = -1 \), i.e. for fixed end walls and walls with load eccentricity on opposite sides (double curvature).
### TABLE 7.6.3.2.
**DESIGN LOADS (IN POUNDS) FOR FIXED END WALLS**

<table>
<thead>
<tr>
<th>Code</th>
<th>Basic Stress</th>
<th>SLENDERNESS RATIO</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>p.s.i.</td>
<td>10 (7.5)*</td>
</tr>
<tr>
<td></td>
<td>Axial Flex-</td>
<td>e/t</td>
</tr>
<tr>
<td></td>
<td>ure</td>
<td>0 1/12 1/6 1/3</td>
</tr>
<tr>
<td>Stress R.F.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Load R.F.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C.P.111 (1970)</td>
<td>298</td>
<td>286 230 176  86</td>
</tr>
<tr>
<td>Load R.F.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Draft C.P.111 (1973)</td>
<td>475</td>
<td>454 332 309 162</td>
</tr>
<tr>
<td>Load R.F.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Australia</td>
<td>330</td>
<td>315 266 200 114</td>
</tr>
<tr>
<td>Load R.F.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>U.S.A.</td>
<td>320</td>
<td>320 288 256 195</td>
</tr>
<tr>
<td>Load R.F.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Canada</td>
<td>400</td>
<td>369 313 236 118</td>
</tr>
<tr>
<td>Switzerland</td>
<td>711</td>
<td>639 495 337  77</td>
</tr>
</tbody>
</table>

*NOTE:* Values in brackets are for C.P.111 and Australian code to take care of end conditions.
It is seen from these tables that for walls up to load eccentricity of \( t/6 \), the Swiss and draft codes are the least conservative. But for the highest eccentricity of \( t/3 \) the draft code and the Canadian codes are least conservative. The high value of permissible load permitted by Swiss code is because of excellent quality of workmanship involved. The draft code also specifies high quality of workmanship. Permissible loads calculated from the draft specification are substantially higher than those calculated on the basis of C.P.111(1970). This is mainly because of difference in quality of workmanship and difference in approach of calculating reduction factors.

On comparing permissible loads of Canadian, American, Australian and British C.P.111(1970) codes the Canadian code is found to be the least conservative. This is because of the high basic stresses permitted in this code which offsets the higher load reduction factors in the American and British codes. This difference in permissible load may be due to variation in the mortar type and quality of workmanship envisaged which may not be exactly the same in the different design codes. For fixed end walls, the American code corresponds to eccentricity ratio of \( e_1/e_2 = -1 \). This ratio is also applicable for the case when there is equal eccentricity on opposite sides at top and bottom. The effective height in this case will be half, because of the double curvature, and the code takes the same effective height for a fixed end wall. This is rather an ideal assumption, which is not valid in an actual situation.
For the eccentrically loaded walls the basic stresses in the British and Australian codes are increased by 25% and in the Canadian code by 28%. In the American code this increase is taken care of in the eccentricity coefficients and as such no increase is made in the basic stress.

7.7. CONCLUSIONS

1. Load reduction factors given in the American and Canadian codes are similar to those in the British code for hinged walls with axial and eccentric loading. For the fixed-end walls, because of difference in choosing effective height, there is a difference in load reduction factors given by these two codes.

2. The test results show that higher load reduction factors are given in the American and Canadian codes for most cases, except for walls with load eccentricity t/3.

3. The Australian code is conservative in comparison to the British code, especially for eccentrically loaded hinged walls.

The test results for all walls have higher load reduction factors except for eccentrically loaded walls with eccentricity t/3.

4. There is fairly good agreement between the British and Swiss codes for 4½ ins. thick single leaf walls for all types of loading, except for walls with load eccentricity of t/3. The British code values are more than twice the Swiss values. The Swiss code does not permit walls having slenderness ratios greater than 16 at load eccentricity t/3, as against the limit of 22 specified in the British code (C.P.111(1970)).
5. Load reduction factor values in the British code are based on the test results of 4\frac{3}{4} ins. single leaf brick walls and 9 ins. square piers and so the values given for bonded walls in the Swiss code are different.

6. Test results have higher reduction factor values than Swiss values for all the types of loading.

7. Compared with the first issue of C.P.111(1948) the present British code has been greatly modified. The load reduction factor values of the current code are comparable with the codes of other countries.

8. Test results show higher values of reduction factors compared with code values for axially loaded walls. There is a wide scatter of experimental points, the code values representing lower bound values rather than characteristic values. Walls with eccentricity of t/6 have higher values than the code values, but for walls with load eccentricity of t/3 the code values are higher. This, however, may be a reflection of the fact that experimental eccentricities were higher than the nominal value.

9. These test results indicate the possibility of further revision of codes. The new draft code is a step in this direction.

10. Other available test results when compared with the draft code show higher values of reduction factor for axially and eccentrically loaded walls, especially at the large eccentricity of t/3. The test results available in this region of eccentricity and slenderness ratio are very few, and so a definite conclusion for this case at the moment is not possible.
11. While comparing the permissible loads of various codes, the Swiss and Canadian and draft codes are found to be less conservative than others.

12. The value of effective height taken in various codes for calculating slenderness ratio is different in different codes.
CHAPTER 8

GENERAL CONCLUSIONS

8.1.

This investigation has attempted to clear the state of confusion that existed regarding the relationship between ultimate strength and slenderness ratio, and has led to the following general conclusions:

1) The test results for single leaf and bonded brick walls and also for light weight concrete (Aglite) block walls axially loaded under flat ended conditions clearly show a decrease in wall strength with the increase in slenderness ratio. The proportions by which they decrease is different for different types of walls.

2) Single leaf walls axially loaded between R.C. slabs also show a decrease in wall strength with increase in slenderness ratio. After taking into consideration the influence of brick strength and rate of loading on the strength of walls, experimental results indicate that better end restraint is provided by these slabs in comparison with that provided by the machine platen.

3) The ultimate strength of axially and eccentrically loaded hinged walls also shows a decrease in strength with increase in slenderness ratio. The ultimate strength of walls loaded with nominal eccentricity of t/3 shows a sharp decrease in strength with increase in slenderness ratio, especially at high slenderness ratios.

4) The mode of failure of axially loaded walls shows a typical failure pattern by vertical tensile splitting, crushing and
spalling. All walls, except those of greater height exhibited strength failure. Tall walls exhibited both strength and stability failures. These modes of failure have also been observed by previous workers. Eccentrically loaded hinged walls failed mainly by excessive bending at the brick mortar interface primarily due to bond failure.

3) Most of the theories formulated to relate the ultimate strength of the walls with slenderness ratio are found to be approximate because of the assumption that the walls have no tensile resistance, and also because of the elastic and non-homogeneous characteristic of masonry. Turkstra's and Angervo's theories are found to be relatively suitable for predicting wall strength.

4) The test results, when compared with C.P.111(1970) and the design codes of other countries, show higher reduction factors than those specified in the codes for axially loaded walls, especially at slenderness ratios above 18.0. For eccentrically loaded walls the reduction factors are nearly the same as those given in C.P.111 (1970). In draft code the load eccentricity is taken at the top only and so draft values are higher than C.P.111(1970).

5) There is a wide scatter of available test results for axially loaded walls. The line representing the C.P.111 (1970) recommendations gives a lower bound rather than characteristic values.

6) The experimental investigation has also paved the way for full scale wall tests to be carried out in order to study the slenderness effect under realistic end conditions.
and thereby arrive at suitable reduction factors for these walls. The number of full scale walls to be tested could be reduced because of the substantial amount of data available from these model tests. Also, improvements in the testing technique could be made as a result of experience gained in the model tests.

7) The stress-strain relationships are found to be non-linear, and there are considerable variations in the results obtained from tests on nominally identical walls. Block walls exhibit a relatively less non-linear relationship, but again show variation between tests on similar walls.
REFERENCES


57. S.I.A. Standard No. 113. Zurich, Switzerland.

58. Sahlin, S. Professor. Chalmers University of Technology Department of Structural Mechanics, Goteborg, Sweden. Private Discussion.


88. Turkstra, C-J. "Resistencia De Muros De Mamposteria Ante Cargas Verticales Excentricas". Patrocinado Por Fondo de Operacion y Descuento Bancario a la Vivenda, 27, Unam, Mexico, October, 1970.


A.1.1. **ECCENTRICITY CALCULATION**

The eccentricity of loading is calculated from the strain measurements. Two cases of loading (1) when the load is within the kern, (2) when the load is outside the kern are considered.

The following basic assumptions are made in derivation of the eccentricity equation from the measurement of strain at the faces of the walls.

1. Modulus of elasticity $E$ is constant, i.e. stress ($\sigma$) is linear function of strain $\varepsilon$.
2. The strain distribution through the wall is linear.
3. Modulus of elasticity $E$ in tension and compression are equal.

**NOTATIONS:**
- $e$ - Load eccentricity
- $M$ - Moment
- $P$ - Load
- $Z$ - Section Modulus
- $\sigma_1, \sigma_2$ - Stresses at wall faces
- $t$ - Wall thickness
- $S$ - Distance from the neutral axis to the face of the wall.

**A.1.1.1. Case I:** Load is within the kern

![Diagram](image)

Fig.A.1.1.1.

= $\frac{\sigma_2}{\sigma_1}$
For a wall of thickness "t", having stresses at its face as $\sigma_1$ and $\sigma_2$ and the bending moment $M$, the load eccentricity $e$ is

$$e = \frac{M}{P}$$

$$M = \sigma Z$$

$$M = \frac{-\sigma_1 - \sigma_2}{2} x Z \quad \text{Bending Stress}$$

$$P = \frac{\sigma_1 + \sigma_2}{2} x b x t \quad \text{Direct Stress}$$

Also from two similar triangles in Fig. (A.1.1.1) we have

$$\frac{\sigma_1}{(t+S)} = \frac{\sigma_2}{S}$$

$$\sigma_2 = \frac{\sigma_1}{(t+S)} x S$$

$$e = \frac{M}{P} = \frac{\sigma_1 - \sigma_2}{\sigma_1 + \sigma_2} \times \frac{b x t^2}{6b x t}$$

substituting for $\sigma_2$ in the above equation and solving it we get

$$e = \frac{t^2}{t + 2S} \times \frac{1}{6}$$

A.1.1.2. Case II:- Load is outside the kern

\[ \text{fig. A.1.1.2.} \]
For the combined stress condition the sum of the forces on the section = Axial Force.

\[ \frac{1}{2} \times P/t(1 + \frac{6e}{t})S \times 1 - P/t(1 - \frac{6e}{t})t - S) \times 1 = \frac{P}{t} \times 1 \times 1 \times t \]

or \[ S = \frac{3}{4}(t - 2e) \]

\[ e = \frac{t}{2} - \frac{S}{3} \]

Also from Fig. (A.1.1.2c)

\[ e = \frac{t}{2} - \frac{S}{3} \]

Knowing the thickness "t" and intercept "S", the eccentricity of loading of various walls have been calculated. Fig. A.1.1.3. to (A.1.1.4) shows strain reading diagrams of some walls.
A.2.1. DEMEC GAUGE

Specifications:
Manufactured by W.H. Mayes and Son Ltd.,
Gauge length obtainable from 2 to 80 inches.
Price range: gauge, gauge setting out bar and invar reference bar from £60 to £150.
Stainless steel demec studs 112 at £2.30.
Calibration factor by Cement and Concrete Association.
2 inch gauge $2.48 \times 10^{-5}$ strain per division
8 inch gauge $1.01 \times 10^{-6}$ strain per division
12 inch gauge $6.6 \times 10^{-6}$ strain per division
24 inch gauge $3.33 \times 10^{-6}$ strain per division

Description

The "Demec" gauge is a demountable strain gauge (Plate (A.2.1.)) developed by Cement and Concrete Association, being obtainable at various gauge lengths. Its main components consists of an Invar main beam with two conical gauge points, one fixed at one end and the other pivoting on a knife edge. This pivoting movement is transmitted to a dial gauge (graduated in $10^{-4} \text{ ins.}$) mounted on the beam. An invar reference bar is provided as a check. Reference bar readings are usually taken before and after a test. The correct gauge length will be obtained by the use of a stainless steel stud to the surface of the structure. Durofix is a suitable adhesive for cementing the studs to the surface of the structure.
Using demec gauges the reading accuracy of $3 \times 10^{-6}$ can be obtained. A small temperature correction could be applied to the gauge.

When measuring strain with the Demec gauge just enough pressure should be applied to the gauge to provide good contact. The reading is repeated to improve accuracy. The gauge is always held in the same way for a particular position. When measuring vertically the fixed point should be in the lower disc. Throughout a test only one person should take readings for a particular measuring point.

**A.2.2. DIAL GAUGES**

Specifications: Baty Dial Gauge costing approximately £10 each.

- 1 division = 0.0001 inch.
- Range 0.2 inch or 0.5 inch.

**A.2.3. HYDRAULIC JACK**

Detail is given in Section 4.4.3.1. Chapter (4).

**A.2.4. LOAD CELL**

The load cell is discussed briefly in Section 4.4.3.2. Chapter (4).

They consist basically of electrical resistance strain gauges bonded to the steel column of load cell and connected as 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 applied load is produced. A critical feature of this type of load cell is the height to diameter.
ratio which should be made greater than 12 to achieve 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 cell has self aligning cap to ensure concentric loading, and the upper surface which comes in contact with the ram, is a ferobestos disc dressed with molybdenum disulphide based grease. It is seated on 1 1/2 ins. thick steel plate with machine surfaces which in turn is seated on the loading beam. It is designed to withstand at least 50% overload above the normal rating without electrical or mechanical damage. Deflection on the vertical axis under load is generally less than 0.012 ins. (0.3 m.m.) Reference (86) gives detail of the load cell.

A.2.5. **DIGITAL VOLTMETER**

This is also discussed briefly in Section 4.4.3.3. Chapter (4).

The operational range of 0 - 1000 Volt is covered in six ranges, with a maximum sensitivity of 10 $\mu$V and an accuracy of $\pm$ 0.05% of reading $\pm$ 0.05% of range full scale. The voltmeter has internal calibration, filter and repetitive/single-shot sampling facilities. The detail of it is given in Reference (87).

A.2.6. **ELECTROLEVEL**

The Electrolevel type EL/10 is a portable, remote reading instrument for precision levelling. The sensitive element is a spirit-level filled with a conducting liquid and provided with a system of electrodes from which the
bubble position is read as an electrical signal from a suitable indicator.

Specifications:

- Length: 10 ins.
- Width: 7 ins.
- Height: 5\(\frac{1}{2}\) ins.
- Weight: 11 lb.

Three scale ranges are provided reading from 0.05 thous. per inch (10 arc seconds) to 10 thous. per inch.

This instrument can be applied to the determination and control of small angular displacements where a remote read out is desired.

A.2.7. TALYVEL

This instrument was not used in the present investigation. However, it can be used in place of electrolevel.

The 'Talyvel' instrument is a new type of precision level in which the usual spirit bubble is replaced by a pendulum co-operating with transducers which provide an electrical displacement signal. This signal is amplified to feed a centre-zero meter.

The instrument is battery operated and comprises:

1. The 'Talyvel' level unit which engages the surface under test.
2. A Meter Unit which contains the battery and a transistorised amplifier, and provides storage space for the 'Talyvel' unit and lead.

The meter is scaled both in angular measure (minutes and seconds) and in gradient (inches per inch and mm per metre), to indicate precisely the amount of tilt of the 'Talyvel'
unit. One second is approximately 0.000 005 inches per inch, or 0.005 mm per metre.

The pointer comes to rest in about one second of time. Three ranges of sensitivity are shown by the Meter. These are selected by a combined ON-OFF and Range switch. Calibrated levelling knobs provide a fourth range.
APPENDIX III

A.3.1. TESTING OF PIERS

In all the theories the basic strength of masonry is taken as the strength of the pier of slenderness ratio between 4 and 6. The modulus of elasticity is also determined by testing piers having the same slenderness ratio. As has been mentioned in Chapter (6), the modulus of elasticity value is an important parameter in the determination of masonry strength. Ideally this value of modulus should be based on a large number of tests. However in this study, because of limited time, not many piers have been tested. The Yokel et al. (40)(41) theory takes into account the effect of strain gradient. This gradient can be obtained by testing axially loaded and eccentrically loaded piers. The following section describes the testing method for these piers.

A.3.2. MATERIALS

The materials used in the construction of piers are similar to those used in the walls in Chapter (5).

A.3.3. TEST PROGRAMME

Twenty-one piers of 9.5 ins. x 8.75 ins. x 1.47 ins. dimensions were tested under axial and eccentric loading of t/6 and t/3.

A.3.4. EXPERIMENTAL PROCEDURE

A.3.4.1. Construction of piers

Piers were built in a similar way to the walls.

A.3.4.2. Pier tests and measurements

The piers were taken to the Avery machine manually and were capped in the same way as the wall described in
Chapter (5). The dimensions of the loading beam were like those for the walls, except the length, which was 10 ins. instead of 20 ins.

A.3.4.2.1. Application of loads

For axially loaded piers the loads were applied at 1 ton intervals up to 4 ton and then at 2 ton intervals up to failure. The average rate of loading was 60 p.s.i. per minute. For piers loaded with $e = t/6$ eccentricity the loads were applied at 1 ton interval up to failure. The average rate of loading was 40 p.s.i. per minute.

For piers loaded with $e = t/3$ eccentricity, the loads were applied at $\frac{1}{3}$ ton intervals up to failure. The average rate of loading was 17 p.s.i. per minute.

A.3.4.2.2. Strain measurements

The strain was measured by using Demec gauge of 2 ins. gauge length. The stress-strain curve is shown in Fig. (A.3.1.).

A.3.5. RESULTS

A summary of results are given in Table (A.3.1.).

A.3.6. DISCUSSION

A.3.6.1. Mode of failure

The axially loaded piers $P_0$ failed mainly due to crushing and vertical tensile splitting, and partly due to spalling. In all cases the first crack appeared at 70% of the failure load.

The eccentrically loaded pier $P_{1/6}$ having load eccentricity of $t/6$, generally showed their first cracks at 80% of the ultimate load. They failed due to spalling, crushing and bending, and in some cases there were a few tensile
Fig. A.3.1(a)
COMPRESSIVE STRESS
Vs.
VERTICAL STRAIN
(ALS LOADED PIERS)

Fig. A.3.1(b)
COMPRESSIVE STRESS
Vs.
VERTICAL STRAIN
(ALS LOADED PIERS)
<table>
<thead>
<tr>
<th>Pier No.</th>
<th>Age Days</th>
<th>Failure Load Tons</th>
<th>Average Stress p.s.i.</th>
<th>Mean Average Stress p.s.i.</th>
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<td>18</td>
<td>17.0</td>
<td>2810</td>
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<tr>
<td>P₀ - 2</td>
<td>34</td>
<td>17.2</td>
<td>2828</td>
<td>2725</td>
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<td>21</td>
<td>15.4</td>
<td>2538</td>
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<td>37</td>
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<td>38</td>
<td>4.1</td>
<td>681.0</td>
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* Load eccentricity less than t/3.
cracks as well. Piers P-1/6-6-2 had peculiar failure patterns in which, in addition to spalling and crushing, there was a horizontal tensile crack at the mid depth of the brick along the whole length of the pier. This may be due to the presence of good bond between the mortar joint and the bottommost course, which may have prevented the bottommost course from rotation, and thus result in tensile crack.

The piers P-1/3 having load eccentricity of t/3 generally failed by spalling and bending. The maximum bending occurred at about 4 ins. from the top. In a few cases there was some crushing involved, and vertical tensile splitting in the thickness of the wall.
A review of research carried out on masonry walls shows that there is a scarcity of test data on the effect of slenderness ratio on the compressive strength of walls. The investigation described in this paper was carried out on one third scale model walls. It is intended as a pilot study to determine whether reduction factors prescribed in various codes are conservative: if they are, the background investigation at model scale will permit the determination of revised values on the basis of a comparatively limited programme of full scale tests. Model walls have been tested with axial and eccentric loading and with various end conditions. The results are compared with the provision of various national codes and implications are discussed. As far as the British Code (CP.111 (1970)) is concerned there would seem to be scope for revision of the reduction factors at higher slenderness ratios in the direction of giving higher design strengths.


Soweit die englische Vorschrift (Britisch Code) CP 111 (1970) betreffen ist so scheint es, dass darin genügend Spielraum gegeben ist, die Reduktionsfaktoren bei größeren Schlankheiten zu überprüfen oder aber höhere Druckfestigkeit für Mauerwerk zuzulassen.
L'INFLUENCE DE L'ÉLANCIEMENT ET DE L'EXCENTRICITÉ SUR LA RÉSISTANCE DES MURS À LA COMPRESSION

Quand nous jetons un regard sur l'ensemble des essais effectués sur des murs en maçonnerie, nous sommes frappé par le fait qu'il y a, et peu de résultats d'essais concernant le rapport entre l'élançement et la résistance des murs à la compression.

La recherche décrite dans cette étude a été exécutée sur des maquettes à l'échelle un tiers. L'intention était de savoir par cette étude si les facteurs de réduction, décrits dans différents règlements, pouvaient être maintenus ; si c'est le cas, la recherche primaire sur modèle réduit permettra de déterminer les valeurs revus basées sur un programme de comparaison limité d'essais à échelle normale.

Des maquettes ont été testées avec charges axiales et excentriques et dans de différentes conditions d'enfouissement. Les résultats sont comparés aux prescriptions de nombreux règlements nationaux et les conclusions sont discutées. En ce qui concerne la Norme Britannique (CP. 111 (1970)) il semble qu'il y a une tendance à la révision des facteurs de réduction pour des murs très élançés et dans le sens d'augmenter les tensions de calcul.
1. INTRODUCTION

The combined effect of S.R. and eccentricity are the important parameters affecting the strength of brick walls yet they have not been adequately studied. There is scarcity of test data and available test results are conflicting. This paper describes very briefly tests carried out on model scale brick walls and the results are discussed in light of previously published results and code provisions. Model scale model testing of brickwork has been used extensively in the author's laboratory and results have previously been validated against full scale tests.

2. TEST PROGRAMME

Table 1 gives the details of the programme.

3. MATERIALS

3.1 Bricks

One third scale bricks were used. Bricks were supplied in batches so their strength varied. They were tested in accordance with B.S. 3921 - 1969 Part 2.

3.2 Sand and Cement

In all tests, dry Leighton Buzzard 25/52 sand was used in conjunction with rapid hardening Portland cement (Ferrocrete).

3.3 Mortar

The mortar mix was 1:3 cement:sand by weight. With each wall six 25.4 mm mortar cubes were made by hand compaction. The water cement ratio varied from 0.8 to 0.85.

4. EXPERIMENTAL PROCEDURE

Details of experimental procedure is given in Reference 5. All walls were built in wooden jigs except B1W8, B1W8 and B1W12. Wall B1W8 had higher strength than smaller walls because of better vertical alignment and workmanship as they were built in wooden jigs. Walls were cured under polythene cover for a minimum of 7 days before being tested.

4.1 Application of Load

For all walls except those in WS group the load was applied by an Avery testing machine. For WS group walls the load was applied by means of an hydraulic jack through a load cell in a specially built rig. Plate 2.

4.2 Strain Measurements

Vertical Strains were measured by compressometer and Dernec gauges of different gauge length depending upon the height of the walls.

4.3 Lateral Deflection

The lateral deflection was measured by dial gauges of 0.00254 mm sensitivity.

4.4 End Rotation

This was measured at the top and bottom most course by fixing brackets and dial gauges. Fig. 1 (a).

5. RESULTS

Tables 2, 3 and 4 give details of the test results.

6. DISCUSSION

6.1 Modes of Failure

In all walls except walls of Wo, W1 and W3 series, the first hairline crack appeared between 50 - 60% of failure load and enlarged with further increase in load. The general mode of failure of the walls was vertical splitting accompanied by crushing and spalling of various courses of brick.

In walls W - 1 - 25 and all walls of W - 1 group failure occurred at mortar-brick interface due to breakdown of bond between the mortar and the brick at the time of maximum deflection.

6.2 Deflection

The lateral deflection measurements for WM, BW and WS group of walls showed that none of the walls deflected to the same extent. Some of the walls had a deflection curve similar to a sine curve e.g. walls of Wo, W1 and W3 group.

6.3 End Rotation

Fig. 6(b) shows load and rotation relationship. There does not seem to be a consistent relationship between the two, because of the variable nature of masonry.

6.4 Slenderness Ratio

6.4.1 Wo, BW and WS Wails

Fig. 6 shows authors' test results and test carried out by other investigators. The S.R. of the smallest walls are different for different tests so that the value of the R.F. calculated will also be different. While comparing different test results this aspect should be kept in view. Walls of WM series show a decrease in strength of wall with the increase in S.R. except for WM12 wall. WM walls have higher R.F. than Canadian tests on 102 mm thick full scale walls up to S.R. of 15.0, above this value the R.F. was lower than in American tests. This could be attributed to differences in workmanship which becomes important in the case of more slender walls. Thomas' test results1 on 114.3 mm thick full scale tests show almost no decrease in strength with increase in S.R.

6.4.2 Wo, W1 and W3 Wails

Fig. 6 shows a comparison of test results carried out by SCPI, Haller and the author. There is a slight variation in the basis of the R.F calculation. However on comparison, with the exception of wall of S.R. 22.5, the R.F. of the other SCPI test walls are about 10% lower than those of Wo walls. The eccentrically loaded walls of W-1 series have nearly the same R.F. as the SCPI test walls, whereas W-1 walls have R.F. which is approximately 25% higher than SCPI tests. For all these walls the maximum S.R. did not exceed 25.0 whereas SCPI tests had walls up to S.R. of 48.1. Torsion walls above S.R. of 30 is more of academic than of practical interest. Haller's test results are about 10% lower than the author's for the walls of Wo series. In the case of walls with eccentricity of loading ε = t/6. Haller's test results are lower than the author's results.
Fig. (4) shows the results of the test carried out by Watstein and Allen and Grave and Motteu, and the author's test. The lowest wall in Watstein and Allen's tests had a S.R. of 12.4 and so the R.F. is based on the ultimate strength of this wall. Similarly the lowest wall in the Grave and Motteu test had a S.R. of 3.0. Unlike the author's test the eccentricity in the Watstein and Allen test was only at the top of the wall and the bottom end of the wall was loaded axially. The R.F. of Watstein and Allen's tests for all the three types of loadings were higher than the author's wall test. This was expected because of the difference in calculating R.F. and load eccentricity. The R.F. in Grave and Motteu's tests are also higher than author's wall of Wo series. In this test the wall having lowest S.R. of 9.0 was a bonded wall of (11.8") 250mm and (5.6") 140mm, strength of the wall is found to decrease with the increasing in thickness. This reduction in strength becomes more pronounced if it is a bonded wall. Haller's test shows that strength of an axially loaded 125mm single leaf wall can be 46% greater than that for a 250mm bonded wall when the slenderness ratio of both are equal to 10.0. This is the reason for Motteu's test walls having higher R.F.

Difference in rate of loading and method of testing etc. may be responsible for the variation in results of different tests. Although Grave and Motteu tested a number of walls they changed various parameters such as type of loading and condition and thickness thus restricting comparison of their remaining test results with those of the authors.

6.4.3 Comparison of Tests with Code Provision

Figure (2) shows comparison of test results with CP 111 (1970) and draft code. Except wall WS-14 all remaining walls of Dw, WH, WS and Wo series have higher R.F. than given in CP 111 (1970) and in the British draft code. Eccentrically loaded walls of W-3 series have higher R.F. than that given in CP 111 (1970). The actual load eccentricity occurs when the intended eccentricity of t/b and so if test values are compared with the code of corresponding eccentricity it will be still higher.

Walls of W-3 series have lower R.F. than those given in CP 111 and the draft code for load eccentricity of e = t/3. But when compared with the code for load eccentricity of e = t/2, the test values are higher, except for wall W-0.5. In all cases of walls having eccentric loading at the top and the axial loading at the bottom, the draft code gives uniform reduction factors up to S.R. of 13 and 16.5 for load eccentricity of t/b and t/3 respectively. This seems strange but the few available test results justify the draft values.

6.4.4 Comparison of Permissible Loads

Tables (5) and (8) show comparison of permissible loads in various codes, while calculating these loads the following assumptions have been made:

1) Brick Strength = 28 N/mm² (4000 P.S.I.)
2) Mortar used is one or other of the following mixes:
   (a) Cement: Lime: Sand = 1:1:3
   (b) Cement: Sand = 1:3
   (c) M-type Mortar = as per SOP and Canadian Code. This is the mix which is the nearest equivalent to the above two mixes.

3) For eccentrically loaded walls the average stress has been calculated.
4) Inspected Workmanship
5) Area of the Wall in unity
6) In draft code the partial safety factor γm is taken as 2.0

7. CONCLUSIONS

1) Fig. (2) shows the wide scatter of the available test results for axially loaded walls.
2) The line representing the U.K. CP 111 (1970) values give lower bound and not characteristic values.
3) Test results show significantly higher reduction factors than those given in CP 111 (1970) and draft code above a slenderness ratio of 16.0, but the number of test points is comparatively few. More tests are therefore required in this region.
4) The model tests show scope for further revision of various codes. The draft values can further be modified based on limited number of full scale tests.

REFERENCES


**ACKNOWLEDGEMENT**

The work was sponsored by the British Ceramic Research Association and carried out in the laboratories of the Civil Engineering Department of the University of Edinburgh.

**TABLE I**

<table>
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<th>Wall No.</th>
<th>Slenderness Ratio</th>
<th>Eccentricity of loading</th>
<th>End Condition</th>
<th>Remarks</th>
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**NOTE:**
S.R. means Slenderness Ratio
R.F. means Reduction Factor
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<th>Coeff. of variation (%)</th>
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### TABLE 3
**TEST RESULTS. WS WALLS. (BETWEEN R.C. SLAB)**

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<th>Slender-ness ratio $h/t$</th>
<th>Age of mortar cube days</th>
<th>Mortar strength $1:3$ce $N/mm^2$</th>
<th>Age of walls days</th>
<th>Ultimate compressive stress $N/mm^2$</th>
<th>Ultimate compressive stress $N/mm^2$</th>
<th>Average Ultimate compressive stress $N/mm^2$</th>
<th>Test Reduction factor of CP 111</th>
<th>Reduc- tory factor of $E$</th>
<th>Modulus of Elasticity $E$ $ultr. stress \times 10^2 N/mm^2$</th>
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<td>14.3</td>
<td>13.9</td>
<td>0.75</td>
<td>0.78</td>
<td>5.5</td>
<td>t/22</td>
</tr>
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<td>14.0</td>
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<td>22.7</td>
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<td>2.4</td>
<td>13.7</td>
<td>13.9</td>
<td>0.75</td>
<td>0.78</td>
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<td>t/21.4</td>
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<td>WS14-3</td>
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<td>22.8</td>
<td>64</td>
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<td>13.7</td>
<td>13.9</td>
<td>0.75</td>
<td>0.78</td>
<td>5.5</td>
<td>t/22</td>
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<td>0.62</td>
<td>5.6</td>
<td>t/27</td>
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<tr>
<td>WS19-2</td>
<td>19.8</td>
<td>51</td>
<td>22.3</td>
<td>50</td>
<td>2.2</td>
<td>12.6</td>
<td>12.98</td>
<td>0.70</td>
<td>0.62</td>
<td>4.58</td>
<td>t/31</td>
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<td>WS19-3</td>
<td>15.8</td>
<td>60</td>
<td>20.7</td>
<td>58</td>
<td>2.96</td>
<td>16.1</td>
<td>16.1</td>
<td>0</td>
<td>0</td>
<td>4.97</td>
<td>t/30</td>
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**Fig. 1(a)**
<table>
<thead>
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<tr>
<td>S.No.</td>
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<td>2</td>
</tr>
<tr>
<td>3</td>
</tr>
<tr>
<td>4</td>
</tr>
</tbody>
</table>

Fig. 1(b)
### TABLE 5
DESIGN LOADS (IN NEWTONS) FOR HINGED END WALLS

<table>
<thead>
<tr>
<th>Code</th>
<th>Basic Stress N/mm²</th>
<th>Axial Stress N/mm²</th>
<th>Flexure Stress N/mm²</th>
<th>Slenderness Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>10</td>
<td>15</td>
<td>20</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>e/t</td>
<td>e/t</td>
<td>e/t</td>
<td>e/t</td>
</tr>
<tr>
<td></td>
<td>1/12</td>
<td>1/6</td>
<td>1/3</td>
<td>1/12</td>
</tr>
<tr>
<td>Stress R.F. C.P.111 (1970)</td>
<td>2.06</td>
<td>2.8</td>
<td>2.6</td>
<td>2.06</td>
</tr>
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<td>Load R.F. C.P.111 (1970)</td>
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<td>2.8</td>
<td>2.6</td>
<td>2.06</td>
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<td>3.3</td>
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<td>Load R.F. Australia</td>
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<td>2.85</td>
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<td>2.27</td>
</tr>
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<td>2.2</td>
<td>2.2</td>
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</tr>
<tr>
<td>Load R.F. Canada</td>
<td>2.76</td>
<td>3.53</td>
<td></td>
<td>2.76</td>
</tr>
<tr>
<td>Switzerland</td>
<td>4.8</td>
<td>-</td>
<td></td>
<td>4.8</td>
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</table>

**NOTE:** For U.S. and Canadian codes, the load R.F. values correspond to $e_1/e_2 = -1$, i.e. for fixed end walls and walls with load eccentricity on opposite sides (double curvature).

### TABLE 6
DESIGN LOADS (IN NEWTONS) FOR FIXED END WALLS

<table>
<thead>
<tr>
<th>Code</th>
<th>Basic Stress N/mm²</th>
<th>Axial Stress N/mm²</th>
<th>Flexure Stress N/mm²</th>
<th>Slenderness Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>10 (7.5)x</td>
<td>15 (12)x</td>
<td>20 (15)x</td>
<td>30 (22.5)x</td>
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<tr>
<td></td>
<td>e/t</td>
<td>e/t</td>
<td>e/t</td>
<td>e/t</td>
</tr>
<tr>
<td></td>
<td>1/12</td>
<td>1/5</td>
<td>1/3</td>
<td>1/12</td>
</tr>
<tr>
<td>Stress R.F. C.P.111 (1970)</td>
<td>2.1</td>
<td>2.6</td>
<td>2.1</td>
<td>2.1</td>
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<tr>
<td>Load R.F. C.P.111 (1970)</td>
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<td>2.6</td>
<td>2.1</td>
<td>2.1</td>
</tr>
<tr>
<td>Load R.F. Draft C.P.111 (1973)</td>
<td>3.3</td>
<td>-</td>
<td>-</td>
<td>3.3</td>
</tr>
<tr>
<td>Load R.F. Australia</td>
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<td>2.8</td>
<td></td>
<td>2.27</td>
</tr>
<tr>
<td>Load R.F. U.S.A.</td>
<td>2.2</td>
<td>2.2</td>
<td></td>
<td>2.2</td>
</tr>
<tr>
<td>Load R.F. Canada</td>
<td>2.76</td>
<td>3.5</td>
<td></td>
<td>2.76</td>
</tr>
<tr>
<td>Switzerland</td>
<td>4.0</td>
<td>4.4</td>
<td></td>
<td>4.0</td>
</tr>
</tbody>
</table>

**NOTE:** Values in brackets are for C.P.111 and Australian code to take care of end conditions.
Plate 1: MI 25 Wall ready for test (Flat ended)

Plate 2: Test rig for XS Walls (Between R.C. slabs)

Plate 3: Failure of MI 25 Wall (Flat ended)

Failure of $4 - \frac{1}{4} = 25$ Wall (Rings end)
**Fig. 2**

REDUCTION FACTOR VERSUS SLENDERNESS RATIO

- **Fig. 3**

COMPARISON OF TEST RESULTS

- **Fig. 4**

COMPARISON OF TEST RESULTS

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NOTE: LRF calculated by taking SR = 12 as the basis for WATSTEIN & ALLEN results. SR = 9 basis for GRAVE & MOTTED tests.
HENDRY A.W.
HASAN S.S.

EFFECT OF SLENDERNESS RATIO
ON COMpressive STRENGTH OF WALLS
EFFECT OF SLENDERNESS RATIO ON THE COMpressive StRENGTH OF WALLS
by
S.S.Hasan, A.W.Hendry
University of Edinburgh

1. Introduction

There appears to be a scarcity of experimental information on the effect of slenderness ratio on the compressive strength of brick walls. A programme of tests was therefore undertaken on 1/3rd scale model walls to provide further data against which to examine the provisions of design codes. Model scale testing has been used extensively in the author's laboratory and the results have previously been validated against full scale tests /1/.

2. Test Programme

Table 1 gives the details of the programme.

3. Materials

3.1. Bricks

One third scale bricks were used. Bricks came in batches so their strength varied. They were tested in accordance with BS3921:1969 Part 2/a/.

3.2. Sand and Cement

In all tests dry Leighton Buzzard 25/52 sand was used in conjunction with rapid hardening Portland Cement /Ferrocrete/.

3.3. Mortar

The mortar mix was 1:3 cement: sand by weight. With each wall, six 25.4 mm mortar cubes were made by hand compaction. The water cement ratio varied from 0.8 to 0.95.

4. Experimental Procedure

4.1. Construction of Walls

The walls were built in wooden jigs. The thickness of the mortar bed was scaled down to 3.2 mm by drawing guide lines on the wooden backing of the jig to mark each course. Walls were
<table>
<thead>
<tr>
<th>Wall No.</th>
<th>Slenderness Ratio</th>
<th>Eccentricity of loading</th>
<th>Condition</th>
<th>Remarks</th>
</tr>
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<tbody>
<tr>
<td>28.1 mm thick x 480 mm wide in stretcher course</td>
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<td></td>
<td></td>
<td></td>
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<tr>
<td>WM6</td>
<td>5.4</td>
<td>0</td>
<td>Flat ended</td>
<td>Slenderness Ratio calculated as 0.9 H/t where H = height of wall t = actual thickness</td>
</tr>
<tr>
<td>WM12</td>
<td>10.8</td>
<td>0</td>
<td>&quot;</td>
<td>&quot;</td>
</tr>
<tr>
<td>WM18</td>
<td>16.2</td>
<td>0</td>
<td>&quot;</td>
<td>&quot;</td>
</tr>
<tr>
<td>WM25</td>
<td>22.5</td>
<td>0</td>
<td>&quot;</td>
<td>&quot;</td>
</tr>
<tr>
<td>J6.2 mm thick x 380 mm wide (English-Bond)</td>
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<td></td>
<td></td>
<td></td>
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<tr>
<td>B1W6</td>
<td>5.4</td>
<td>0</td>
<td>&quot;</td>
<td>&quot;</td>
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<td>B1W8</td>
<td>7.2</td>
<td>0</td>
<td>&quot;</td>
<td>&quot;</td>
</tr>
<tr>
<td>B1W12</td>
<td>10.8</td>
<td>0</td>
<td>&quot;</td>
<td>&quot;</td>
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<td>B1W18</td>
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<td>0</td>
<td>&quot;</td>
<td>&quot;</td>
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<td></td>
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<td>WS4</td>
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<td>R.C.C.Slab</td>
<td>S.R. calculated as 0.75 H/t</td>
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<td>WS6</td>
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<td>0</td>
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<td>WS9</td>
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<td>&quot;</td>
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<td>WS19</td>
<td>19</td>
<td>0</td>
<td>&quot;</td>
<td>&quot;</td>
</tr>
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<td>38.1 mm thick x 480 mm wide in stretcher course</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Wo-6</td>
<td>6</td>
<td>0</td>
<td>Hinged</td>
<td>S.R. calculated at H/t</td>
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<tr>
<td>Wo-12</td>
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<td>&quot;</td>
<td>&quot;</td>
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<td>0</td>
<td>&quot;</td>
<td>&quot;</td>
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<td>&quot;</td>
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<tr>
<td>W - 1/6 - 6</td>
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<td>t/6</td>
<td>Hinged</td>
<td>S.R. calculated as H/t</td>
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<td>&quot;</td>
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<tr>
<td>W - 1/6 -18</td>
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<td>W - 1/6 -25</td>
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<td>&quot;</td>
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<tr>
<td>W - 1/3 - 6</td>
<td>6</td>
<td>t/3</td>
<td>Hinged</td>
<td>S.R. calculated as H/t</td>
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<td>W - 1/3 -12</td>
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<tr>
<td>W - 1/3 -18</td>
<td>18</td>
<td>&quot;</td>
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<tr>
<td>W - 1/3 -25</td>
<td>25</td>
<td>&quot;</td>
<td>&quot;</td>
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</table>
4.2. Wall Tests and Measurements

All walls except the WS groups were tested in an Avery Universal Testing machine. Prior to each test, a steel beam of appropriate cross-section, depending upon the thickness of the wall was bedded with a 1:1 cement: sand mortar on top of the wall and then levelled in two perpendicular directions.

Walls of the WS groups /Figure 1/ were tested in a specially designed rig. They were bedded in 1:1 cement: sand mortar between 38 mm thick reinforced concrete slabs. The bottom slab was bedded on four courses of brickwork and its two ends were supported on steel angles. The top RC slab was bedded on the wall in a similar way. The load was applied through a loading beam bedded on two courses of brickwork on the upper slab.

4.2.1. Application of Load

For all walls except those in the WS group the load was applied by an Avery Machine. For WS group walls, the load was applied by means of a hydraulic jack and load cell.

4.2.2. Strain Measurement

Vertical strains were measured by compressometer and Demec gauges of different gauge lengths depending upon the height of the walls. The average of four readings gave the final strain value for each load.

4.2.3. Lateral Deflection

The lateral deflection was measured by dial gauges of 0.00254 mm sensitivity.

FIGURE 1

LOAD

SPREADER BEAM

40mmx50mm

75mm

2-COURSE BRICK WORK

ANGLE SECTION

VARYING HEIGHT

38mm THICK

38mm RCC SLAB

1:1 MORTAR MIX

125mm

4-COURSE BRICK WORK

STEEL BASE

SCALE 1:5
5. Discussions

5.1. Mode of Failure

In all walls except walls of Wo, \( W_6^1 \) and \( W_3^1 \) groups, the first hair line crack appeared between 50-60% of failure load and enlarged with further increase in load. The general mode of failure of the walls was vertical splitting accompanied by crushing and spalling of various courses of brick. There were vertical cracks in the thickness of the walls as well.

Walls of Wo group failed mostly by tensile vertical splitting up to slenderness ratios of 12. Above slenderness ratios of 12 the failure was both due to tensile vertical splitting and bending. Maximum bending usually occurred at the mid height of the wall.

In walls of \( W_6^1 \) group the failure up to slenderness ratios of 18 was due to the combined effect of tensile splitting, spalling and bending. Walls of slenderness ratio of 25 failed mainly by bending which occurred between 12th and 14th course from top. There was a complete absence of tensile splitting and spalling.

Walls of \( W_3^1 \) group mainly failed by bending except in the walls \( W_3^1-6-1, 2 \) and \( 3 \) which had some spalling as well.

5.2. Strain Readings

Strain readings on both faces of the axially loaded walls were not greatly different from each other implying a fairly uniform distribution of load on the walls. The difference in strain readings was higher in the initial stages but this reduced with the increase in load.

5.3. Modulus of Elasticity

A stress strain curve for finding the value of the modulus of elasticity \( E \) was plotted on the basis of an average of four strain readings for all walls. The \( E \) values fluctuated during the lower range of loading, but afterwards at about 25-40% of ultimate load it started decreasing with the increase in load.

5.4. Deflection

The lateral deflection measurements for \( W1, B, W \) and \( WS \) group of walls showed that none of the walls deflected to the same extent. Some walls had a deflection curve similar to a sine curve e.g. walls of Wo, \( W_6^1 \) and \( W_3^1 \) group.
5.5. Slenderness Ratio

Murthy and Hendry /1/ have established that the model test results can be applied to full-scale brickwork structures provided thickness of mortar joint and size of mortar cubes used for mortar strength determination are scaled down. The following section compares the model test results with other full-scale test results.

5.5.1. WN and B1W Group

Taking reduction factors for the squat walls /i.e. walls having a slenderness ratio of 6/ as 1.0, the stress reduction factors of different walls can be compared. On this basis the reduction factor for WM12 is greater than 1.0 and for WM18 and WM 25 is less than 1.0, as was expected. Figure /2/ shows that these tests when compared with American tests /2/ on 102 mm thick full scale walls had higher reduction factors up to the slenderness ratio of 19, above this value the reduction factor was lower than in the American tests. This could be attributed to differences in workmanship, which becomes important in the case of more slender walls. The reduction factors in the American tests have been calculated on the basis of the lowest wall having a slenderness ratio of 4.3 and not 6 as with the tests described here and in CP111. CP111 /6/ assumes no reduction in strength up to a slenderness ratio of 4.3 but in American tests the strength reduces by 10% at a slenderness ratio of 6.0. This when compared with CP111 gives a higher difference in reduction factors than it would be if the reduction factor is taken to be 1.0 at a slenderness ratio of 6.0. This aspect should be kept in mind while comparing different codes and tests.

The test results of Sinha and Hendry /3/ /Fig. 2/ on a one sixth scale model wall of 114.3 mm equivalent thickness had the same reduction factor as that for the WM18 wall but had higher reduction factors for walls of slenderness ratio above 18.0.

The bricks used for the wall WM25 had compressive strength 20% lower than the bricks of walls WM6, WM12 and WM18. This would result in a lower strength reduction factor. The values of Reduction Factor have been accordingly adjusted. The results of the 114.3 mm thick full scale tests by Thomas /4/ show almost no decrease in reduction factors with increase in slenderness ratio /Fig. 2/.

In B1W wall tests the ultimate stress of B1W18 is found to be greater than that of B1W6, B1W8 and B1W12 walls. This is due to
better vertical alignment of wall as compared to other walls of these groups. Also there is better distribution of load as is evident from the strain readings.

The reduction factors given in various codes are found to be lower than the reduction factors of the B₁W walls and 203 mm thick full-scale American wall tests /2/.

The deflection curves for the flat ended walls vaguely indicate a point of inflection between 0.9 and 0.95 times actual height. The Australian code recommends the effective height to be 0.85 times actual height /5/. This condition is approximately similar to flat ended walls, hence instead of taking effective height for flat ended walls as \( \frac{3}{4} \) of the actual height as given in CP111 an effective height equivalent to 0.9 times actual height is taken.

However if slenderness ratio of B₁W and WM walls are calculated on the basis of \( \frac{3}{4} \) actual height, the stress reduction factors so obtained are higher than those given in CP111.

5.5.2. WS Group

The reduction factors for WS group of walls are higher than those specified in CP111 except for wall WS-14. This wall was not well built in comparison to other walls and due to a fault in the loading system, the rate of loading was lower than the other walls. This may be the reason for the low strength of this particular wall.

The R.F.'s of WM and B₁W walls are higher than of WS walls, although the WS walls should have the same reduction factors if not higher. The brick strength of WS walls are lower than the brick strength of WM and B₁W walls. This may have had some effect on the reduction factors. Thomas' test results /4/ indicate that walls of weak brick have lower reduction factor than the walls made of strong bricks.

5.5.3. Wo, \( W_0^1 \) and \( W_3^1 \) Group

The stress reduction factors of Wo and \( W_0^1 \) walls are higher by about 20% than those specified in CP111.

The reduction factors for the \( W_3^1 \) walls are very much lower than those of CP111. The eccentricity of loading calculated from strain measurements are found to be higher than was intended. Also, low strength bricks were used in this group of walls. These factors may be responsible for low reduction factors.
The eccentricity of loading for Wo and \( W_{\frac{1}{6}} \) group of walls was also greater than intended. These reduction factors will actually be higher if the eccentricity is not allowed to exceed the desired value.

The ultimate strength of \( W_{\frac{1}{6}} \) and \( W_{3} \) walls are very sensitive to the absorptive properties of bricks. Haller's tests /8/ indicate a substantial decrease in the strength of slender brick walls made up of highly absorptive bricks especially under eccentric loading. It is probable that the bricks used in these walls had different absorption capacities at the moment of laying in spite of the efforts made to ensure a constant soaking time for all bricks. Fig. /2/ also shows a comparison between load reduction factors of the draft for revision of CP111 issued in Oct.1973 and CP111 /1970/. This draft suggests a uniform reduction factor for eccentrically loaded walls up to a slenderness ratio of 18.5, depending upon the magnitude of eccentricity. This is not in accordance with the test values or with the CP111 /1970/ recommendations. It does not seem reasonable to have the same strength for a wall of slenderness ratio 6 and a wall of slenderness ratio 18 particularly with as high an eccentricity of loading as \( t/3 \). The test results, however, seem to fall below draft specification values for eccentrically loaded walls and above them for axially loaded walls.

To sum up it is seen that the reduction factors in CP111 are lower than the test values particularly at higher slenderness ratio. This high safety factor at high slenderness ratio is probably intended as a precaution against the effects of poor workmanship in very slender walls. Figure /3/ compares reduction factors of the various codes /10/.

6. Conclusions

Figure /2/ shows the wide scatter of the available test results for axially loaded walls.

The line representing the U.K. CP111 /1970/ recommendations give lower bound and not characteristic values.

Test results show significantly higher reduction factors above a slenderness ratio of 18, but the number of test points is comparatively few. More tests are therefore required in this region.

A revision of CP111 /1970/ values based on these tests supplemented as required should give characteristic values which will have higher reduction factors than those given at present.
The reduction factors given in CP111 /1970/ for walls of t/6 eccentricity are in agreement with the test results. Reduction factors for walls of t/3 eccentricity are higher than the test values. This is because of the eccentricity of loading as calculated from strain measurements being higher than the nominal eccentricity of t/3.

REFERENCES

/7/ B.S. CP111 draft revision of code issued in 1974, B.S.I.
/10/ Macchi, C. "Safety considerations for a limit state design of brick masonry" - SIBMkC Proc. 1970 - BCRA - Stoke-on-Trent.

ACKNOWLEDGEMENTS:
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FIGURE 2
REDUCTION FACTOR VERSUS SLENDERNESS RATIO

FIGURE 3
COMPARISON OF CODES