

ASPECTS OF THE DESIGN OF THE MULTISTOREY BUILDINGS IN
LIGHT-WEIGHT CONCRETE BLOCKWORK

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ABSTRACT

The work presented in this thesis is concerned with design aspects of multistorey buildings in light-weight concrete blockwork.

The investigations concern the ultimate strength of wall panels under axial compression, the ultimate shear strength of single storey shear-wall structures with openings, and an investigation into the behaviour of a five-storey shear-wall structure subjected to lateral loading.

In order to ascertain the suitability of model blocks, tests are carried out on a number of full-scale and one-third scale storey height panels built with four varieties of light-weight aggregate concrete blocks covering a wide strength range. The test results show that it is possible to reproduce, with reasonable accuracy, the strength of full-scale blockwork by means of model tests. Other relationships and properties such as stress-strain, Young's Modulus of Elasticity, Poisson's ratio tensile strength, ultimate load and mechanism of failure have been investigated.

A comparison with CP 111:Part 2:1970, shows that the present provisions of the code are adequate for the design of wall panel under axial loading.

Phase 2 of the investigation concerns the shear strength of blockwork. Preliminary tests were carried out on one-third scale concrete block masonry triplets to find the effect of precompression on the ultimate shear strength. The test results indicate that the shear strength of blockwork triplets is the sum of bond shear and frictional resistance between the block and mortar. A statistical analysis of the test's results shows that /

shows that the coefficient of friction between the block and mortar used was 0.73. It has also been observed that precompression increases the shear strength of the triplets.

Further, direct tensile tests on a number of one-third scale, one half-block couplets proves that, for the material used the shear strength of triplets at zero precompression is about 1.3 times the tensile bond strength between the block and the mortar.

Previously the blockwork shear wall was treated as an isolated element with a concentrated restraining load on the top rather than a realistic uniform load from the slab.

A study is made of the ultimate load behaviour of single-storey coupled shear wall structures containing door openings and subjected to precompression. Several racking tests are carried out on one-third scale single-storey coupled-shear walls connected through slabs.

Deflections, strains, ultimate shear strength and the failure mechanism under a set range of precompression are studied. From the results of the experimental work, it was observed that blockwork shear wall, depending on the amount of applied precompression exhibit two distinct types of failure: (1) by shear failure, a combination of bond and frictional resistance due to precompression. (2) tensile failure, governed by the maximum tensile strength of blockwork.

The structure is analysed using the finite element method and the equivalent frame analogy. Both methods give close estimate of the stresses and deflections at high precompression. The work is further extended to investigate the behaviour of five-storey shear wall structure in one-third scale blockwork subjected to lateral loading. A brief review of the current analytical methods for the solution of idealized shear/

idealized shear wall structures, including a survey of previous work on multistorey brickwork is outlined in Chapter 5.

Experimental stresses and deflections, due to lateral loading, are compared with the results of analysis using the cantilever method, shear continuum approach, wide column frame analogy, equivalent frame analogy, and the finite element method. While no method conveys an accurate representation of the structure, the equivalent frame analogy gives the best approximation to the lateral deflection; stresses are overestimated which is safe from design point of view.

CONVERSION FACTORS

	Imperial Units	SI Units
Length	1 ft	0.3048 m
	1 in	25.4 mm
Area	1 in ²	645.2 mm ²
	1 ft ²	0.0929 m ²
Section Modulus	1 in ³	16.39 x 10 ⁻⁶ m ³
Second Moment of Area	1 in ⁴	0.4162 x 10 ⁻⁶ m ⁴
Mass	1 lb	0.4536 kg
Density	1 lb/ft ³	16.02 kg/m ³
Force	1 lbf	4.448 N
	1 ton f	9.964 kN
Force/unit length	1 lbf/ft	14.59 N/m
	1 tonf/ft	32.69 kN/m
Pressure	1 lbf/in ²	6.895 kPa (kN/m ²)
	1 lbf/ft ²	47.88 Pa (N/m ²)
Moment of Force	1 lbf in	0.1130 N m
	1 lbf in/ft	0.3707 N m/m

CHAPTER 1

1.1 INTRODUCTION

In recent years, the need to build as economically as possible and to utilise materials to their fullest potential has encouraged rapid development in the building industry. Considerable changes have been revealed by a comparative study of modern masonry construction and that of fifty years ago. The modern masonry building no longer stands solely as a monument to traditional structural craftsmanship but, like steel and reinforced concrete, it now falls within the field of calculated design.

In a modern building, walls have many functions to perform. They are required to provide enclosure, to resist sound and heat penetration, to be fire-resistant, and to withstand gravity and lateral forces. Concrete blocks seem to be an adequate building materials satisfying these requirements (56).

Recently, in the United States of America (61,101) increasing number of designers and builders have employed load-bearing concrete blockwork in the construction of 'high-rise' buildings ranging from three to thirteen storeys. These buildings form 80% of all permanent masonry construction (including brick), and are designed and constructed either as reinforced or as non-reinforced masonry structures, depending on the code of practice, building height and seismic condition.

The Canadians are considered to be most advanced in the field of load-bearing blockwork structures. The first outstanding example is a thirteen storey apartment building (42) erected in 1964. The 259 units consists of concrete block load-bearing walls and a combination of bricks and blocks/

and blocks on the exterior.

In this chapter the historical background of concrete-block industry in Great Britain is briefly reviewed and the various types and the advantages are discussed.

1.2 HISTORICAL BACKGROUND

Although attempts had been made on a small scale to introduce the concrete block during the latter half of the 19th century, by men such as Joseph Gibbs⁽⁸⁾ about 1850, it was not until the turn of the century that the concrete block became of any significance. The manufacturing of concrete blocks coincided with the growth of the cement industry about 1910, and the rapid development of technology in this field. Its acceptance was slow at first, hampered by the competition of the fired clay brick. However, before long the concrete block, namely Clincker or "Breeze" block^(67, 68, 69) proved itself to be a more viable proposition. They were more economical and convenient to produce, construction time was reduced and 60% of the amount of mortar required for a brick wall of the same thickness saved. In Scotland, Wales, the Channel Islands, and the West Country acceptance was rapid and widespread. This was primarily due to an uneven distribution of suitable brick material and the fact that concrete block offered a more convenient building material than the local natural stone.

As the concrete block industry developed the variety increased. During the First World War period dense and lightweight blocks went into production. In 1924 there was an attempt to introduce aerated blocks but this particular type proved non-viable due to inadequate control of the curing process and the fact that the blocks were not autoclaved. The split blocks appeared in use in Scotland in 1932 (Torphin Bridge, Midlothian)⁽⁸⁾.

These blocks are still in production today and are widely used, as a recent example the bridge on Preston-Lancaster motorway M6⁽³⁷⁾ can be mentioned.

World War II led to a boom in the manufacture of concrete blocks. Most brickworks were closed down at the beginning of the War but building materials were badly needed for military blocks, defence projects and factories. Although it gave an impetus to the amount produced, in actual fact the war period rendered the development of the concrete blocks stagnant in terms of variety and experimentation. The main production was of the dense aggregate blocks, solid and hollow which suited all demands and purposes.

Building activity after the war had a slow start. There was difficulty in obtaining new plants. However, from 1945 onwards the use of lightweight blocks, which had been somewhat neglected during the war period in favour of the dense aggregates, was extended and by 1950, the former was being used for partition, and inner leaf of cavity construction.

The lightweight block was cheaper and more readily available than brick and they had better thermal insulation and acoustic properties.

The next significant step in the concrete block development came in 1950 with the mechanisation of production mainly in the form of egg-laying type machines. By 1955 this form of mechanisation was widespread and machines were producing blocks to their full capacity. It is reported that the annual production of five million square yards in 1950 increased to 65 million square yards in 1965^(68, 102). The introduction of the automatic pallet type machine since then represents another step forward in the mechanisation of production. The block industry is now developing into a /

into a highly mechanised and efficient one, having a high degree of control over its products, and producing blocks with pleasing appearance and high dimensional accuracy which makes them a favourable material for construction of 'high-rise' structures.

1.3 TYPES OF CONCRETE BLOCK

The choice of concrete blocks for structural and facing purposes is wide, ranging from glossy smooth to a natural rugged texture and profile^(23,24,60) and in colour from white to black, with a range of grey in between.

Broadly speaking, the range of facing blocks available are the plain blocks, split-blocks, profile blocks, pierced blocks, exposed aggregate blocks and glazed blocks Fig. (1.1). Using facing blockwork internally eliminates the need for surface finish, and hence is an important development in speeding of construction. Their strength and absorption and texture are uniformly controlled, and shrinkage is reduced to the minimum. More generally, concrete blocks may be dense (150 lb/cu.ft), lightweight (made of expanded slate, shale or clay up to 130 lb/cu.ft depending on the strength), solid or hollow depending on the structural need.

The cores in hollow blocks may be used for reinforcement or services. The standard size is conforming to BS 2028 (1968)⁽¹⁰⁾ are commonly 18 inches by 9 inches by 3, 4, 6 or $8\frac{5}{8}$ inch thick, fig (1.2). Modular block designed to the standard 4-inch module are available. This has face dimensions of 16 inches by 8 inches Fig.(1.3). The wide range of densities and thicknesses of concrete block provide flexibility in design. In order to achieve efficiently any reduction in sound transmission from one room to another, aerated blocks are chiefly used as partitioning materials which offer excellent sound and heat insulation properties. In Chapters 3, 4 and 6 the structural properties of lightweight concrete blockwork will be more deeply studied.

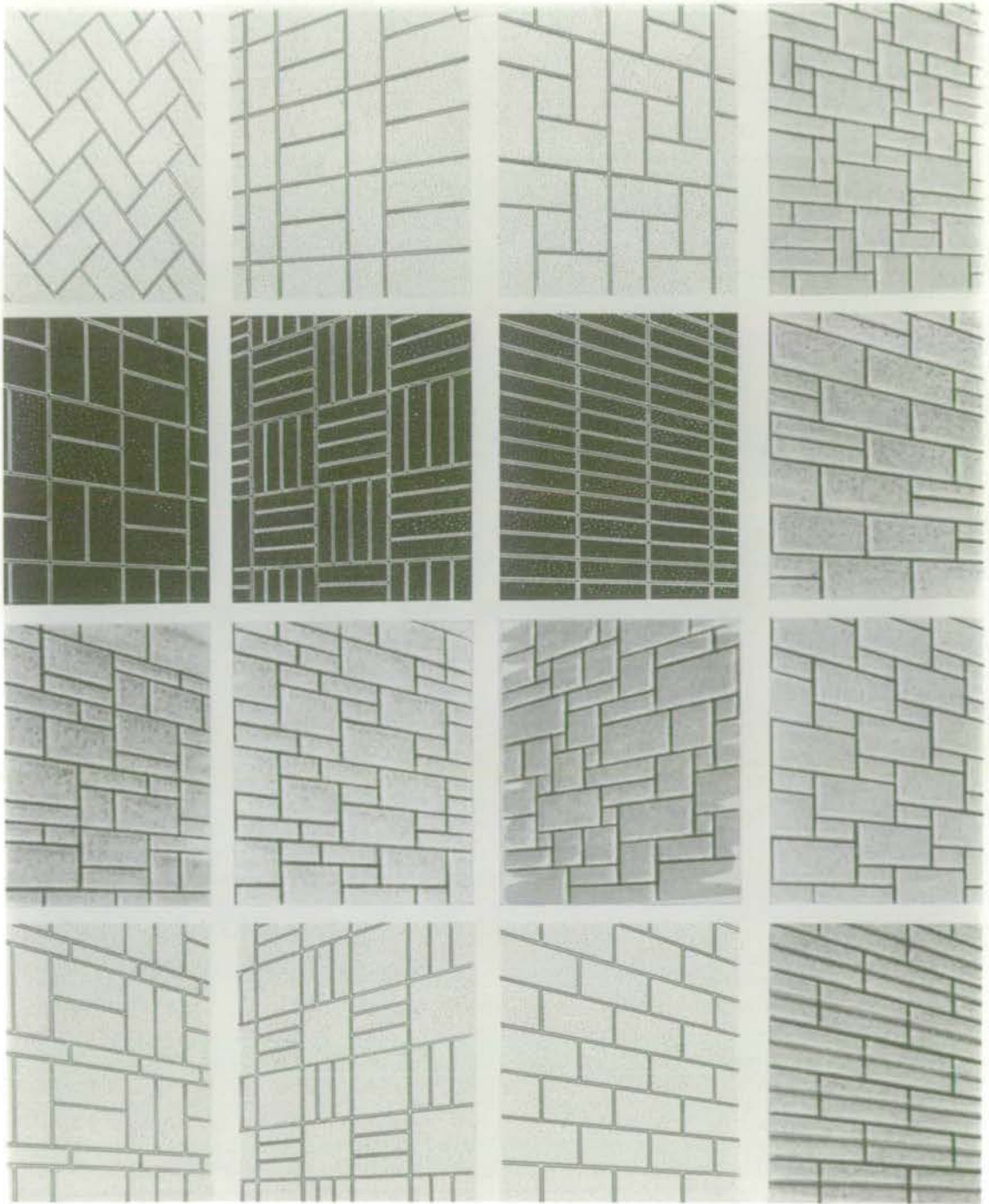
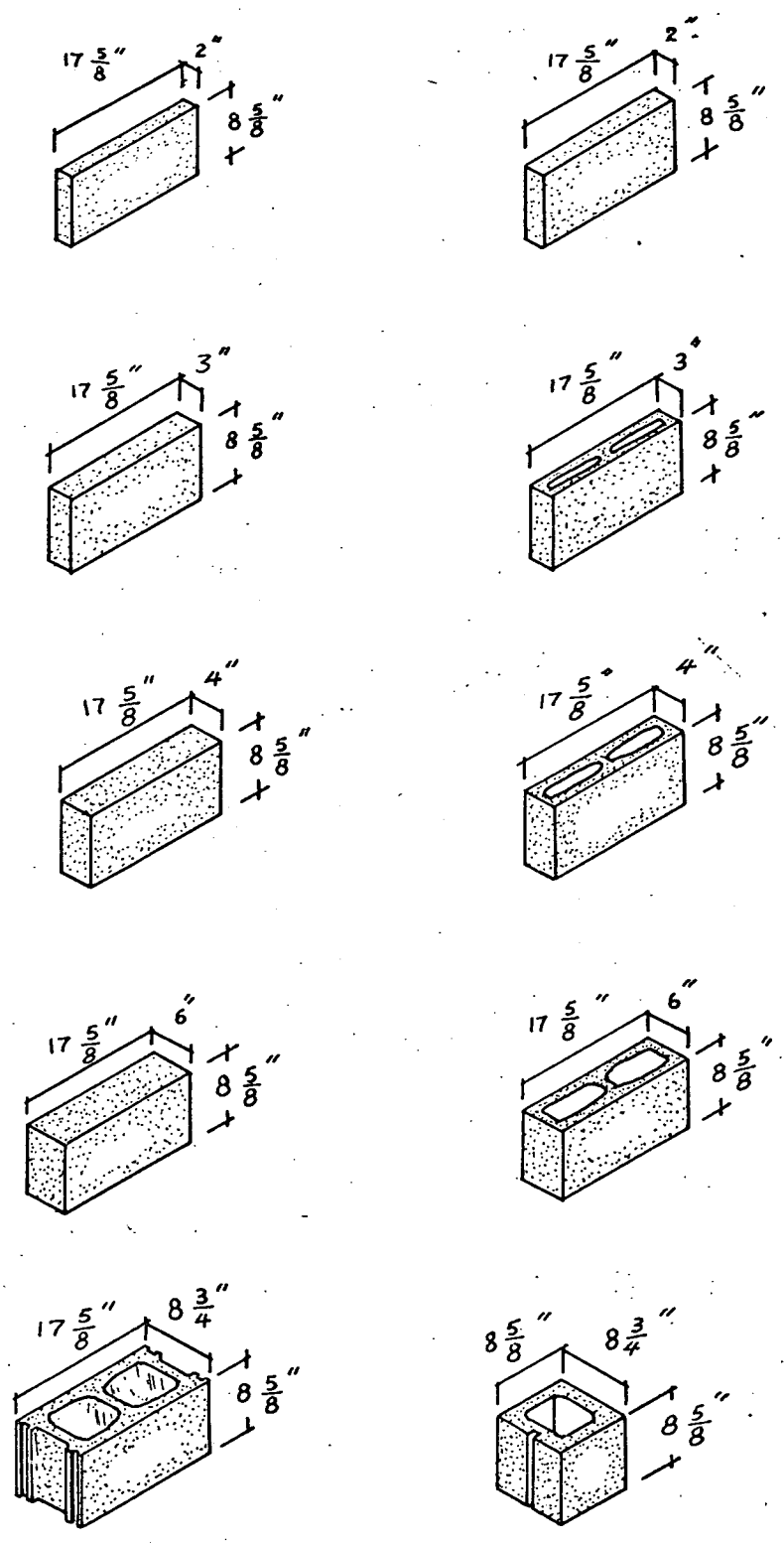


Plate 1.1 DIFFERENT FACING PATTERNS WITH VARIOUS
TYPES OF BLOCKS



Solid & Hollow Concrete Blocks.

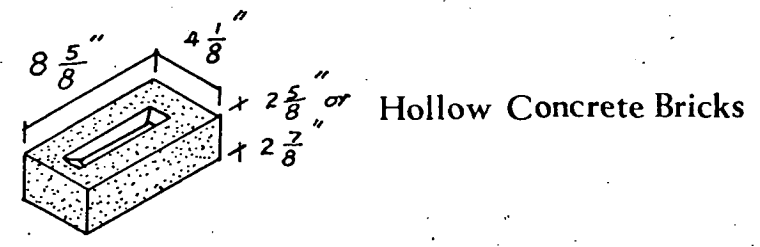


Fig. 1.2. Blocks & Bricks To British Standard Sizes.

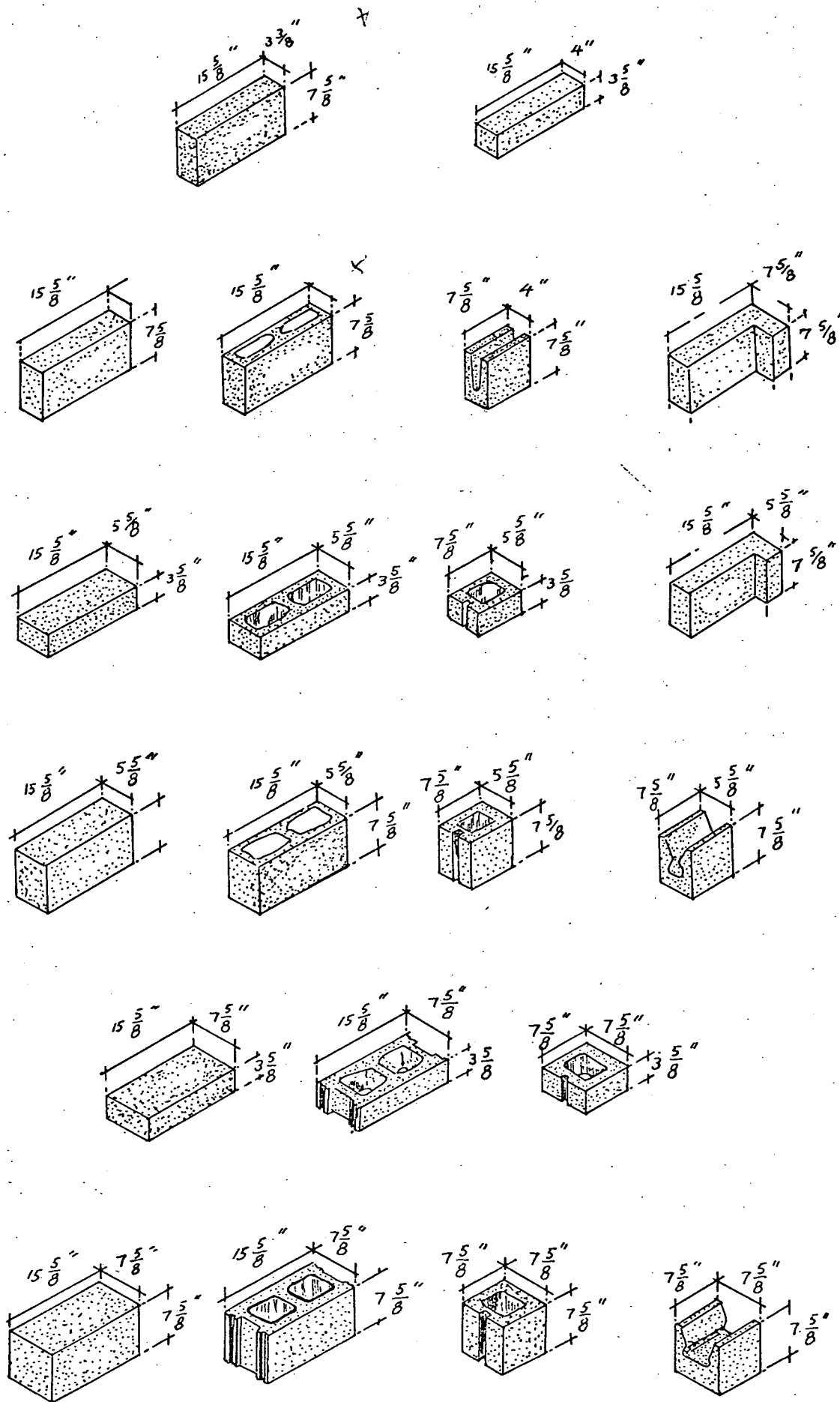


Fig.1.3. Solid & Hollow Blocks To American 16" x 8" Modular Sizes

1.4 FUTURE DEVELOPMENT OF CONCRETE BLOCK INDUSTRY.

Rapid changes have occurred in the making of concrete blocks which have raised it to the level of a major industry. The development of the concrete block must progress further in terms of quality of components, quantity, availability and research. Future development entirely relies on the need of building industry and therefore production must meet requirements such as quality and structural performance.

Recently, interest in structural blockwork has become widespread because of a combination of factors rather than due to any one reason. Amongst these may be included: the latest changes to CP 111 (1964), structural recommendations for loadbearing brick and blockwork⁽¹⁵⁾; the revision of British Standard 2028, 1364, (1968)⁽¹⁰⁾ for precast concrete blocks, and the increasing ease with which concrete blocks of a high standard could be obtained. Amongst others, efforts of the brick industry directed towards expanding the use of load-bearing walls through research and development⁽¹⁶⁾; the realisation by many people of the visual rigidity and lack of beauty of normal bricks; cost-benefit analysis bringing out the fact that blockwork has most of the economic advantages of brickwork together with some of its own; and lastly as a solution to some of the present day problems of stability and realisation that blockwork can be easily reinforced.

CHAPTER 2 - REVIEW OF LITERATURE RELATED TO CONCRETE

BLOCKWORK

A general literature survey related to the properties of concrete blocks and blockwork was published by Cement and Concrete Association⁽⁸⁵⁾, in which the compressive and flexural strength of concrete blockwork has been briefly mentioned. This chapter reviews more comprehensively literature and research information about the structural performance of blockwork, covering the compressive strength (under axial and eccentric loads), racking strength, flexural strength and bond strength.

Most of the research work on concrete block masonry has been carried out in the United States of America dating back to 1932, when a series of tests was conducted by Richart et al⁽⁸²⁾ to:

1. Establish a relationship between physical properties of masonry units, mortar and walls.
2. Determine various factors which may affect the masonry strength.
3. Study the stability and the strength of concrete masonry walls under axial and eccentric loading.

Dense hollow blocks (16 in. x 8 in. x 8 in.) having three oval cores with strengths ranging from 550 to 1570 lbf/in² were used. The mortar was generally stronger than the blocks with cylinder crushing strengths ranging from 670 to 2200 lbf/in². A total of 42 wallettes (32 in. long x 48 in. high) were tested to establish a scale factor in relation to full-size wall panels. Sixty nine 6 ft. 0 in. long by 2 ft. 6 in. high full-scale walls were tested. The walls were all cured for 32 days prior to the test. The average ratios of wall to wallettes strength and/

strength and of the wall to block strength were found to be 0.91 and 0.53 respectively. The latter ratio was found to be constant. The initial tangent modulus elasticity for the walls varied from 0.3×10^6 to 1.17×10^6 lbf/in². When the walls were tested eccentrically a 24% reduction in wall strength was observed for a load eccentricity of 1/3 of the section thickness. In almost all cases failure took place by splitting off the faces of the concrete block units adjacent to the horizontal mortar joints.

Following Richart et al work, Copeland and Timms⁽²⁶⁾(1932), tested 108 concrete masonry wallettes using hollow units ranging in strength from 320 to 4180 lbf/in², and mortars ranging in strength from 150 to 4800 lbf/in². The purpose was to study the effect of the strength of mortar and units on the strength of concrete masonry. The strength ratio ranged from 0.33 to 0.79. They concluded that for a given mortar strength, wall strengths increases linearly with the block strength and that the potential strength of a wall can be obtained by using mortar at least as strong as the blocks.

Kristen and Schulze⁽⁵⁷⁾(1936) carried out similar tests to those of Richart et al, on walls and wallettes made with various types of blocks. A value of 0.81 was obtained as the strength ratio of the large to the small walls. Walls were 5 ft. long x 10 in. high and wallettes were 2 ft. 8 in. long x 2 ft. 8 in. high, both having the same thickness. In 1938, Whittemore, Stang and Parsons^(104a) conducted tests to assess the structural properties of low-cost housing construction. Dense hollow blocks of 1760 and 2880 lbf/in² crushing strength (on net area) were used. Mortar strength varied from 630 to 3220 lbf/in², As the second part of the same investigation^(104b) in 1939/

in 1939 they tested 12 cavity walls 8 ft. 3 in. high, 4ft. 0 in. long and 10 in. thick with a 2 in. cavity. In 1939 as the final part of the investigation^(104c) tests were carried out on 48 cavity walls, 36 of which had brick facing with a concrete block backing while the remaining 12 walls were built in concrete blocks. Walls were tested under compressive, transverse, concentrated, impact and racking loads. In the compression tests single-leaf walls failed by crushing of two to four of the top courses and vertical splitting. In the case of cavity walls failure appeared in the backing. Failure in transverse loading was by rupture of the bond between the facing and backing at bed joints. In racking tests loads were applied near the upper end of each wall specimen to a bearing-plate covering both the facing and backing, the stop was also in contact with both. At the ultimate load failure in walls built with block and mortar of approximately the same compressive strength was by crushing of blocks in both facing and backing and diagonal shear along a diagonal path through the blocks. The calculated shear strength was 12 lbf/in² on gross area. Single leaf and cavity walls constructed with mortar weaker than the block failed by rupture of the bond between the mortar and the block in stepwise cracks along the bed and head joints diagonally from load to a stop in both facing and backing. Herrmann⁽⁴⁷⁾ in 1943 using numerous test results for a variety of blocks, both hollow and solid arrived at the expression

$$w = k \sqrt[3]{m \cdot s^2}$$

where w = blockwork strength

m = mortar strength

s = block strength

k = block characteristic constant.

The value/

The value of k for solid blocks and $\frac{s}{m}$ ratios 4.4, 7.5 and 9.4 were 0.85, 0.71 and 0.66 respectively. For walls constructed with hollow blocks values of k were found to be 0.57, 0.57 and 0.55 for the respective $\frac{s}{m}$ ratios of 6.3, 3.8 and 3.9.

In 1944 Nylander⁽⁷⁷⁾ used a variety of solid and hollow blocks for the construction of his test walls. The object of this investigation was to determine the strength of walls built with blocks and mortar other than the ones he used in his tests. Walls were 3ft.3in. long and 9 ft. 10 in. high. Three walls from each specimen were built and tested in axial compression and eccentric loading applied at the top and bottom, in opposite and in the same directions. Blocks tended to be much stronger than the mortar used. By studying the mechanism of failure a formula was suggested for the wall strength based on block and mortar strength. In all tests failure occurred by vertical splitting of the blocks. Direct measurements by Hast⁽⁴³⁾ (1943) had shown that compressive stresses are set up in the mortar at right angles to the direction of compressive load causing tensile stresses in the block and ultimately failure in the wall by tensile splitting. Assuming that the tensile strength of concrete block is proportional to:

$$\sqrt[3]{(\text{block strength})^2}$$

and knowing that the tensile stresses in the block may be proportional to the wall compressive strength minus the compressive strength of the mortar Nylander arrived at the following equation;

$$w = m + K \sqrt[3]{s^2}$$

where k being a constant characteristic of the block. The above expression only/

expression only holds if other properties of blockwork such as thickness of joints, type of block and its dimension are kept constant. Development of an expression to include the above variable entails an extensive investigation. Since the above formula is based on the assumption of a tensile failure, the formula suggested applies only to the cases where the ratio of block strength to mortar strength or vice versa is great. For blocks of 1180, 2020 and 2902 lbf/in², K was calculated, being 1.26, 1.18 and 1.21 with 7% variation. For block strengths of 1607, 1124 and 1152 lbf/in², K was 1.10, 1.00 and 0.96 respectively the variation being 14%. Mortar strength was not greater than 285 lbf/in².

Fishburn⁽³⁵⁾ in (1961) tested 114 masonry walls, 58 of these were constructed with hollow blocks and the remainder of composite construction with a brick facing having a concrete block backing. The programme included a study of the physical properties of mortar, bond strength between mortar and blocks, and the strength of masonry walls in compression. Walls tested under racking and flexure loads were 4 ft. to 8 ft. long and 8 ft. to 8.67 ft. high, cured in the laboratory and tested after fifteen days. Block compressive strength was 2100 lbf/in². Under compressive load failure was by vertical splitting and the average compressive strength on the net area of the walls ranged from 1030 to 1240 lbf/in² depending on the grade of mortar used. The strength ratio ranged from 0.49 to 0.58 with an average of 0.54. In general, mortar strength had little effect on the strength of blockwork. The secant modulus of elasticity at 100 lbf/in² compressive load on gross area ranged from 0.53/

from 0.53×10^6 to 1.18×10^6 lbf/in², decreasing with increasing loads. The values did not change greatly for a stronger mortar. Fishburn's test arrangement for the racking load and a typical shear failure is shown in Fig. (2.1). Loading was applied diagonally through steel brackets. Failure in all walls was primarily caused by bond failure between blocks and the mortar allowing a stepped path passing through the vertical and horizontal mortar joints, and along all parallel to the wall diagonal. His test results show an increase in shear strength from 66 lbf/in² for a 'BN' mortar (680 lbf/in²) to 103 lbf/in² for a 'CS' mortar (2200 lbf/in²). It was concluded that the bond strength of mortar was the principal factor affecting the shear strength of the blockwork, and the mortar compressive strength affected the bond strength. He also found that the shearing modulus of elasticity at 14 lbf/in² average shear stress was 0.14×10^6 lbf/in². In his final 34 walls flexure tests with lateral line loads applied at quarter points of simply supported wall. Tests were done with the wall bed joints perpendicular and parallel to the span. In general 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, showing that the tensile strength of mortar was greater than its bond strength to the masonry.

Determination of tensile bond strength between block and mortar resulting from flexure test of walls, wallettes or simple beams is reported by many investigators. These tests in many cases are not practical, and due to possible existence of/

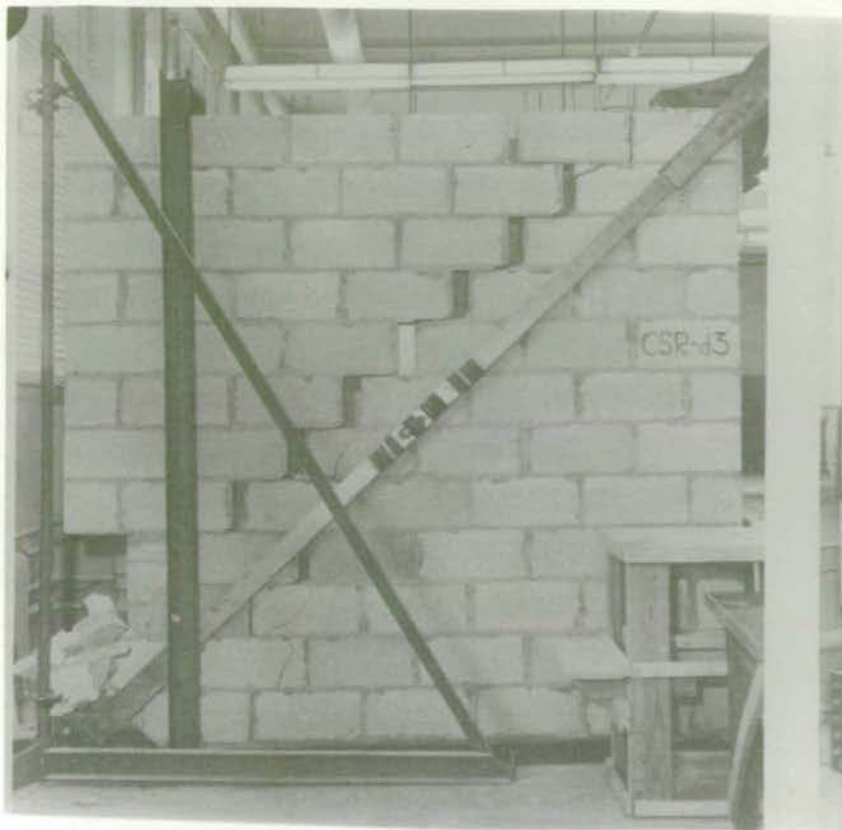


Plate 2.1

FISHBURN'S ARRANGEMENT FOR RACKING TEST

existence of stress concentrations they may give a misleading answer as to the actual value of tensile bond strength of mortar to the masonry. Other methods of determining the tensile bond strength are reported by various investigators such as Polyakov⁽⁷⁹⁾, Seamann⁽⁹⁴⁾, Ritchie⁽⁸³⁾, and Kuenning⁽⁵⁸⁾ which utilize use of brick or block couplets, and a direct method of testing.

Copeland and Saxer⁽¹¹⁰⁾ in their extensive work tested 592 concrete block couplets in order to cover all variables and to provide data on the individual variable factors. They employed a flexural testing procedure, producing flexural stress normal to the horizontal mortar joints, using two block assembly. The tests revealed that the type of portland cement, lime and the use of admixture in mortars, specimen storage, re-tempering of mortar, weighting of joints during hardening all had a minor effect on the tensile bond strength.

However, other factors such as compressive strength of mortar and its consistency especially, method of curing had important effect on the tensile bond strength. Damp curing increased the bond and the tensile bond of the epoxy joints exceeded the tensile strength of concrete. Tensile bond strength of mortar to a variety of concrete blocks e.g. dense and lightweight were obtained from flexure tests; inconsistent results were obtained.

Palmer and Parsons⁽⁷⁸⁾(1934), Seamann⁽⁹⁴⁾(1955), Fishburn⁽³⁵⁾(1961) and Hedstrom⁽⁴⁵⁾(1961), they all 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.

Kuenning⁽⁵⁸⁾ tested crossed-brick couplet using tripod method as described in A.S.T.M. E 194-66⁽²⁾. He demonstrated that due to the high/

the high magnitude of bending in the course of experiment, the specimen undergoes non-uniform loading. Due to this loading the couplet is more highly stressed at the edges of the mortar joint than at its centre, Fig.(2.2), developing larger tensile stresses at the ends and resulting an early failure.

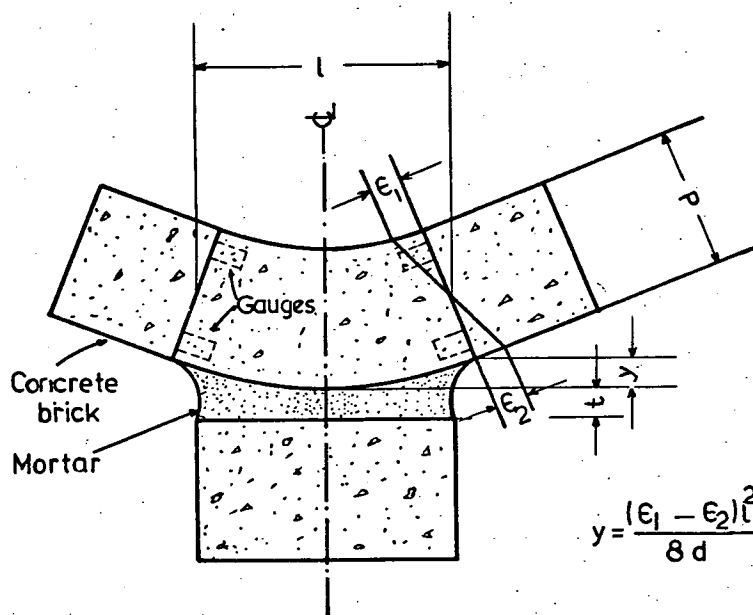


Fig. 2.2 Schematic representation of section through crossed brick couplet during test with tripods after Kuenning.

He also used a direct tension test method. His test arrangement for testing a half-block couplet assembly in tension is similar to the one used by the author in Chapter 4. In this method a more realistic simulation of longitudinal forces in the couplet is achieved and because of the existence of bond between the steel blocks and the couplet no bending stresses are induced in the assembly and under load the couplet will be in a state of uniaxial tension.

Hedstrom⁽⁴⁵⁾ (1961) investigated the compressive and flexural strength of concrete/

of concrete blockwork laid in nine different patterns and compared the results obtained with that of the conventional running bond pattern. Walls were 4 ft. wide by 8 ft. high. Lightweight aggregate concrete blocks 8 in. by 8 in. by 16 in. with three cores and mortars of two different strengths (1290 and 2500 lbf/in²) were used. Considering the 10% variation in wall strength for the two different mortar strengths he concluded that the compressive strength of the blockwork depends on the block strength and is little affected by the strength of mortar. For all walls tested the factor of safety based on allowable compressive stress of 85 lbf/in² ranged from 4.2 to 7.6 proving that the various patterns were satisfactory for load bearing purposes.

In 1966 as special investigation for the Cement and Concrete Association⁽¹⁷⁾, Building Research Station conducted axial loading tests on six block walls of storey height. Solid and hollow blocks of 1560 to 4640 lbf/in² crushing strength and 1:1:6 cement:lime:sand mortar mix was used. Mortar strength varied from 850 to 1090 lbf/in². Walls were 6 ft. long by 9 ft. high with thicknesses ranging from 3 inches to 6 inches. They were cured under polythene sheets for three days and tested within 28 to 39 days. Failure in all walls was by splitting and local crushing of the upper portion of the test wall. The wall to block strength ratio varied from 0.52 to 0.78. The following approximate relationship was suggested for the modulus of elasticity.

$$E = 850 / \mu b$$

where μb is block strength (on gross area).

Erntroy and/

Erntroy and Weeks ⁽³³⁾(1970) also reported the results of their axial tests on walls constructed of hollow concrete blocks with a plain or reinforced concrete infill. Twenty two walls were tested in order to assess the effect of wall strength when hollow cores are filled with in-situ plain or reinforced concrete. 8 in. by 8 in. by 16 in. blocks of 1370 and 2400 lbf/in² with a 1:½:3 mortar of 5360 lbf/in² crushing strength were used. The following conclusions were drawn.

1. Infilling increased the wall strength by about 50%.
2. Vertical steel increased the wall strength by 25% but did not have any effect when lower strength blocks were used for wall construction.
3. Horizontal steel in addition to vertical reinforcement caused a further 10% increase in wall strength.

Yokel et al. ^(106, 107)(1971) published a report which is based on the results of 192 full-scale wall tests of which a total of 114 were concrete masonry, solid, hollow, reinforced and cavity construction. A classic analysis determined the wall lower strength limit enabling the prediction of the strength of slender walls under eccentric compressive load, and a combination of eccentric and transverse loads using a load moment interaction curve, assuming that the compressive strength in bending is equal to the compressive strength of an axially loaded prism, and further, by applying a strength reduction factor for slenderness on the basis of the additional moment induced by the wall deflection.

Yokel ⁽¹⁰⁸⁾(1971) derived equations giving upper limits for elastic deflection and the lower limits for the critical load for compression members in/

members in deriving his formula he made the following assumptions:

1. Members to be prismatic with rectangular cross-sections.
2. To behave elastically under load with a linear stress-strain relationship.
3. Tensile strengths to be neglected.

The theoretical solution was used to predict results of tests on 39 walls tested at the Structural Clay Products Institute. The trend of the test results was predicted by the theory but the load carrying capacity was only approximately estimated.

Following the construction of a 700 ton.f. testing rig at the Cement and Concrete Association (Research and Development Station) in January 1972, tests were conducted by Read and Clements⁽⁸⁰⁾.

A total of 11 walls were built with a 1: $\frac{1}{4}$:3 mortar mix of 1335 to 5610 lbf/in² and were tested to assess the suitability of the testing frame for testing masonry walls with the maximum dimensions of 8 ft. 10 in. high by 6 ft. 6 in. wide. The tests carried out at B.R.S.⁽¹⁷⁾ were repeated at C. & C.A. in order to compare the performance of the two testing machines. Failure in all walls was by splitting, and the tests on two groups of four identical walls showed a coefficient of variation of 6%. The C. & C.A. results were 10 and 25% higher than the B.R.S. results for the low-strength and high-strength walls respectively. The difference 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 1.12×10^6 to 2.33/

to 2.33×10^6 lbf/in². Block strength varied from 1360 to 3755 lbf/in². The lowest value of "E" was obtained for walls built in high-strength hollow blocks while the highest "E" value was obtained for walls built in lowest strength cellular blocks.

Read and Clements⁽⁸¹⁾ (1972) 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 ratios, stiffness of units and walls and failure characteristics were investigated. Suitability of two-block couplet as a control specimen was also studied. Walls were approximately 8 ft. 6 in. high by 5 ft. 11 in. wide. 1: $\frac{1}{4}$:3 mortar mix of relatively high crushing strength with various types of blocks, solid and hollow of 855 to 4365 lbf/in² crushing strength were used for the wall construction. Wall preparation and tests were conducted according to the BS 2028 1968⁽¹⁰⁾ and CP 111 Part II, 1970⁽¹⁵⁾. For all solid walls tested, mortar strength was higher than the block strength the difference in strength in some cases amounted to 2600 lbf/in². In conclusion, the use of single block seems to be more advantageous as a control specimen over the couplet, since the former was easier to prepare. Their results were compared with the code of practice CP 111, 1970 part II, and the proposed revision of CP 111, 1970. A large increase in the basic permissible stress was observed when the wall panel tests results were used in accordance with section 5 of CP 111, Part II, 1970, Appendix C of the Draft Code.⁽¹⁶⁾

Converse⁽²⁵⁾/

Converse⁽²⁵⁾ (1946) reported a series of racking tests on nine reinforced concrete masonry walls with three variations in proportion of horizontal to vertical reinforcing. His loading rig was such that the wall was subjected to pure shear and the failure occurred as the opening of a diagonal crack joining the loaded corners. The major result was that the failure load increased as the increase in reinforcing steel area, whether this steel was placed horizontally or vertically.

Schneider⁽⁹⁰⁾ (1959) carried out shear tests for a series of full-scale reinforced grouted masonry walls of clay brick, hollow concrete blocks, and tied double-leaf concrete brick walls. All walls failed in diagonal tension. In the case of the hollow concrete blocks little difference existed between the ultimate loads sustained by similarly constructed walls reinforced on the basis of 0.002 or 0.003 x the gross cross-sectional area of the wall, indicating that the lower quantity of reinforcing was sufficient to develop the ultimate shear resistance of the grouted masonry. However, the load at which the first crack appeared was noticeably lowered by a reduction in the amount of wall steel. This was due to the fact that masonry separation cannot occur until the reinforcement in the region of high stress has yielded. He also discovered that there was no basic difference between the shear resistance of the stack and running bond concrete block masonry, and that intermittently filled-cell walls resisted a total load of about 67% of that sustained by solid grouted walls.

Scrivener^(93a) (1966), tested reinforced concrete masonry walls and concluded that/

concluded that the amount and distribution of vertical reinforcement are the critical factors in determining the stiffness and failure load. He further noticed that increase in the amount of vertical steel would increase both the strength and failure load, and the most effective position for this reinforcement would be the periphery of the wall. By assuming yielding of all vertical steel and that the centre of compression is the toe of the wall, the failure load may be predicted. The resisting moment of a wall was then the sum of the moment of the yield forces in each bar times the lever arm, distance from the toe of the wall to the centre line of the bar. The horizontal reinforcement did not contribute to the resisting moment and therefore it did not alter the stiffness or failure load significantly.

Following this work Scrivener^(93b) reports results of further investigation on concrete masonry shear wall. The object of the tests were to get information to aid design of load-bearing reinforced block masonry walls subjected to earthquake loads, using peripheral vertical and horizontal reinforcement. Test arrangements were similar to the previous with the difference that the wall was anchored along its base by welding mild steel vertical deformed bars to $\frac{3}{4}$ inch diameter screwed rods projecting from the reinforced concrete base. Loads (in vertical and horizontal directions) were applied simultaneously and the ratio of the two loads was kept constant and equal to 0.9 in order to balance the overturning moment of the racking load about the wall toe. Failure load was greater with higher percentage of reinforcement up to 0.3% of the gross cross-sectional area/

sectional area of the wall, above this the addition of reinforcement had a very little effect. The average shear stress of 170 lbf/in^2 on gross area was obtained and both vertical and horizontal reinforcement were found effective in providing the satisfactory failure load and crack behaviour.

Schneider⁽⁹¹⁾ investigated the behaviour of full-size field constructed concrete masonry piers subjected to lateral forces. He studied its mode of failure, amount and orientation of the web-reinforcement, jamb-reinforcement and the effect of axial pre-compression. From the mode of failure exhibited by piers, vertical reinforcement did not restrain the propagation of the cracks and therefore did not significantly increase the pier resistance in terms of shear strength. The spread of diagonal cracks however, was prevented by horizontal reinforcement, and piers were strengthened. Application of 400 lbf/in^2 precompression increased the shear strength by 51% for one pier; on the other hand, the difference was insignificant for two other piers tested under the same condition.

A study of the behaviour of dense concrete block masonry walls under axial compression was made by Ibrahim⁽⁵¹⁾. A total of nine walls were tested. From his test results he concluded that the blocks were adequate for multi-storey construction.

Hassan⁽¹⁴⁾ investigated slenderness ratio and its effect on the ultimate strength of masonry wall built in lightweight aggregate concrete blocks. He compared from his test results the stress reduction factor and its variation with slenderness ratio with various existing codes. His results indicate that the values of reduction factor given in CP 111, Part II (1970) are conservative, and suggests that further axial tests should be carried out to study the effect of end rotations and deflections on the effective height.

CHAPTER 3 - COMPARATIVE STUDY OF THE BEHAVIOUR OF $\frac{1}{3}$ AND FULL-SCALE
LIGHT-WEIGHT AGGREGATE CONCRETE BLOCK MASONRY WALLS
UNDER AXIAL COMPRESSION

3.1 INTRODUCTION

As a tool of structural design and research, model testing has been developed over the past half century. The increasing use of complex structural forms has called upon the need for structural model testing. These types of structures do not readily lend themselves to analysis by existing theories of structural analysis. This is particularly the case in the fields of shell structures, slender arch dams, and complex three-dimensional building structures.

Basically two types of models exist:

- (1) "Elastic Models" in which material is homogeneous and isotropic. This type of model test may be used to establish, modify or simplify theoretical solution based upon classical theory of elasticity.
- (2) "Realistic Models" in which case the material used in the model is identical with or related to that used in the prototype. The present investigation is based upon the second method of model testing.

3.2 REVIEW OF SIMILAR WORK

All the relevant existing literature are based on investigations into structural performance of brickwork, and to the author's knowledge so far no work has been conducted on blockwork utilising model technique. The following survey outlines briefly similar work conducted on model brickwork under compressive load.

Speer⁽⁹⁹⁾ (1953/4) used photoelastic technique to study the stress distribution in the brickwork. His studies revealed the existence of tensile or compressive stresses in the horizontal direction due to deformation of bricks and mortar. In order to investigate the basic principles of model tests for brickwork Vogt⁽¹⁰³⁾ (1957/58) carried out tests on $1/10$ -scale pillars of 6 cm x 6 cm x 30 cm, using mortars of different strength. He concluded that the tensile strength of the brick is the determining factor governing the compressive strength of the brickwork. Murthy^(74,75) (1964) extended the application of model technique for testing brickwork. He carried out a large number of tests on $1/3$ and $1/6$ th-scale brick piers in order to compare results obtained on full-scale brickwork at B.R.S., taking into account the effects of mortar strength, eccentricity and slenderness ratio. His test results were given as non-dimensional quantities in order to make a comparison with full-scale and model test results, Fig (3.1). Murthy's tests demonstrated conclusively that the scaled brick could be used to investigate the behaviour of full scale brickwork structure under compressive load. Following Murthy's work, Sinha⁽⁹⁶⁾, Kalita⁽⁵²⁾, and Khoo⁽⁵⁵⁾ used scaled brick to investigate various structural properties of load-bearing brickwork.

3.3 OBJECT OF PRESENT WORK

The tests of light-weight aggregate concrete block masonry walls described in this Chapter were carried out in order to develop a model technique for the investigation of the strength of blockwork by establishing a relationship between $1/3$ -scale model walls and corresponding/

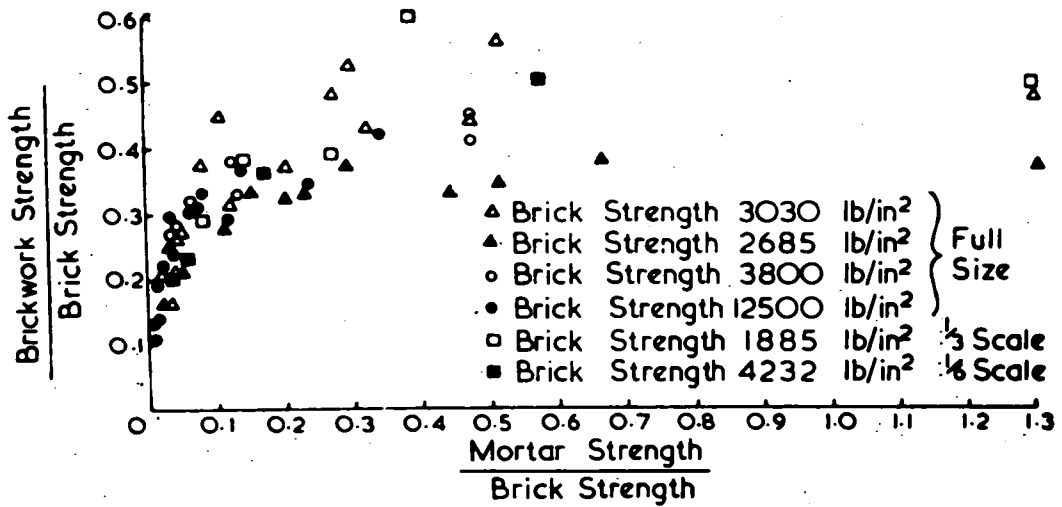


FIGURE 3.1-Test results: brickwork strength v. mortar strength

and corresponding full size walls built with the same type of block and mortar. The investigation was extended to determine the strength of blockwork, their elastic constants, to provide relevant data for the design, and to examine the mechanism of failure in blockwork by studying the behaviour of wall panels representative of storey-height building under axial compressive load.

3.4 MATERIALS

3.4.1 Blocks

Four types of light weight aggregate concrete blocks commercially known as: Thermalite, Leca, Lytag and Aglite blocks⁽⁶⁰⁾ covering a compressive strength range of 500 to 4000 lbf/in² were chosen for these tests. Full scale blocks were 18 in. long x 9 in. high x 4 in. thick, whereas blocks for construction of model walls were cut in the laboratory from full scale samples to the required dimension (6 in. long x 3 in. high x 1 $\frac{1}{3}$ in. thick) by a "Clipper" machine, using an abrasive blade and employing wet cutting technique.

$\frac{1}{3}$ -scale samples were cut from 3 inch thick full-size blocks for the following reasons:

- (a) To minimize the cutting labour.
- (b) To obtain bearing surfaces exactly similar to those of full-size blocks.

3.4.2 Cement

"Ferrocrete" a rapid hardening Portland Cement was used to give an early desired mortar strength. Compression tests on 3-inch cement mortar cubes were conducted according to BS 12, 1958, the average strength/

average strength at seven days being 4830 lbf/in² against the minimum of 4000 lbf/in² recommended by the British Standard Institution.

3.4.3 Lime

Hydrated lime "Class A" was used conforming to BS 890 (13).

3.4.4 Sand

Ordinary building sand, being available locally, was used for blockwork construction, conforming with the limits of BS 1200, 1955⁽¹⁴⁾, and its grading is given in Table (3.1). For the construction of all $\frac{1}{3}$ -scale and nine full-scale walls the sand used was thoroughly dried to eliminate all moisture and to give controlled test conditions. For the remaining four full size walls it was wet, but the moisture content was measured and hence its weight was corrected.

3.4.5 Mortar

Both model and full-scale blockwork were built in 1:1:6 cement:lime:sand (by volume), the actual proportioning was by weight, being 1:5:6.7 cement:lime:sand per batch. For construction of $\frac{1}{3}$ -scale walls the sand was sieved through a No. 14 sieve to eliminate large particles.

Initially trial mixes were made with varying quantities of water, and for the mix to have a workable consistency, the water/cement ratio was adjusted to 1.92. This water/cement ratio was used throughout the wall construction, except that the mason was allowed to use additional water when necessary. Each batch of mortar was mixed by hand, and from each batch two 3-inch cubes were cast and vibrated for two minutes.

All mortar/

All mortar cubes were kept in moulds for 48 hours, being covered by damp sacks, after which they were demoulded. Half of these were cured in a curing tank at constant temperature, and the remainder were cured alongside the wall at the laboratory temperature, and humidity till they were tested with the walls. Results are shown in Tables (3.7 to 3.11).

3.5. DETERMINATION OF PHYSICAL PROPERTIES OF BLOCK

The required physical properties for full size and $\frac{1}{3}$ -scale blocks were determined as follows:

3.5.1 Density

The density of full size blocks was determined according to BS 1364: 2028 (1968). The density of $\frac{1}{3}$ -scale blocks calculated in the following manner: Ten $\frac{1}{3}$ -scale cut specimens were chosen at random from each groups, using calipers and rule, the thickness of each specimen was measured at the six edges of the block. Also the length and the height of each specimen were measured accordingly and the density was calculated. Results for full and $\frac{1}{3}$ -scale blocks are given in the Tables (3.2 and 3.3).

3.5.2 Compressive Strength

The compressive strength of blocks were obtained from tests on the individual blocks, the mean value for a sample of twelve blocks being taken. Twenty four blocks, were chosen at random from the available stock, twelve of these were prepared and tested in accordance with BS 2028 (1953). The remaining twelve being prepared in the same way and were kept in the laboratory tested after they were completely dry. The crushing strength, coefficient of variation and the range of strength for the various types of blocks full/

TABLE 3.1 - SIEVE ANALYSIS OF ORDINARY BUILDING SAND USED
IN CONSTRUCTION OF FULL AND $\frac{1}{3}$ -SCALE BLOCKWORK
(BS 1200, 1955)

BS Sieve No.	% Passing by Weight	% Retained
1/8	98.5	1.5
7	98.0	.5
14	96.7	1.3
25	92.9	3.8
52	63.4	9.5
100	17.3	46.1

Finess Modulus = .627

TABLE 3.2 - DENSITY OF FULL SCALE BLOCKS BS 2028

Block Type	Density pcf	Coefficient variation %	Range %
Thermalite	51.5	1.32	4.1
Leca	58.3	1.4	5.7
Iytag	84.6	5.2	17.6
Aglite	93.4	23.5	6.9

TABLE 3.3 - DENSITY OF ONE-THIRD SCALE BLOCK

Block Type	Density pcf	Coefficient variation %	Range %
Thermalite	49.1	4.1	15
Leca	57.8	3.1	7.6
Iytag	80.1	8.5	7.8
Aglite	97.0	22.1	6.2

TABLE 3.4 - CRUSHING STRENGTH OF FULL SCALE BLOCKS

Block Type	Crushing Strength lbf/in ²	Coefficient of variation %	Range %	Mode of Failure
Thermalite	500	13.7	44.21	Shear
Leca	733	9.98	29	Shear
Lyttag	1607	21	70	Shear
Aglite	2161	7.8	27.5	Shear

TABLE 3.5 - CRUSHING STRENGTH OF ONE-THIRD SCALE BLOCKS

Block Type	Crushing Strength lbf/in ²		Coefficient of variation %	Range %	Mode of failure
	*	/			
Thermalite	517	565	11.7	31.5	Shear
Leca	763	824	7.8	21.7	Shear
Lyttag	1674	1804	11.1	13.1	Shear
Aglite (1)	2717	2939	14.4	55.8	Shear
Aglite (2)	3931	4257	12.5	39.3	Shear

* Cured and tested according to BS 2028

/ Cured in the laboratory atmosphere

TABLE 3.6 - TENSILE SPLITTING STRENGTH OF 4-INCHES DIAMETER CYLINDERS
BORED FROM 4 INCH THICK CONCRETE BLOCKS

Material	Tensile splitting strength lbf/in ²	Coefficient of variation %	Range	Splitting strength/ crushing strength
Thermalite	47	3.1	8.81	.094
Leca	64	6.2	14.8	.087
Lyttag	168	5.7	14.4	.104
Aglite (1)	194	3.8	10.3	.090
Aglite (2)	354	4.4	11.2	.090

(1) Aglite Blocks of 2717 lbf/in² Compressive strength

(2) Aglite Blocks of 4257 lbf/in Compressive strength

blocks full and $\frac{1}{3}$ -scale are given in Tables (3.4 & 3.5)

3.5.3 Tensile Strength

The indirect 'Brazilian' splitting test was employed to determine the tensile strength of each block type. A sample of six 4 in. by 4 in. cylinders were bored from six full-size blocks taken at random from each batch of blocks. The specimens were prepared and tested in a dry condition. Test results are given in Table (3.6).

3.6 CONSTRUCTION OF WALLS

3.6.1 Full Scale Walls

A total of 13 full size walls were built, all being 4-inches thick and 8 ft. 3 in. high x 4 ft. 6 in. long. From each type of block three walls were constructed except for Lytag blocks, in which case four walls were built. To facilitate subsequent handling, the test walls were constructed in the test rig and on a previously levelled concrete base by a professional mason. They were built in running bond using typical construction techniques which are described. A full mortar bed was laid on the supporting platform in the 600 ton multi-purpose testing rig ⁽⁹⁾ Plate (3.1), at the base of each wall. The face of the wall nearer to the mason was kept in alignment by him and the mortar joints in both faces were cut flush and were not tooled. The head joints were staggered by using cut stretchers at the ends of alternate courses. The mason was instructed to keep up the same workmanship and to use mortar with the same consistency throughout the wall construction, keeping the mortar bed joints thickness to three-eighths inches in/

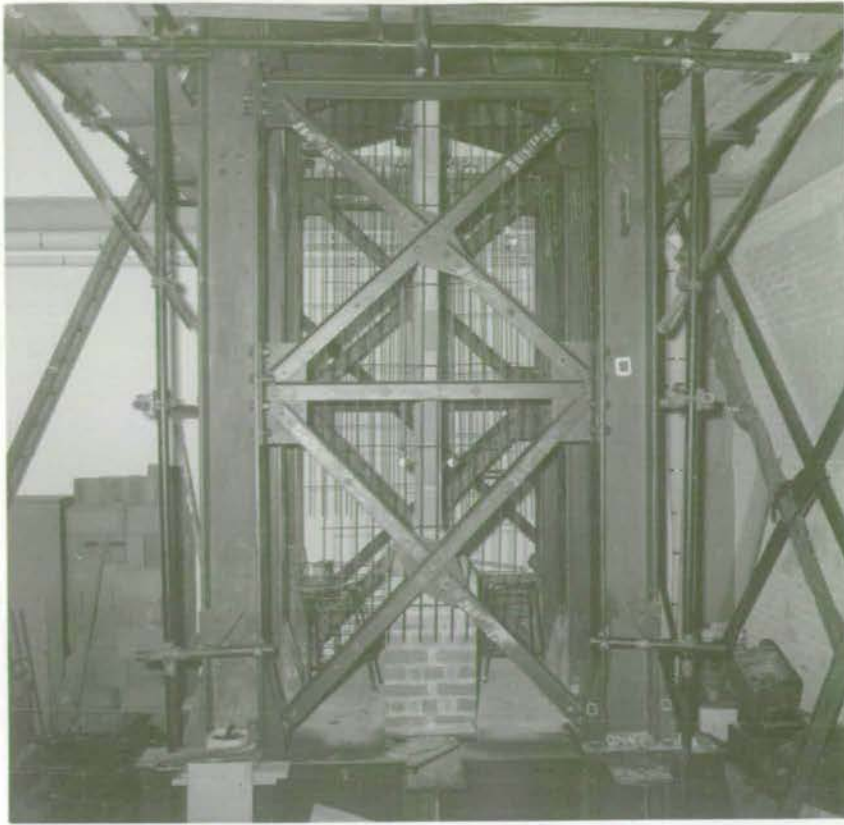


Plate 3.1 600-TON CAPACITY MULTI-PURPOSE TESTING RIG

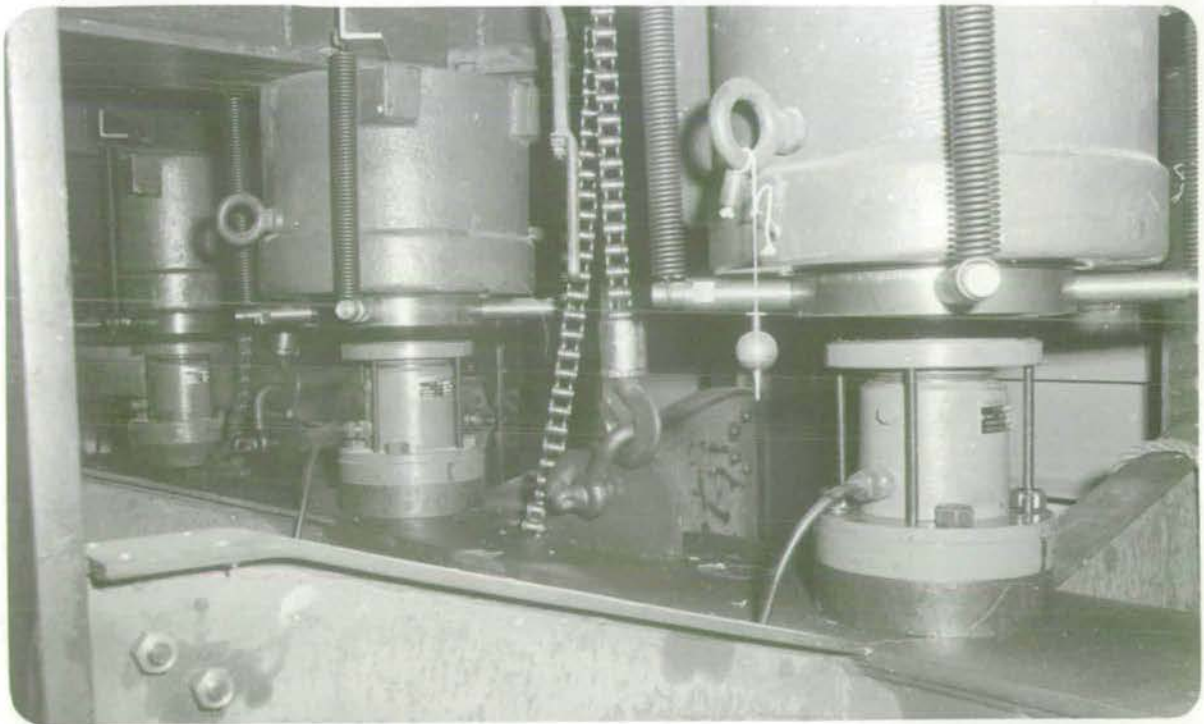


Plate 3.2 200-TON CAPACITY HYDRAULIC RAMS

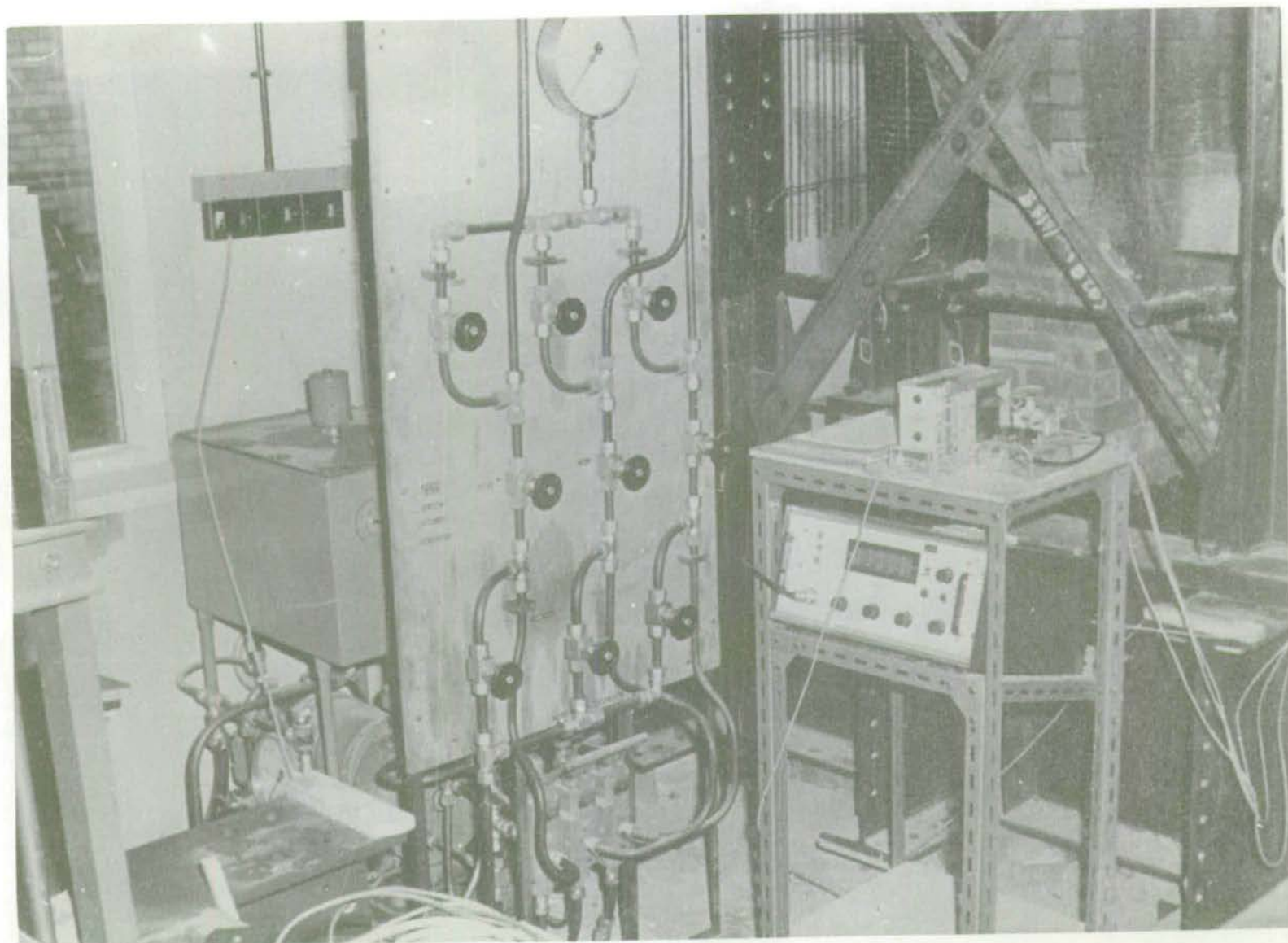


Plate 3.3 LOADING EQUIPMENTS FOR FULL-SCALE WALL TESTS

inches in all cases. For this purpose a storey-height coursing rod was supplied to the mason. For construction of each full size wall three batches of mortar were used and from each batch two 3-inch mortar cubes were cast. Walls were allowed to cure in the laboratory for 15 days. A check on the plumbness of walls was made prior to the test, to ensure a uniform loading. All walls were capped by 1:1 Fondu cement:sand mortar, a $\frac{1}{2}$ -inch mortar bed was laid on top of the wall and the spreader beam was lowered into this bed and levelled in position.

3.6.2 Model Walls

The same number of $\frac{1}{3}$ -scale walls were built. They were $1\frac{1}{3}$ -inches thick, $3\frac{1}{2}$ inches high and 18 inches long. To facilitate accurate construction in the laboratory, jigs were made to the required dimensions and fixed plumb onto a flat steel plate, (Plates 3.5 & 3.6). The thickness of the mortar beds was scaled down to one-eighth inches and the mason was instructed to keep it constant throughout. For this purpose guide lines were drawn on the wooden backing of the jigs to mark each course of blockwork. Three model walls were built at a time from each material, six 3-inch mortar cubes were cast from each batch and cured till they were tested with the walls.

3.7 EXPERIMENTAL PROCEDURE

3.7.1 Full-Scale Wall Test and Strain Measurement

The full-scale walls were constructed within the loading frame Plate (3.1). During the curing period being a minimum of 14 days/

days the wall was prepared for the test. Four compressometers of 72-inches gauge length were fixed by means of brackets to each corner of the wall (Plate 3.4) to measure vertical shortening under axial load. "Demec" studs were fixed on the blocks and the mortar joints on the 5th, 6th and the 7th courses in horizontal and vertical directions. The horizontal deformations of all full-scale walls were measured by a 24 inch "Demec" gauge and four horizontally mounted compressometers. The axial load was applied to the wall by three hydraulic rams each of 200 tons capacity (Plate 3.2) through a welded box spreader beam 8ft. long made up of 24 inches by 12 inches by 100 lbf/ft universal beam. Where the ultimate strength of wall was expected to be low, least number of jacks were used giving better accuracy in the readings. Three load cells previously calibrated in an Avery Testing Machine, were placed directly under the rams, the load cell readings were measured with a digital voltmeter capable of reading to an accuracy of 5-microvolts (Plate 3.3 Appendix 1). The reading of digital voltmeter was recorded for each pre-determined load. To ensure that the compressometers were in working order the wall was initially loaded to half a ton and movements of the dial gauges were observed. The load was then removed and the initial zero reading was taken. Readings of compressometers and "Demec" gauges were taken on both sides of wall for each load increment. Readings were recorded up to about half the expected failure load, at which dial gauges were removed and the load was then allowed to increase at a constant rate, about $45 \text{ lbf/in}^2/\text{min}$ up to the failure.

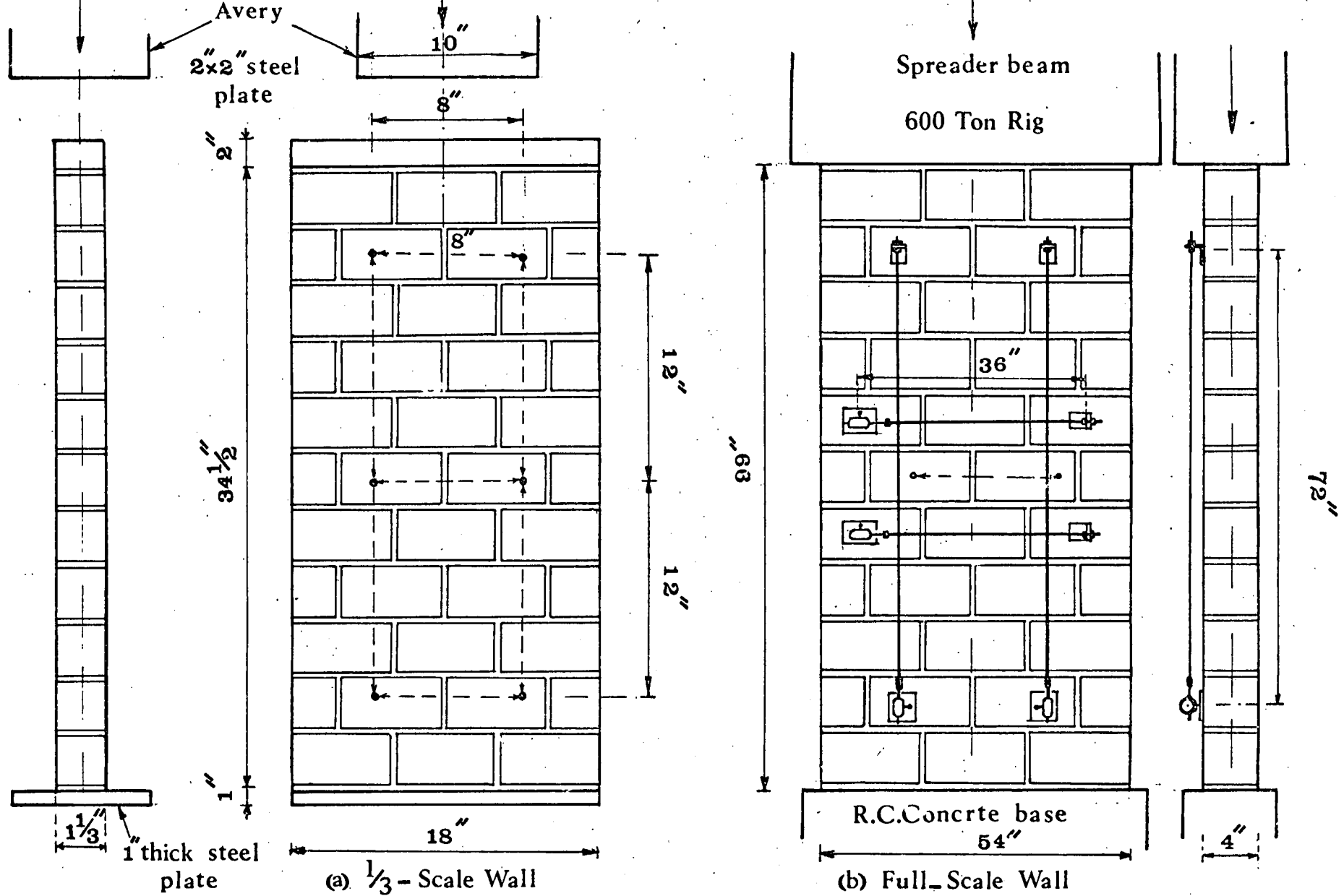


Fig. 3.1(a&b) Position of Demecs & Comprossometers in the 1/3rd & Full - Scale Concrete Blockwork

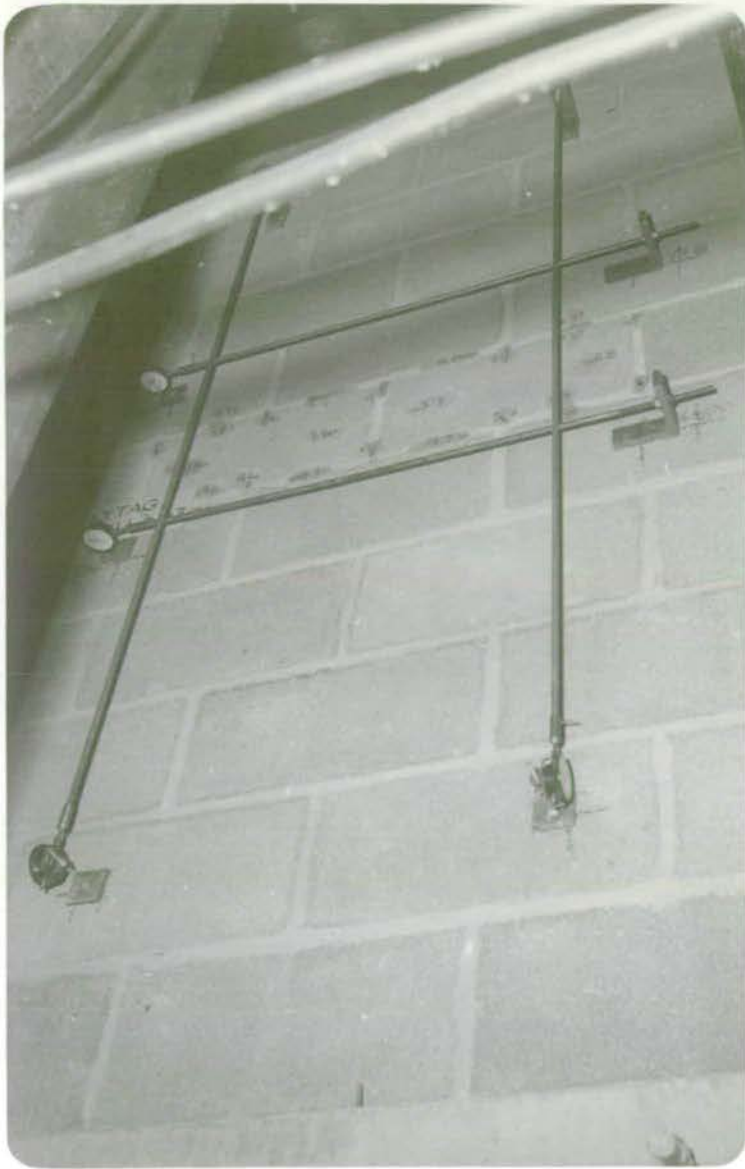


Plate 3.4

SHOWING THE POSITION OF HORIZONTAL AND
VERTICAL COMPRESSOMETERS

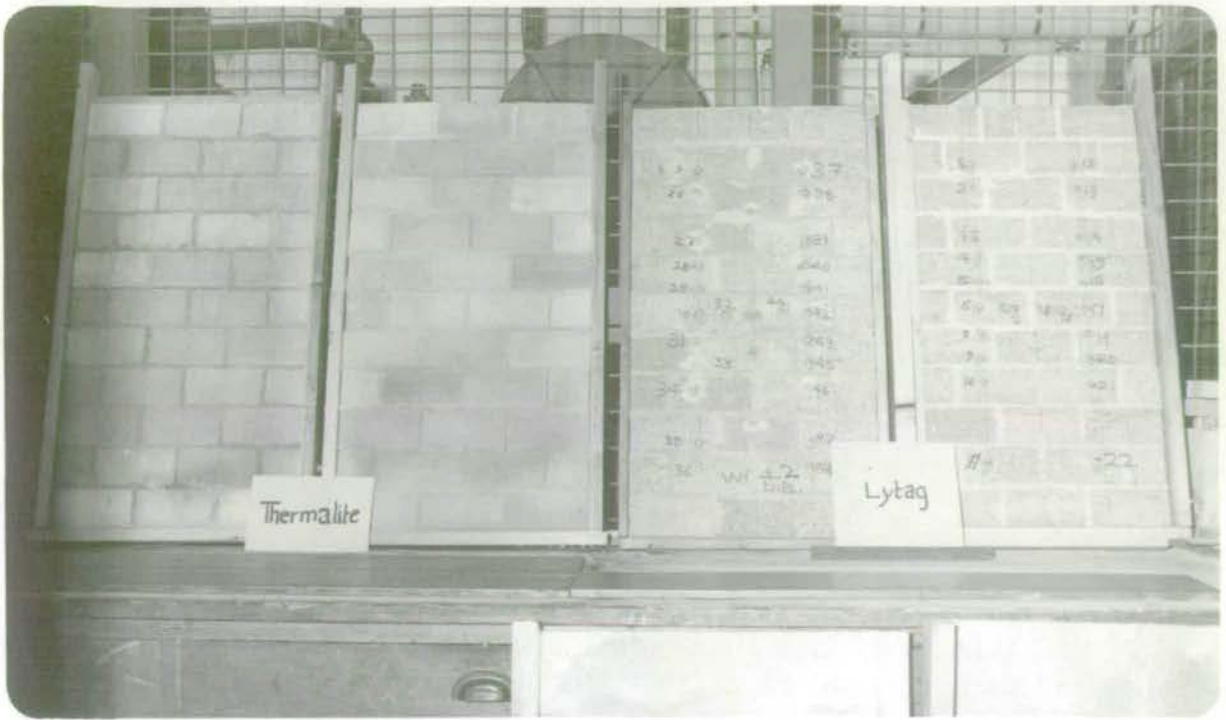


Plate 3.5 ONE-THIRD SCALE BLOCKWORK



Plate 3.6 HANDLING AND PREPARATION OF ONE-THIRD SCALE BLOCKWORK FOR THE COMPRESSION TEST

3.7.2 One-Third Scale Wall Tests

The one-third scale walls were tested in an Avery Universal Testing Machine Plate(3.6). Prior to each test, a 2 in. x 2 in. steel beam was bedded with 1:1 Fondu cement:sand mortar on top of the wall and accurately levelled in order to ensure a uniform distribution of the applied load over the entire section of the test wall. 24 and 12 inch "Demec" gauges were used to measure vertical and horizontal deformations of walls. Rate of loading was kept constant about $45 \text{ lbf/in}^2/\text{min}$.

3.8 DISCUSSION OF THE RESULTS

3.8.1 Stress-Strain Relationship

Figures (3.2 to 3.13) give stress v. strain, stress v. lateral strain and lateral strain v. normal strain relationship for all full and one-third scale walls tested. From the uniformity of results it may be concluded that model and prototype behave very similarly under axial compressive load, and the variation in the deformation between corresponding model and prototype is within the scatter of experimental results, although it is seen that the deformation in the one-third scale walls is slightly higher than the full-scale walls.

3.8.2 Relationship between Wall Strength and Block Strength

The relationship between wall strength and block strength is presented in Figure(3.14). From this figure the relationship between wall strength and block strength appears to be non-linear. It is seen that, the curve in Figure (3.14) passes through the origin, although the block strength is that of saturated blocks. This suggests/

This suggests that the existence of a pore water pressure in saturated blocks is absent or of insignificant influence. On account of the porous nature of light-weight concrete blocks, it is probable that a rapid dissipation of pore water pressure in saturated blocks takes place upon loading.

The mean strength ratio (wall ultimate strength/block crushing strength) for all full-scale and one-third scale walls are given in Tables (3.7 to 3.12). The average values obtained ranged from 0.56 to 0.71 for full-scale and 0.43 to 0.67 for one-third scale walls. The difference in the ratio between full-scale and model walls may be due to the slight difference in the slenderness ratio, section 3.89. These results reasonably agree with the findings of other investigators.

Richart et al⁽⁸²⁾ in their tests on 110 walls and wallettes obtained values for strength ratio ranging from 0.34 to 0.65, with an average value of 0.53. Fishburn⁽³⁵⁾ reports an average value of 0.54. The same ratio from Copeland and Timms⁽²⁶⁾ investigation ranged from 0.34 to 0.79, and they observed that the blockwork strength increases linearly with block strength.

It may be seen from tables (3.7 to 3.12), where the strength difference between the block and the mortar is large the strength ratio is low. The reason for this is that when the difference in the strength between block and mortar is large the lateral tensile stress in the block induced by the weak mortar joint is high, resulting in a lower ultimate strength. Hence the strength ratio will be lower. Read and Clements^(80, 81) also obtained low values, for strength ratio for high strength block and relatively low strength mortar.

It is for this reason that in brickwork when the strength difference between brick and mortar is usually greater than that for blockwork, the strength ratio is lower. Also in brickwork the ratio of mortar thickness to brick depth is higher than the corresponding value for blockwork. In brickwork using imperial size brick the ratio is $1/7$, in the blockwork used in these tests this ratio is $1/23$. The lateral tensile stress induced by the mortar joint in a deeper block is lower.

3.8.3 Mortar

The wall tests were conducted under conditions that were relatively unfavourable in developing mortar strength. The walls were cured in the laboratory atmosphere and at a low percentage of relative humidity, so that the only water available for hydration of the cement was that initially contained in the mortar.

The initial absorption of water from the mortar tends to increase the strength by lowering the water/cement ratio; however, since much of the water absorbed by the block owing to their porous nature is quickly lost to the atmosphere, the mortar does not receive the benefit of further curing.

3.8.4 The Ultimate Strength of blockwork

By studying the variation of wall strength with the strength of blocks and by comparing the results obtained with the results from other investigations^(17, 51) it is possible to predict the ultimate strength of blockwork built in solid block and 1:1:6 mortar provided that the ratio of block strength/mortar strength exceeds a value of 1.7.

Following Herrmann's work⁽⁴⁷⁾ the ultimate strength of blockwork may be/

may be estimated from the equation:

$$W = K \sqrt[3]{m \cdot s^2}$$

where m = mortar strength (cured in the water)

s = block strength (BS 2028)

K = characteristic constant found to be 0.9.

Table (3.13) shows the results of this investigation. For comparison the data and the calculated values of blockwork strength from other investigations are given in the same table.

3.8.5 Relationship between Crushing and Tensile Strength of Blocks

The standard formula for tensile splitting strength $\sigma_f = \frac{2P}{\pi DL}$

where: σ_f = tensile splitting strength of the block (lbf/in²)

p = applied load (lb.)

D = diameter of specimen (in.)

L = length of specimen (in.)

was used to calculate the results given in table (3.6). Fig (3.15) shows the relationship between B.S. Compressive Strength and the tensile strength of blocks derived from these tests. The relationship is essentially linear and indicates that the tensile to compressive strength ratio does not vary with the block strength, being approximately $1/11$ for all blocks tested.

3.8.6 Modulus of Elasticity

All walls were tested at a constant rate of loading of about 45 lbs/sq.in/min. The time lapse between individual set of strain readings was between 7 to 10 minutes and duration of test for various walls ranged from 1 to $1\frac{1}{2}$ hours. The stress-strain relationship for/

relationship for individual walls, and the typical variation of secant modulus of elasticity with compressive stress for all walls are given in figures (3.16 to 3.33). The first part of the stress-strain curves, up to about 30-40% of ultimate strength, is reasonably straight. Beyond this stress level, the stress-strain relationship is not linear. For concrete the stress-strain relationship is a function of time⁽⁷⁶⁾. The work of Rusch⁽⁸⁹⁾ has shown that the stress-strain behaviour and ultimate strengths of concrete are also affected by the rate of loading. Stress-strain relationship is affected by creep during which strain increases with time. With slow rates of loading the creep strain becomes significant even at small proportion of ultimate load. Therefore the static modulus of elasticity has been defined in terms of secant values⁽⁷⁶⁾, at a certain stress level and for the specific rate of loading. Creep was observed in almost all tests and was especially pronounced for masonry built in weak mortar and blocks with low density and low crushing strength such as Thermalite. The individual and the mean values of secant modulus of elasticity at $\frac{1}{4}$ of ultimate load for all walls are given in tables (3.7 to 3.12). The mean values for corresponding model and prototype are about the same decreasing with increasing load. The difference between the individual values is probably due to the characteristics of the materials, test arrangement, duration of loading and the workmanship.

3.8.7. Variation of E with Block Strength

Figure (3.25) shows the relationship between "E" values (secant modulus elasticity over/

elasticity over $\frac{1}{4}$ of ultimate load) and ultimate strength of masonry walls tested. From this plot it is evident that the Modulus of Elasticity of blockwork increases with the strength of the wall and hence with the strength of blocks. Close relationship between full and one-third scale walls is apparent. The relationship is non-linear.

3.8.8 Poisson's Ratio

Values of Poisson's Ratio for all walls were calculated from the deformation curves and at $\frac{1}{4}$ of ultimate load using the relationship

$$\nu = \frac{\epsilon_x}{\epsilon_y}$$

where:

ν = Poisson's Ratio

ϵ_x = strain in horizontal direction

ϵ_y = strain in vertical direction

The values could also be calculated from the two longitudinal and transverse deformation at various stages of stress (Figs.3.10 - 3.13). The results for the individual wall both full and one-third scale are tabulated in tables (3.7 to 3.12). The average values varied from 0.19 to 0.24 for full-scale walls and from 0.18 to 0.20 for one-third scale walls. The variation may be due to the difference in the efficiencies of the gauge lengths used for measuring deformations.

3.8.9 Slenderness Ratio

For all full-scale walls tested the slenderness ratio was 18.6, corresponding values for all models were 19.5, based on the effective height of the walls and their effective thickness. The difference in Slenderness Ratio for full and one-third scale walls was due to the difference/

THERMALITE WALLS

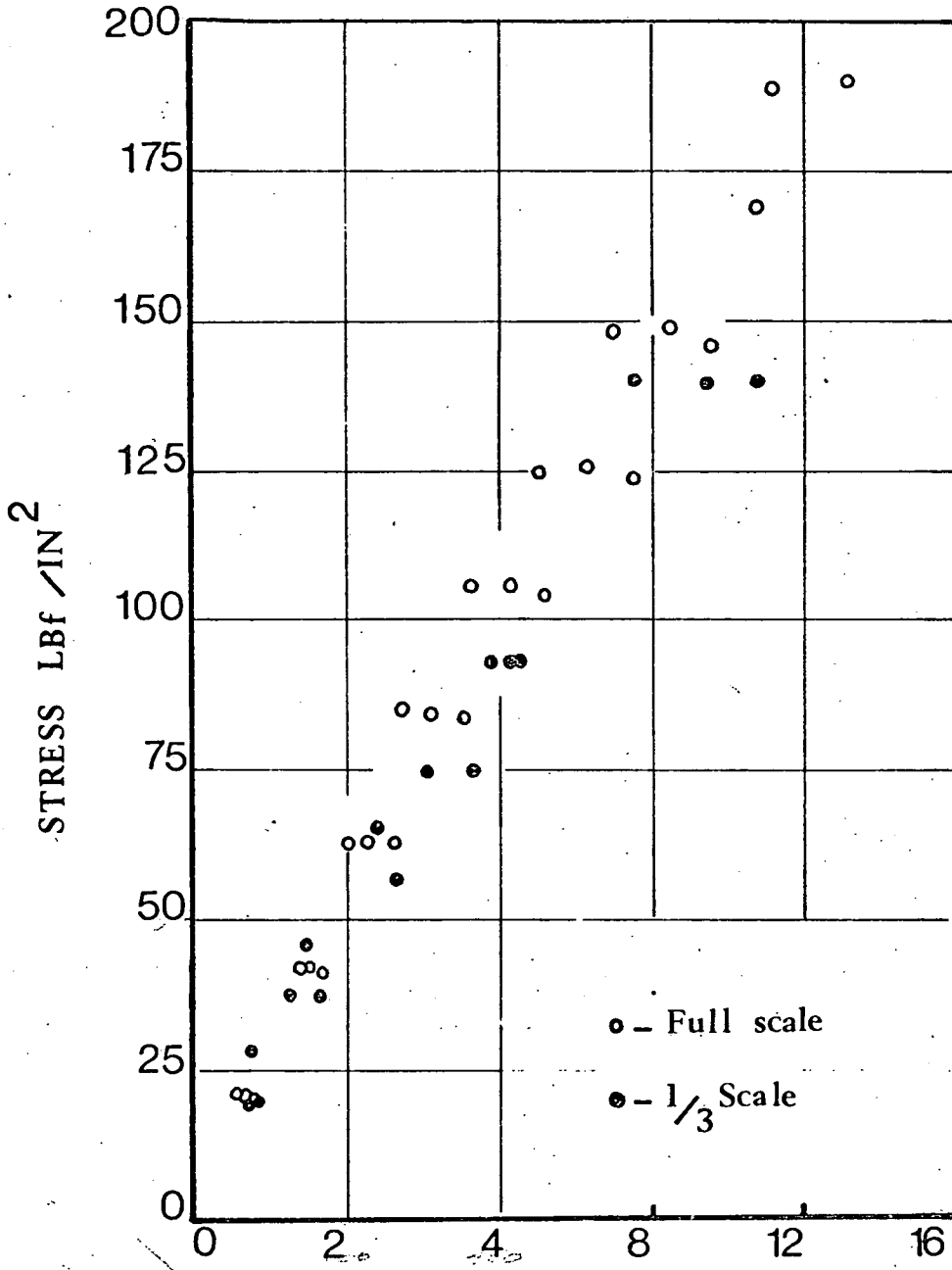


FIG. 3,2 STRAIN IN / IN × 10⁻⁴

LECA WALLS

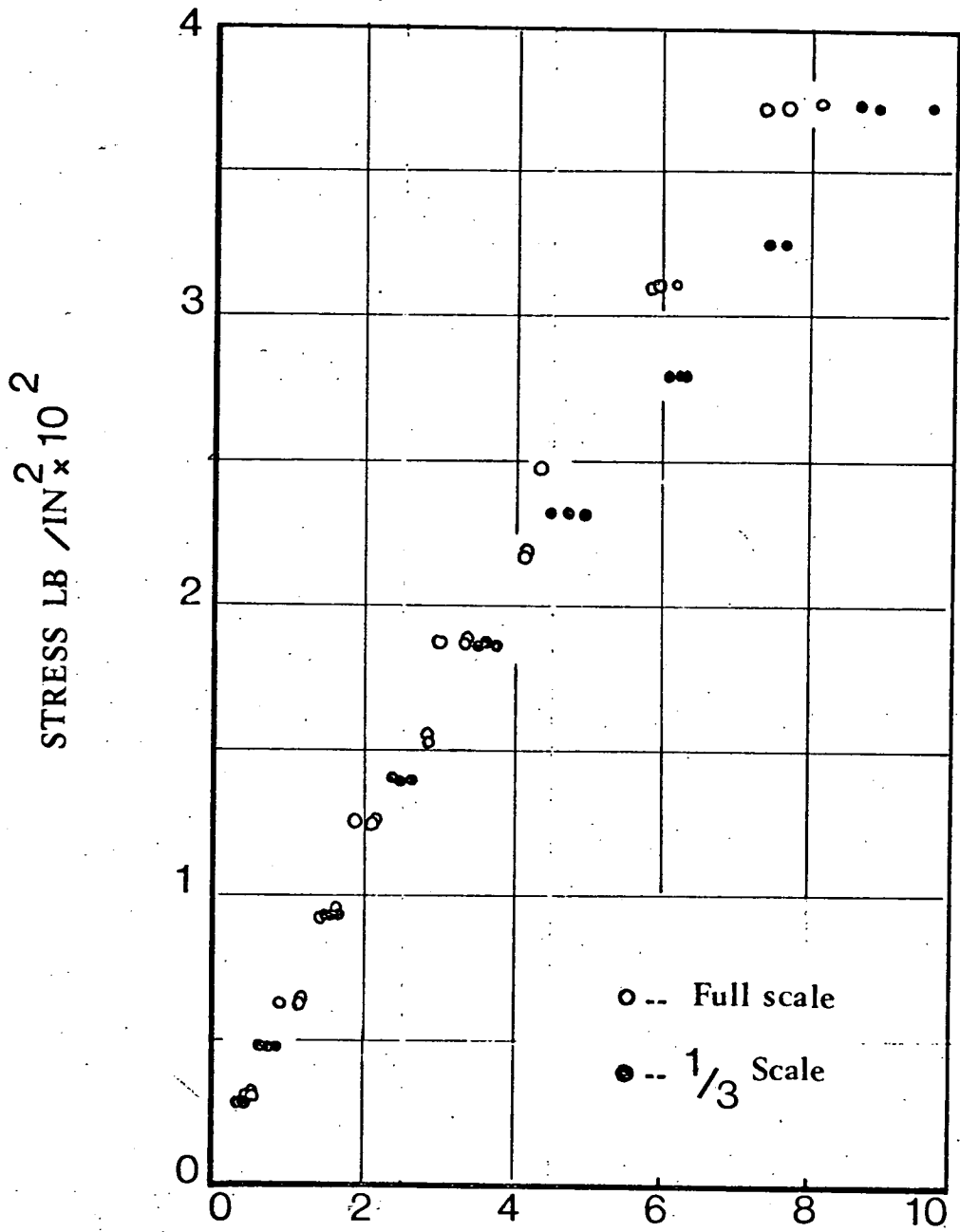


FIG. 3.3 STRAIN IN/IN × 10⁻⁴

LYTAG WALLS

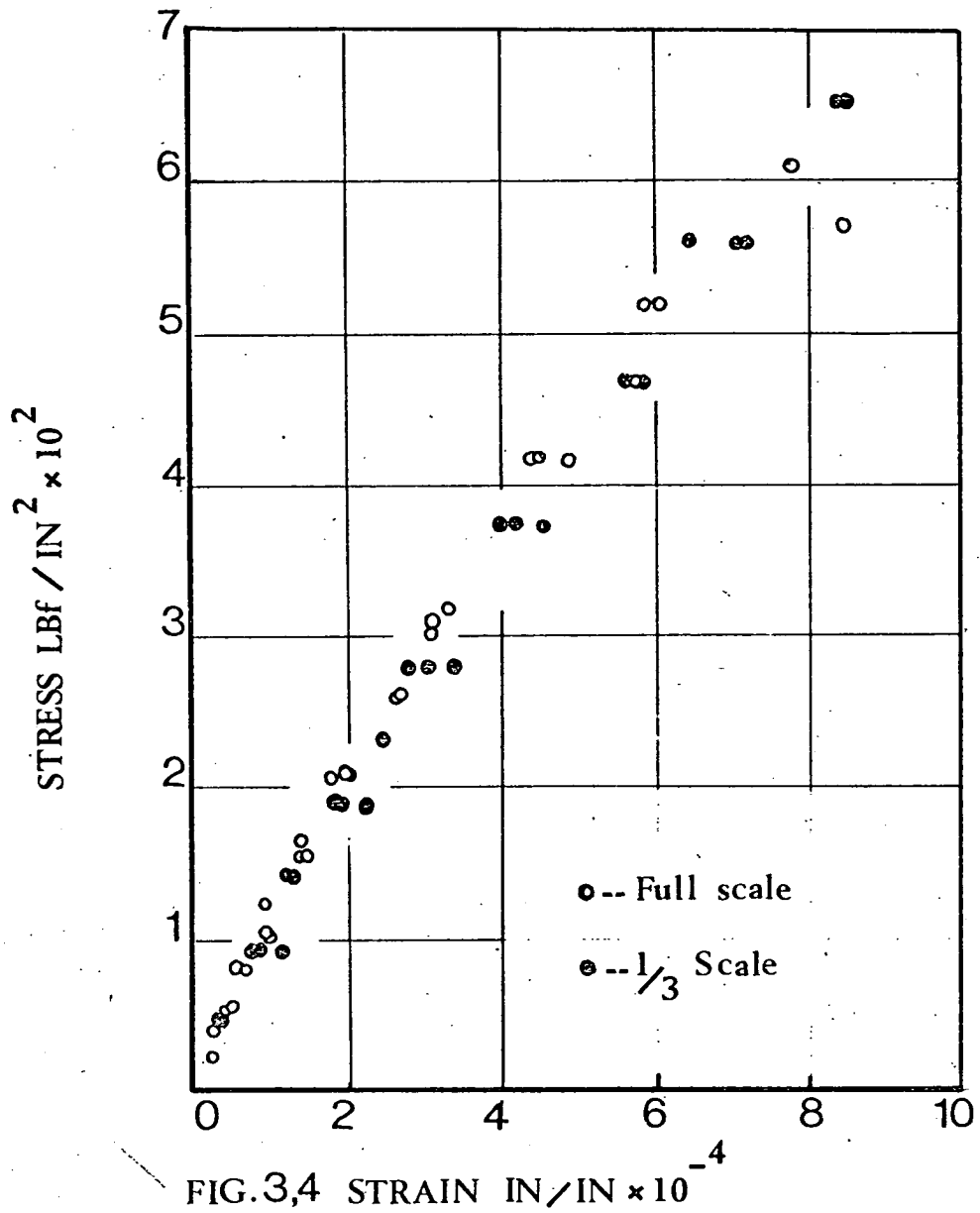


FIG.3,4 STRAIN $\text{IN}/\text{IN} \times 10^{-4}$

AGLITE WALLS

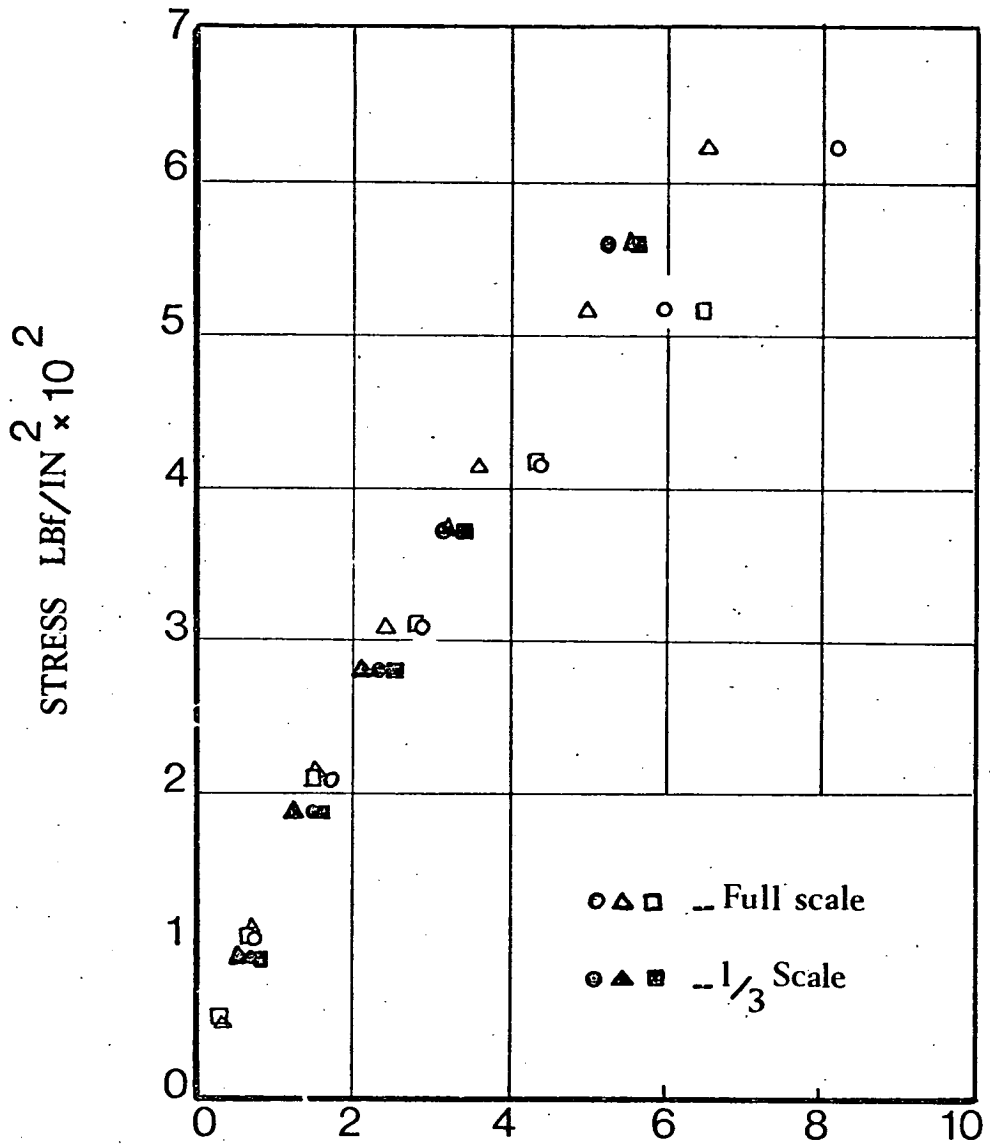


FIG. 3.5 STRAIN IN/IN × 10⁻⁴

THERMALITE WALLS

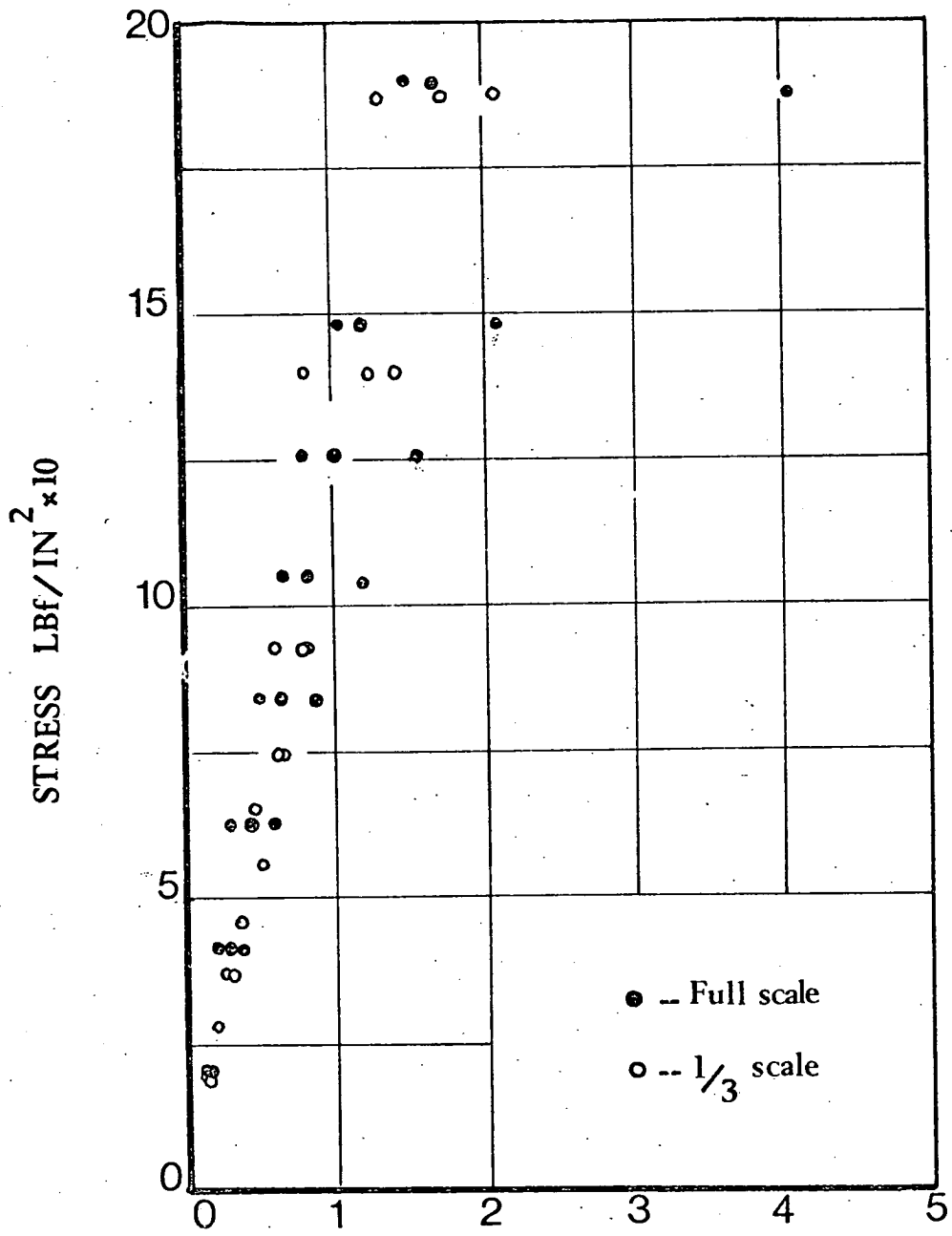


FIG. 3.6 LATERAL STRAIN IN/IN * 10⁻⁴

LECA WALLS

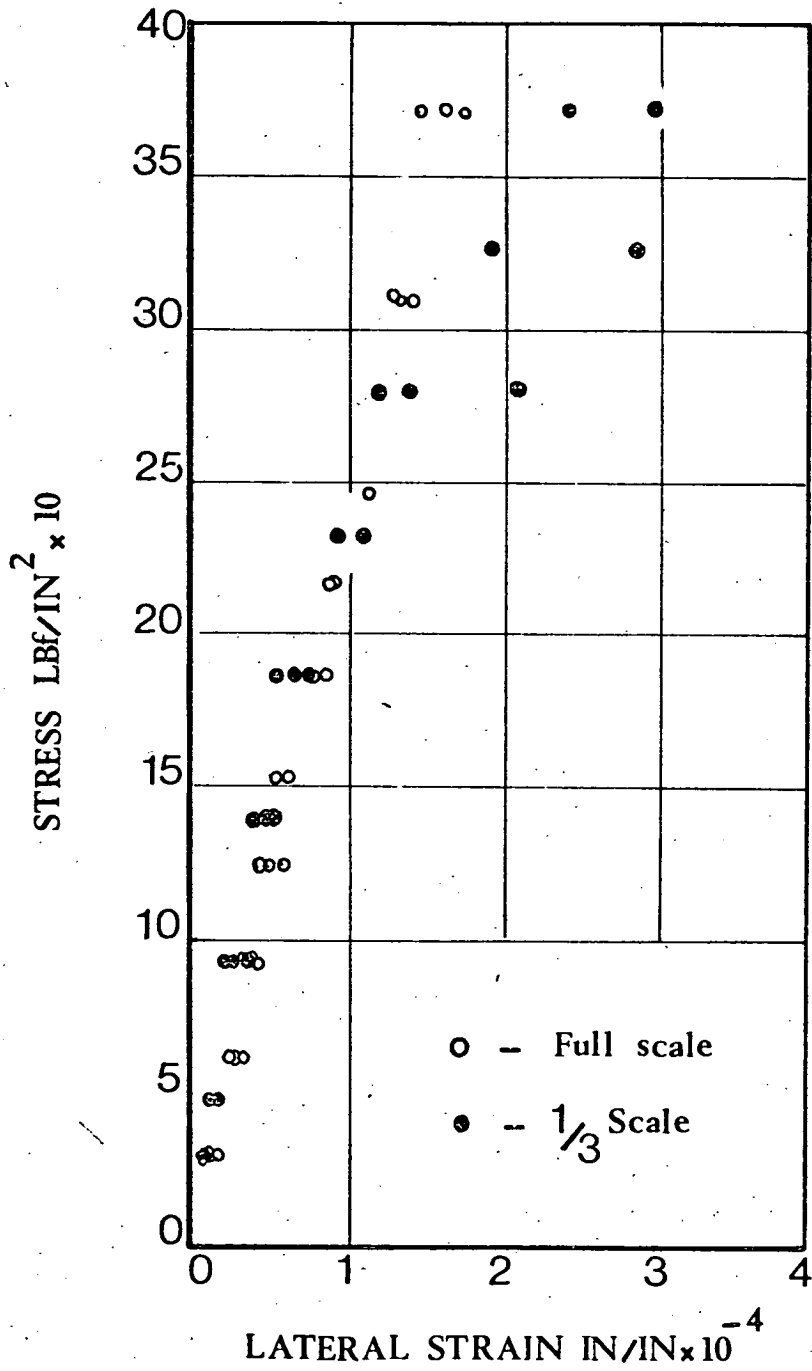


FIG. 3,7

LYTAG WALLS

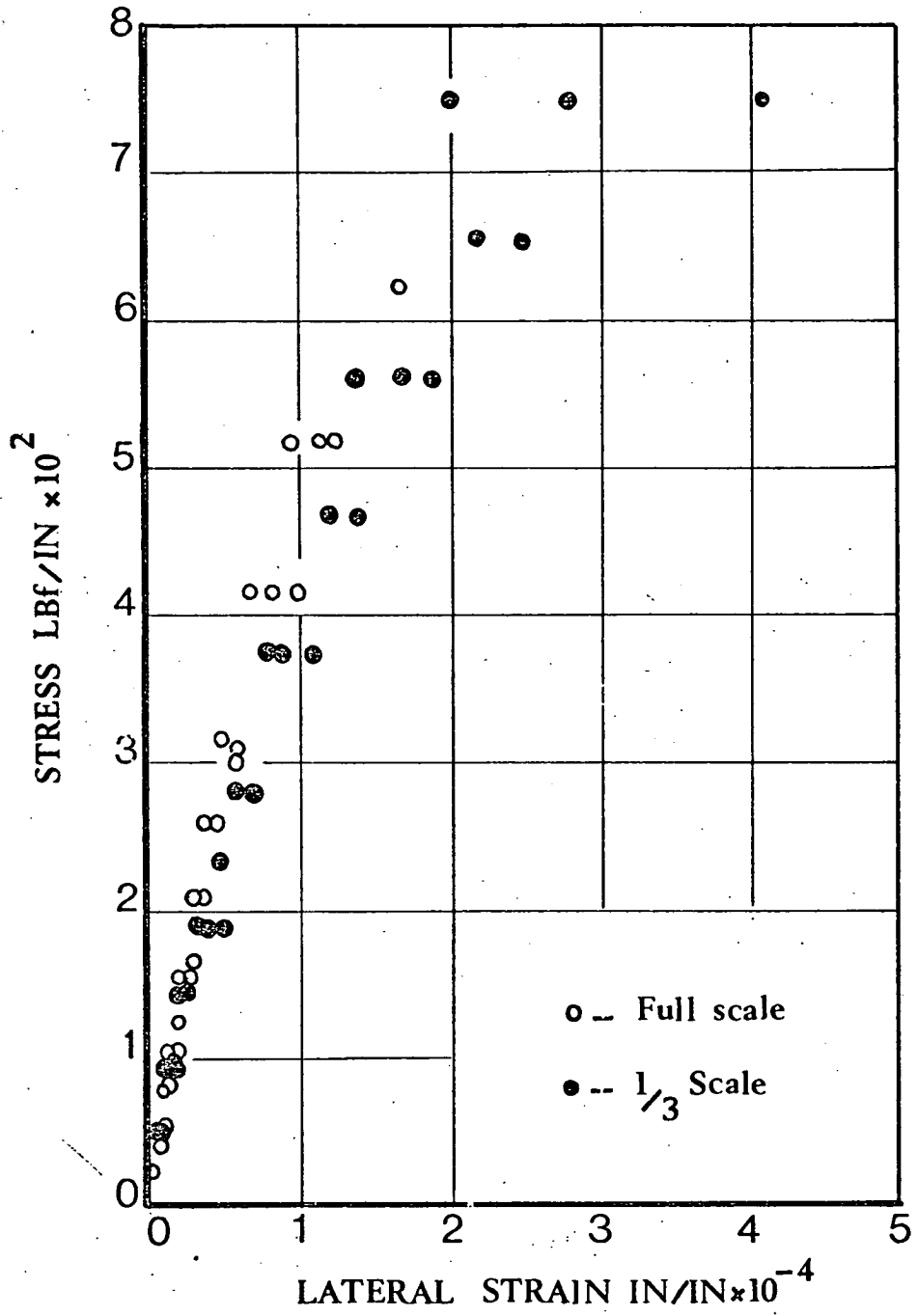


FIG. 3,8

AGLITE WALLS

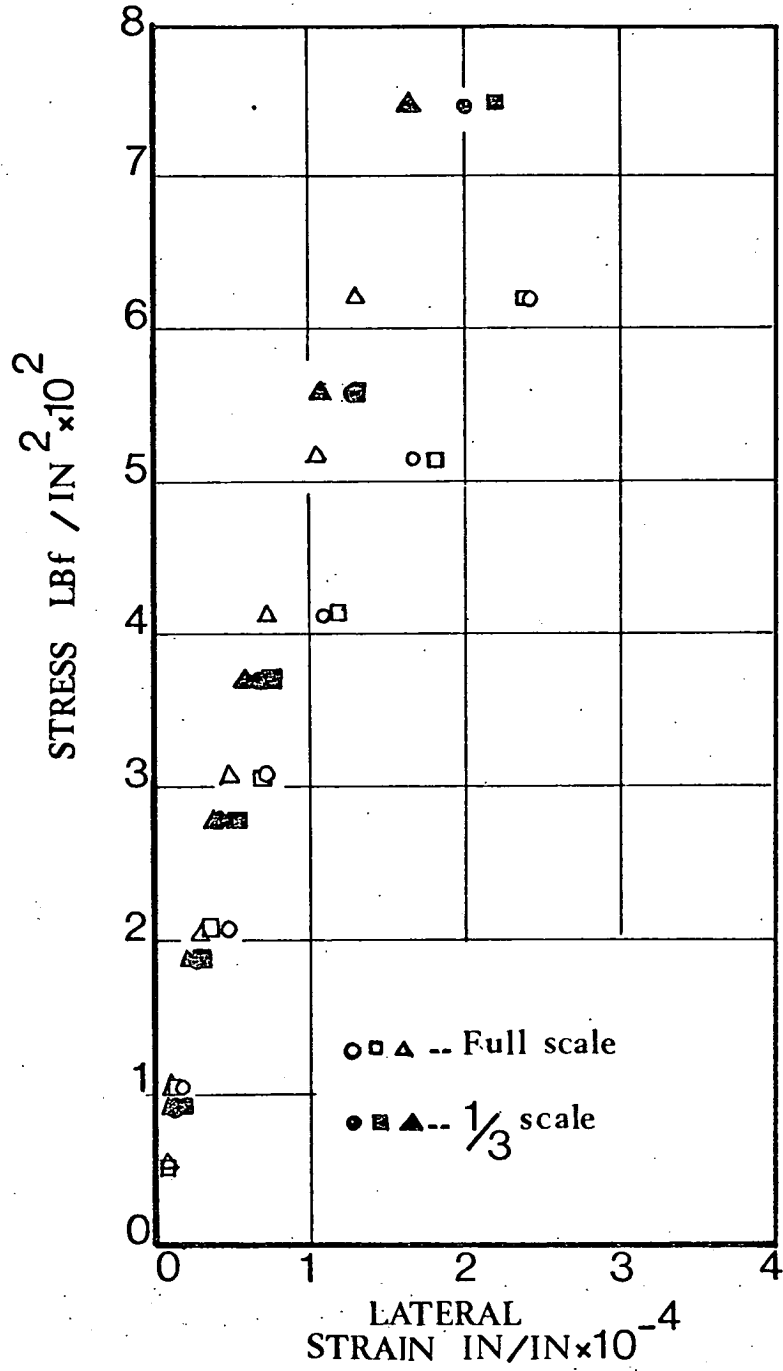


FIG. 3,9

THERMALITE WALLS

Full & $\frac{1}{3}$ scale

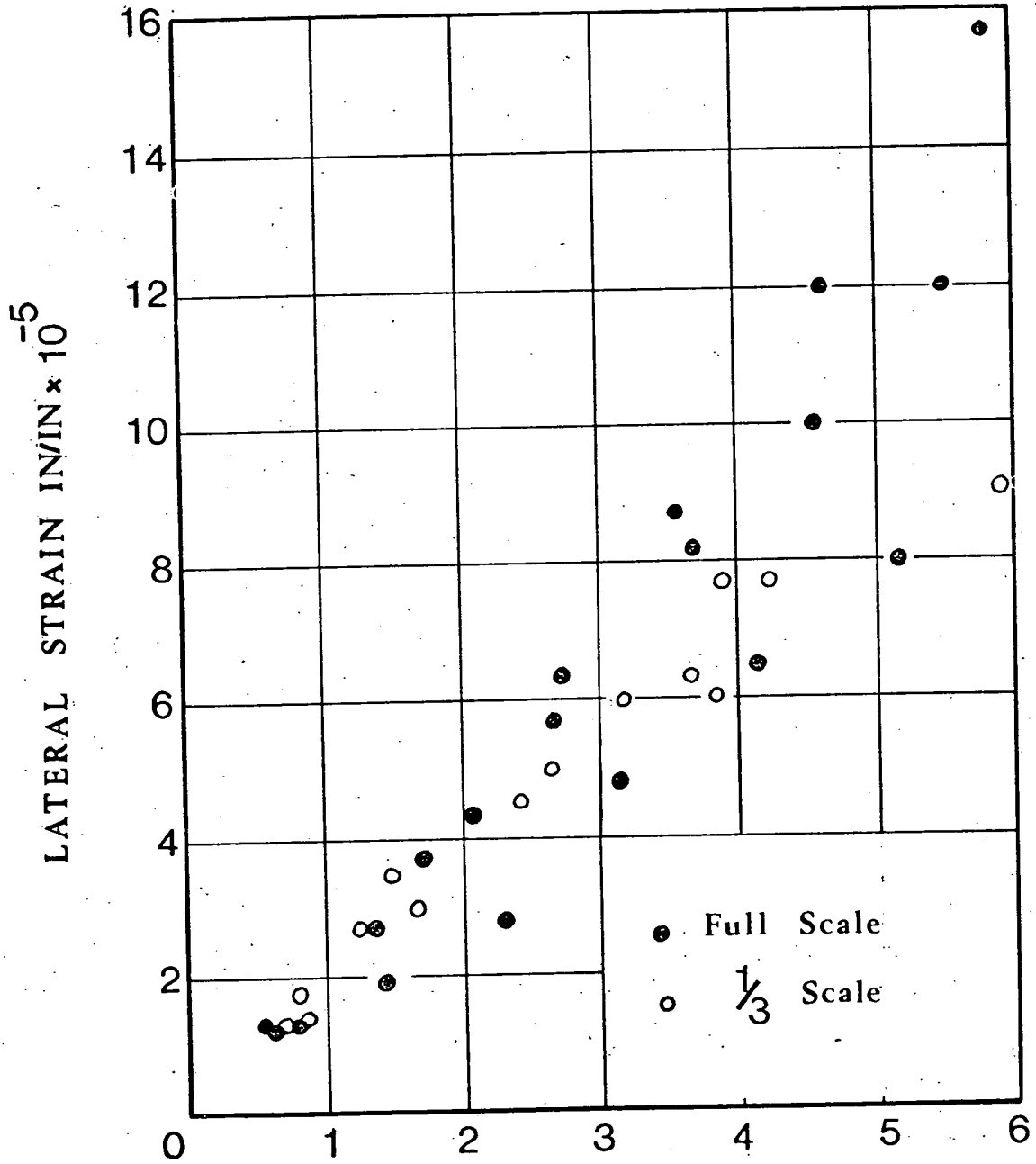


FIG.3.10 STRAIN IN/IN $\times 10^{-4}$

LECA WALLS
 Full & 1/3 scale

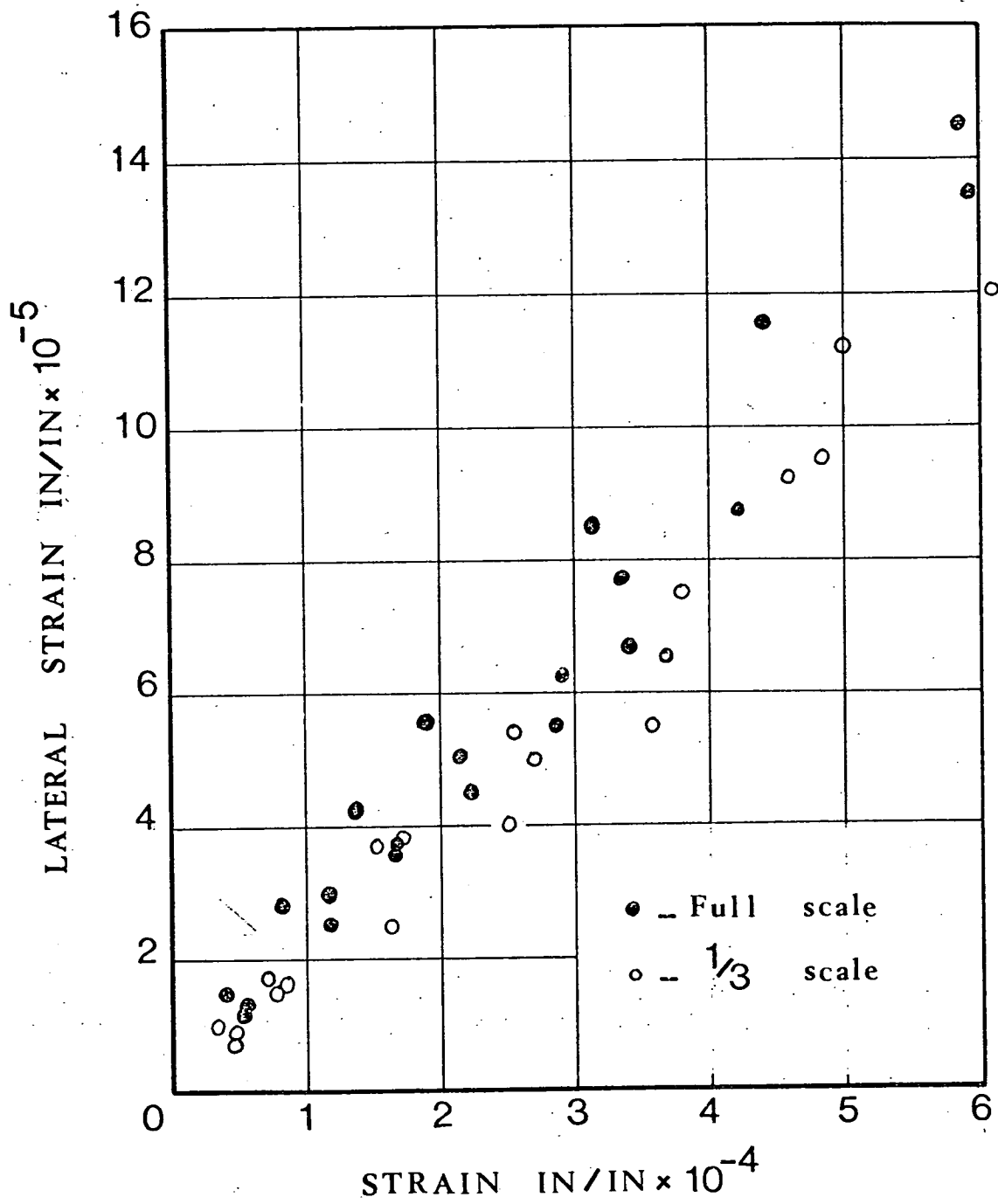


FIG. 3.11

LYTAG WALLS
 FULL & 1/3 SCALE

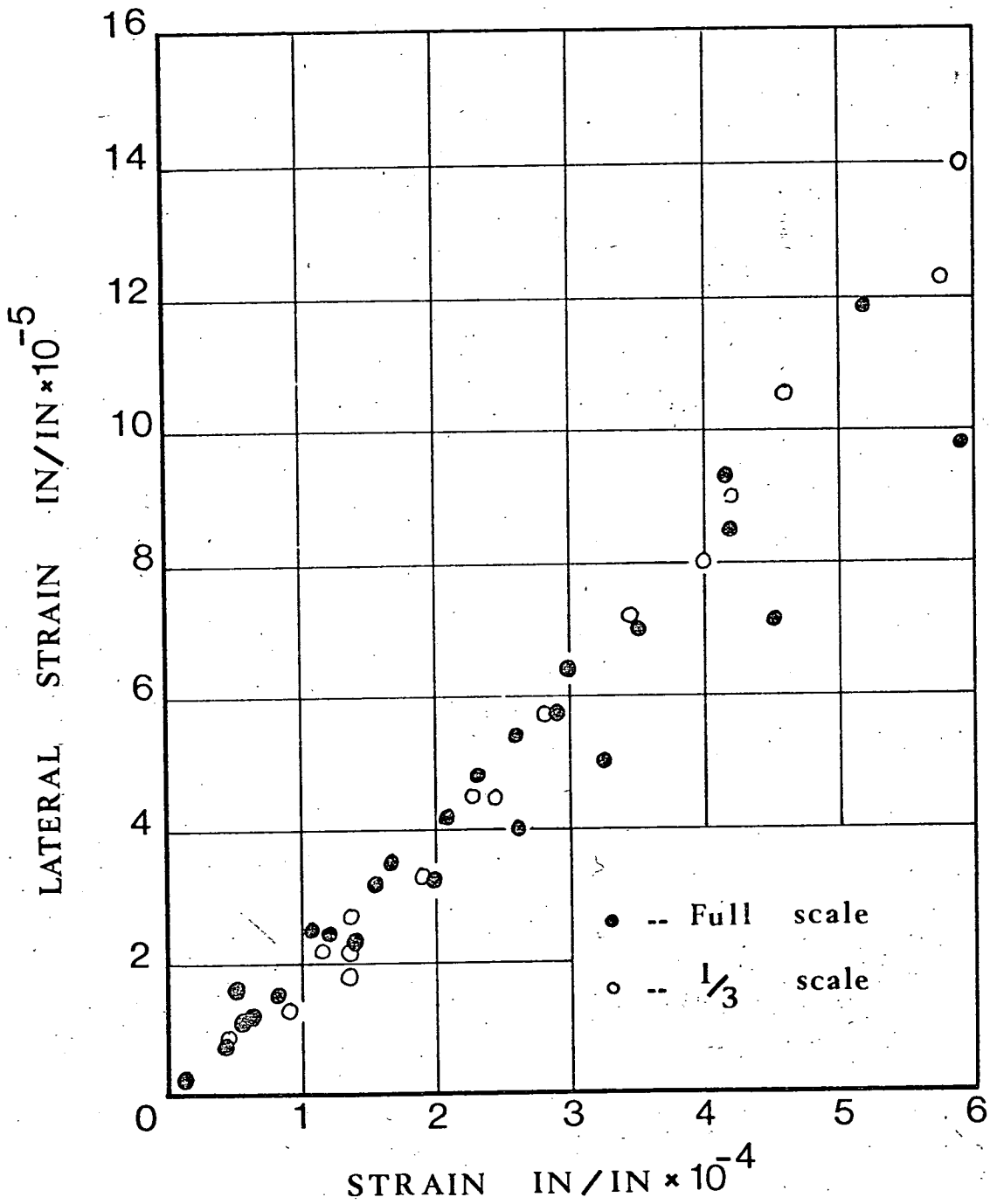


FIG. 3,12

AGLITE

FULL & 1/3 SCALE WALLS

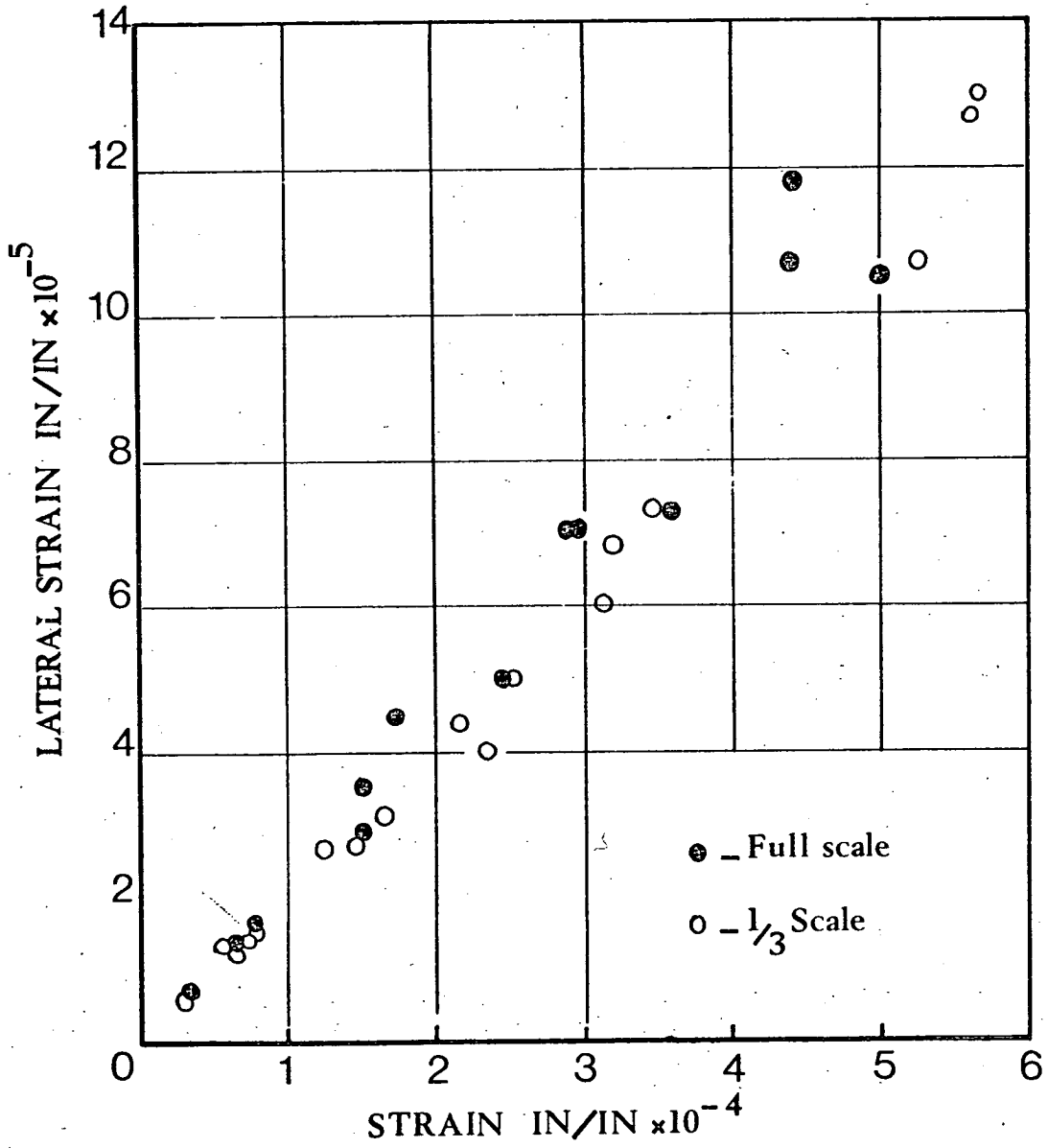


FIG.3.13

BLOCKWORK STRENGTH LBF / IN.²

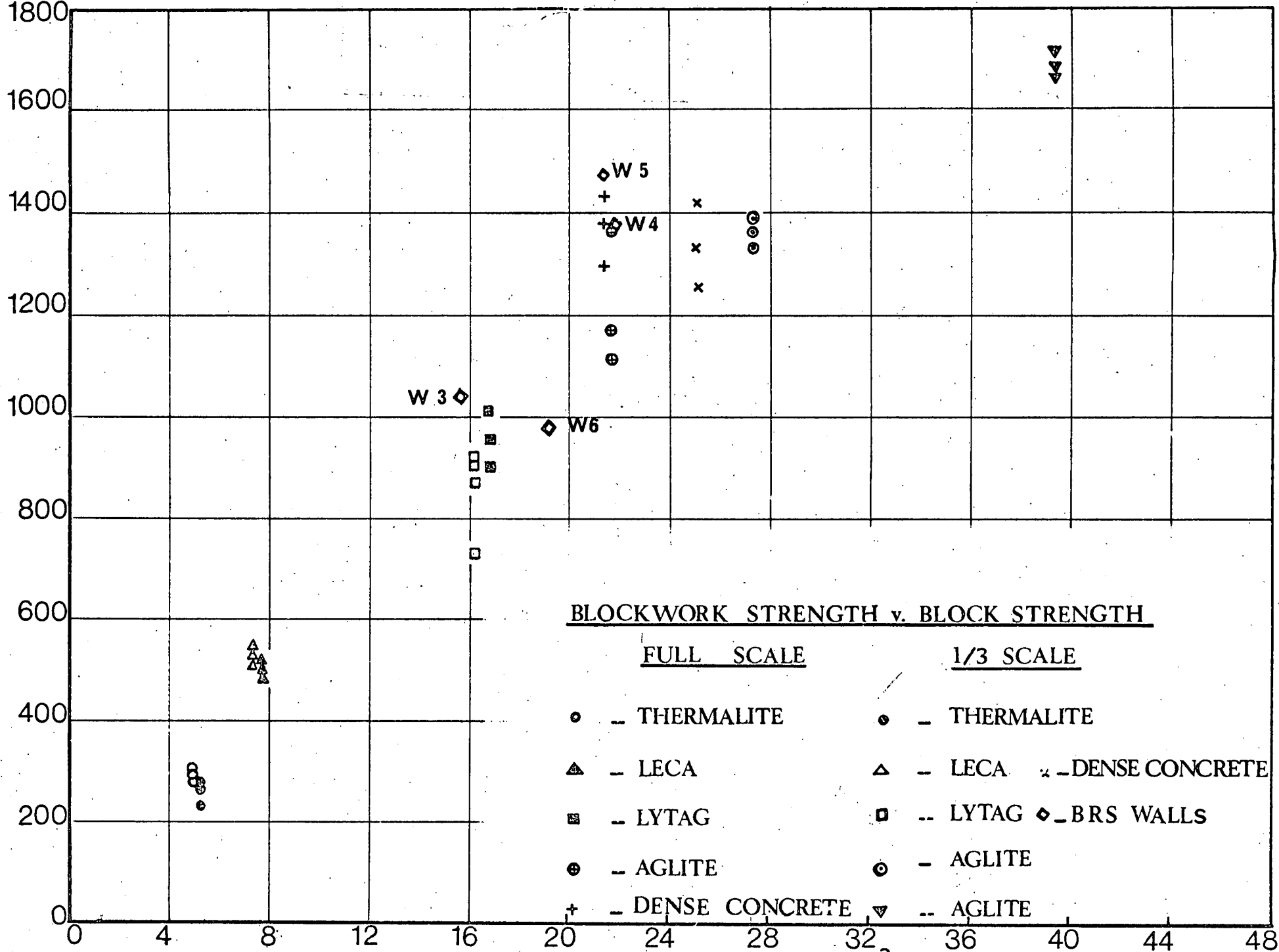


FIG 3.14 BLOCK STRENGTH LBF / IN.²

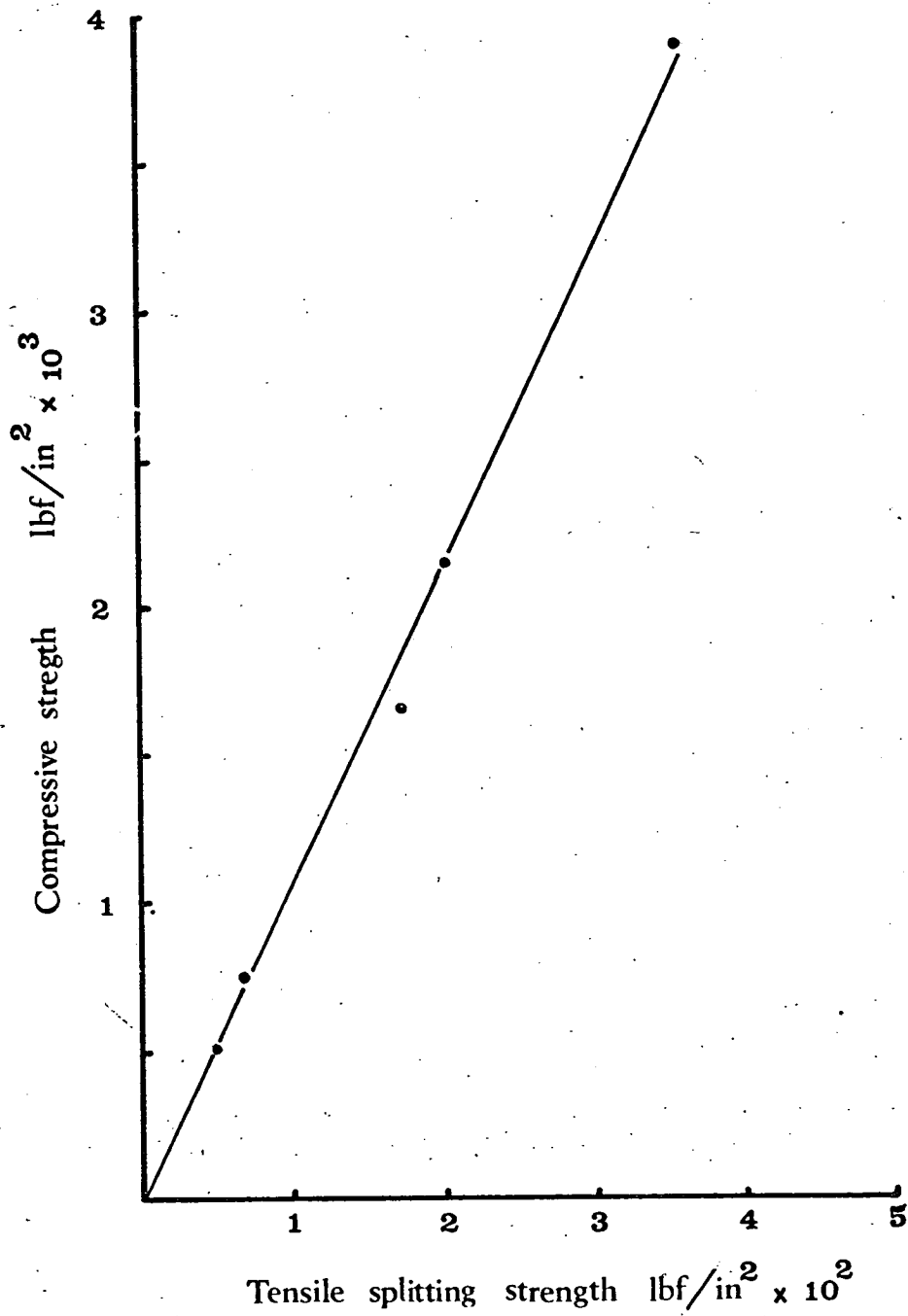


Fig.3.15 Compressive/Tensile strength of the light-weight concrete blocks

THERMALITE MODEL WALLS

1/3 SCALE

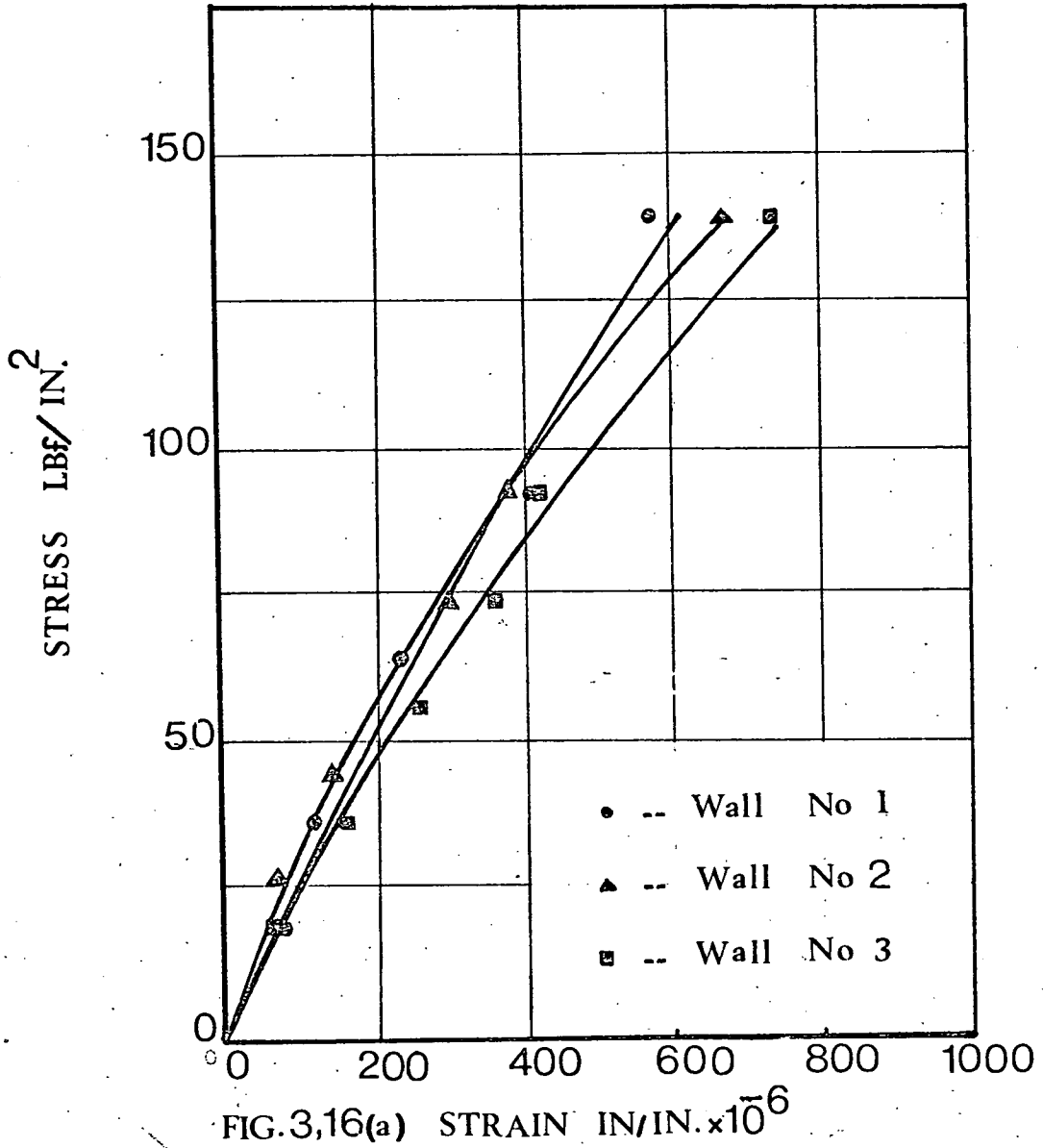


FIG. 3, 16(a) STRESS LBF/IN.²

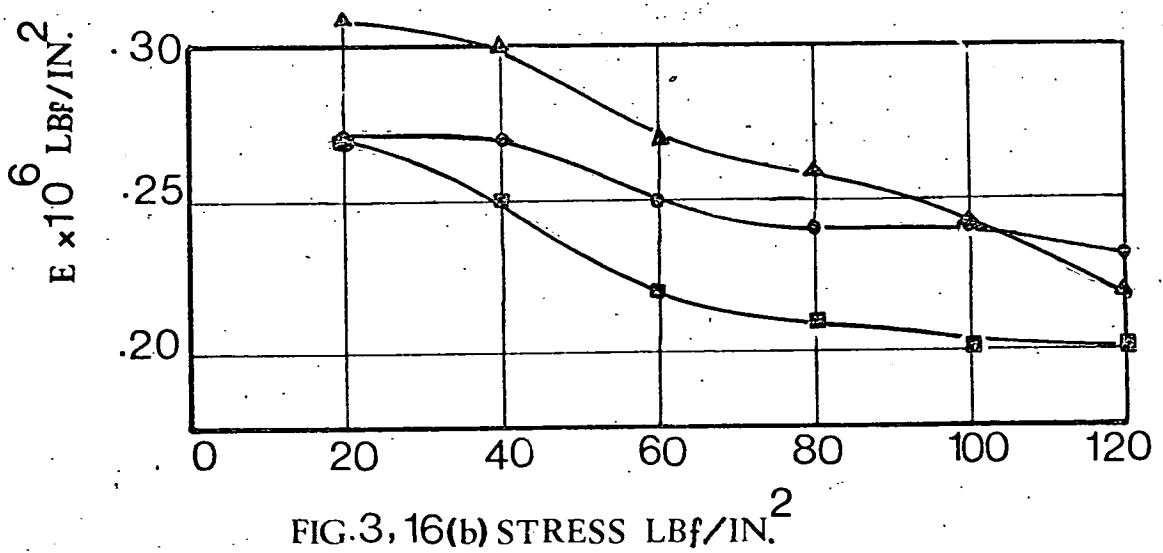


FIG. 3, 16(b) STRESS LBF/IN.²

THERMALITE WALLS, FULL SCALE.

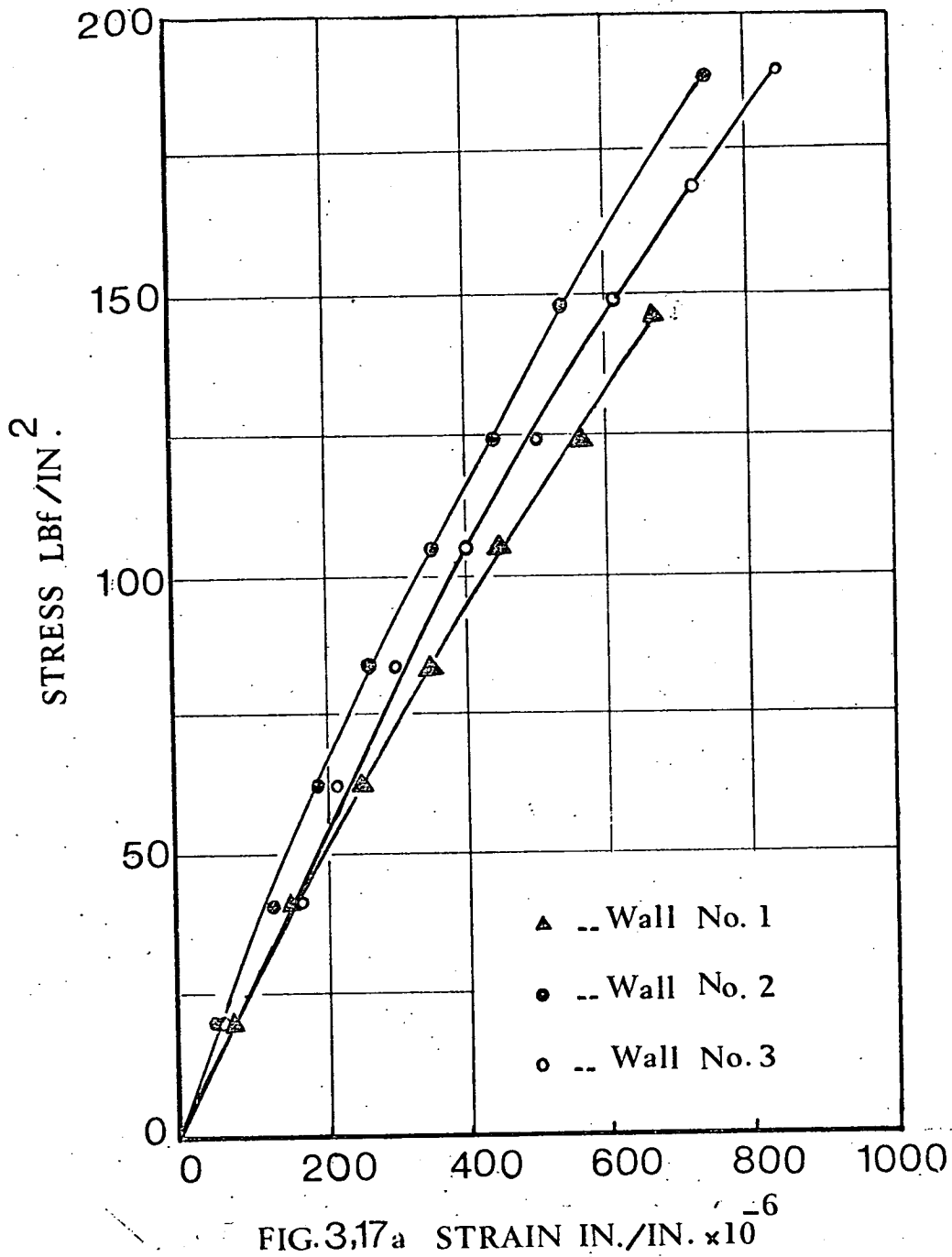


FIG. 3,17a

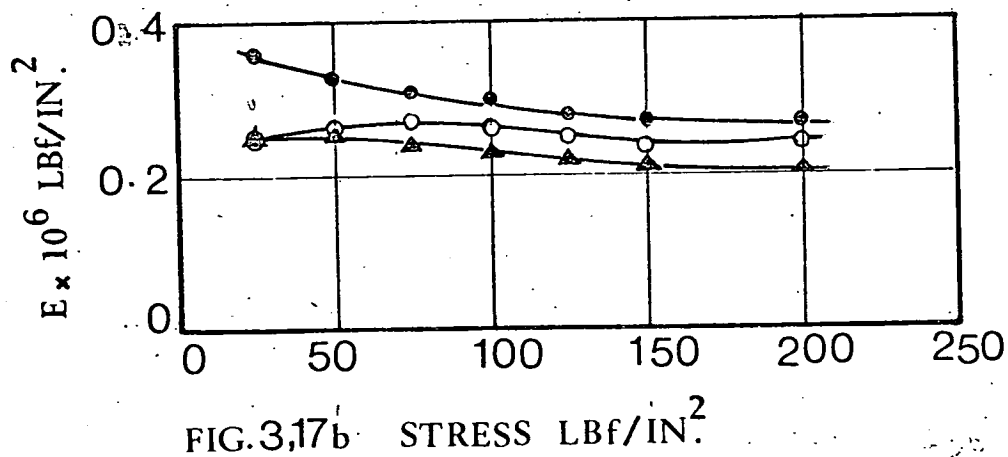


FIG. 3,17b

LECA MODEL WALLS

1/3 SCALE

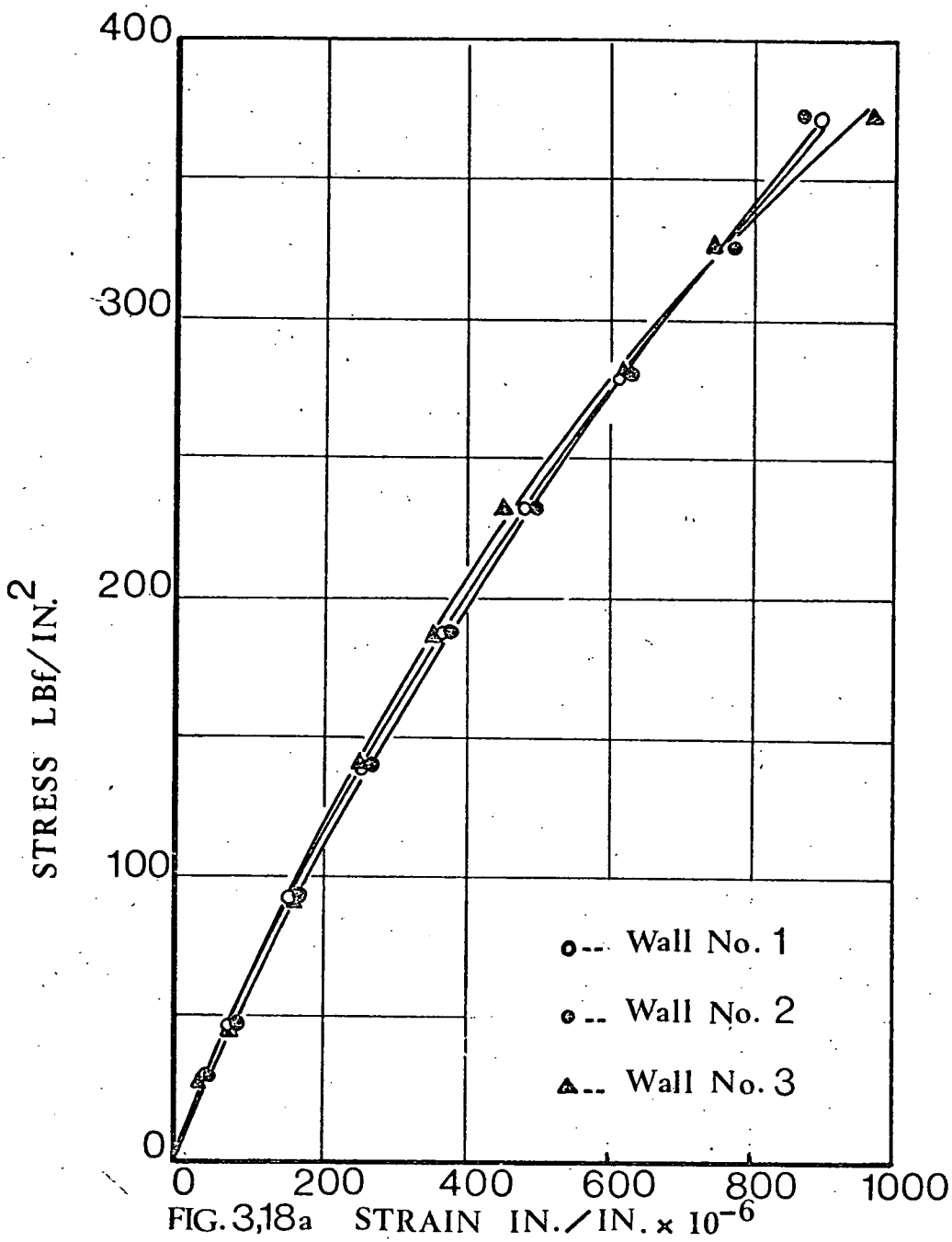


FIG. 3,18a

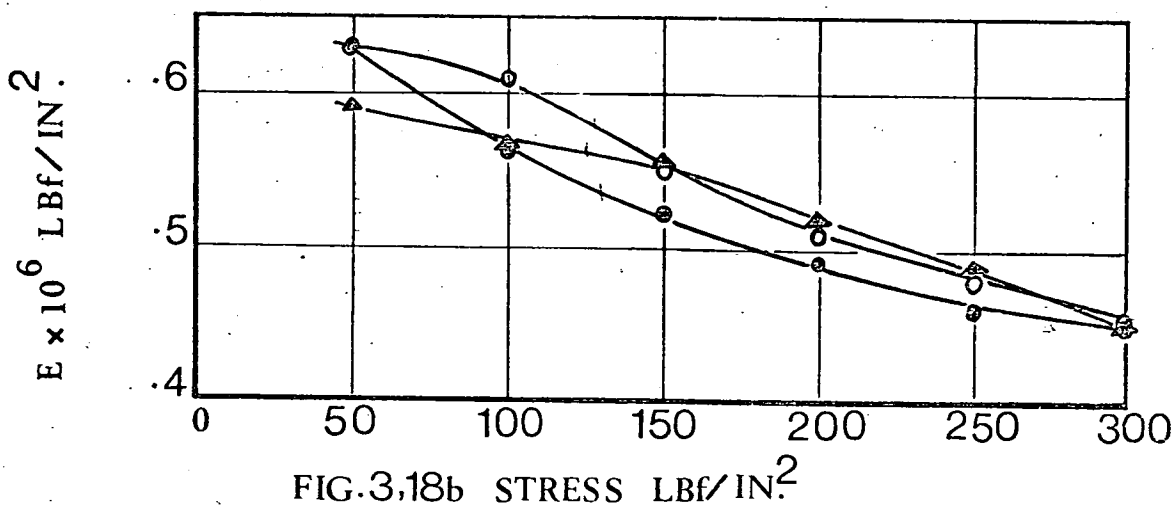
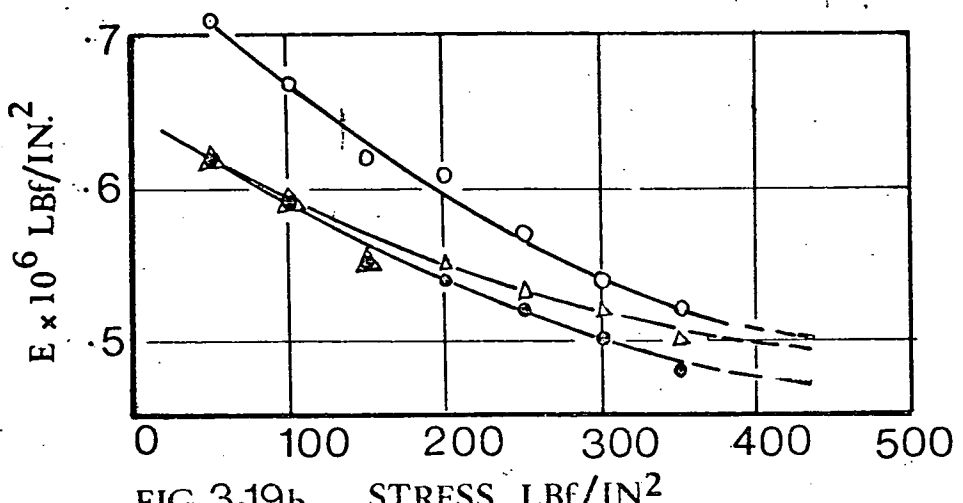
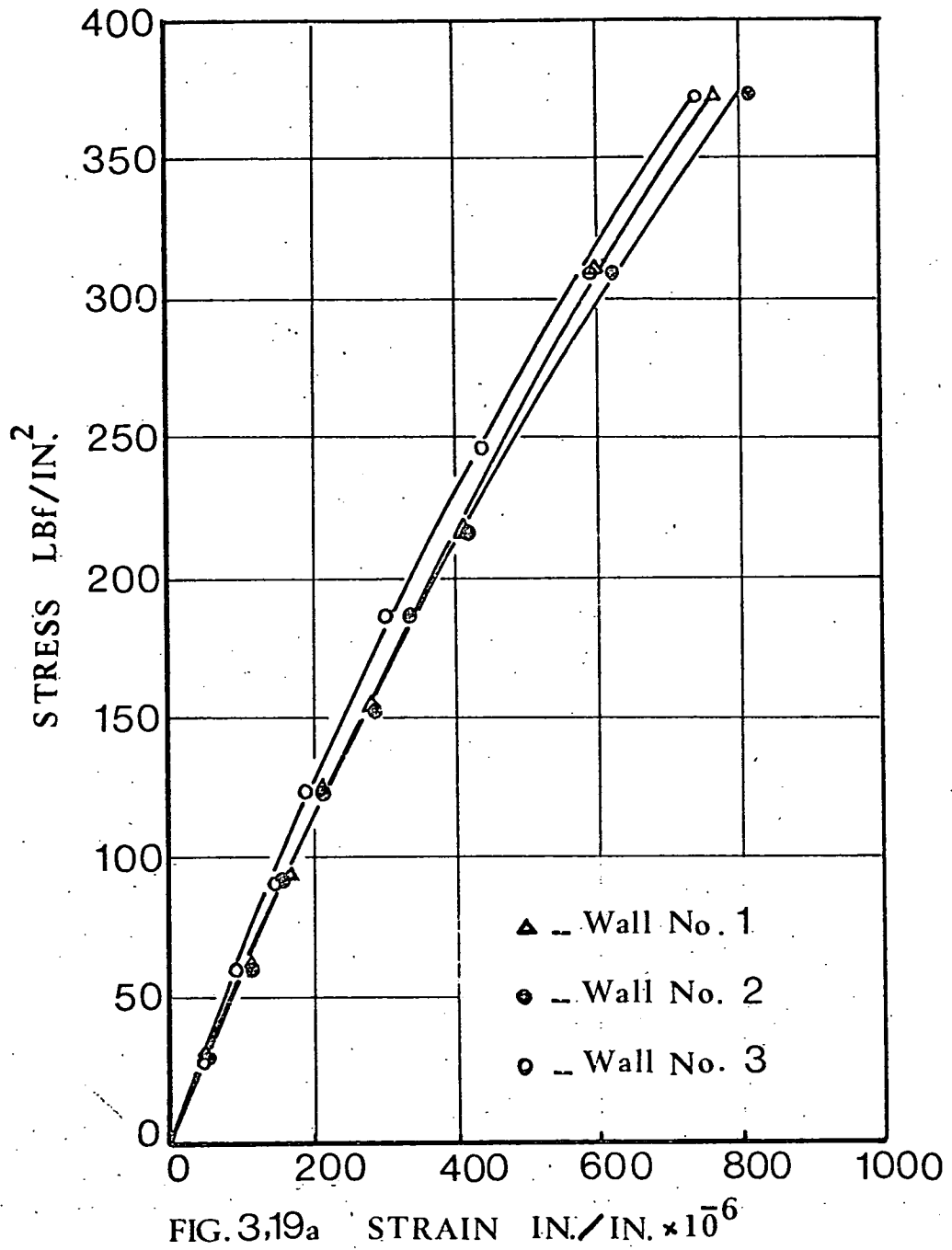


FIG. 3,18b

LECA WALLS

FULL SCALE



LYTAG MODEL WALLS

1/3 SCALE

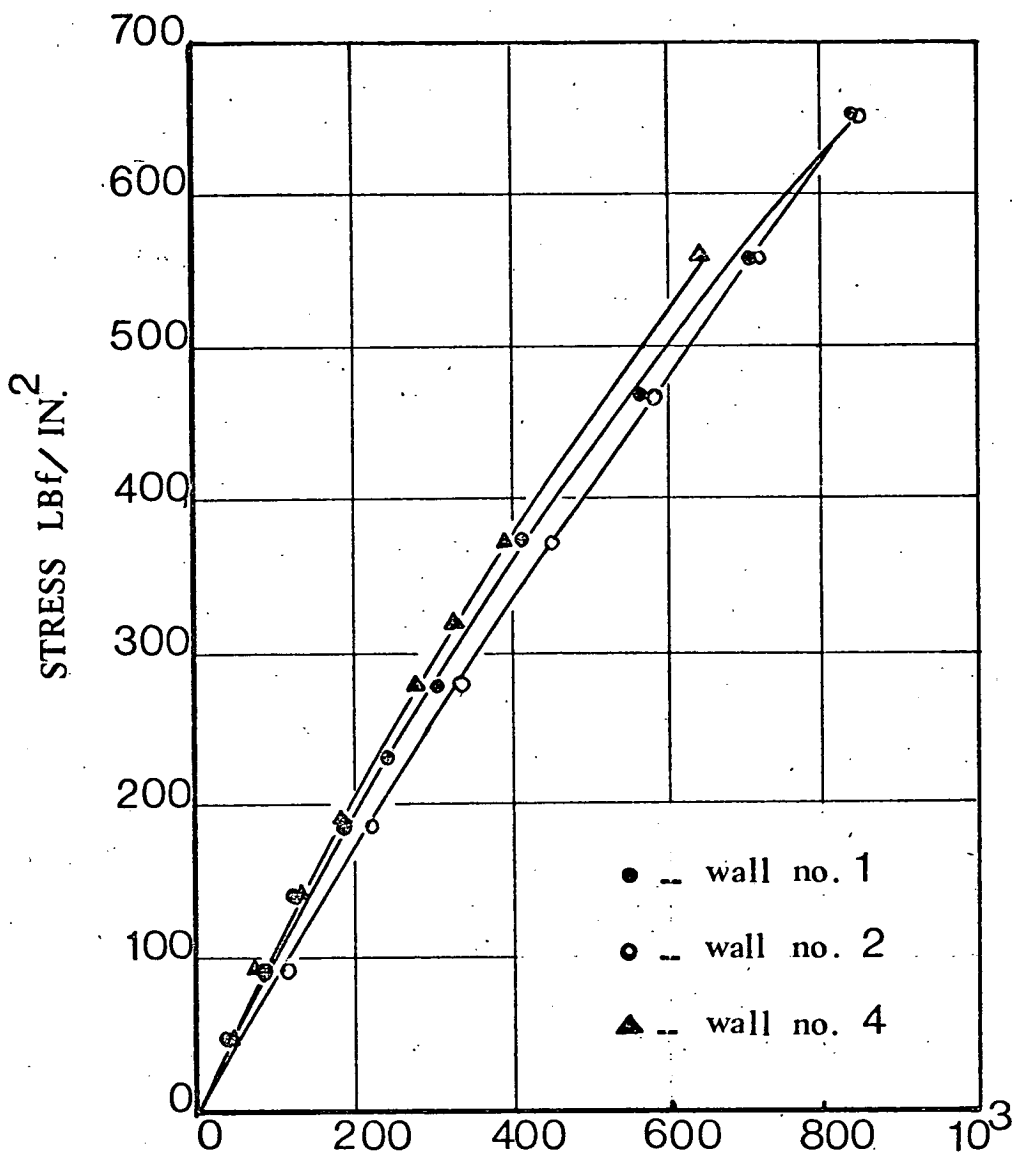


FIG. 3.20a STRAIN IN./IN. × 10⁻⁶

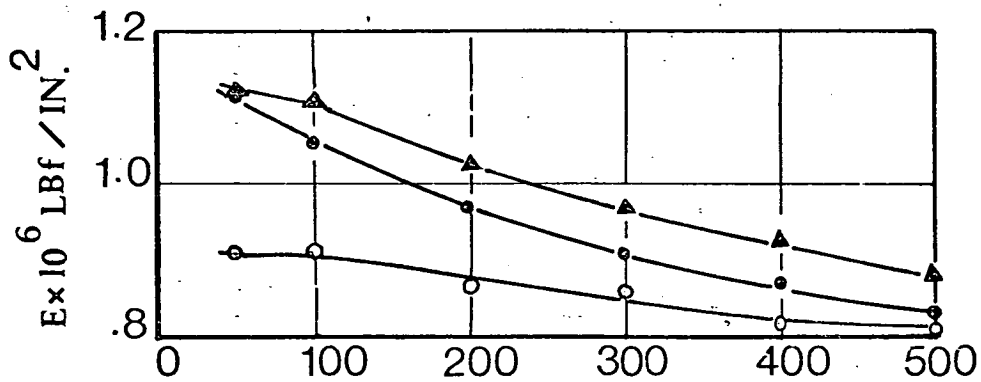
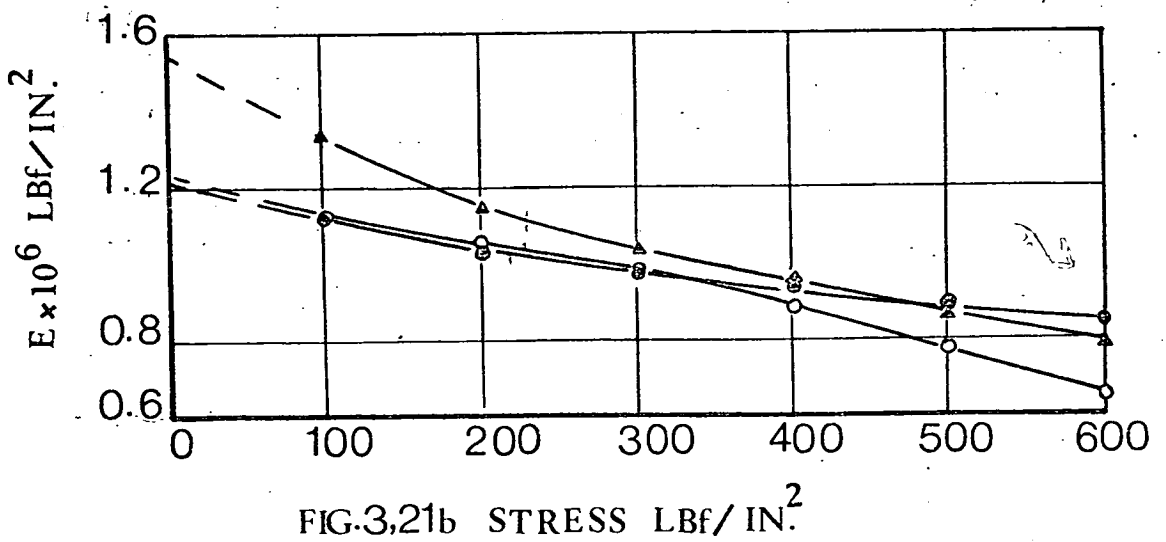
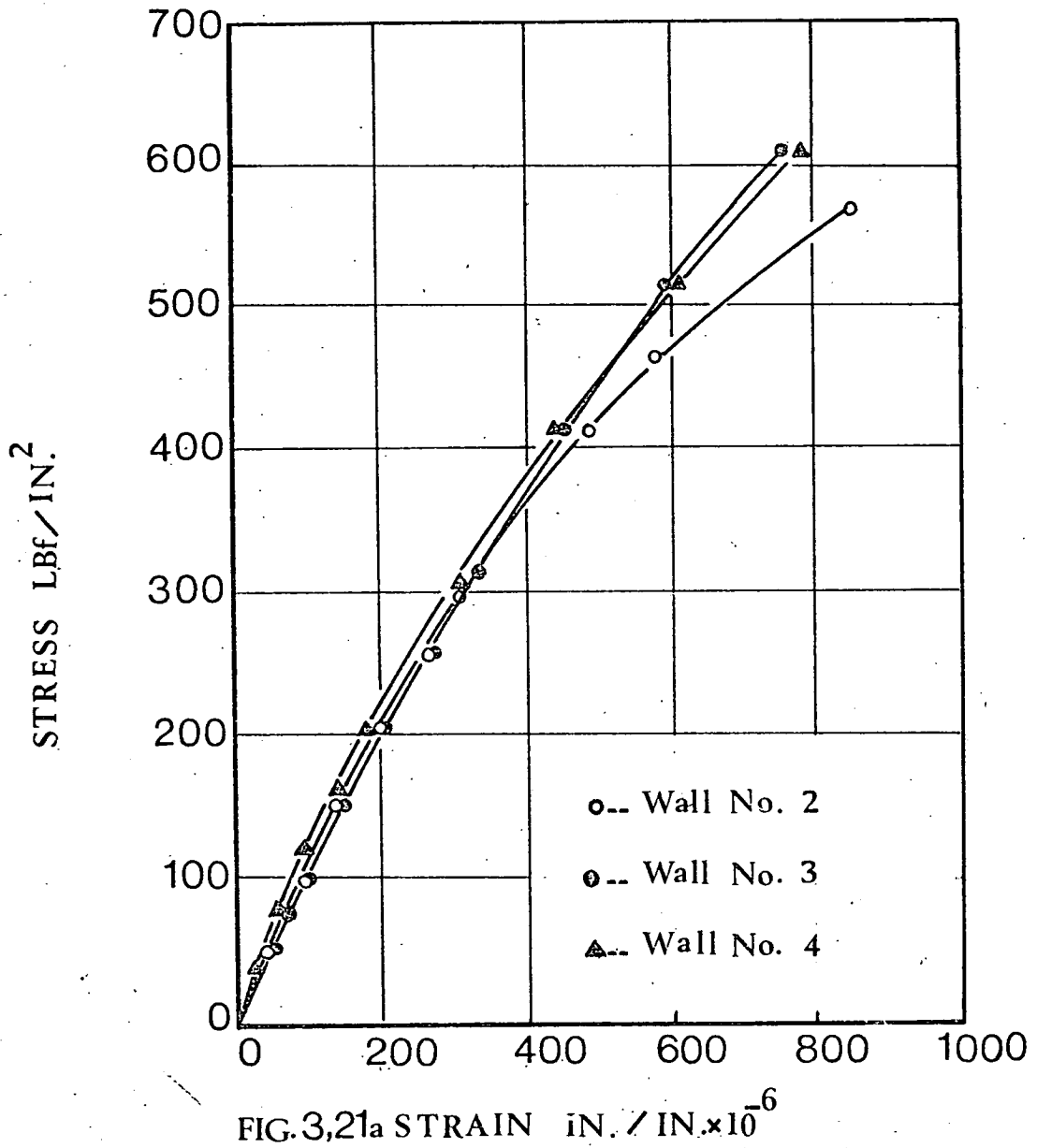


FIG. 3.20b STRESS Lbf/IN.²

LYTAG WALLS

FULL SCALE



AGLITE MODEL WALLS

1/3 SCALE

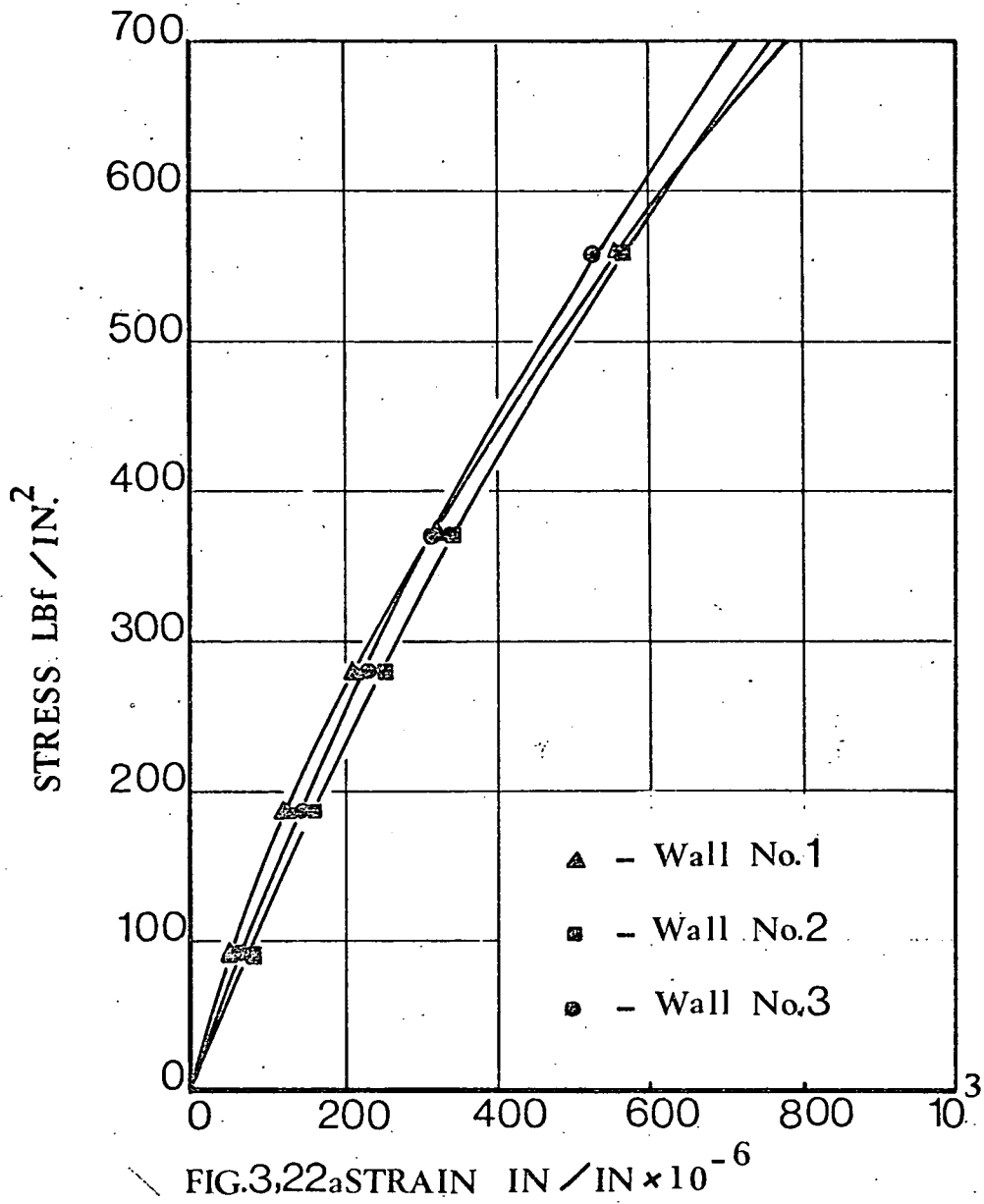


FIG. 3.22a

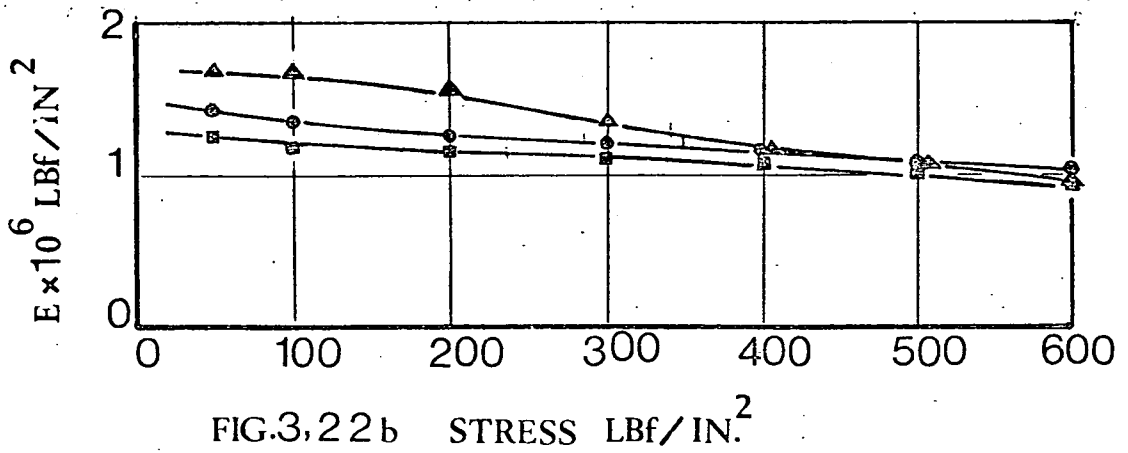


FIG. 3.22b

AGLITE MODEL WAELS

1/3 SCALE

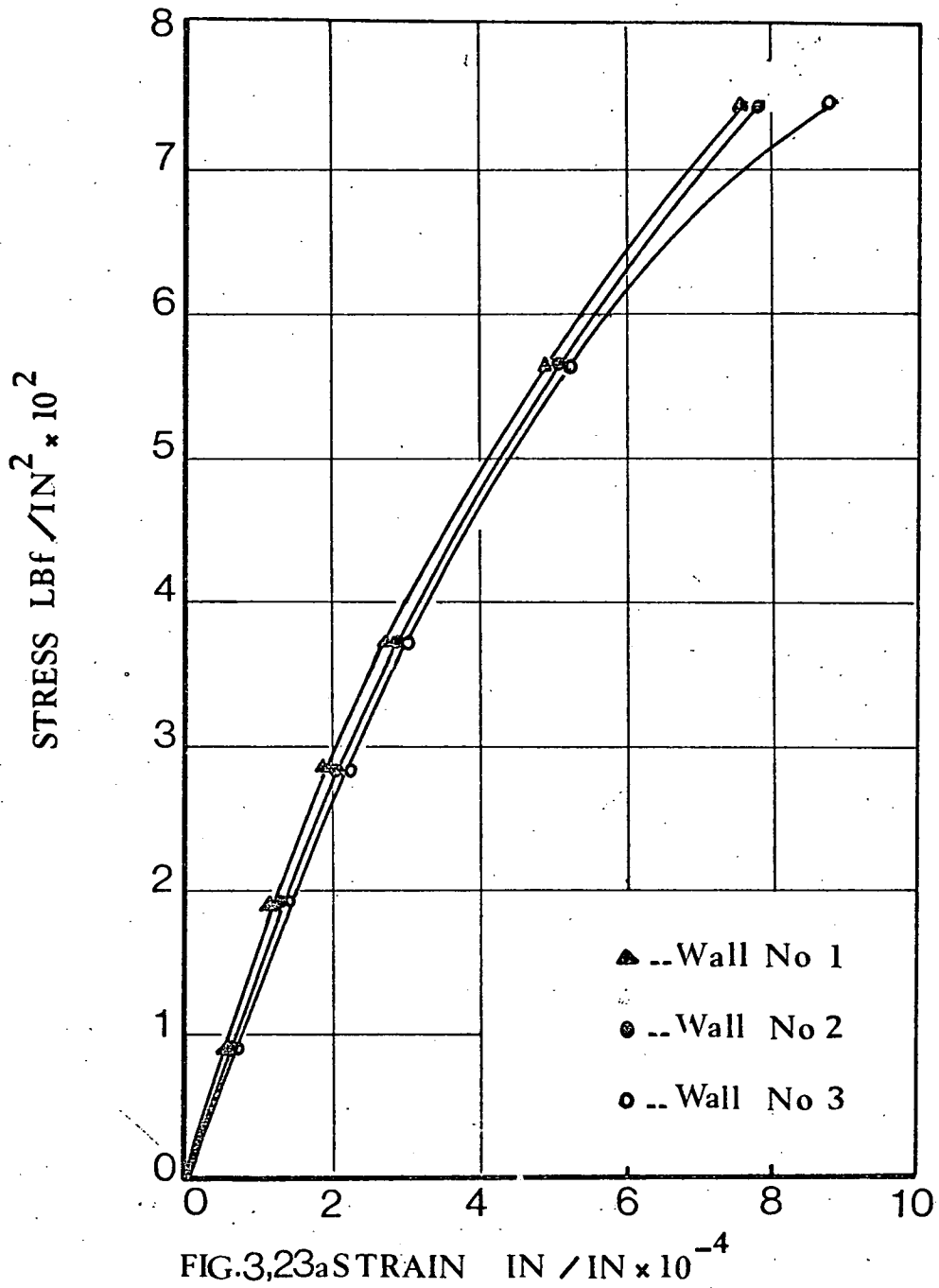


FIG. 3,23a

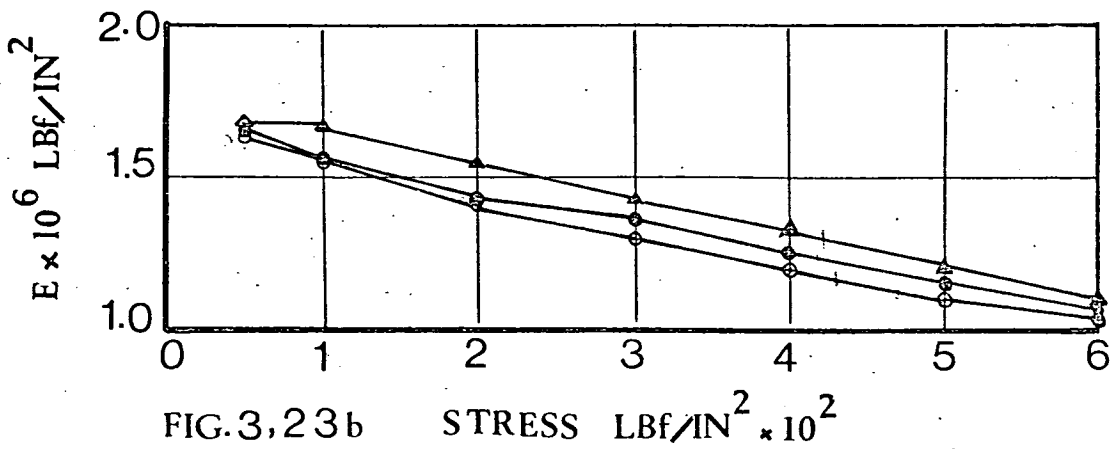
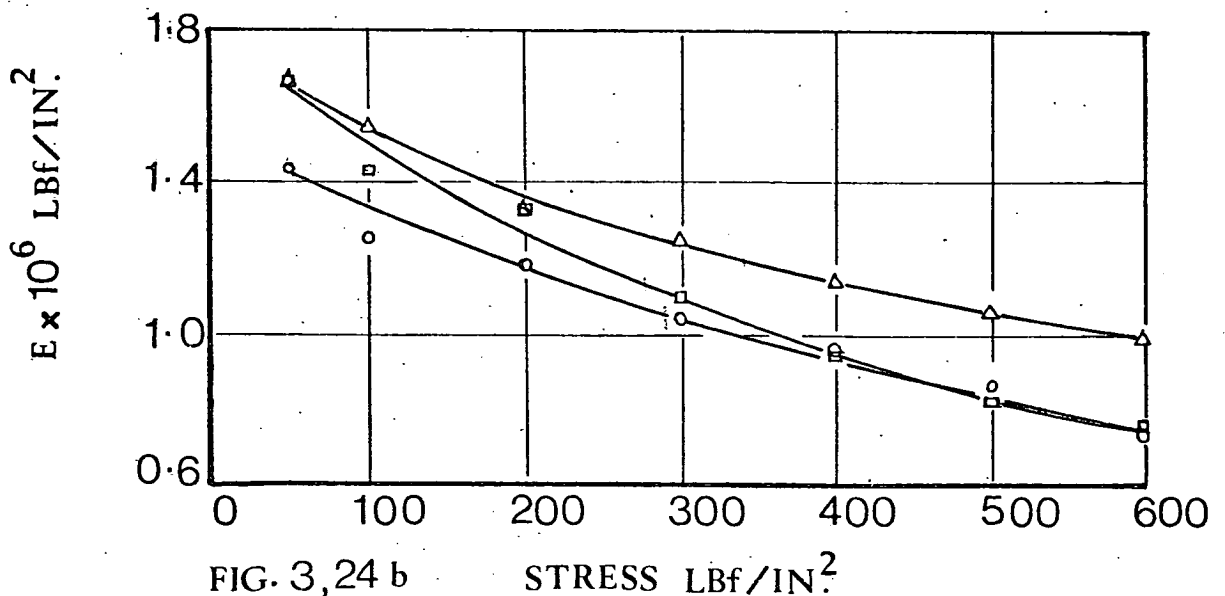
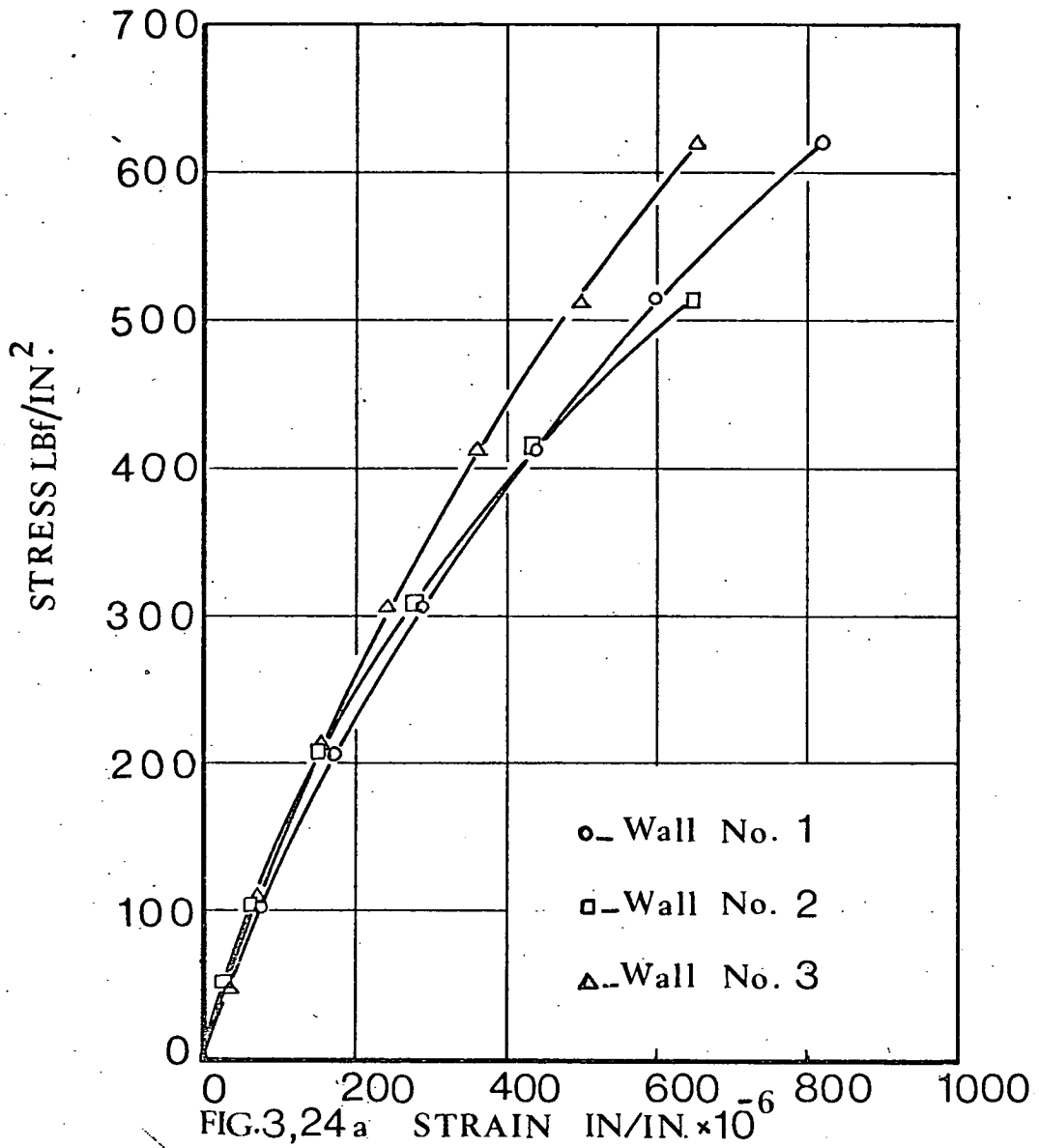


FIG. 3,23b

AGLITE WALLS
FULL SCALE



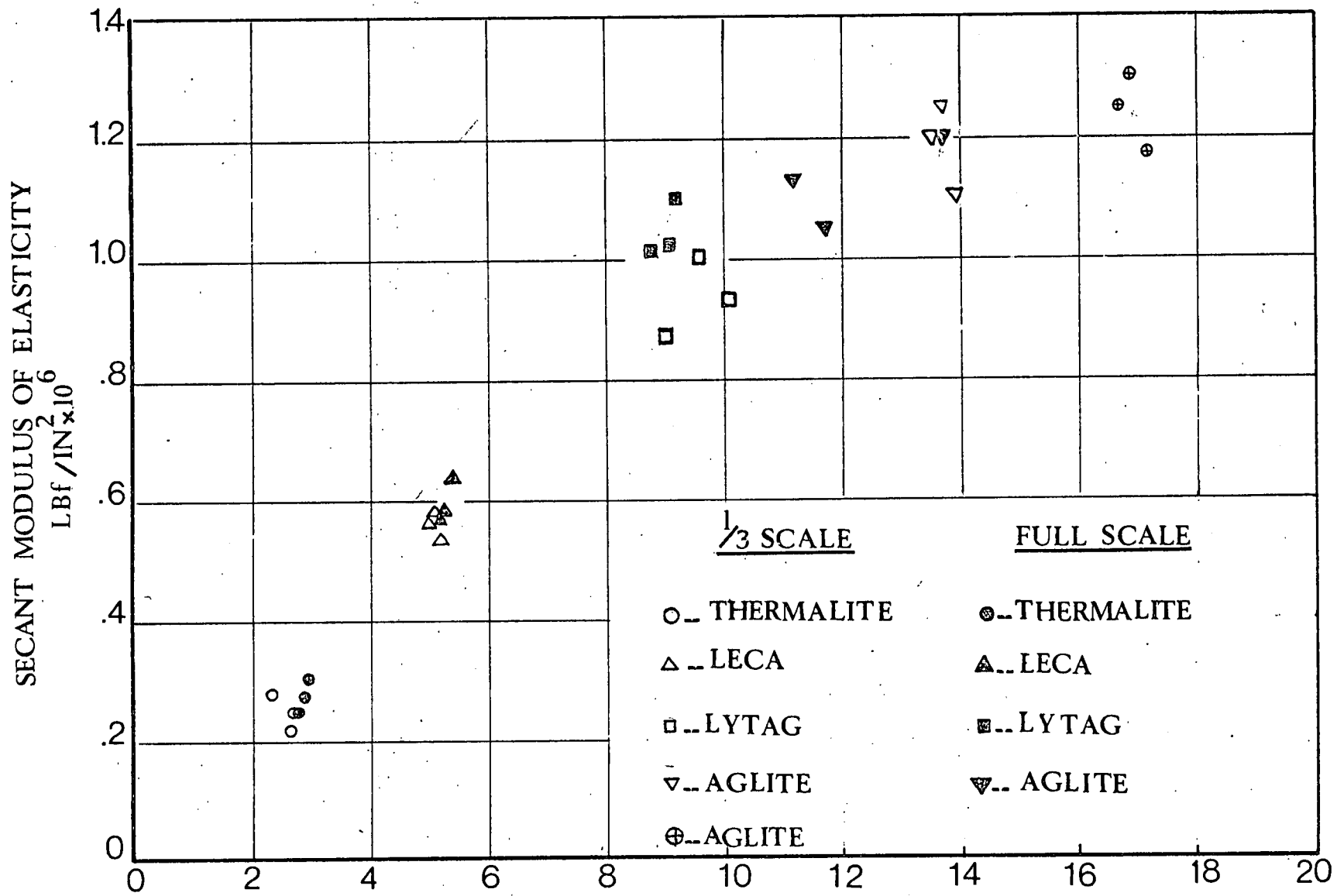


FIG.3,25 WALLS ULTIMATE STRENGTH LBF IN² × 10²

TABLE 3.7 - RESULTS OF TESTS ON FULL AND ONE-THIRD SCALE WALLS BUILT IN

"THERMALITE BLOCKS AND 1:1:6 MORTAR

Wall No.	Age at test (days)	Mortar strength (lbf/in ²)	Block strength (lbf/in ²)	Wall strength (lbf/in ²)	Strength Ratio wall/block	Secant Modulus of Elasticity at $\frac{1}{4}$ of ultimate load (lbf/in ²)x10 ⁶	Poisson's Ratio at $\frac{1}{4}$ ultimate load
FW1	19	456	500	282	0.56	0.25	0.22
FW2	14	409	500	294	0.58	0.30	0.21
FW3	14	366	500	286	0.57	0.27	0.15
Ave.	-	410	500	287	0.57	0.24	0.19
SW1	14	356	517	271	0.53	0.245	0.15
SW2	19	356	517	233	0.45	0.273	0.21
SW3	28	356	517	266	0.51	0.215	0.2
Ave.	-	356	517	257	0.5	0.244	0.18

Wall designation : FW = full-scale walls
 SW = one-third scale walls

TABLE 3.8 - RESULTS OF TESTS ON FULL AND ONE-THIRD SCALE WALLS BUILT

IN "LECA" BLOCKS AND 1:1:6 MORTAR

Wall No.	Age at Test (days)	Mortar Strength (lbf/in ²)	Block strength (lbf/in ²)	Wall ultimate strength (lbf/in ²)	Strength ratio (wall/block)	Secant Modulus of Elasticity of $\frac{1}{4}$ of ultimate load (lbf/in ²)x10 ⁶	Poisson's Ratio at $\frac{1}{4}$ of ult- imate load
FW1	15	470	733	522	0.71	0.58	0.23
FW2	17	455	733	520	0.70	0.57	0.20
FW3	15	446	733	537	0.73	0.64	0.29
Ave.	-	457	733	526	0.71	0.59	0.24
SW1	15	425	763	509	0.67	0.580	0.21
SW2	17	425	763	523	0.68	0.54	0.18
SW3	18	425	763	500	0.65	0.560	0.16
Ave.	-	425	763	511	0.67	0.56	0.18

Wall designation : FW = Full-scale walls

SW = one-third scale walls

TABLE 3.9 - RESULTS OF TESTS ON ONE-THIRD AND FULL-SCALE WALLS BUILT IN

"LYTAG" BLOCKS AND 1:1:6 MORTAR

Wall No.	Age at Test (days)	Mortar strength lbf/in ²	Block strength lbf/in ²	Wall strength lbf/in ²	Strength Ratio Wall/block	Secant Modulus of Elasticity at $\frac{1}{4}$ of ultimate load (lbf/in ²)x10 ⁶	Poisson's Ratio at $\frac{1}{4}$ of ultimate load
SW1	15	416	1674	1013	0.60	0.93	0.18
SW2	17	416	1674	904	0.54	0.87	0.19
SW3	16	456	1674	957	0.57	1.00	0.20
Ave.	-	430	1674	958	0.57	0.93	0.19
FW1	15	456	1608	908	0.57	1.02	0.21
FW2	21	438	1608	872	0.54	1.01	0.17
FW3	17	529	1608	921	0.57	1.10	0.21
Ave.	-	474	1608	900	0.56	1.04	0.20

Wall designation : SW = one-third scale walls

FW = full-scale walls

BUILT IN AGLITE BLOCKS AND 1:1:6 MORTAR

Wall No.	Age at Test (days)	Mortar strength (lbf/in ²)	Block strength (lbf/in ²)	Wall ultimate strength (lbf/in ²)	Strength Ratio (Wall/block)	Secant Modulus of Elasticity at $\frac{1}{4}$ of ultimate load (lbf/in ²)x10 ⁶	Poisson's Ratio at $\frac{1}{4}$ of ultimate load
SW1	20	377	2161	1167	0.54	1.05	0.24
SW2	14	361	2161	1109	0.51	1.13	0.27
SW3	14	405	2161	1365	0.63	1.20	0.20
Ave.	-	381	2161	1214	0.56	1.13	0.24
FW1	15	427	2717	1366	0.50	1.25	.20
FW2	16	427	2717	1350	0.50	1.20	.20
FW3	17	427	2717	1390	0.51	1.10	.20
Ave.	-	427	2717	1368	.53	1.18	.20

Wall Designation : SW = one-third scale walls
 FW = full scale wall

TABLE 3.11 - RESULTS OF TESTS ON ONE-THIRD SCALE WALLS BUILT IN
"AGLITE" BLOCKS AND 1:1:6 MORTAR

Wall No.	Age at test (days)	Mortar strength (lbf/in ²)	Block strength (lbf/in ²)	Wall strength (lbf/in ²)	Strength Ratio (wall/block)	Secant Modulus of Elasticity at $\frac{1}{4}$ of ultimate load (lbf/in ²) $\times 10^6$	Poisson's Ratio at $\frac{1}{4}$ of ultimate load
1	22	395	3931	1689	.43	1.30	.23
2	30	395	3931	1670	.43	1.25	.21
3	35	395	3931	1722	.44	1.17	.19
Ave.	-	395	3931	1694	.43	1.24	.21

Wall Designation : SW = one-third scale walls

FW = full scale walls

TABLE 3.12 - RESULTS OF TESTS ON FULL AND ONE-THIRD SCALE DENSE CONCRETE

BLOCKWORK BUILT IN 1:1:6 MORTAR Ref. (47)

Wall No.	Age at Test (days)	Mortar strength (lbf/in ²)	Block Strength (lbf/in ²)	Wall strength (lbf/in ²)	Strength ratio (wall/block)	Secant Modulus of Elasticity at $\frac{1}{4}$ of ultimate load (lbf/in ²)x10 ⁶	Poisson's Ratio at $\frac{1}{4}$ of ultimate load
FW1	31	546	2142	1659	.77	1.43	-
FW2	21	605	2142	1556	.73	1.30	0.12
FW3	14	338	2142	1348	.63	0.95	0.11
Ave.	-	496	2142	1521	.71	1.23	0.12
SW1	30	728	2502	1307	0.52	0.73	0.31
SW2	30	728	2502	1633	0.65	1.27	0.20
SW3	21	600	2502	1447	0.58	0.93	0.14
Ave.	-	685	2502	1462	.58	.98	.21

Wall Designation : FW = full scale walls
SW = one-third scale walls

TABLE 3.13 - COMPARISON OF THE CALCULATED AND EXPERIMENTAL
VALUES OF BLOCKWORK ULTIMATE STRENGTH

Wall Designation	Block strength BS 2028 (lbf/in ²)	Ave. Mortar Strength (lbf/in ²)	Blockwork Strength	
			Experimental lbf/in ²	Calculated lbf/in ²
$\frac{1}{3}$ -scale Leca	763	420	511	556
Full-scale Leca	733	420	526	541
$\frac{1}{3}$ -scale Lytag	1674	420	958	948
Full-scale Lytag	1608	420	900	923
$\frac{1}{3}$ -scale Aglite	2717	420	1368	1310
$\frac{1}{3}$ -scale Aglite	3931	420	1694	1676
Full-scale Aglite	2161	420	1214	1124
$\frac{1}{3}$ *-scale Dense	2502	637	1462	1427
*Full-scale Dense	2140	637	1521	1287
B.R.S. Wall No.3	1560	900	1030	1169
B.R.S. Wall No.4	2170	850	1530	1428
B.R.S. Wall No.5	2130	1040	1670	1509

* Tests conducted by IBRAHIM (51)

All results are average of three tests except the B.R.S. Walls⁽¹⁷⁾ (one test).

The calculated values of wall ultimate strength in this investigation are based on an average mortar strength of 420 lbf/in²

the difference in the height of the blocks. Full size blocks were $8\frac{5}{8}$ inches high while one-third blocks were 3 inches high. The slight difference in the results could be partially due to the difference in the slenderness ratio; however, more data is necessary and work on this aspect (on light-weight concrete blockwork) is currently in progress in the department and more data will be available in the near future.

3.8.10 Effect of Eccentricity

The strength of one-third scale "Thermalite" wall No. 2 was 12% and 14% less than similar walls No. 1 and 3. Prior to the test, this wall was checked for plumbness and it was observed to be about $\frac{1}{4}$ inch out of plumb. Under load it failed by buckling. "Aglite" full scale walls No. 1 and 2 were found to be off plumb by $\frac{1}{4}$ and $\frac{3}{8}$ inches respectively. These eccentricities resulted a lower ultimate strength and the walls were about 14.5% and 21% weaker than similar wall No. 3.

Comparison of the stress reduction factors for axially and eccentrically loaded walls given in the British Standard Code of Practice CP 111 1970 ⁽¹⁵⁾ shows stress reduction factor for axially loaded wall having slenderness ratio of 19.4 to be 0.632, the same factor for $\frac{1}{4}$ inches eccentricity i.e. (1/6th of the wall thickness) is 0.525 i.e. 17% reduction in strength when there is $\frac{1}{4}$ inch eccentricity. Similarly reduction in strength for $\frac{3}{8}$ inch eccentricity of load ($\frac{3}{32}$ of the wall thickness) would be 28% suggesting that the actual reduction is somewhat less than allowable value provided for by the Code.

3.8.11 Mode of Failure

The mode/

The mode of failure in axially loaded blockwork is determined principally by the strength and deformation characteristics of the two constituents materials, viz block and mortar acting compositely. The behaviour of blockwork under axial compression is best discussed by reference to a short stack-bonded blockwork. This eliminates the need to consider the secondary influence of mortar perpends and effect of slenderness ratio on the stress distribution in blockwork.

Fig. 3.26a shows a blockwork prism acted upon by an axial compressive load. In addition to a vertical compressive stress, lateral stresses are induced in the blocks material and in the mortar joint owing to the different strain characteristic in the two materials. Where the lateral deformation of the mortar joint under load is greater than that of the block material, the mortar joint tends to expand laterally more than the block and as the results imparts a bilateral tensile stress in the block which in reaction a bilateral compressive stress is induced in the mortar joints. Hence, the state of stress in a block unit in blockwork under axial compression is a combination of vertical compression and a bilateral tension, as indicated in Fig.(3.26b). The corresponding state of stress in a horizontal mortar joint is one of triaxial compression, consisting of vertical compression and bilateral compression, as indicated in fig. (3.26c).

The state of stress in these two elements, block and mortar joints, are reversed if the lateral deformation in the mortar joint is less than that of the block material. In a unique and ideal instance where the lateral deformation in these two materials under load is identical, then the bilateral stress vanish and the composite blockwork under/

blockwork under axial compression behaves as a homogeneous material.

An elastic analysis of lateral stress distribution in axially loaded brickwork was undertaken by Stafford Smith and Carter⁽¹⁰⁰⁾.

Using the method of finite element analysis for a plane stress condition, they analysed the lateral stress distribution within an axially loaded brick wall of various height width ratios.

Fig. (3.27) shows the lateral stress distribution in a brick wall of $h/w = 2.4$. They found that the maximum tensile stresses occurred in the vertical mortar joints, which initiated vertical cracking in the wall through these joints. The greater the height:width ratio of a wall, the larger the lateral stresses. This is because Smith's and Carter's analysis of the lateral stress distribution included the influence of the load platen restraint. However, in view of the existence of stress concentration in the brick and mortar element in axially-loaded brickwork, it is doubtful whether a finite element analysis applied to a complete wall for the determination of the lateral stress distribution, as undertaken by Smith is of much quantitative use. An elastic analysis of the lateral stress distribution in axially-loaded brickwork was undertaken by Khoo⁽⁵⁵⁾.

In this investigation when the brick is stronger than the mortar which tends to yield laterally more than the brick under load, it was shown that the bilateral stresses which act along the interfaces of the brick and mortar are concentrated sharply at the edges of the interfaces producing high lateral tensile stresses in the brick units in these regions./

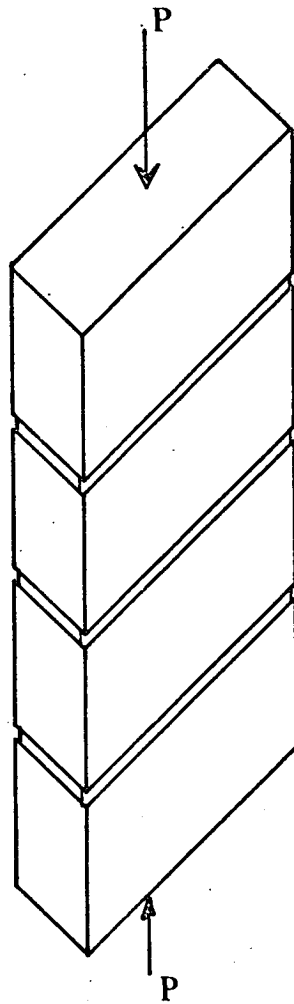


Fig. 3.26 a Concrete Blockwork Prism under Axial Load.

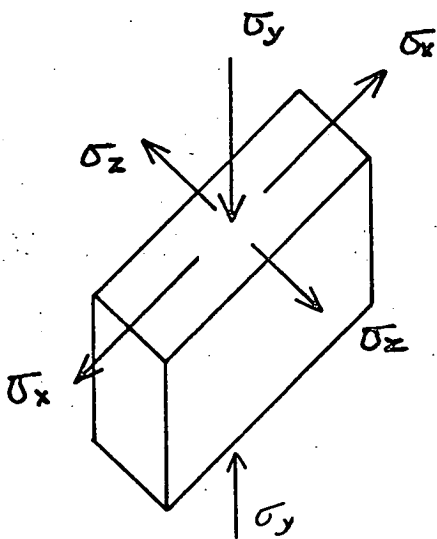


Fig. 3.26 b Block Element under Vertical Compression & Bi-Lateral Tension.

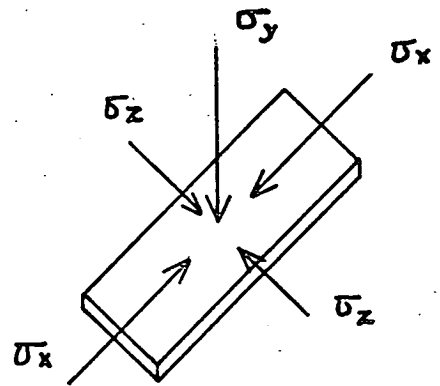
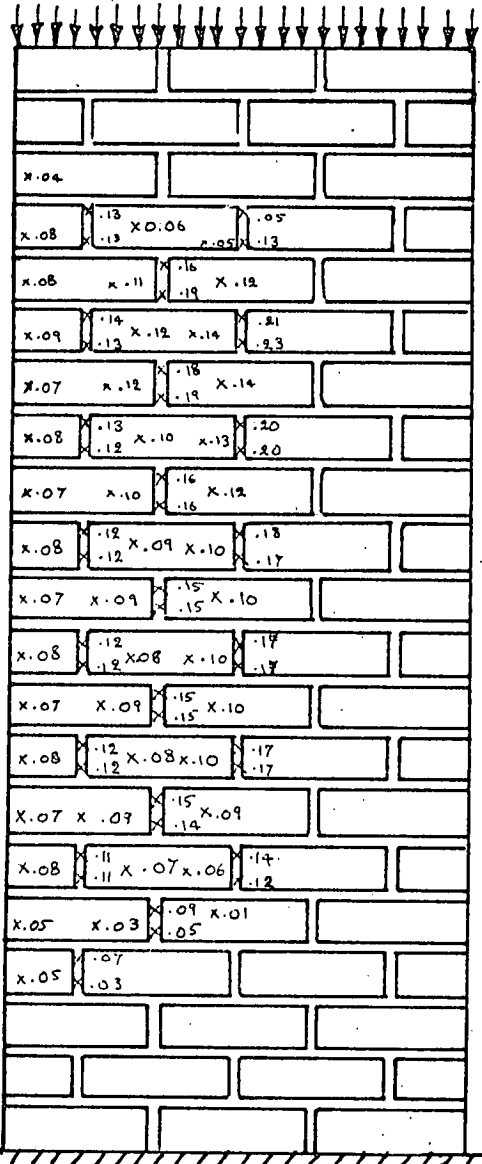


Fig. 3,26 c Mortar Joint under Triaxial Compression.

LOAD 400 lbf/ft.



WALL 4" THICK
 $E_B/E_M = 1.5$
 $\nu = 0.18$

SPOT VALUES OF TENSILE STRESS IN lbf/in²

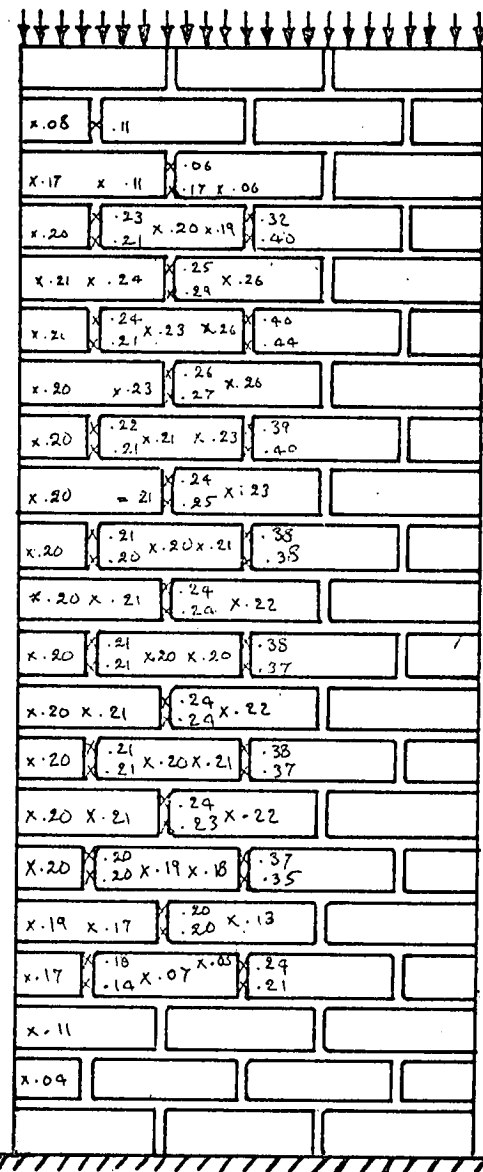
SPOT VALUES OF TENSILE STRESS IN lbf/in²

AVERAGE VALUES OF VERT. COMP. STRESS = 8.3 lbf/in²
 MAX. VALUE = 9.2 lbf/in² IN BOTTOM CORNERS

$R/L = 2.4$ $E_B/E_M = 1.5$ horizontal principal tensile stresses.

FIG. 3.27

LOAD 400 lbf/ft



WALL 4" THICK
 $E_B/E_M = 6.0$
 $\nu = 0.18$

AVERAGE VALUE OF VERT. COMP. STRESS = 8.3 lbf/in²
 MAX. VALUE = 9.1 lbf/in² IN BOTTOM CORNERS

$R/L = 2.4$ $E_B/E_M = 6.0$ horizontal principal tensile stresses.

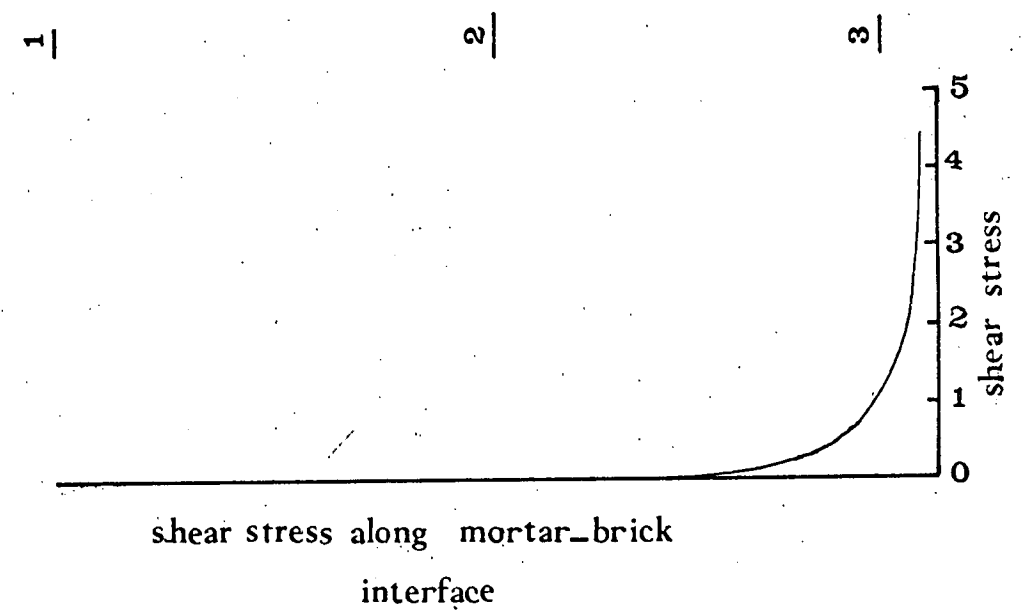
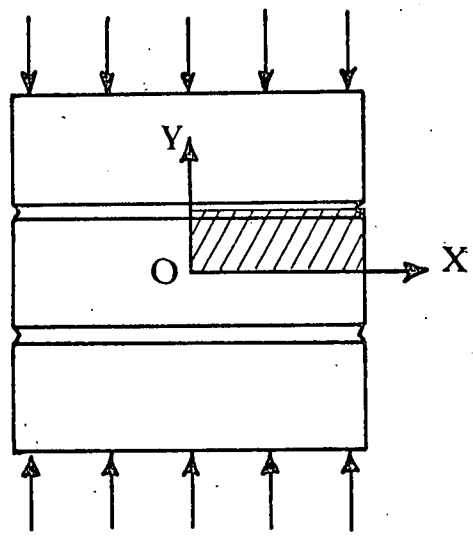
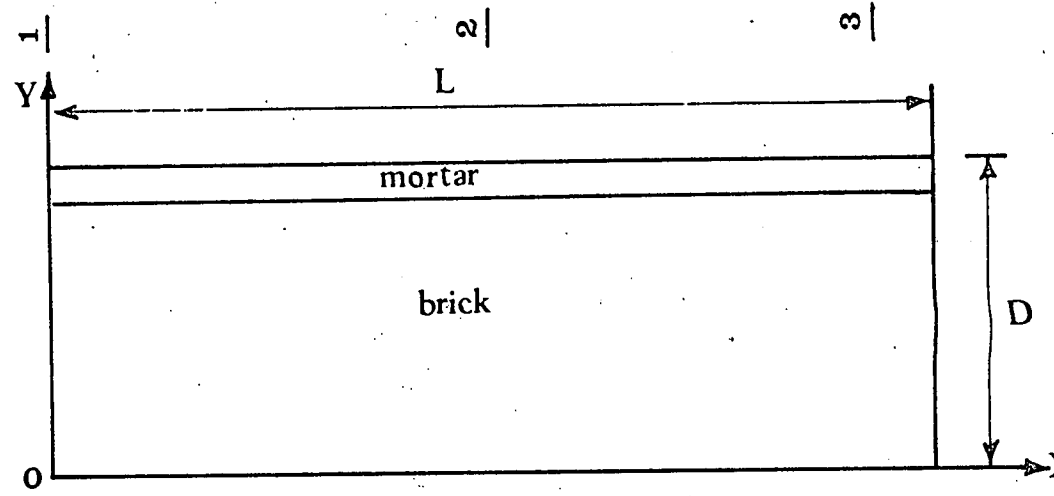
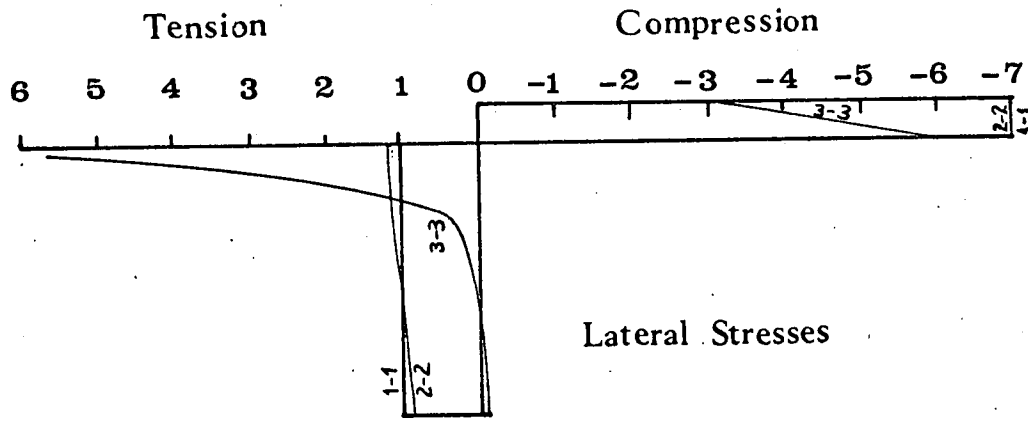


Fig 3.28 Stress Distribution in Brickwork under Compression $L/D = 2.9$

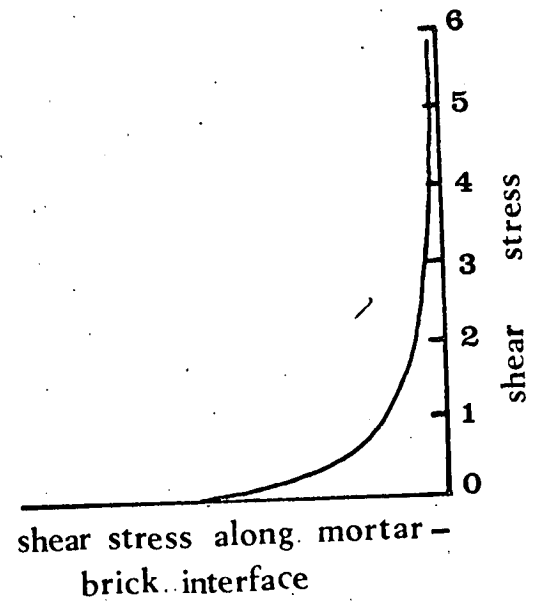
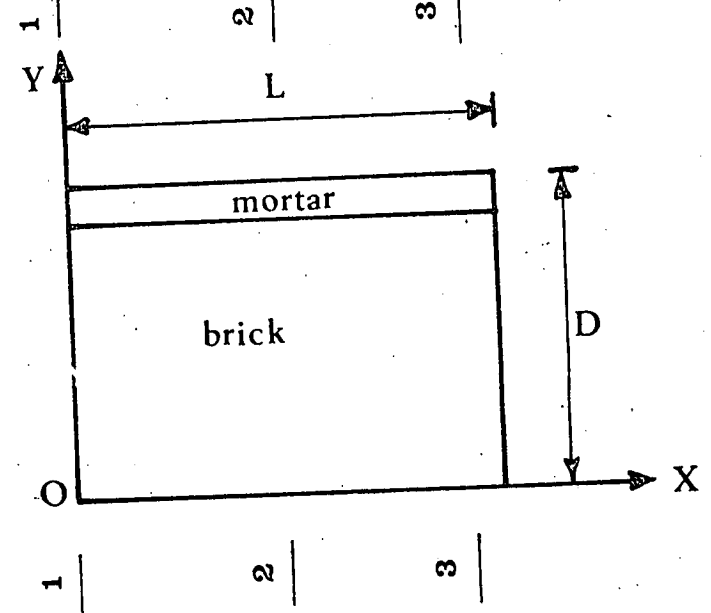
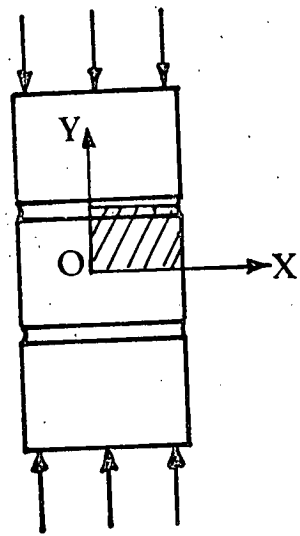
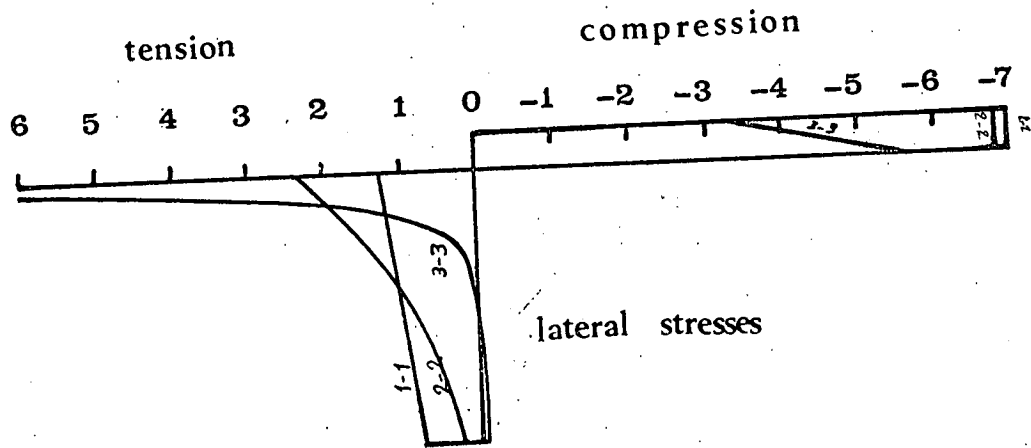


Fig.3,29 Stress Distribution in Brickwork under Compression.

$$L/D = 1.4$$

these regions. The stresses being more critical in an end-face section (see Figs. 3.28 to 3.29) than in a front-face section of the brickwork, although it is accepted that in reality, brick and mortar being inelastic materials to some degrees, considerable stress redistribution takes place approaching ultimate stress. Failure occurs in the brickwork when the combined compression and bilateral tension in the brick reaches a critical value characteristic of the material and the mode of failure is that of vertical tensile splitting in the brick across a plane situated near the edge of the end-face of the brick in the direction of its length. This mode of failure has often been described as spalling failure in brickwork.

A blockwork prism built with strong block material and weak mortar will fail in a similar manner as that described above for brickwork. The mode of failure for this category of blockwork is seen in Plate (3.8), which shows a typical vertical tensile splitting in the block material at the end-face in one-third scale blockwork prisms. These particular prisms were built with "Aglite" blocks of crushing strength 1430 lbf/in^2 bonded with mortar mix of cube strength 400 lbf/in^2 . All ten specimens at ultimate load exhibited same type of failure. However, unlike brickwork where the strength of brick is usually higher than that of mortar, blockwork may sometimes consist of a weak block material and a strong mortar. In this event, it was pointed out earlier that the mortar exerts a bilateral compressive restraint on the block unit, and the block unit is therefore in a state of tri-axial compression. The mode of failure in blockwork in this case is that of shear failure across an inclined plane in the block unit, and an/

and an example of this type of failure is seen in Plate (3.7) which shows one-third scale blockwork prisms built with a block material of crushing strength 1430 lbf/in^2 and a mortar of cube strength 2140 lbf/in^2 .

Further examples of the mode of failure in axially-loaded blockwork may be seen in Plates (3.9 to 3.13) which show the crushing of full scale and one-third scale blockwork walls.

In other crushing tests of blockwork wall, the mode of failure is not so discernable, being often complicated by the presence of mortar perpendes.

3.8.12 Load Factors

For axially loaded walls the average load factor (based on the Code of Practice CP 111 (1964) with 1970 Amendment) ranged from 5.6 to 6.6 for full-scale walls, and from 6.4 to 7.7 for one-third scale walls with the exception of "Thermalite" walls, Table (3.13).

The values were calculated for:

- a) Slenderness Ratio of 18.5 for full-scale and 19.4 for one-third scale walls based on $\frac{3}{4}$ of the actual wall height.
- b) Reduction factor for slenderness ratio 0.654 for full-scale and 0.635 for one-third scale walls.
- c) Stress reduction factor for area effect being 0.858 for full-scale and 0.762 for one-third scale walls.
- d) Modification factor for the shape of the units being 2.0. Load factors have also been calculated according to the Draft Revision of the Code of Practice CP 111, 1970. Average values ranging from 4.8 to 5.8 for full-scale and from 5.1 to 5.7 for one-third scale walls.



Plate 3.7 SHEAR FAILURE IN BLOCKWORK
PRISM

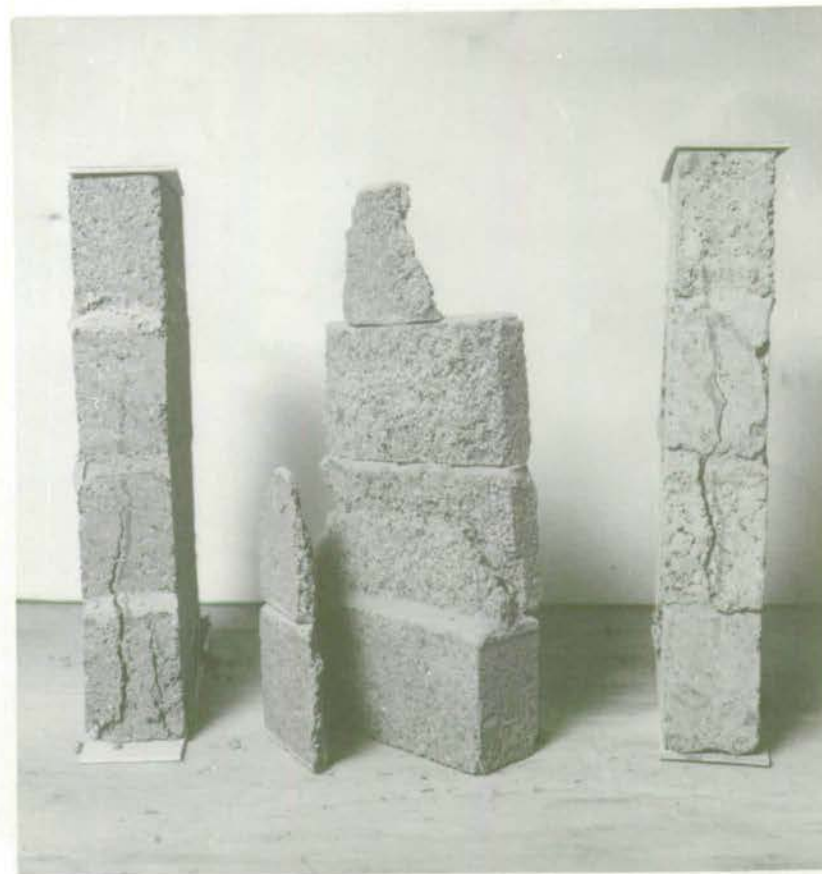


Plate 3.8 VERTICAL TENSILE SPLITTING
IN BLOCKWORK PRISM

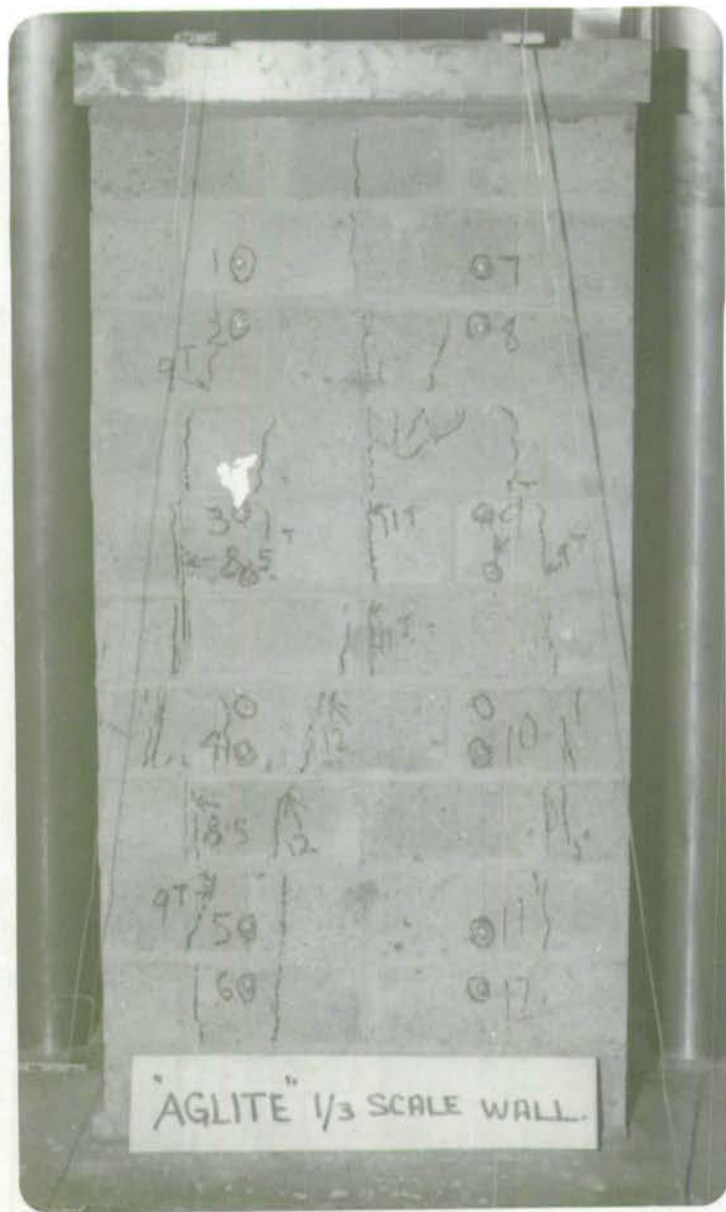


Plate 3.9 a

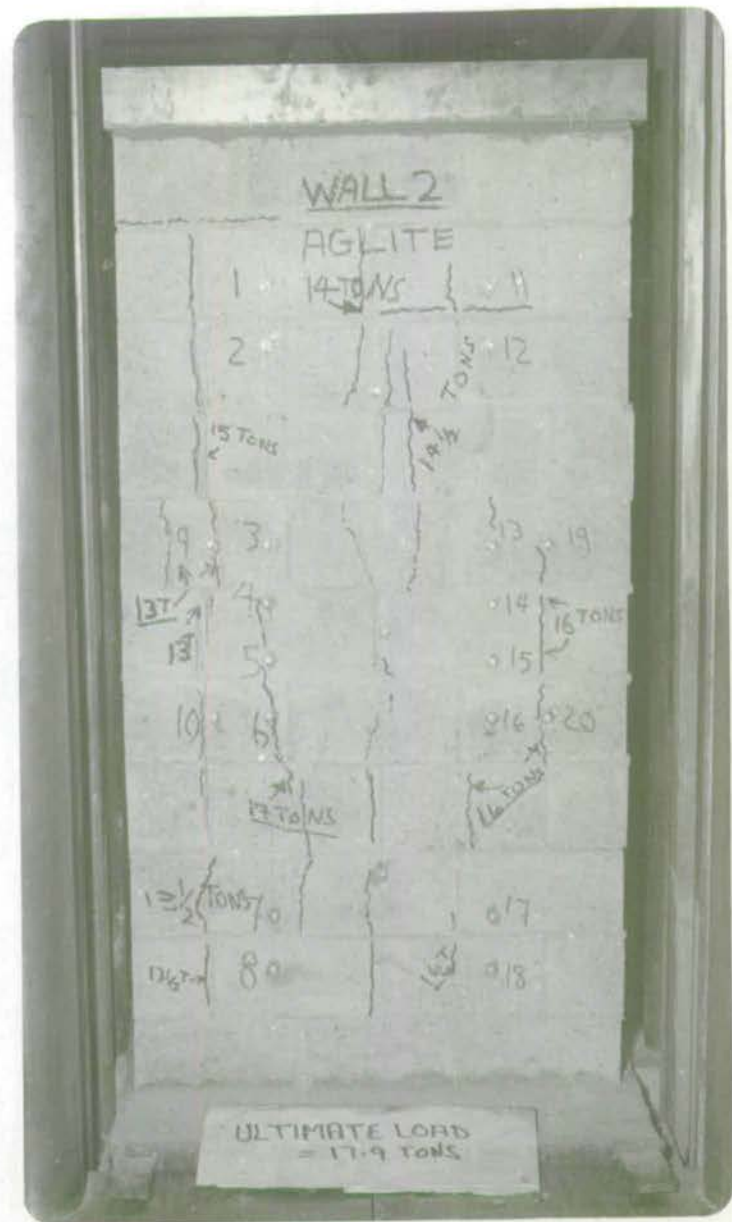


Plate 3.9 b

TENSILE SPLITTING FAILURE IN ONE-THIRD SCALE "AGLITE" BLOCKWORK

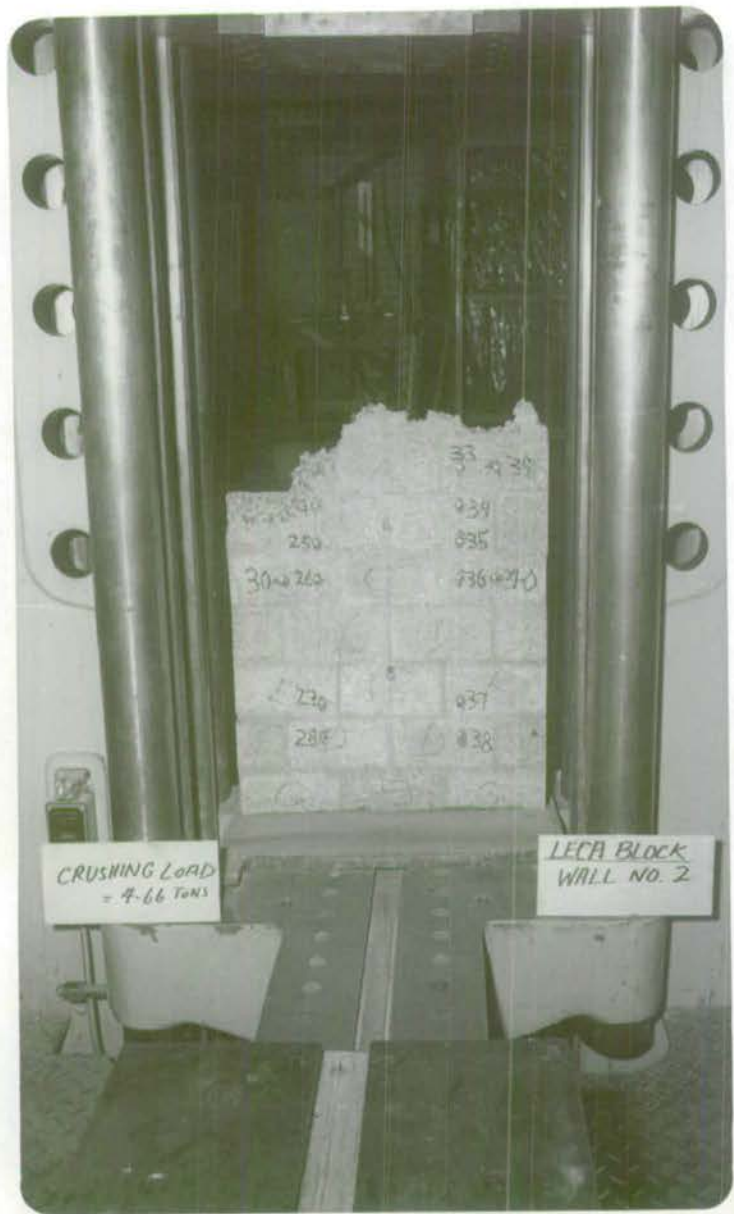


Plate 3.10 a



Plate 3.10 b

SHEAR FAILURE IN ONE-THIRD SCALE "LECA" BLOCKWORK

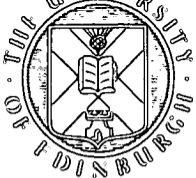


Plate 3.11 a



Plate 3.11 b

TENSILE SPLITTING FAILURE IN FULL-SCALE "LYTAG" BLOCKWORK



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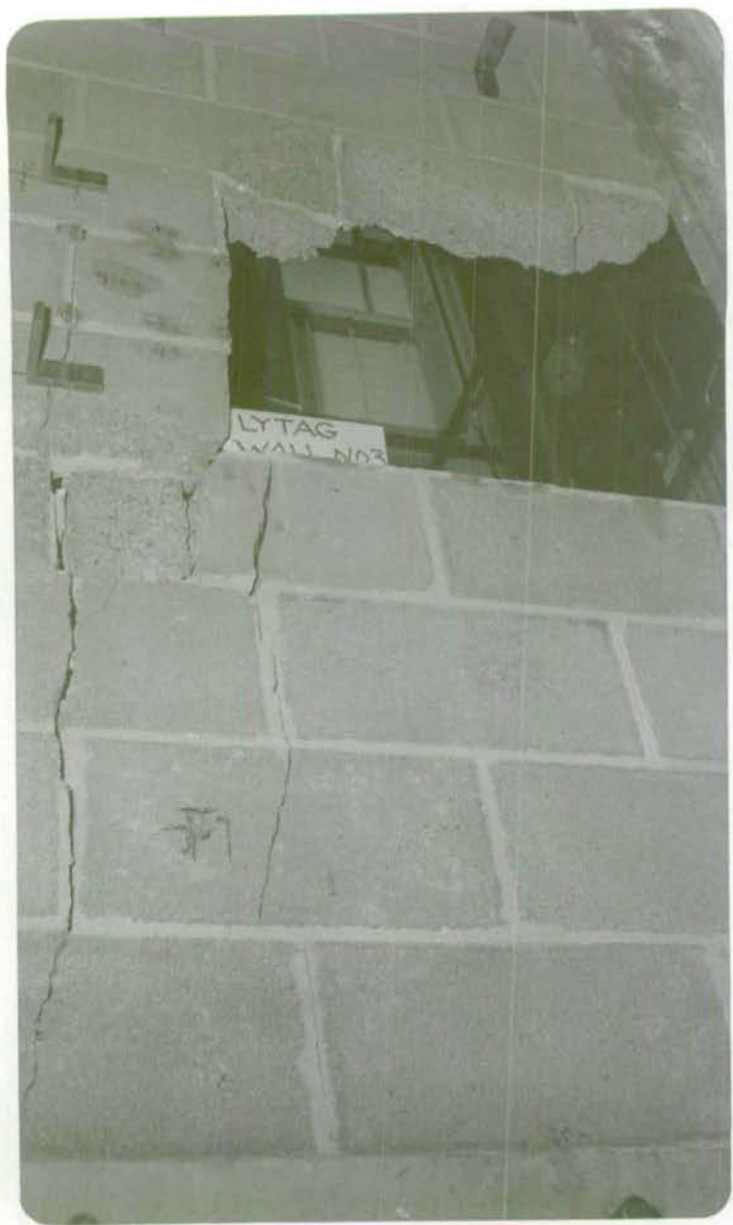


Plate 3.12 a



Plate 3.12 b

TENSILE SPLITTING FAILURE IN FULL-SCALE "LYTAG" BLOCKWORK



Plate 3.13 TENSILE SPLITTING FAILURE IN FULL SCALE
"AGLITE" BLOCKWORK

They were calculated for the same slenderness ratio using Table (3.3) of the Draft Code, giving Basic Compressive stresses for members built with solid blocks of height/thickness ratio between 2.0 and 4.0.

Table (3.14) gives the load factors for all walls tested. The low value of load factor for the "Thermalite" wall No. 2 is due to the eccentricity of load and for the remaining other two "Thermalite" walls may be due to the fact that the crushing strength of the material was relatively low, and the effect of slenderness is perhaps more severe in walls built with weak components. Mortar used in construction of the walls was also of low strength and the average mortar crushing strength for one-third scale "Thermalite" walls was 356 lbf/sq.in. and 11% less than the minimum recommended value by the Draft Revision Code.

The allowable value for the load factor recommended by the Draft Code of Practice is 6.7, for most walls tested, these values were less.

This could be due to the following factors:

1. Rate of loading - Rate of loading as recommended by the draft Code is being 101.5 lbs/sq.in/min. in the present investigation loads had to be held constant at intervals for strain measurement, and the increments were at the rate of 45 lbs./sq.in/min. The influence of rate of loading in the ultimate strength of brickwork walls was observed by Bradshaw⁽⁹⁾. He concluded that the ultimate strength of walls tested over a period of 1½ hours is about 20% less than walls loaded over half-hour.
2. Method of curing, Draft Code recommends the walls should be covered by polythene for 3 days after construction.

3./

TABLE 3.14 LOAD FACTORS

Wall Type	Load Factor CP.111 1970		Load Factor Revised Code	
	Full Scale	1/3 Scale	Full Scale	1/3 Scale
Thermalite Ave.	5.0	5.4	3.8	3.7
	5.2	4.6	4.0	3.2
	5.1	5.3	3.8	3.6
	5.1	5.1	3.9	3.5
Leca Ave.	6.3	6.9	5.2	5.0
	6.3	7.1	5.2	5.2
	6.5	6.8	5.3	5.0
	6.4	6.9	5.25	5.1
Lyttag Ave.	6.3	6.4	4.9	5.5
	5.6	6.2	4.7	4.9
	5.9	6.6	4.9	5.2
	5.6	6.4	4.8	5.2
Aglite Ave.	6.4	(1) 7.8	5.6	5.7
	6.1	7.6	5.3	5.6
	7.5	7.7	6.5	5.8
	6.6	7.7	5.8	5.7
Aglite Ave.	-	(2) 7.5	-	5.8
	-	7.3	-	5.6
	-	7.6	-	5.8
	-	7.5	-	5.7

(1) Blocks with 2717 lbf/in² crushing strength

(2) Blocks with 3930 lbf/in² crushing strength

Ave. mortar strength 2140 lbf/in²

3. Age at test. Minimum of 20 days specified in the Draft Revision Code.
4. Mortar strength not less than 580 lbf/in^2 for 1:1:6 Cement:lime:sand mortar as recommended in the Draft Code.
5. Workmanship.

A load factor of about 5.0, however would seem to be adequate for all design purposes.

3.9 CONCLUSIONS

From the foregoing discussion, the following conclusions may be drawn:

- (1) The strength of full-scale blockwork for a given strength of block, and mortar can be reproduced by means of model tests, provided:
 - (a) Mortar joints are scaled down.
 - (b) Sand used in mortar for construction of Model Walls is sieved and coarse fraction is discarded.
- (2) The appreciable difference in the values of load factors obtained from CP 111 (1970), and the new Draft Code is mainly due to the area effect. The new Draft Code does not allow for a reduction in stress for wall areas over 194 sq. in. or $.125 \text{ m}^2$, whereas in CP 111 (1970) the limiting values for area is 500 sq. in. ($.4 \text{ m}^2$).
- (3) CP 111 (1970) indicates the degree of mortar strength by the proportion of Cement:lime:sand (by volume).
It would seem more appropriate to indicate a minimum cube strength for each grade of mortar mix as is recommended in the Draft Revision Code.

- (4) The relationship between wall strength and block strength may be assumed non-linear for the test's range. Formula given in the section (3.8.4) gives a close estimate of wall ultimate strength for S/m ratio greater than 1.75 and 1:1:6 mortar.
- (5) The values of Poisson's Ratio for all one-third scale walls were consistent and varied from .18 to .21. The variation for full scale walls ranged from .19 to .24.
- (6) The average secant modulus of elasticity at $\frac{1}{4}$ of ultimate load for all walls varied from $.244 \times 10^6$ (for thermalite walls) to 1.24×10^6 (for high strength Aglite walls) indicating that strong walls possess a higher Modulus of Elasticity.
- (7) Mode of Failure was by tensile splitting or shear failure depending on the relative strength of block and mortar used in the construction of the walls.
- (8) Stress strain relationship for all walls tested were reasonably linear up to $\frac{1}{4}$ of U.L.
- (9) Poor workmanship may introduce accidental eccentricity in the walls which would reduce the ultimate strength of the blockwork.
- (10) Loadbearing lightweight aggregate concrete blocks are adequate for construction of high rise structures for residential purposes.

CHAPTER 4 - SHEAR TESTS ON SINGLE STOREY CROSS WALL STRUCTURES
WITH OPENINGS IN BLOCKWORK

4.1. INTRODUCTION

Following a series of comparative tests on the strength and structural behaviour of one-third and full-scale lightweight concrete blockwork, and the development of model prototype relationships, an investigation has been carried out on the structural behaviour of single storey shear-wall structures with openings built in one-third scale "Aglite"⁽⁶⁰⁾ lightweight concrete blocks, under lateral and precompressive loads. The programme included tests on seven one-third scale storey height model blockwork shear-wall structures. Precompression varied from one test to another covering the range of 50 - 250 lbf/in². The primary object of these tests was to investigate the shear strength of a single storey shear-wall structure with door openings under combined shear and precompressive loads.

In conjunction with these tests some exploratory investigations were carried out on one-third scale blockwork couplets and triplets. The bond strength of block and mortar was found by testing one-half block couplets in pure tension, and the relationship between bond shear and normal compressive stress was established. Due to application of shear force along the mortar joint no shearing stresses were developed in the blocks, and hence the failure in triplets always occurred along the mortar joint. These tests, however, did not represent closely the effect of lateral loads on shear walls but they gave some indication of ultimate shear strength of the masonry.

Work presented in this chapter is divided into two separate parts:

1. a) Direct tensile testing of blockwork couplets.
 - b) Testing of blockwork triplets under combined precompression and axial loading.
2. An experimental investigation on the structural behaviour of one-third scale single-storey shear-wall structure with openings in one-third scale "Aglite" blocks subjected to precompression and lateral loading.

PART ONE

4.2. INVESTIGATION OF BOND TENSION, BOND SHEAR AND THE EFFECT OF PRECOMPRESSION ON THE SHEAR STRENGTH OF ONE-THIRD SCALE BLOCKWORK ASSEMBLIES.

Failure in blockwork subjected to in plane lateral forces normally occurs in tension or shear along the mortar joint. This is due to the fact that the point of lowest tensile or shear strength in masonry wall is usually the bond between mortar and the unit. For the design of masonry blockwork, therefore, it is essential to consider the bond strength between block and mortar. In general development of a stronger joint will improve the structural integrity and performance of masonry which in turn should simulate progress towards more economical masonry design.

4.3. MATERIALS

4.3.1 Blocks

One-third scale "Aglite" blocks, 6 x 3 x 2 inches thick were used. The mean crushing strength of these blocks according to BS 2028, 1364, 1968⁽¹⁰⁾, was 1436 lbf/in² and their density was 76.1lb/cu.ft.

4.3.2 Sand

Ordinary building/

Ordinary building sand was used in construction of all assemblies, conforming with table (1) BS 1200, 1055⁽¹⁴⁾. The grading of the sand is given in table (3.1).

4.3.3 Cement

The cement used was "Ferrocrete"⁽¹¹⁾ a rapid hardening Portland cement giving an early strength.

4.3.4 Lime

Hydrated lime, class 'A' was used to conform with BS 890⁽¹³⁾.

4.3.5 Mortar

The mortar was 1: $\frac{1}{4}$:3 cement:lime:sand (by volume) and all proportioning was done by weight. Initially trial mixes were made and water/cement ratio was adjusted for the mix to have a workable consistency, this ratio was about 2.0. The same water/cement ratio was used throughout the model construction, except that the mason was allowed to use an additional amount of water when necessary. Six 3-inches mortar cubes were made from three different batches of mortar and were cured in the water tank until they were tested with the model. The mean compressive strength of these cubes is given in Table (4.2).

a). 4.4. BOND TENSION TEST

4.4.1 Test Specimen

This consisted of two half-blocks (Plate 4.1) 3 x 3 x 2 inches which were cemented together by mortar of $\frac{1}{8}$ inches bed joint thickness. The specimens were cured in the laboratory atmosphere and prepared for test after 21 days.

4.4.2 Test Arrangement

Plate (4.2) shows the test arrangement. It consists of a pair of 3 x 2 x 1 inch steel blocks glued to the ends of the couplet by a quick setting/

quick setting epoxy resin commercially known as Cataloy paste.

A template and a pair of clamps were provided to keep the two steel blocks in a correct position when glueing them to the specimen, so that the bars attached to the end blocks are in the same axis, thus ensuring an axial loading. The bars are then attached to the Instron testing machine through a universal joint allowing a uniform distribution of the tensile load. Results of these tests are given in table (4.1).

Table (4.1)

Results of Bond Tension Tests

Specimen No.	Area of blocks sq. in.	Tensile bond strength lbf/in ²
1	6.00	49.0
2	5.89	61.9
3	6.00	63.5
4	6.00	52.5
5	5.90	54.1
6	5.92	61.8
7	5.95	53.2
8	6.00	60.5
9	6.05	58.2
10	6.04	57.1
11	6.05	56.0
12	5.95	56.1
Mean:		57.0

Coefficient of variation : 7.22%

Mortar strength : 2150 lbf/in²

Block strength : 1436 lbf/in²

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b). 4.5 BOND SHEAR TESTS

Three blocks (triplets) assemblies fig. (4.1) were tested in order to determine the shear strength of the blockwork. Materials used in construction of these assemblies were similar to those used in construction of couplets for bond tension. Triplets were tested in the Instron testing machine where they were subjected only to pure shear along the mortar bed joints. The results are shown in Fig. (4.2). The failure occurred at the contact area between the mortar and the concrete block.

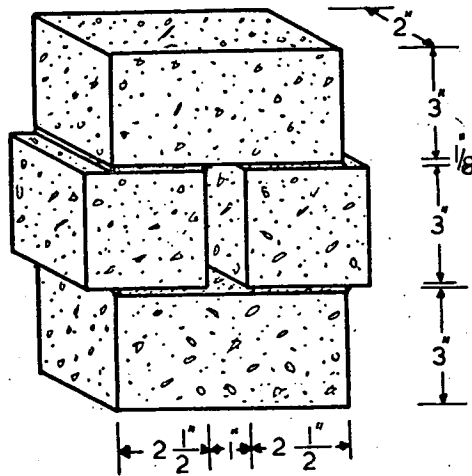


Fig. 4.1 BLOCKWORK TRIPLET

4.6 RELATIONSHIP BETWEEN BOND SHEAR AND BOND TENSION IN BLOCKWORK

For the particular block and mortar used in these tests, the following relation between bond shear and bond tension may be established.

$\tau_o = K \tau_{bt}$ (4.1)

where: τ_o = bond shear.

τ_{bt} = bond tension.

K = characteristic constant for the mortar and the block type used.

The value/

The value of 'K' in this investigation is found statistically to be equal to 1.27. Murthy⁽⁷⁴⁾ suggested the value of $K = 2.3$ for a particular brick and mortar. Polyakov⁽⁹⁾ suggested that the ratio of bond shear to bond tension depends on the bond tension as given by the equation

$$\frac{\tau_o}{\tau_{bt}} = 2.25 - 0.5 \tau_{bt} \dots\dots\dots(4.2)$$

where :

$$\tau_{bt} \leq 35.6 \text{ lbf/in}^2$$

Following Murthy's work, Sinha⁽⁹⁾ suggested a general relationship for brickwork given as:

$$\tau_o = 8.8 \tau_{bt}^{0.5}$$

The above relationship agrees reasonably with the test results on blockwork.

4.7 EFFECT OF PRECOMPRESSION ON SHEAR STRENGTH OF BLOCKWORK

A series of tests were conducted on blockwork triplets in order to establish the relationship between shear strength and the applied precompression.

Earlier work^(74,96) on shear strength of brick and block masonry couplets indicated that, the shear strength of brickwork couplets consists of bond shear and frictional resistance as represented by the equation

$$\tau = \tau_o + \mu \bar{\sigma}_n \dots\dots\dots(4.3)$$

where τ = ultimate shear strength of masonry assembly

τ_o = bond shear at zero precompression

μ = coefficient of friction at assemblies interface

and $\bar{\sigma}_n$ = precompression normal to the shearing interface.

In the/

In the absence of bond in the block and mortar the ultimate shear depends on frictional resistance and can be represented by:

$$\tau = \mu \sigma_n \dots\dots\dots(4.4)$$

4.7.1 Shear Strength of Blockwork Triplets Due to Friction

And Bond

To predict the ultimate shear strength of triplet the value of bond shear at zero compressive stress was first determined. Tests were also conducted to find the effect of the varying precompression on shear strength. Tests were conducted in the Avery which provided the precompressive load. A reaction frame together with a 3-ton hydraulic jack operated by hand was used to apply the lateral load Plate (4.3). This load was measured by a 10,000 lbs proving ring, connected to the reaction frame through the 3-ton jack. A load cell and a digital voltmeter were used to measure the precompression. The compressive load was applied prior to the application of the shear load. Results are shown in Fig (4.2).

4.7.2 Shear Strength of Blockwork Triplets Due to Friction Only

These series of tests were conducted to establish an approximate value for the coefficient of friction between block and mortar interface. Knowing the initial value of bond shear without precompression and the coefficient of friction, the strength of blockwork triplet in shear can then be predicted. Tests were done as follows: after the completion of tests described in the section 4.7.1, loads were released. The pieces were reassembled together and tested again, this time without the bond between block and mortar interface. The shear strength obtained from these tests were assumed to be solely due to the frictional resistance. Results are shown in Fig. (4.2).

The coefficient/

The coefficient of friction for the particular block and mortar used was statistically found to be 0.73.

4.8 DISCUSSION OF THE RESULTS

The relationship between the compressive stress and the shear strength of the triplets was found to be linear up to a precompressive stress of 200 lbf/in². Further increase in precompression did not increase the shear strength of masonry and practically it remained constant between 200 - 300 lbf/in² precompression. However, there was an apparent increase in the shear strength once the precompression exceeded the limit of 300 lbf/in² (this is explained in detail in the second part of this chapter). Above this value of precompression the shear strength may be calculated from the frictional resistance of the triplet using the formula:

$$\tau = \mu \sigma_n$$

and assuming the value of 0.73 for the coefficient of friction.

The value of the initial bond shear at zero precompression obtained in this investigation (73 lbf/in²) is close to that obtained by Copeland and Saxer⁽¹¹⁰⁾ (69 lbf/in²) for sand and gravel concrete, and mortar of 2600 lbf/in² crushing strength. The slight variation is probably due to the strength of the different mortar and the nature of the aggregates in the blocks.

4.9 CONCLUSIONS

1. The tensile bond strength between the mortar and the block used in these investigations is 0.78 times their initial bond shear.
2. The shear strength of blockwork masonry triplet due to friction is proportional to the applied precompression.
3. The value of coefficient of friction for the block and mortar used in these investigations was approximately equal to 0.73.

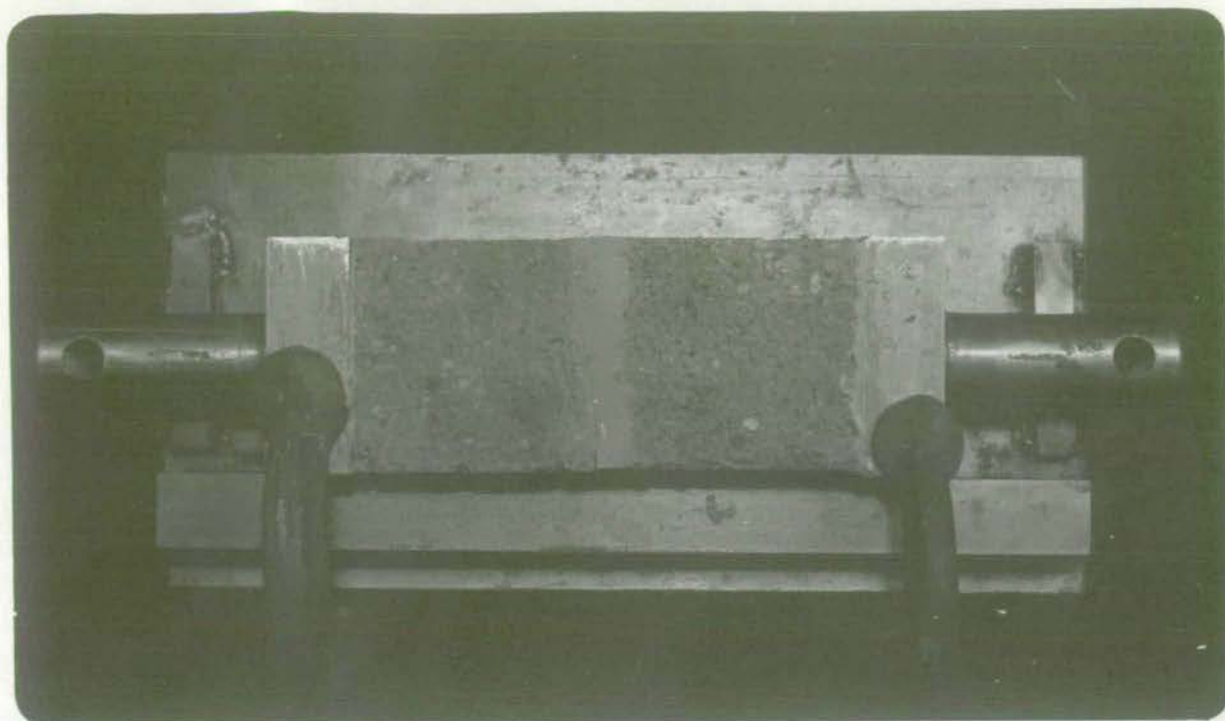


Plate 4.1 PREPARATION OF ONE-HALF BLOCK COUPLET FOR
DIRECT TENSILE TEST



Plate 4.2 DIRECT TENSILE TESTING OF ONE-HALF BLOCK
COUPLET



Plate 4.3 BLOCKWORK TRIPLET UNDER PRECOMPRESSIVE AND LATERAL LOADING

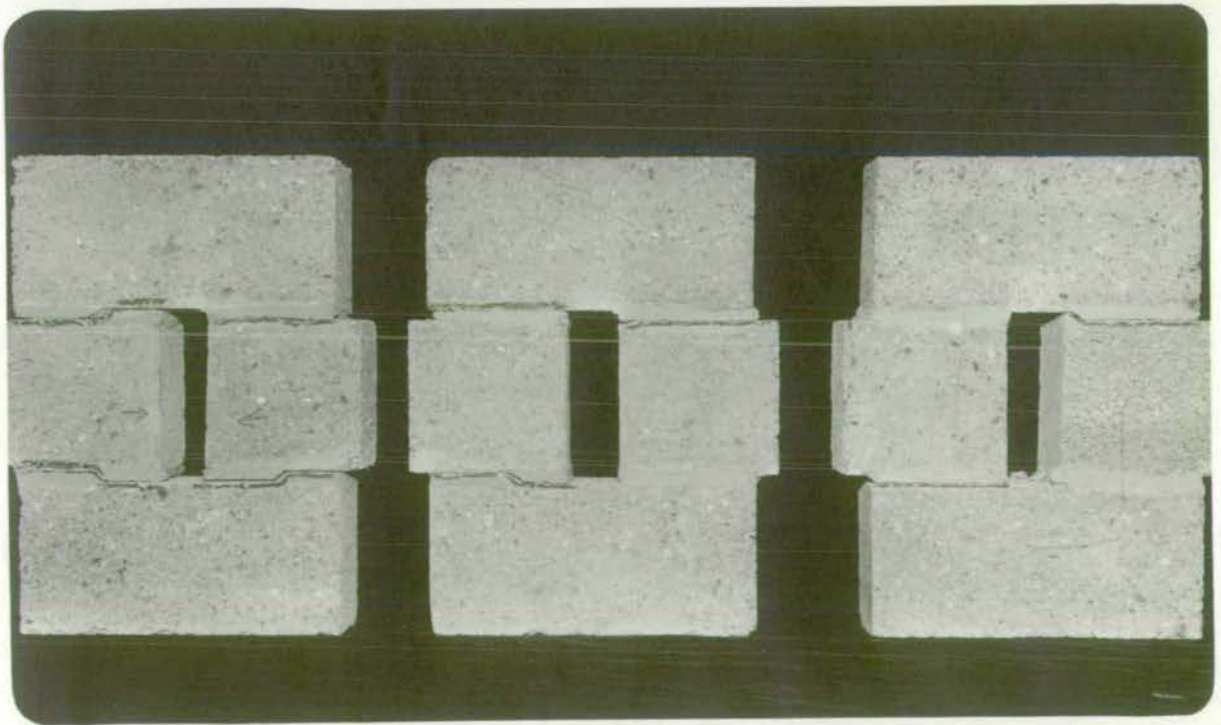


Plate 4.4 BLOCKWORK TRIPLETS AFTER FAILURE

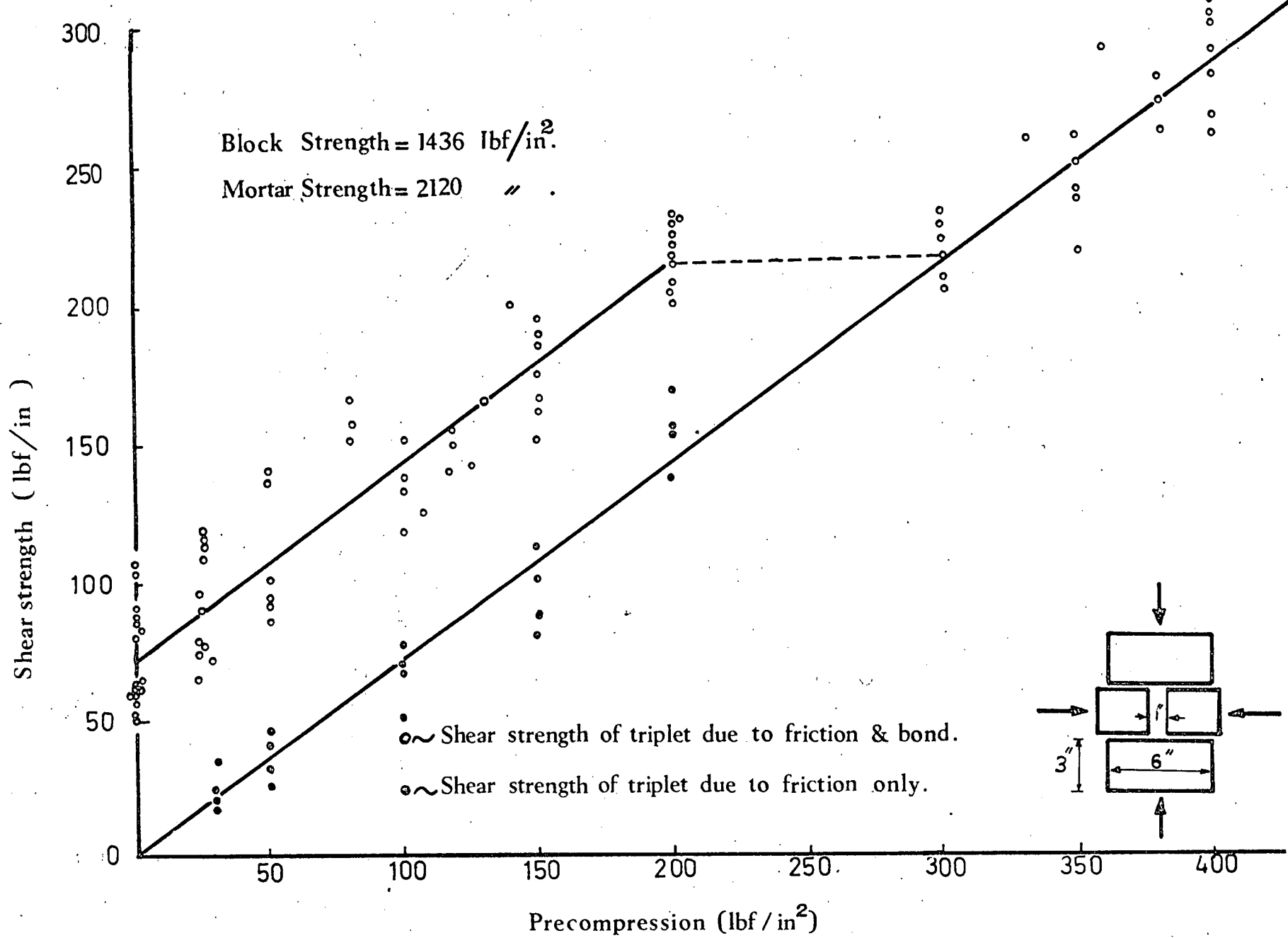


Fig.4.2 Shear strength of blockwork triplets at various precompressions.

PART TWO

4.10 SHEAR TESTS ON STOREY HEIGHT 'AGLITE' BLOCKWORK SHEAR WALL STRUCTURES WITH OPENINGS SUBJECTED TO PRECOMPRESSION

4.11 MATERIALS

The materials used in the following investigation were the same as those given in part one, Section 4.3.

4.12 ROOF SLAB

A doubly reinforced precast concrete slab of 2 inches thickness was set on top of the walls. The concrete mix was 1:1:2 by weight having the maximum aggregate size of 3/16 in. One inch square steel mesh made of 1/8 inch diameter bars was used as reinforcement. The mean crushing strength of 4 inches concrete cubes after 28 days was 7700 lbf/in².

4.13 CONSTRUCTIONAL DETAILS

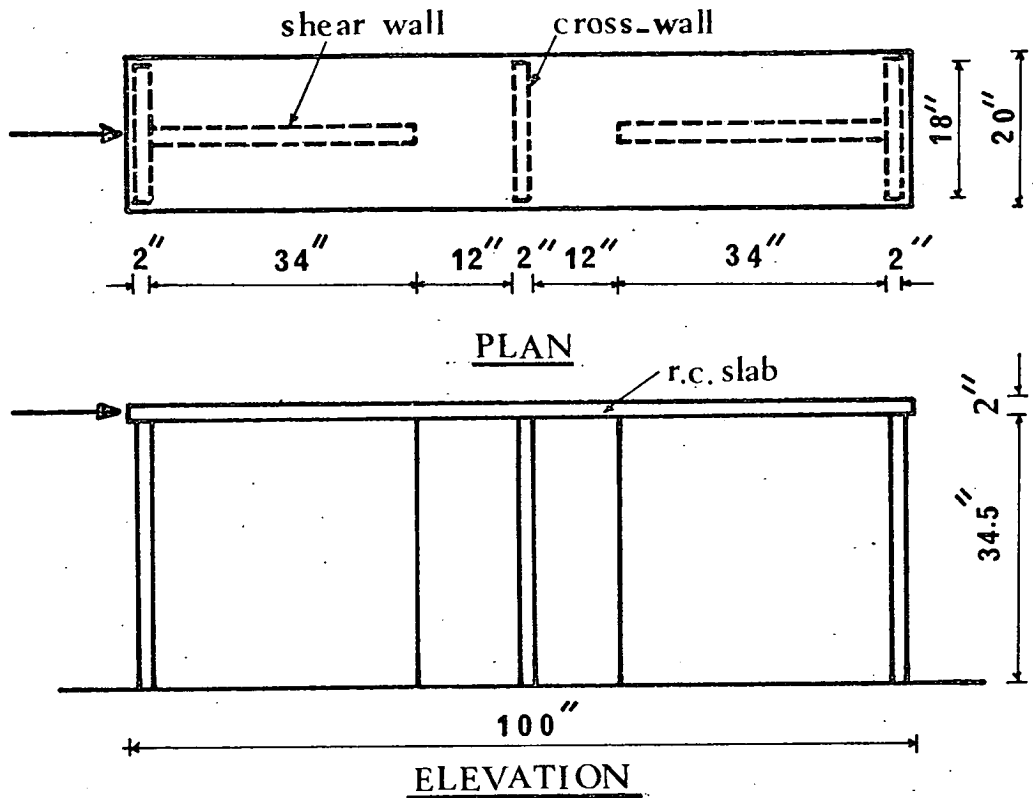
The layout of one-third scale storey-height model structures is shown in Fig. (4.3). All walls were built within the testing frame and on a 4 inch x 2 inch x 7 lbs steel channel bolted into the reinforced concrete floor at equal spacing of 18 inches. The channels were then filled with concrete and the walls were built on the concrete. Shear walls were bonded to the return walls in every other course. The joints between the walls and the reinforced concrete slabs were also made with 1:1/4:3 cement lime mortar. The slab was placed and levelled carefully and the joint was allowed to cure for a minimum of 21 days in the laboratory atmosphere.

4.14 DETAILS OF THE TEST ARRANGEMENT

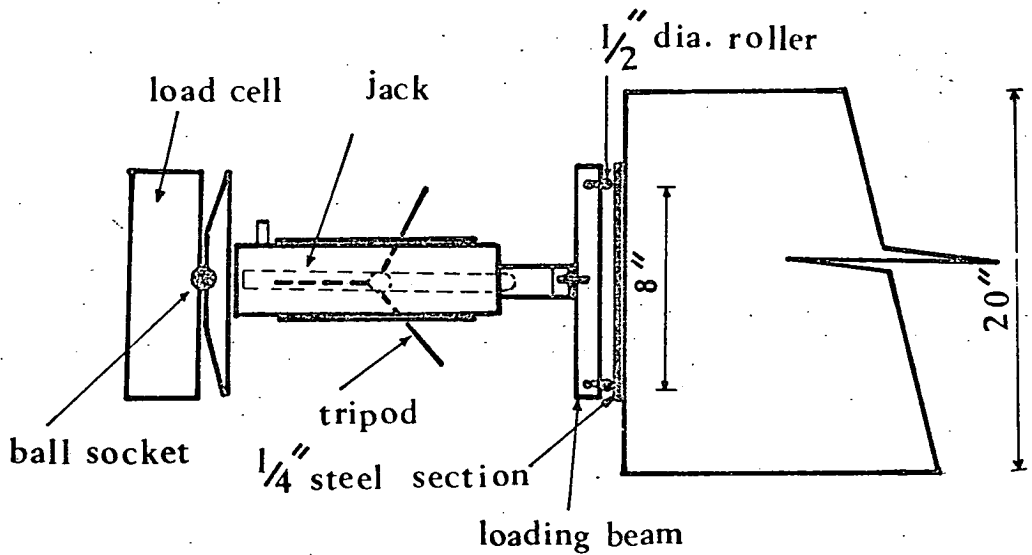
4.14.1 Loading Frame

A loading frame was specially designed and constructed in order to test one-third scale/





LAYOUT OF THE STOREY-HEIGHT SHEAR WALL STRUCTURES



ARRANGEMENT FOR APPLYING RACKING LOAD

Fig. 4.3

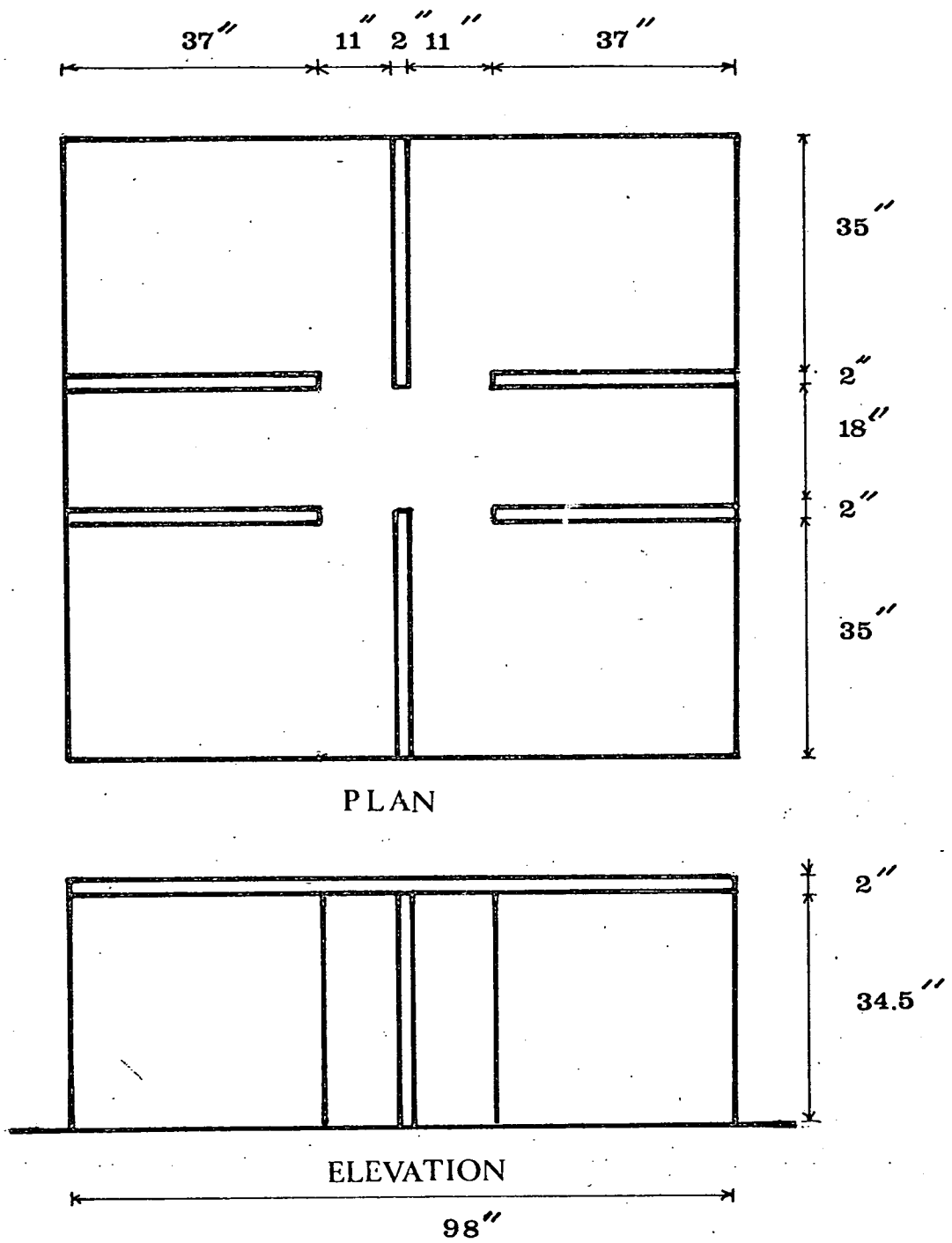


FIG. 4.4 MODEL NO. 7

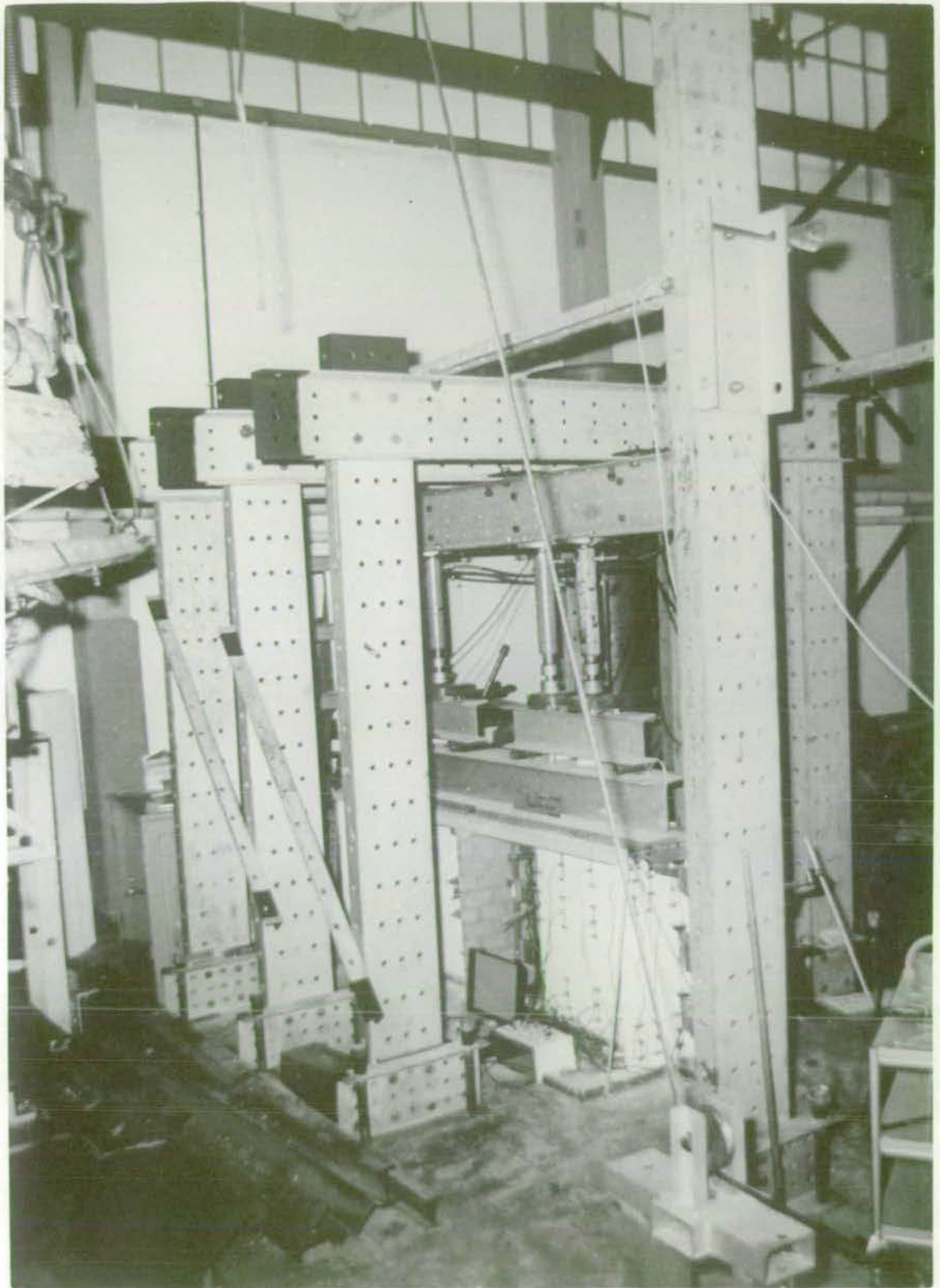


Plate 4.5 LOADING FRAME FOR SINGLE STOREY SHEAR

WALL TEST

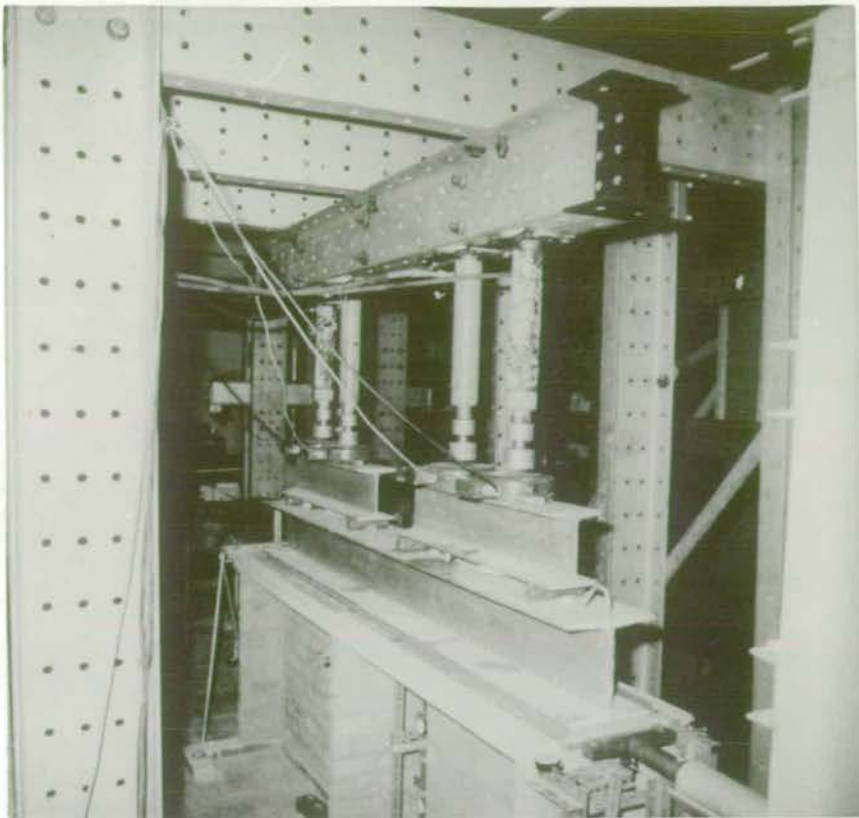


Plate 4.6 LOADING ARRANGEMENT FOR THE APPLICATION OF PRECOMPRESSION

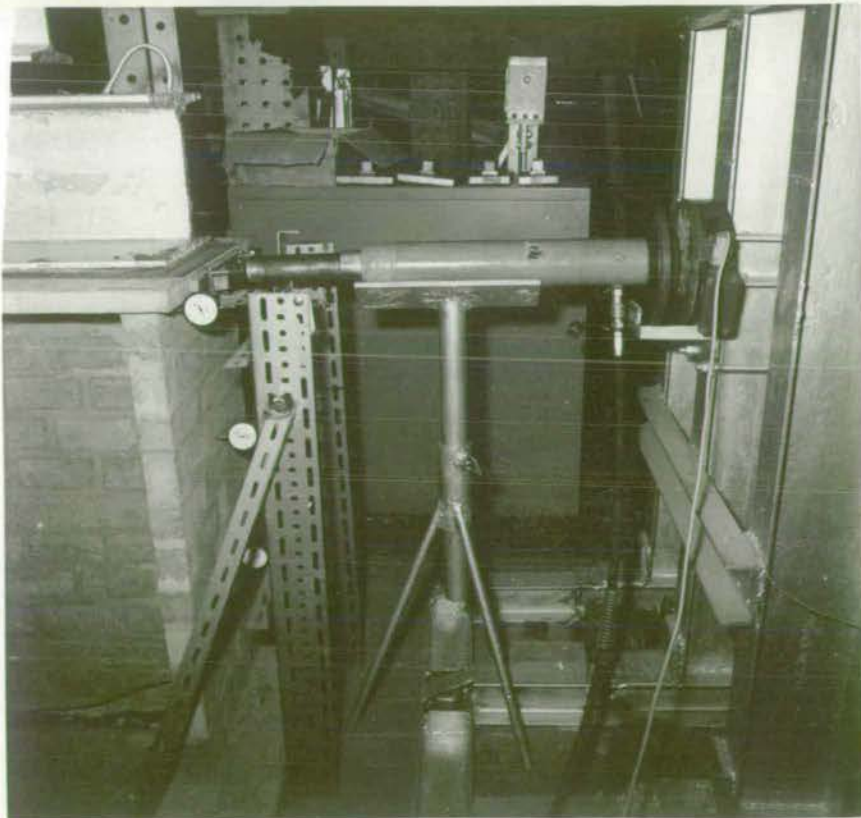


Plate 4.7 LOADING ARRANGEMENT FOR THE APPLICATION
OF THE RACKING LOAD

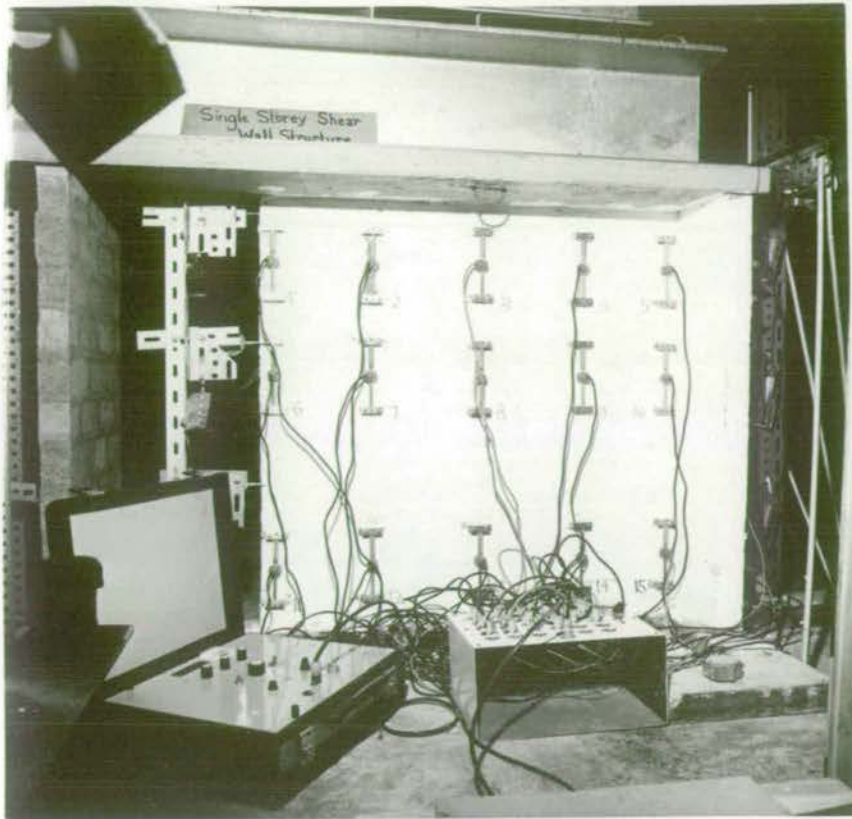


Plate 4.9

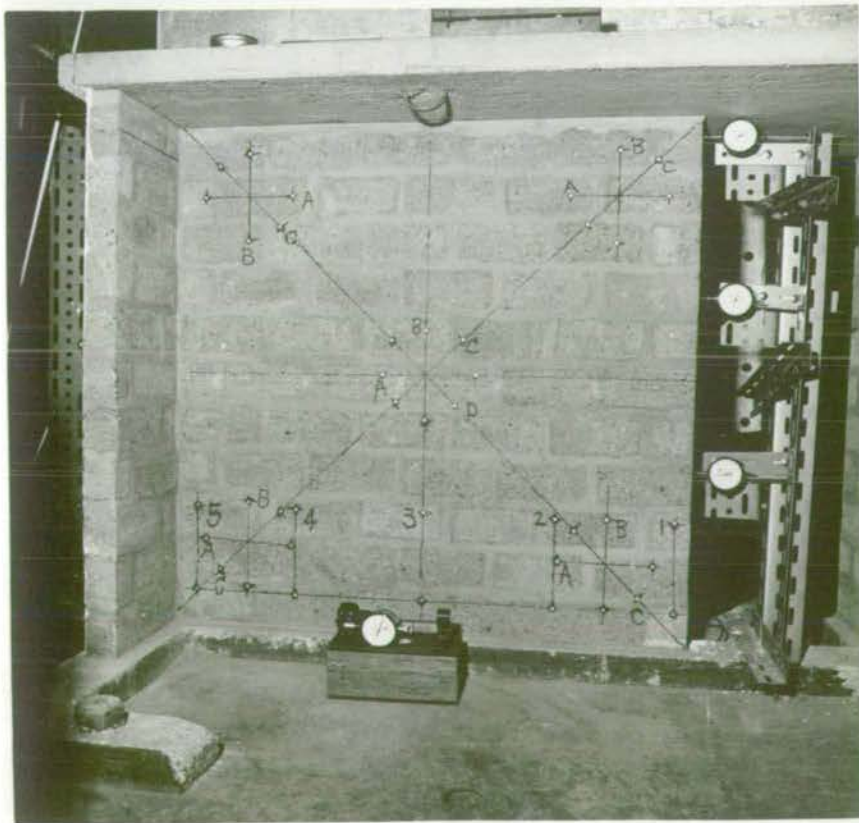


Plate 4.8

POSITION OF DEMEC STUDS AND ARRANGEMENT OF VIBRATING
WIRE STRAIN GAUGES IN SINGLE STOREY SHEAR WALL TEST

one-third scale single storey shear wall structures plate (4.5). This frame was capable of applying horizontal and vertical loads by means of hydraulic jacks. It consisted of two separate and independent parts in order to apply the racking load and the precompression independently. By doing so; no rotation was induced in the frame during the loading and the dial gauges measured the true deflection of the structure. Racking load was applied by a 20-ton hydraulic jack plate (4.7), through a high tensile steel loading beam (made of a rectangular cross-section $1\frac{1}{2} \times 1 \times 10$ inches long) fig. (4.3). The jack was supported by an adjustable tripod and its ends were resting against a 10-ton load cell with a ball socket joint to allow a correct alignment of the jack. This load cell was connected to the reaction frame by an adjustable clamp. The frame was capable of applying 15 tons horizontal load. The load was transmitted to the roof slab through two $\frac{1}{2}$ inch diameter steel rollers 8 inches apart mounted on the loading beam. A mild steel of rectangular cross-section 10 in. x 2 in. x 1.7 lbs. was inserted between the rollers and the edge of the slab to allow a uniform distribution of load along the width of the slab. The frame for the application of vertical load consisted of three rectangular portals made of 12 inches x 4 inches x 31 lbs. channels 9 ft. 6 inches high and 8 ft. 6 inches wide at 3 feet centres. They were braced and connected to each other to form a rigid box. Two 12 inch x 4 inch x 31 lbs. channels were connected back to back and were bolted to the top of the portals to hold the jacks vertically by means of clamps. Precompression was applied by four 20-ton hydraulic jacks via/

jacks via load cells to two steel I - sections (7 inches x 4 inches x 15.5 lbs) being placed on four 2 inches diameter rollers, these in turn transmitted the load to the slab via the main loading beam (10 inches x 10 inches x 112 lbs) Plate (4.6). To ensure an even distribution of precompressive load $\frac{1}{2}$ inch thick layer of dry sand of uniform thickness was placed on the slab and the main loading beam was then placed and levelled along its length and width.

4.14.2 Load Measuring Equipment

All hydraulic jacks were operated from a Losenhausen testing machine. Readings of load cells which were all previously calibrated were taken from a digital voltmeter (reading to an accuracy of 10 microvolts) at precalculated regular intervals of the racking load.

See Appendix (1).

4.15 EXPERIMENTAL INVESTIGATION

Seven one-third scale single storey shear wall structures were tested to destruction with different precompression ranging from 50 to 250 lbf/in². Tests were also conducted at precompressions ranging from 10 to 50 lbf/in² at 10 lbf/in² intervals and within the elastic range of the structures.

4.15.1 Deflection Measurements

Deflections were measured by means of .0001 inch dial gauges connected to rigid Dexion frames which were independent of both model and loading frames. Readings were taken at the precalculated value of lateral load. Twelve dial gauges were fixed at one-third points along the height of the structure at both ends and openings. Four other dial gauges were connected to the four corners of the roof slab in order to give the slab deflection and to ensure no torsion would develop after the application/

the application of the racking load. The positions of the dial gauges are shown in Plates (4.8 and 4.9).

4.15.2 Strain Measurement

Vibrating wire strain gauges (Appendix (1)) of $5\frac{1}{2}$ -inches gauge length with reading accuracy of about three microstrains were used to measure vertical strain in the shear walls and the flanges at sections 6, 21 and 30 inches from the base of the structure, Plates (4.8 & 4.9). A portable strain meter P.S.M. (Appendix (1)) was used to record the strain. Demec studs were also fixed to the model and 6 inches demec gauge was used to measure the strain, Plate (4.8).

4.16 MODE OF FAILURE

Because of masonry's inherent weakness in tension the shear walls exhibit a tendency for a diagonal tensile or sliding failure when subjected to precompression and lateral forces. The mode of failure and the behaviour of each model will be discussed separately.

4.16.1 Model No. 1

Precompression 47 lbf/in^2

After the application of 3 tons lateral load, tensile cracks started to form at the base of the structure, and the sliding failure occurred at about 4 tons, in the form of a diagonal crack in the shear wall at the unloaded end, stepping down the vertical and along the horizontal mortar joints, plate (4.10).

4.16.2 Model No. 2

Precompression 66 lbf/in^2

Prior to this test, it was noticed that the reinforced concrete roof slab had been cracked across its width. At 2 tons racking load, the slab was/

slab was lifted off slightly from the middle cross wall, and the further increase in the lateral load to 3 tons cleared the slab off from this wall. When the lateral load was increased further both shear walls were shifted along their base in the direction of the applied load. This was assumed as the failure load and since the structure was not damaged both lateral and vertical loads were then released. The precompression was then increased to 95 lbf/in^2 and the racking load was applied till the structure finally failed at 6.83 tons. Failure was similar to that of the Model No. 1. The diagonal cracks formed in the shear wall at the unloaded end. The deflection of the structure was slightly higher than expected. This may have been due to the cracks in the base of the structure and in the slab; and it was therefore ignored.

4.16.3 Model No. 3

Precompression was fixed at 100 lbf/in^2 . At a lateral load of 4 tons a slight tensile crack was observed at the base of the flange (return wall) at the loaded-end, at about 5 tons this crack was very distinct. No drop in the racking load was observed. The load was therefore increased further, and finally failure occurred at 6.73 tons by diagonal tension. Cracks occasionally passed through horizontal and vertical mortar joints, but mainly through blocks and mortar. Plate (4.11).

4.16.4 Model No. 4

In this test precompression was kept constant at 150 lbf/in^2 . Slight hair cracks developed in both flanges at about 1 ton shear load. These joint cracks appeared to be due to shrinkage, since they did not develop further/

develop further at higher values of racking load. At about 4 tons slight cracks were formed in both shear walls and finally failure occurred at 6.54 tons. The nature of failure was similar to that of Model No. 3 being a combination of diagonal tensile failure and relative sliding of the blocks, Plates (4.12 a & b).

4.16.5 Model No. 5

Precompression in this test was kept constant at 200 lbf/in^2 . At a shear load of 7.38 tons diagonal cracks started to develop in the shear wall nearer to the loaded-end. These splitting cracks which sometimes were vertical cracks through block and mortar widened as the racking load increased. The structure finally failed at 8.80 tons. Failure was mainly through blocks. Return wall at the unloaded end cracked horizontally across mortar joint between the 4th and the 5th courses. The middle cross-wall cracked at the top between the 1st and 2nd courses and failed by crushing. The return wall at the loaded-end cracked across the mortar joint at the top and split along the middle. Just after the failure of the shear-walls, all the precompressive load was transmitted to the return walls and the slab being supported on the two return walls cracked along its width about 5 inches away from the loaded-end. The severe splitting and cracking failure of both return walls must have been due to this sudden load energy transfer, Plates (4.13 a & b).

4.16.6 Model No. 6

Precompression was maintained at 250 lbf/in^2 . Failure in shear occurred at 10.7 tons. Initial splitting cracks were formed about 6.67 tons/

667 tons shear-load and were widened at about 8.5 tons with formation of further vertical cracks. The new reinforced concrete roof slab also cracked at ultimate lateral load at a distance of 9 inches from its end. The failure mechanism exhibited by this model had the form of multiple cracking pattern, being somewhat modified from those of previous models. Most of the cracks developed in a nearly vertical direction, although a few diagonal cracks also formed, plates(4.14 a & b). Without doubt this arises from the fact that excessive axial pre-compression simply modifies the direction of principal stresses.

4.16.7 Model No. 7

Plan of this model is shown in Fig (4.4). Precompression was applied by placing sufficient weights on the slab (10 lbf/in^2). Failure, at average shear stress of 39 lbf/in^2 , took place at wall-slab joint. Some step-wise cracks developed in the shear walls at the unloaded end close to the wall/slab joint.

4.17 DETERMINATION OF ELASTIC CONSTANTS FOR REINFORCED CONCRETE SLAB AND THE BLOCKWORK

The value of elastic modulus for concrete was found from compression tests of 12 x 6 inch cylinders using vibrating wire gauges of $5\frac{1}{2}$ inch gauge length. The modulus was found to be $4.3 \times 10^6 \text{ lbf/in}^2$. (Fig. 4.12). Assuming Poisson's ratio for concrete as 0.15 the shear modulus was $1.87 \times 10^6 \text{ lbf/in}^2$.

Values of the modulus of elasticity and Poisson's ratio for the blockwork were obtained from axial compression tests. Vertical and lateral strains were measured using 24 and 12 inch demec gauges respectively (Fig. 4.13). Axial compression and flexural tests were also conducted on one-third/



Plate 4.10 FAILURE OF SINGLE STOREY BLOCKWORK SHEAR WALL UNDER LATERAL LOADING AT 50 lbf/in^2 PRECOMPRESSION

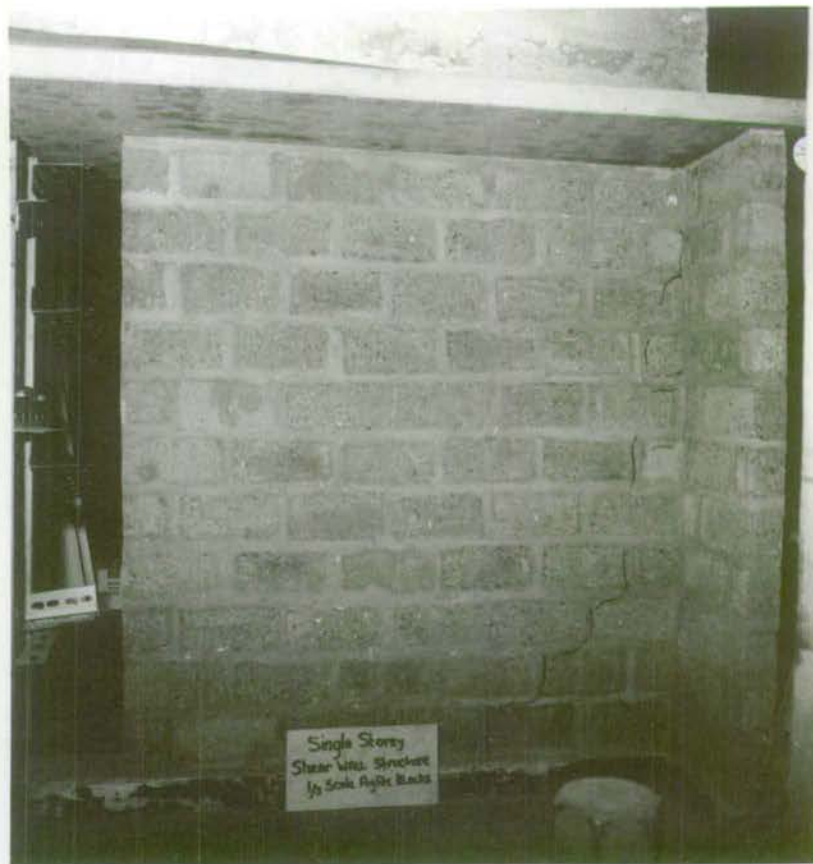


Plate 4.11 FAILURE OF SINGLE STOREY BLOCKWORK SHEAR WALL UNDER LATERAL LOADING AT 100 lbf/in^2 PRECOMPRESSION



Single Storey
Shear Wall Structure
1/2 Scale Flyke Blocks

Plate 4.12 a



Single Storey
Shear Wall Structure
1/2 Scale Flyke Blocks

Plate 4.12 b

FAILURE OF SINGLE STOREY BLOCKWORK SHEAR WALL
UNDER LATERAL LOADING AT 150 lbf/in^2 PRECOMPRESSION



Plate 4.13 a

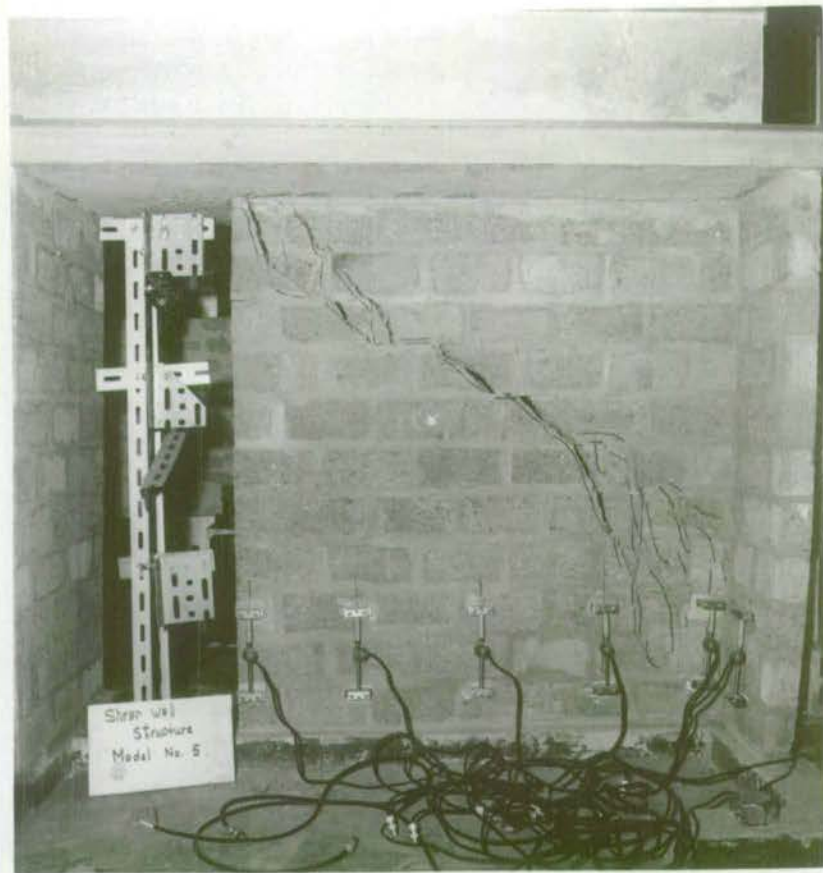


Plate 4.13 b

FAILURE OF SINGLE STOREY BLOCKWORK SHEAR WALL
UNDER LATERAL LOADING AT 200 lbf/in^2 PRECOMPRESSION



Plate 4.14 a

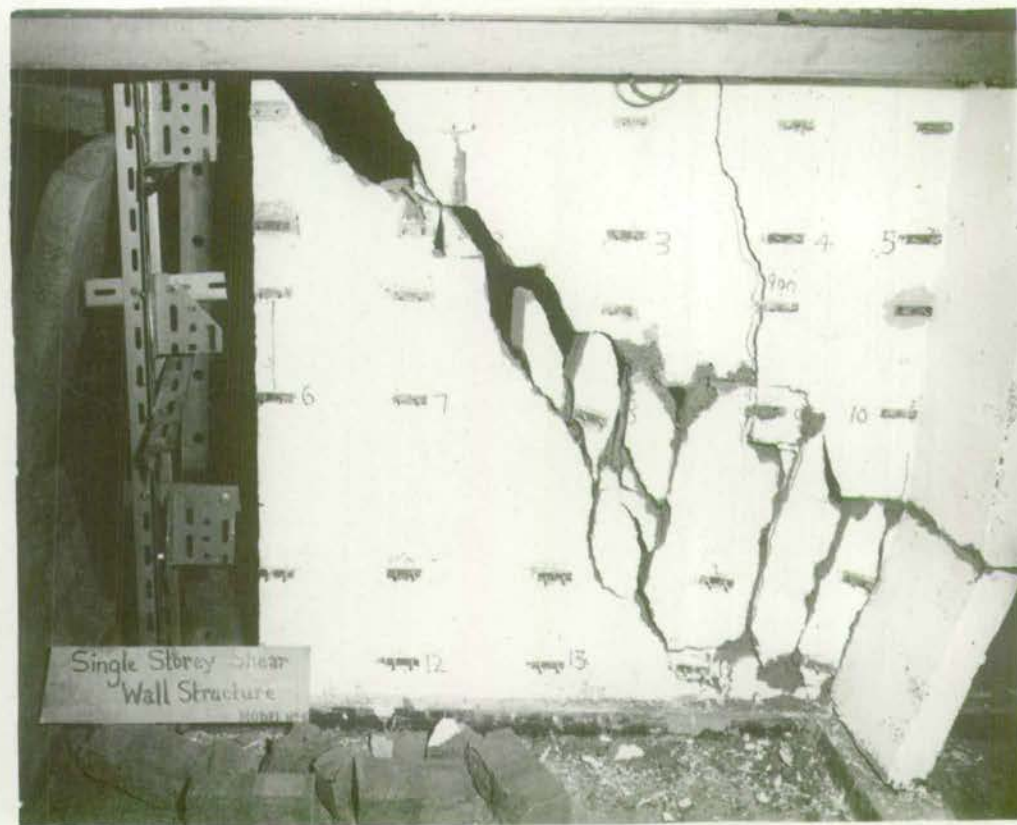


Plate 4.14 b

FAILURE OF SINGLE STOREY BLOCKWORK SHEAR WALL

UNDER LATERAL LOADING AT 250 lbf/in^2 PRECOMPRESSION

TABLE 4.2 - MODULUS OF ELASTICITY OF BLOCKWORK FROM COMPRESSION AND FLEXURE TESTS

Description	'E' from compression test (lbf/in ²)	'E' from flexure test (lbf/in ²)	Poisson's Ratio	Type of strain gauge
Wall 36 x 18½ x 2 in.	1.155 x 10 ⁶	—	0.17	12 & 2½ in. demec gauge
Wall 36 x 18½ x 2 in.	1.00 x 10 ⁶	—	0.185	ditto
Wall 38 x 18½ x 2 in.	1.10 x 10 ⁶	1.09 x 10 ⁶ *	—	5½ in. vibrating wire gauge
Beam 38 x 6 x 2 in.	1.11 x 10 ⁶	1.17 x 10 ⁶ *	—	ditto
Beam 36 x 6 x 2 in.	—	1.16 x 10 ⁶ *	—	ditto
Beam 36 x 6 x 2 in.	—	1.11 x 10 ⁶ *	—	ditto

Block Strength 1436 lbf/in²

Mortar Strength 2290 lbf/in²

Ultimate strength of wall 938 lbf/in²

* See Appendix 3

on one-third scale walls ($36 \times 18\frac{1}{2} \times 2$ inch) and a number of piers ($36 \times 18\frac{1}{2} \times 2$ inch). The deformation was measured using vibrating wire gauges, (Figs. 4.14 & 4.18). Results for the modulus of elasticity from the above tests are given in table (4.2). See plates 4.15, 16, 17

4.18 DISCUSSION OF RESULTS

4.18.1 Load Deflection Relationship

Figs. (4.5. & 4.6) compare the load deflection relationship at 50 and 100 lbf/in² precompression for all models tested. Within the experimental error, it can be seen that the deflection of all models except No. 4 are similar and very close to one another. The slight scatter could be accounted for the variation in workmanship, and mortar strength; deviation of the results for Model No. 4, may be due to the shrinkage cracks existed in the structure.

The typical relationship between the racking load and the deflection at the slab level for precompressions, 10, 20, 30, 40 and 50 lbf/in² is given in Fig. (4.7). The values are average for all models, results for Model No. 2 being omitted. Fig (4.8) shows the same relationships for higher precompressions i.e., 100, 150, 200 and 250 lbf/in².

Fig. (4.9) gives load-deflection relationship for Model No. 5 at the slab level for its ends and openings at 200 lbf/in² precompression.

Deflections at both openings were approximately identical but greater (by 25%) than those obtained for extreme ends of the model.

Fig. (4.10) shows the deflection along the height of the structure at various values of racking load and 100 lbf/in² precompression.

Deflection at various values of racking load up to 1.5 tons (i.e. 34 lbf/in² average shear stress) is given in Fig (4.11). This shows a family/

a family of curves obtained by plotting precompression against deflection at slab level for various value of racking load for all models tested. The relationship being non-linear for low values of precompression, approaching a constant value at about 140 lbf/in^2 precompression.

Clearly precompression increases the rigidity of the structure up to a certain limit, when further increase in the precompression would have no significant effect on the deflection of the structure.

4.18.2 Comparison of Triplet and Shear Wall Results

The ultimate shear strength results obtained from the triplet tests are higher than those obtained for the single-storey structures within the tests range, Figs. (4.2 & 4.25).

As frictional resistance is about the same for both cases, it appears that, the difference is due to the initial bond shear. In the triplets, due to application of shear load which acts along the mortar joint, shear stresses are developed only in the mortar joints, and the triplet is in the state of a pure shear. In large panel structure however, different conditions of loading, workmanship and in some cases existence of shrinkage cracks, may cause a premature failure due to tensile cracking.

4.18.3 Theory Governing the Failure of Blockwork Shear Wall

The internal friction theory, originated by Coulomb, was the first of a group of theories based on the conception of failure as a sliding along planes inclined to the direction of the principal stresses.

The resistance to sliding is assumed to consist of two parts; a constant shearing strength and a resistance of the nature of friction that is proportional to/

proportional to the normal stress on the plane of sliding.

The material is assumed to fail when the actual shearing stress on any plane exceeds the sum of the shearing strength and the frictional resistance.

The material initially deforms elastically under low loads. Until tensile stresses of a certain magnitude are developed the only way for any element to yield, or cease to deform elastically, is by a plastic sliding, or inelastic shearing, in one fixed direction of weakness for that element. The law governing the beginning of sliding failure might be characterized as an "internal friction theory" for each individual element; that is, the resistance of sliding " τ " is taken to be made up of two parts one term, " τ_0 ", representing the shearing strength of the material, and the second " $\mu \sigma_n$ ", a constant times the normal stress, as given by equation:

$$\tau = \tau_0 + \mu \sigma_n$$

When the shearing stress along the plane of weakness of an element of the structure reaches the value " τ ", sliding of the material follows. In addition to the possibility of a sliding failure, it is assumed that "splitting" or "tensile" failure may occur whenever the tensile stress in any direction reaches a limiting value . It is assumed that the structure fails abruptly, without any plastic deformation, across a plane normal to the tensile stress. The "planes of weakness" of the elements have been assumed to have no effect upon the resistance of the material to splitting.

Coulomb's Theory contains more than a simple statement of a criterion of failure. The basis of the theory is a conception of failure as taking place/

taking place through a sliding or shearing along continuous planes inclined to the direction of the main compressive stress. Since the theory is based on a definite type of deformation, it is evident that strain measurements must show such a deformation existing if the conception of failure is correct.

It appears that the departure from elastic action begins as a sliding failure along inclined surface in the material. This sliding may be a bond failure between mortar and block, so that the plane of weakness may evidently pass through horizontal and vertical mortar joints since these are considered as planes of weakness. It seems reasonable that the resistance to bond slip or sliding may be partly of the nature of adhesion and partly as internal friction. With the spreading of this sliding movement with increasing loads, tensile stresses are developed in those parts of the structure still deforming elastically, and three distinct phenomena may follow:

1. With the low normal precompressive stress, as the sliding action continues to spread to an increasing number of joints, a stage will be reached in which the structure as a whole approaches plasticity. Plastic sliding will proceed along a number of surfaces, so that the parts of the material in which no plastic action has occurred will be displaced relative to each other, and cannot further govern the deformation of the material as a whole. As the material becomes more and more of a plastic nature, one direction of sliding motion may become prominent among the many directions along which sliding is proceeding, this one direction being dependent on the external stress, the aspect ratio of the walls and the external condition of the deformation. It is probable that during process of loading/

of loading certain directions of sliding motion may eventually become so much more prominent than all others than a condition may be approached which is similar to that assumed in the theories involving a plane of least resistance. If so, it seems reasonable that at this stage the relation of the external stresses on the structure must be fairly similar to those defined by the internal friction theory given in its original form.

2. Due to high vertical precompressive stress, tensile stresses in some parts of the structure become too great, and splitting of the material will begin. Since the tensile strength varies in different parts, this action will extend through a range of loading, the splitting of the material finally becoming so extensive that the structure cannot resist further load and the maximum load will be passed.

The range through which this tensile failure will extend and the rapidity of action will depend much on the strength properties of the material (See Chapter III). As it was observed this process of failure in blockwork with high strength units, may lead to a sudden failure. A splitting action in weaker blocks leads to a very slow and progressive failure.

3. The two results of diagonal tensile splitting due to tensile stresses, and the sliding failure may occur simultaneously and may influence each other.

For instance at some range of normal precompressive stress, the development of lateral tensile stress in small parts of material in the structure will cause tensile failure of these parts to begin. Cracks will then develop in few blocks and mortar joints. However, as soon as tensile failure occurs in some parts of the material, it will cause a reduction in the normal compression on planes/

on planes in the direction of weakness of other parts, which will cause sliding failure to extend more or less continually through the material in the directions approximately as given by the internal friction theory. Thus the structure under the action of certain precompressive stress which started to fail due to diagonal tension may show a final failure by shearing, or a combination of shearing and diagonal tensile splitting failure. The results of tests carried out in this investigation are given in Table (4.3) and Fig. (4.25). Similar results for brickwork shear wall structures are reported by other researchers (74 and 96). Referring to Plate (4.10) structures under low precompressive load e.g. 10 and 50 lbf/in² the failure occurred in sliding or shear in the interface of the block mortar, cracks stepping down through vertical and horizontal mortar joints following a diagonal path.

At precompression values perhaps ranging from 100 to 150 lbf/in² being known as the transitional phase, Fig. (4.25), the failure took place by the attainment of the maximum tensile strength of the blockwork, diagonal cracks passes through blocks and mortar, Plates (4.12 a & b). Above this range of precompression, failure occurred once the frictional resistance was overcome. In the triplet tests a similar relationship was observed between the ultimate shear stress and precompressive stress (See Section 4.8).

4.18.4 Comparison with CP 111:Part 2:1970

The results of the tests are compared with the Code of Practice as shown in table (4.3) and figure (4.25).

Permissible Shear Stress

The permissible shear stresses in blockwork according to the Code of Practice/

of Practice when using mortars not weaker than 1:1:6 should range from 15 lbf/in² (0.1 MN/m²) when the compressive strength due to the dead load at the level under consideration is zero. The other limit is 75 lbf/in² (0.5 MN/m²) when the compressive stress due to dead load is 360 lbf/in² (2.5 MN/m²). The values are based on the results of shear tests on single panel, full and small-scale brickwork shear wall structures with and without openings.

When comparing ultimate shear strength of blockwork from the above tests with those given by the Code of Practice and its 1971 amendment, the maximum and minimum calculated factors of safety will be 2.54 and 3.35. Assuming a value of 0.73 for the coefficient of friction the initial bond shear works out to be about 35 lbf/in², figure (4.25) with a safety factor of 2.4 when the external precompression is zero.

Considering the results from the above tests CP 111 recommendation seems to be adequate for design when solid blocks are used. It is apparent that the bond shear will be increased when using hollow blocks with cores filled with plain or reinforced concrete. The maximum allowable shear stress for reinforced blockwork is 72.5 lbf/in² (0.5 MN/m²), which merits research work in the future.

4.18.5 Permissible Tensile Stresses

CP 111:Part 2:1970 states that "no reliance should be placed on the tensile strength of blockwork". However, at the discretion of designer it allows account to be taken of tensile stresses in bending for mortars not weaker than 1:1:6, allowing 10 lbf/in² (0.7 MN/m²) when the/

TABLE 4.3 - SHEAR STRENGTH OF ONE-THIRD SCALE STOREY-HEIGHT SHEAR WALL STRUCTURE
WITH OPENING SUBJECTED TO PRECOMPRESSION

Test No.	Ave.Mortar Strength (lbf/in ²)	Precompression	Ult.lateral load (tons)	Ult. shear strength (lbf/in ²)	Max.Perm. shear stresses according to CP111:Pt.2:1970 (with 1971 amendment)	Factor of Safety according to CP 111: Part 2: 1970 (with 1971 amendment)
1	2130	47	4.19	66.5	22.8	2.91
2 _A	1995	67	4.52	70.0	26.4	2.65
2 _B	1995	95	6.62	103.0	30.7	3.35
3	2210	100	6.73	105.0	31.7	3.31
4	2150	150	6.54	101.5	40.0	2.54
5	2090	200	8.80	137.0	47.3	2.90
6	2250	250	10.68	166.0	56.6	2.94
7	2290	10	5.55	42.0	16.5	2.54

The mean crushing strength of blocks tested according to BS 2028, 1364, 1968 was 1436 lbf/in²

when the direction of tensile stress is at right angles to the bed joints and 20 lbf/in^2 (0.14 MN/m^2) when the direction of the tensile strength is at right angles to perpendicular joints. The upper limit is only to be used for blocks having crushing strength in excess of 1520 lbf/in^2 (10.5 MN/m^2).

Based on the direct tensile bond test results for the couplets as given in table (4.1) an average ultimate value of 57 lbf/in^2 was obtained for 1:1/4:3 mortar and Aglite blocks of 1430 lbf/in^2 crushing strength, with a factor of safety of 5.7. This suggests that the present provision allowed by the code is conservative for blockwork. However, more test results with variables such as mortar and block of a wide range of strength are required to justify the above suggestion.

4.19 ANALYSIS OF SINGLE STOREY BLOCKWORK SHEAR WALL

To compare the experimental results with the analytical results, the values of the elastic constants for blockwork and concrete slab are required.

The present investigation and previous research^(52,74,96) indicate that the precompression increases the rigidity of the structure. In the case of brickwork^(53,96) this was attributed to the increase in elastic constants of the material. In blockwork rigidity increases with the precompression; at and above a certain limit of precompression, (i.e. 150 lbf/in^2) the rigidity remains constant (Fig. 4.11). Strain measurements in the walls and piers tested under compressive and flexural loading (Figs. 4.13 - 4.18) showed that there is a unique value for the modulus of elasticity of blockwork. The reason for the variation of the rigidity with precompression may therefore be due to the following factors:

- 1) Degree of fixity at the base of the structure at different pre-compressions.

2)/

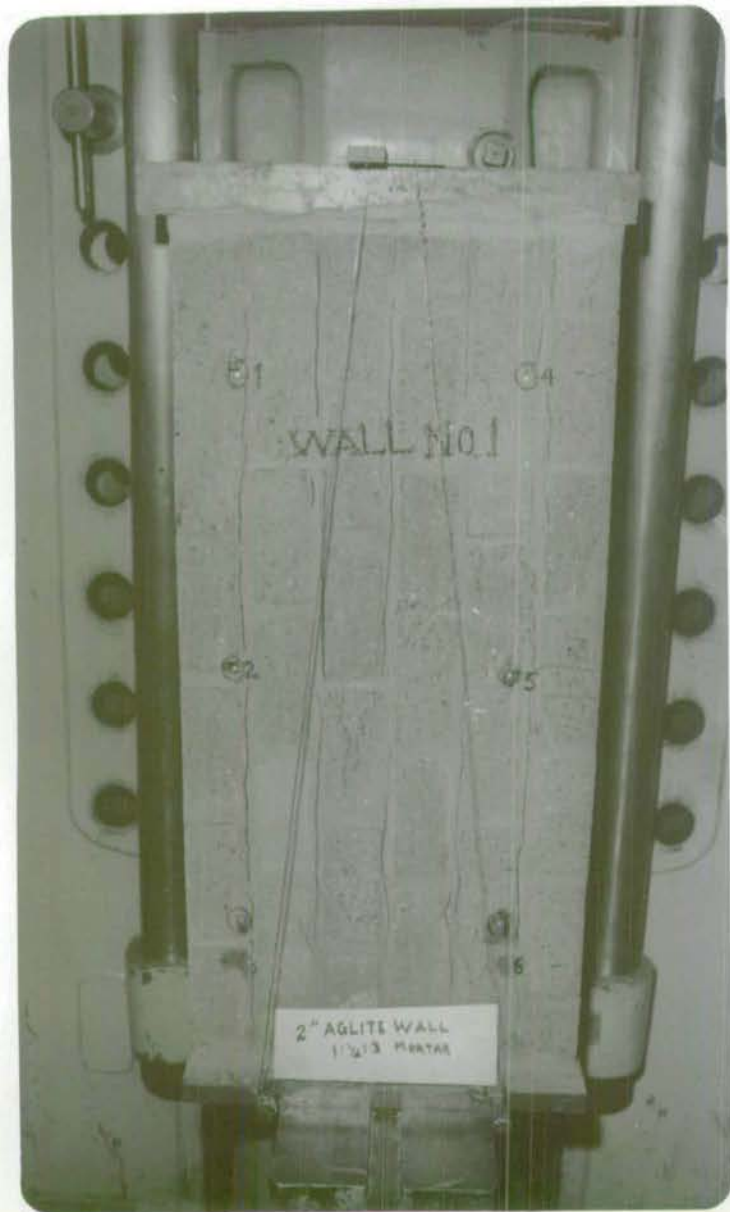


Plate 4.15 a

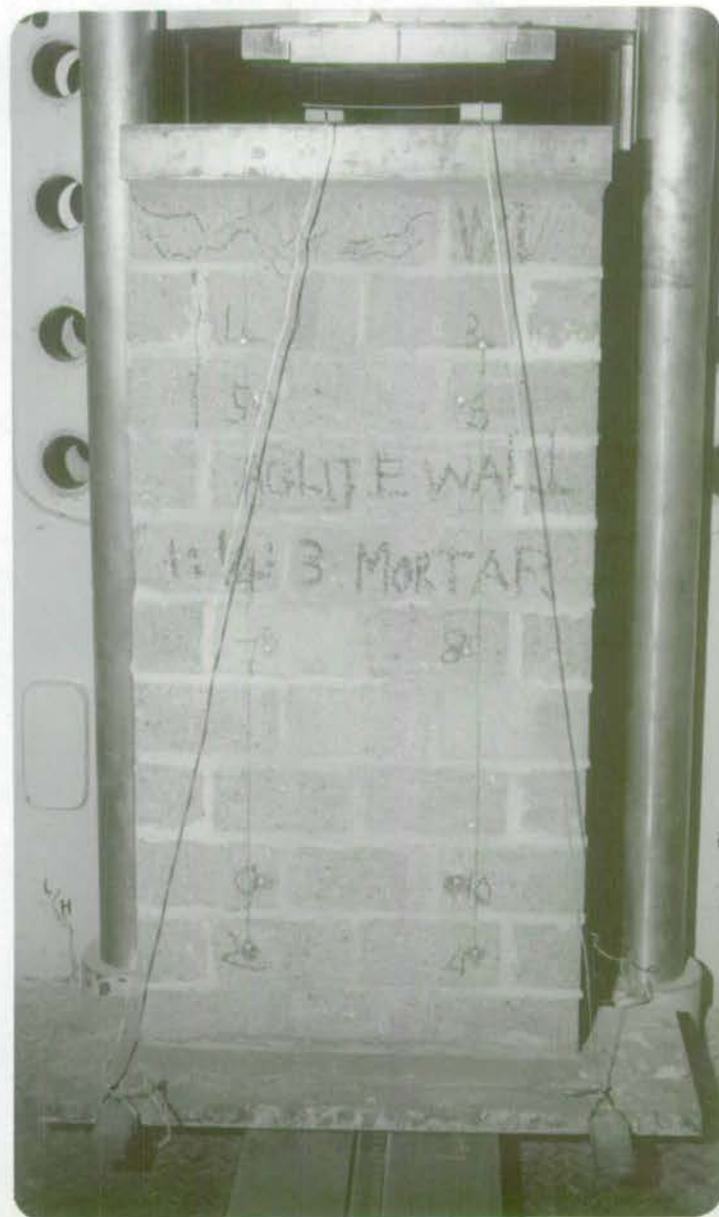


Plate 4.15 b

BLOCKWORK UNDER COMPRESSION FOR MEASUREMENT OF YOUNG'S
MODULUS OF ELASTICITY

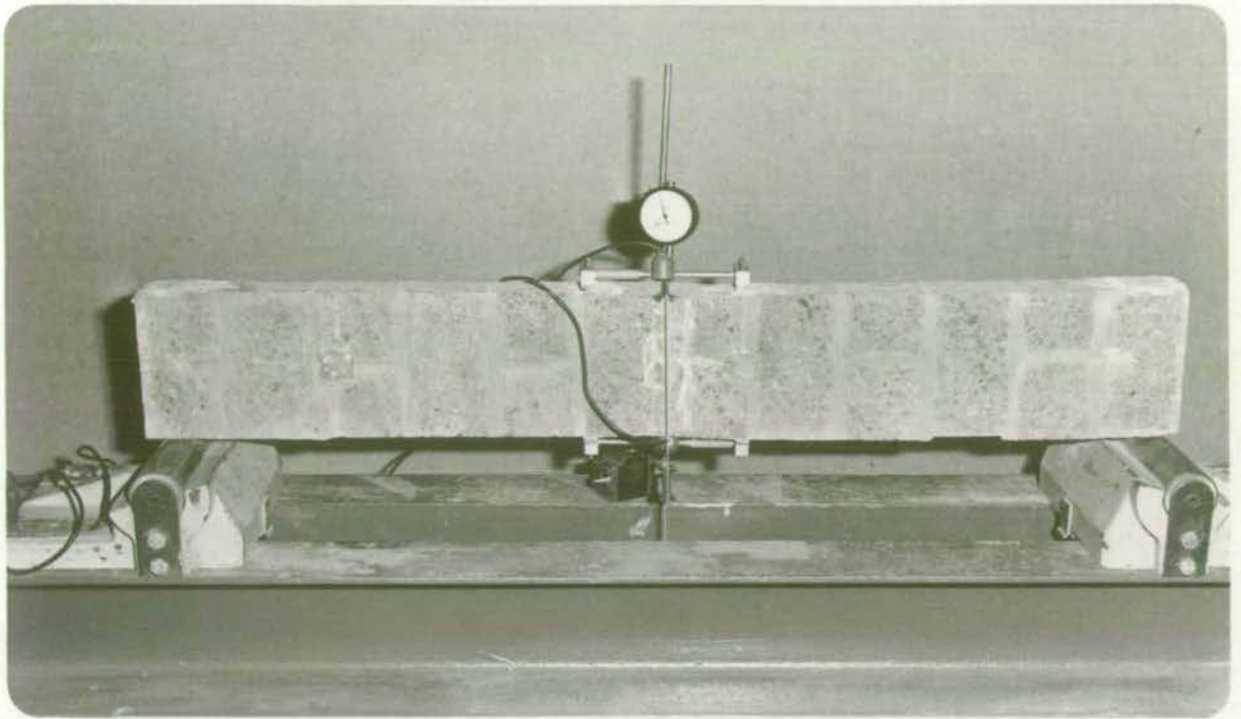


Plate 4.16 FLEXURE TEST ON BLOCKWORK BEAM FOR MEASUREMENT
OF YOUNG'S MODULUS OF ELASTICITY

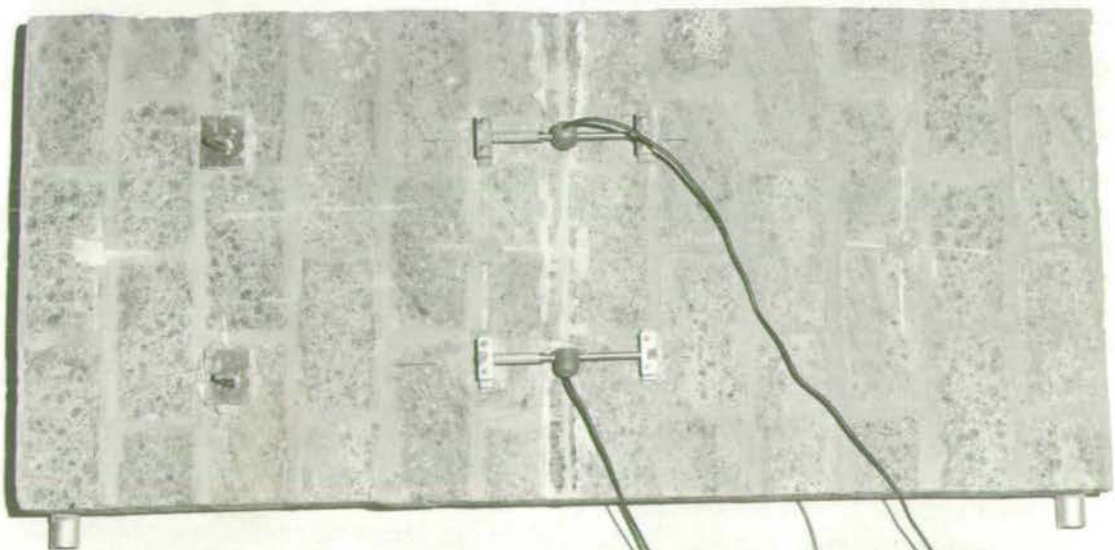


Plate 4.17 FLEXURE TEST ON BLOCKWORK FOR MEASUREMENT
OF YOUNG'S MODULUS OF ELASTICITY

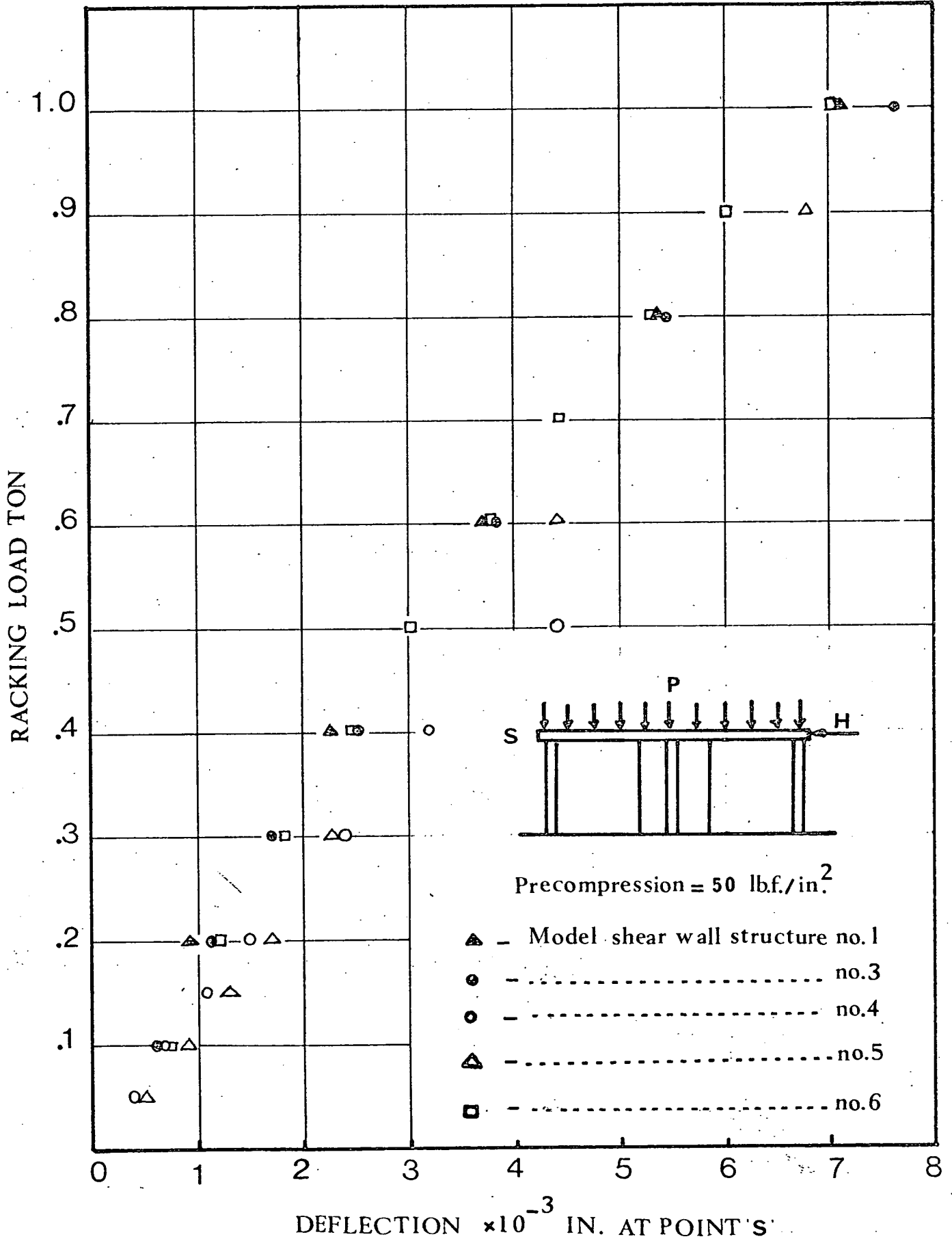


Fig. 4.5

Deflection at slab level

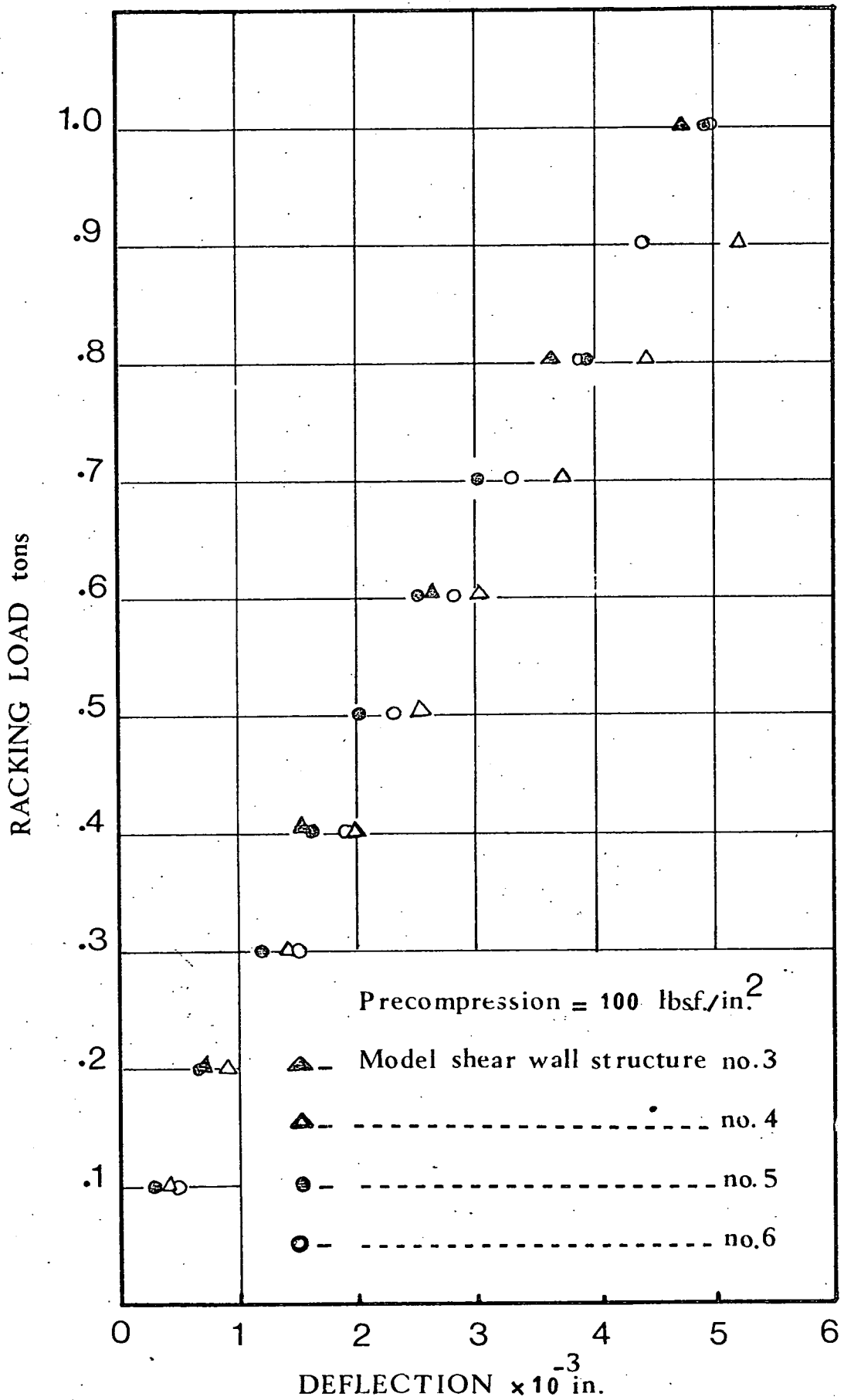


Fig. 4,6

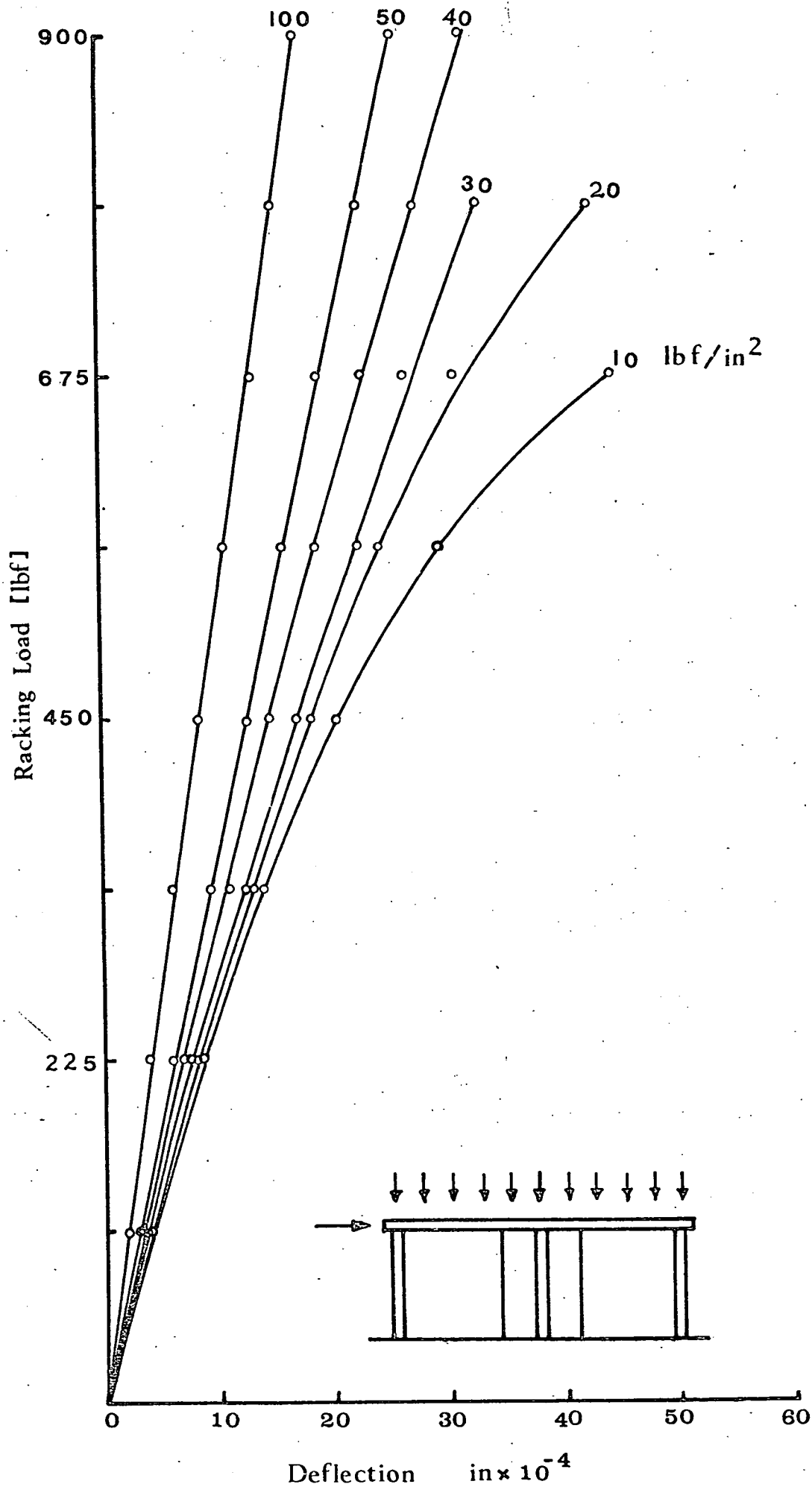


Fig.4.7 Load/Deflection at various precompression

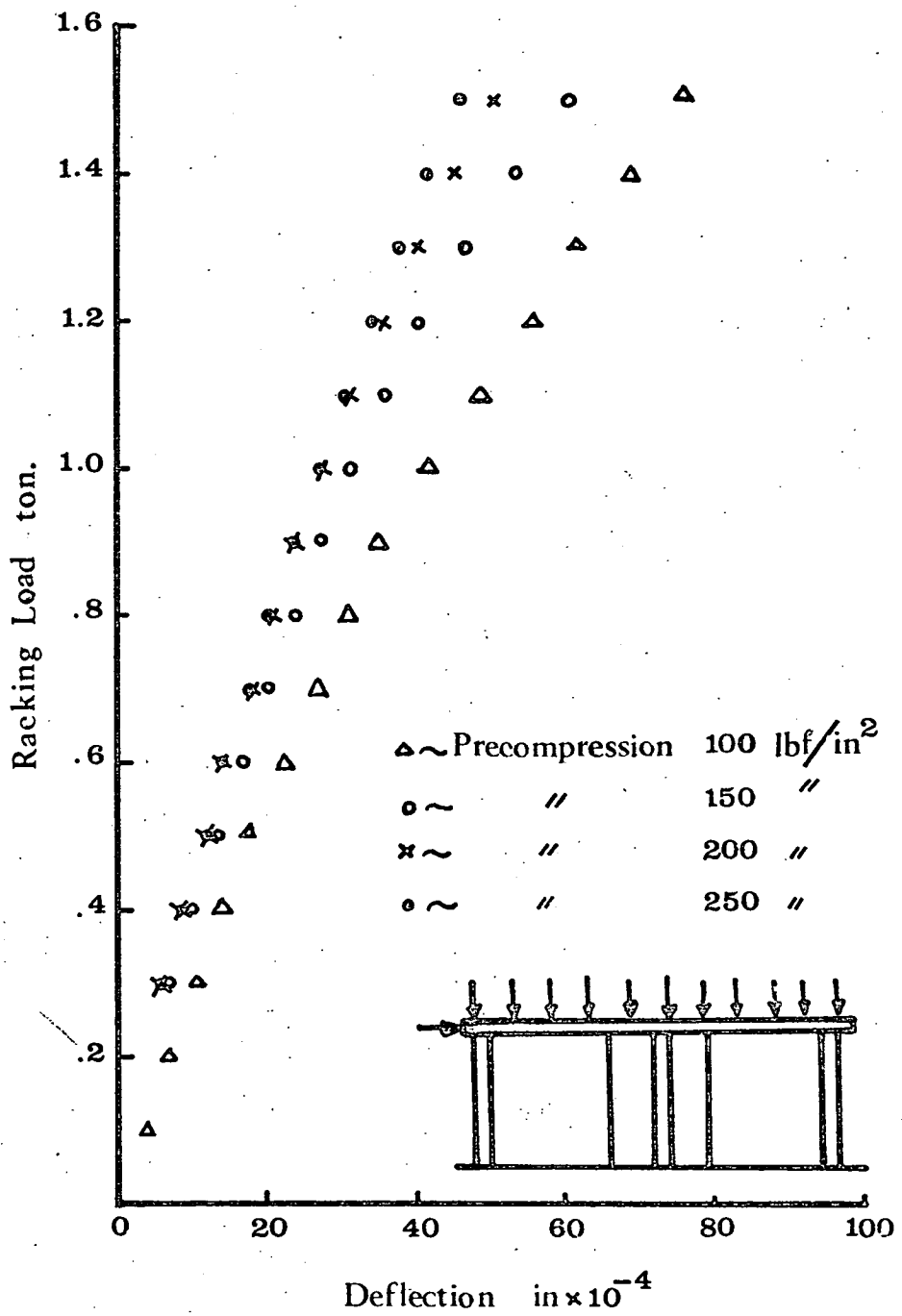


Fig.4.8 Load/Deflection curves at higher precompression

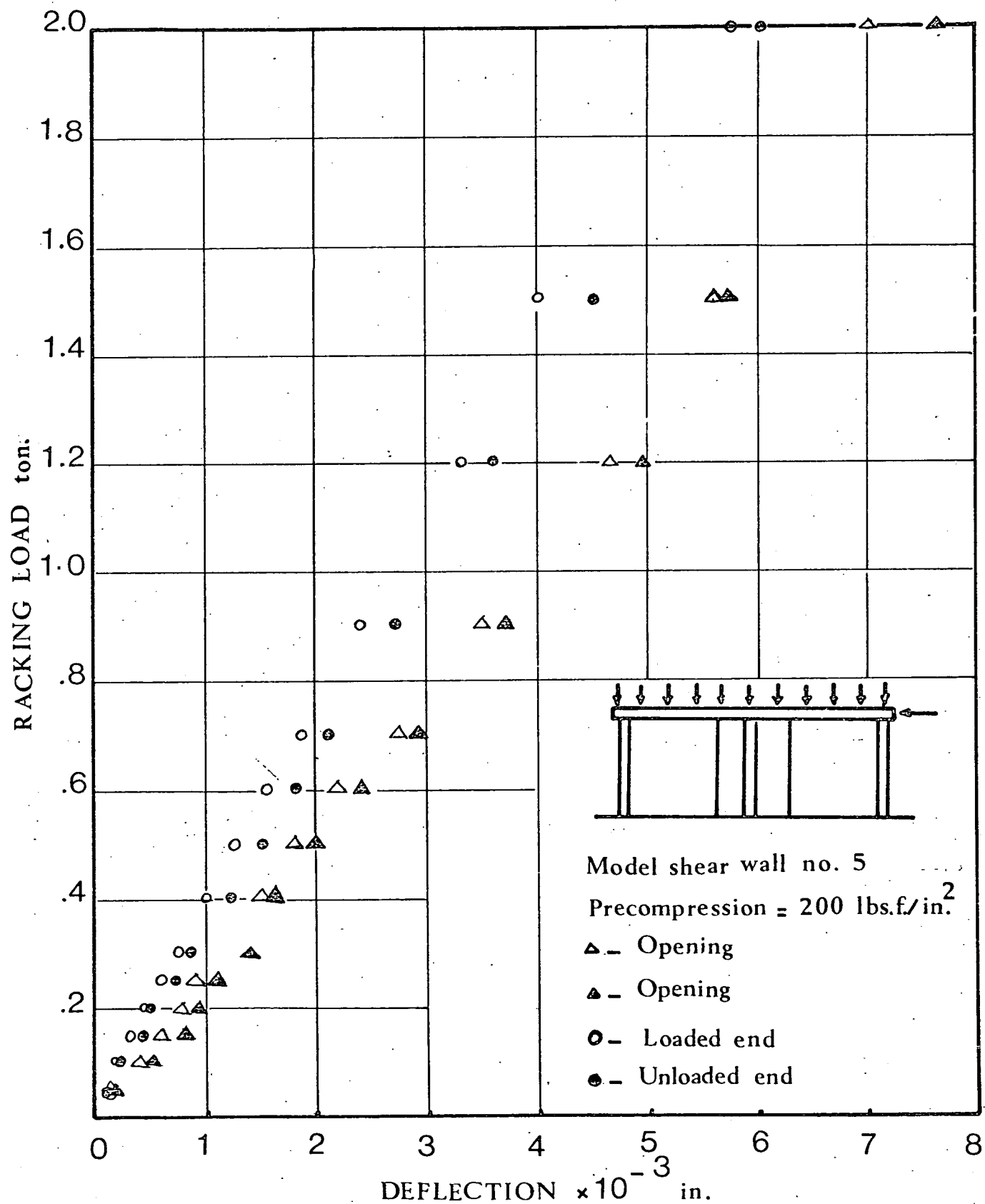


Fig. 4.9

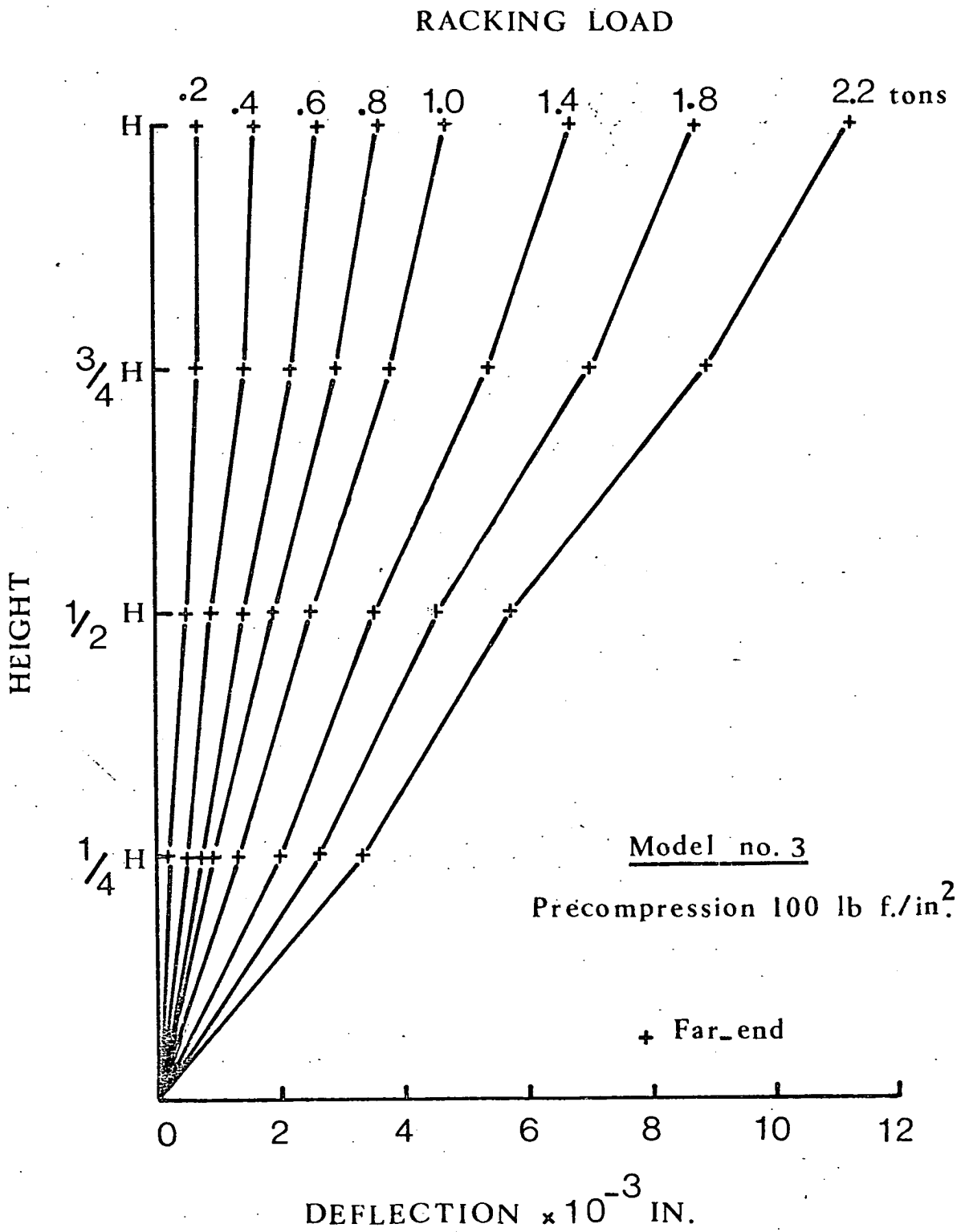


Fig. 4.10.

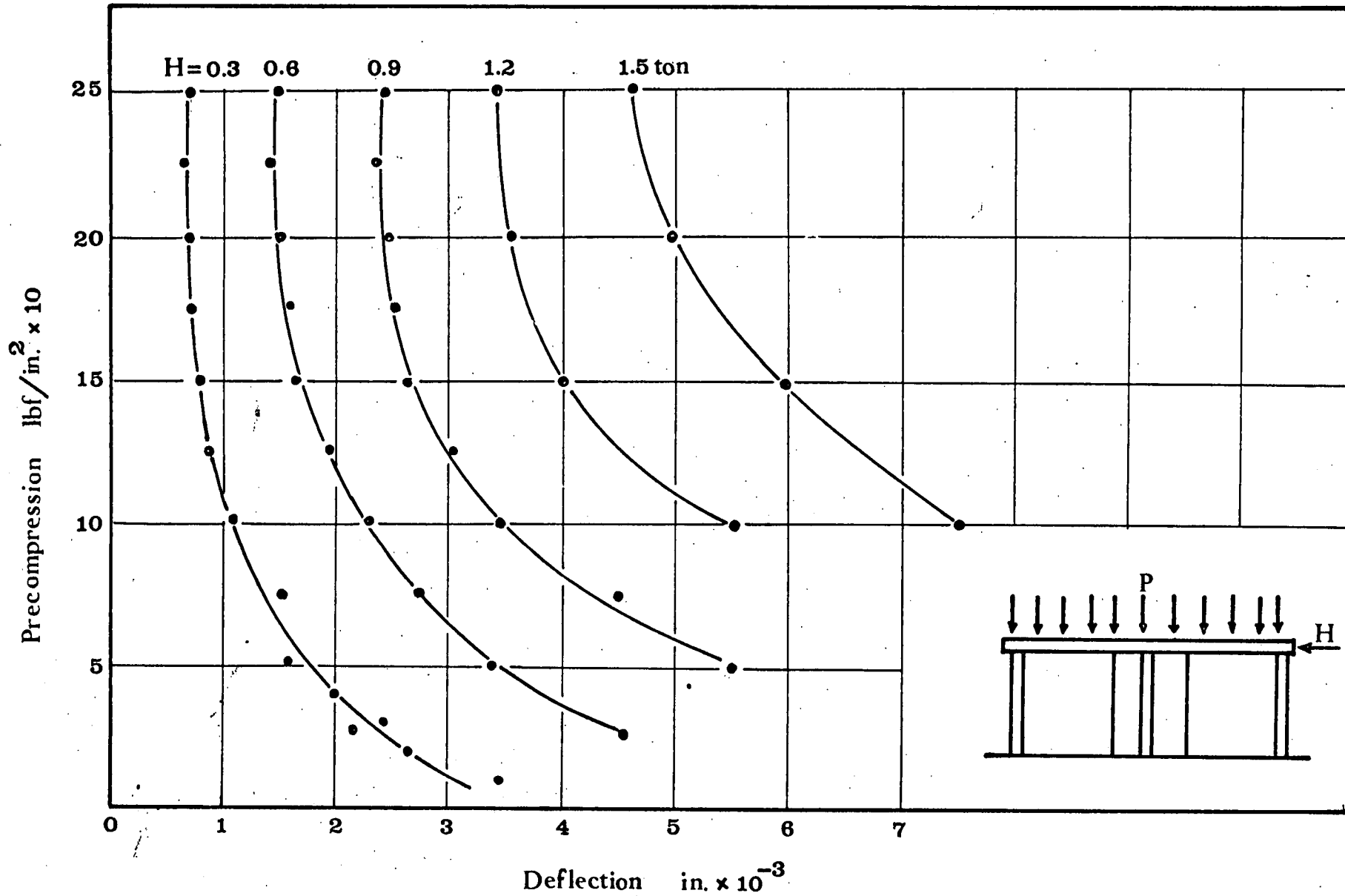


Fig. 4.11 Relationship Between Precompression, Deflection & Racking Load.

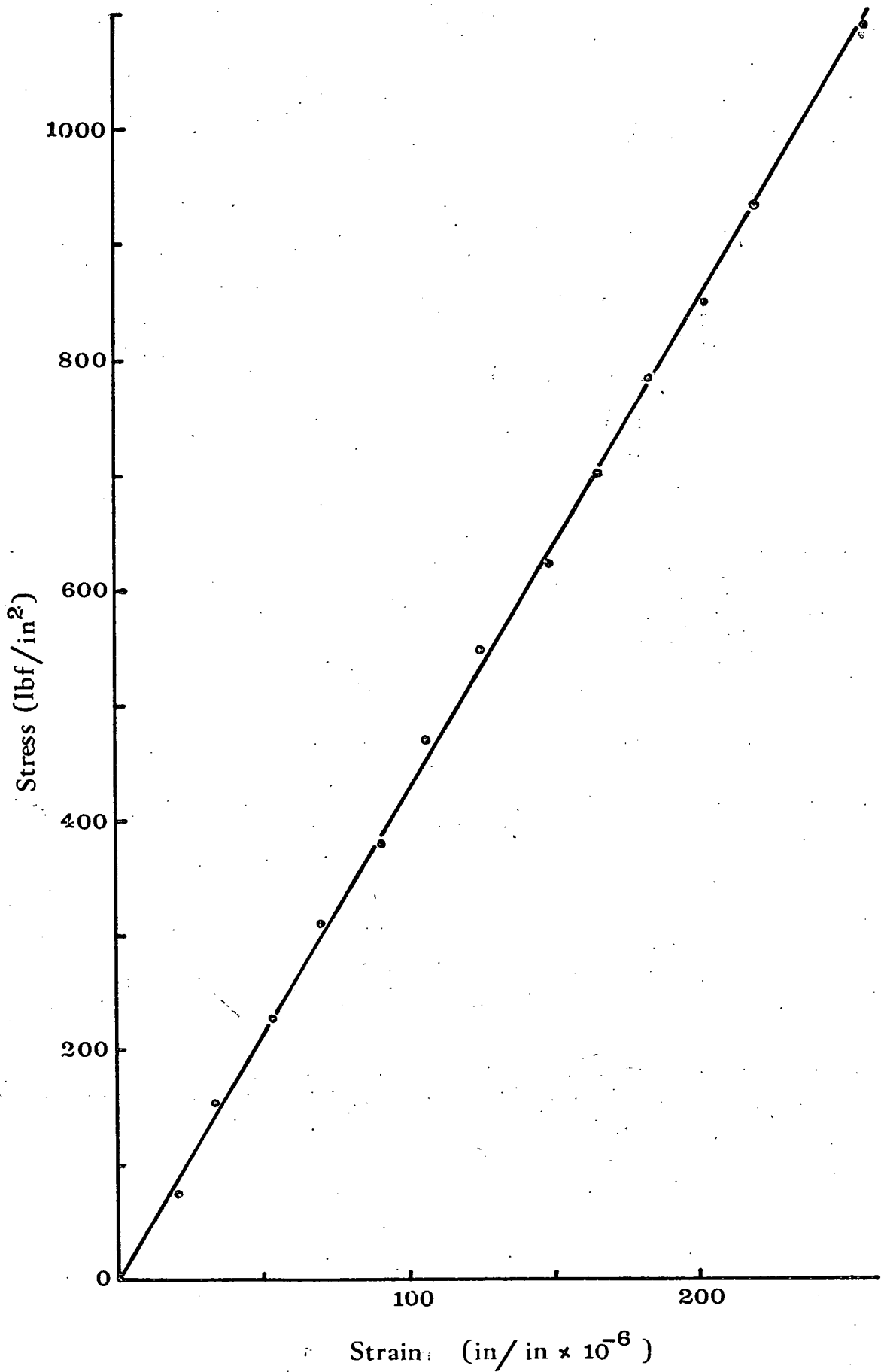


Fig. 4.12 Stress/Strain relationship for concrete cylinder

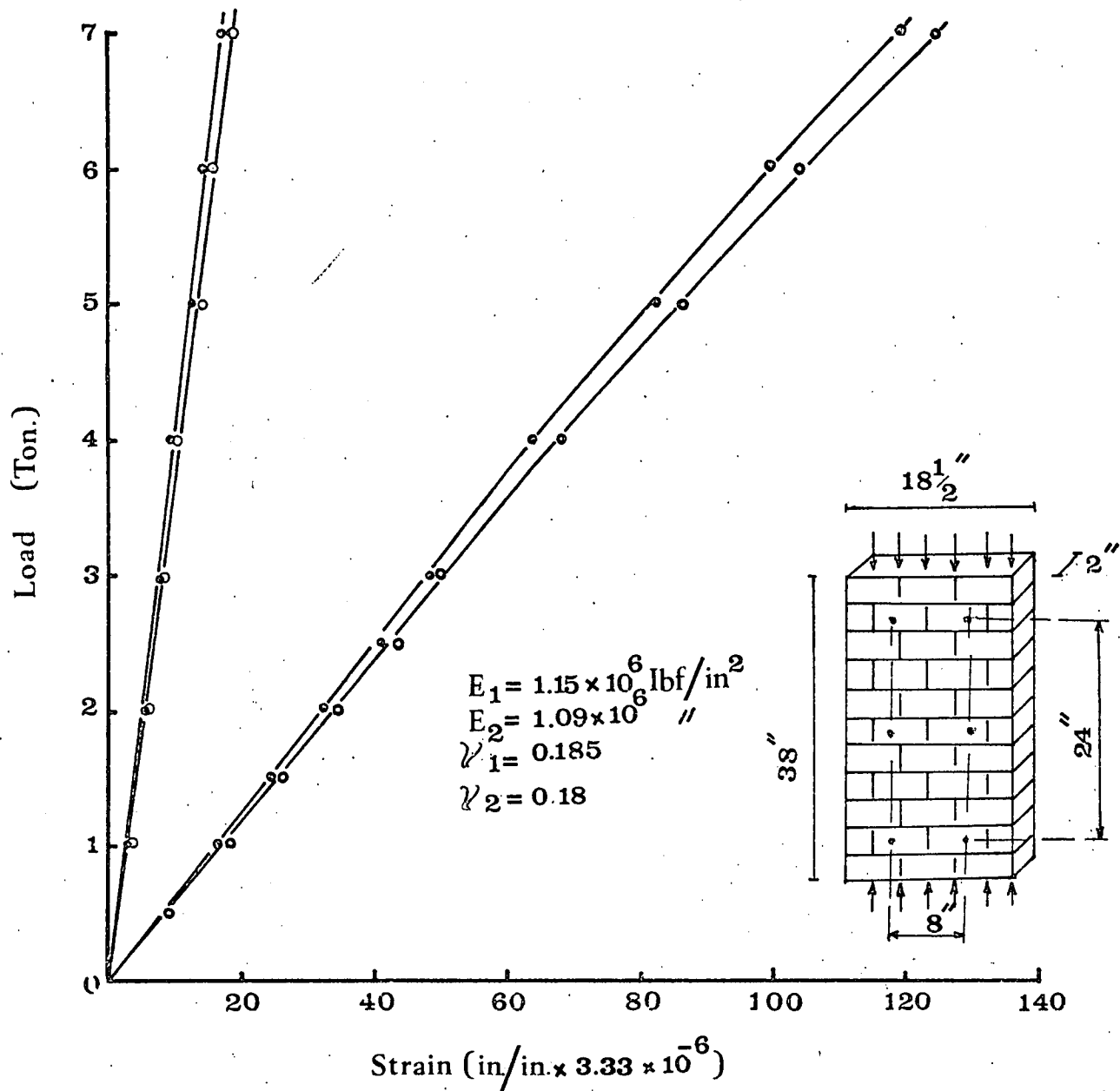


Fig.4.13 Vertical & Lateral deformation of blockwork under axial compression

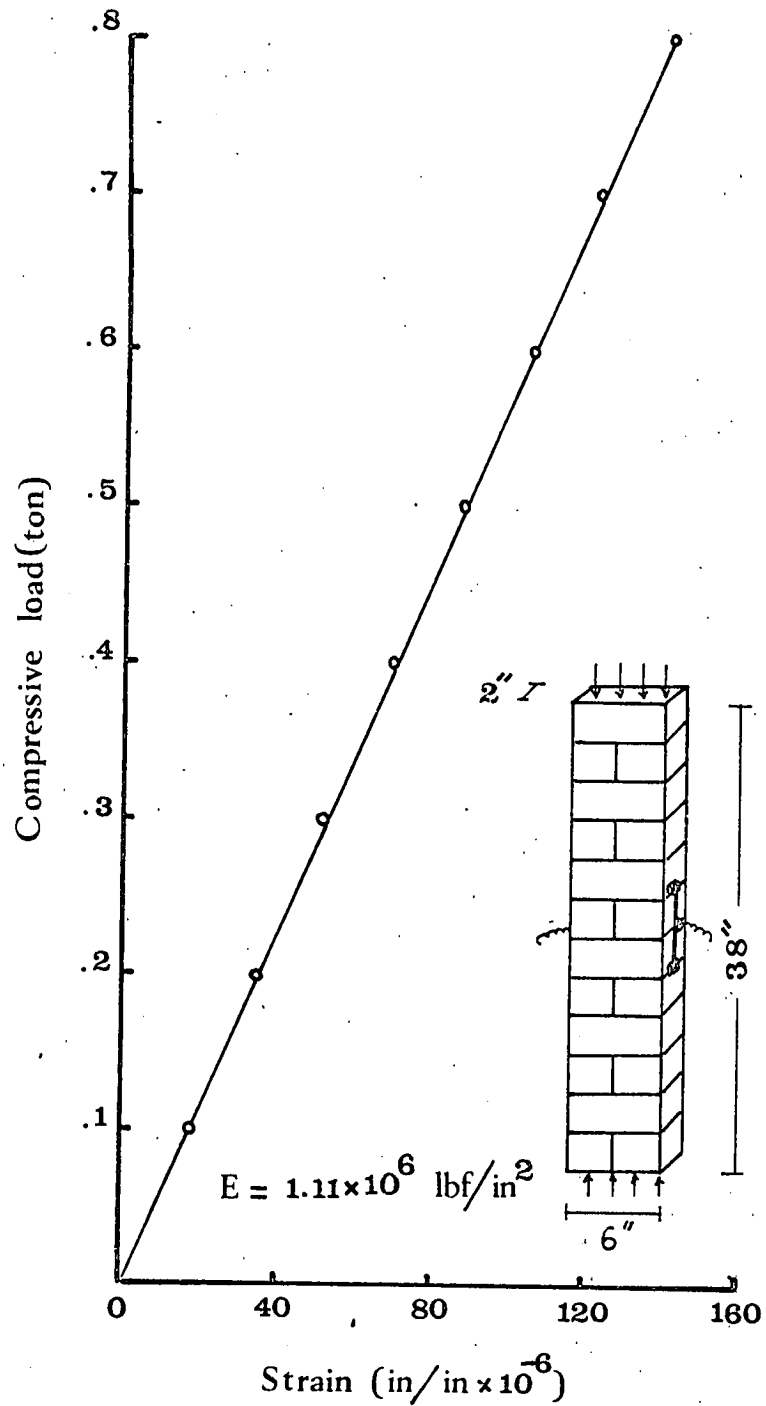


Fig. 4.14 Masonry beam under compression

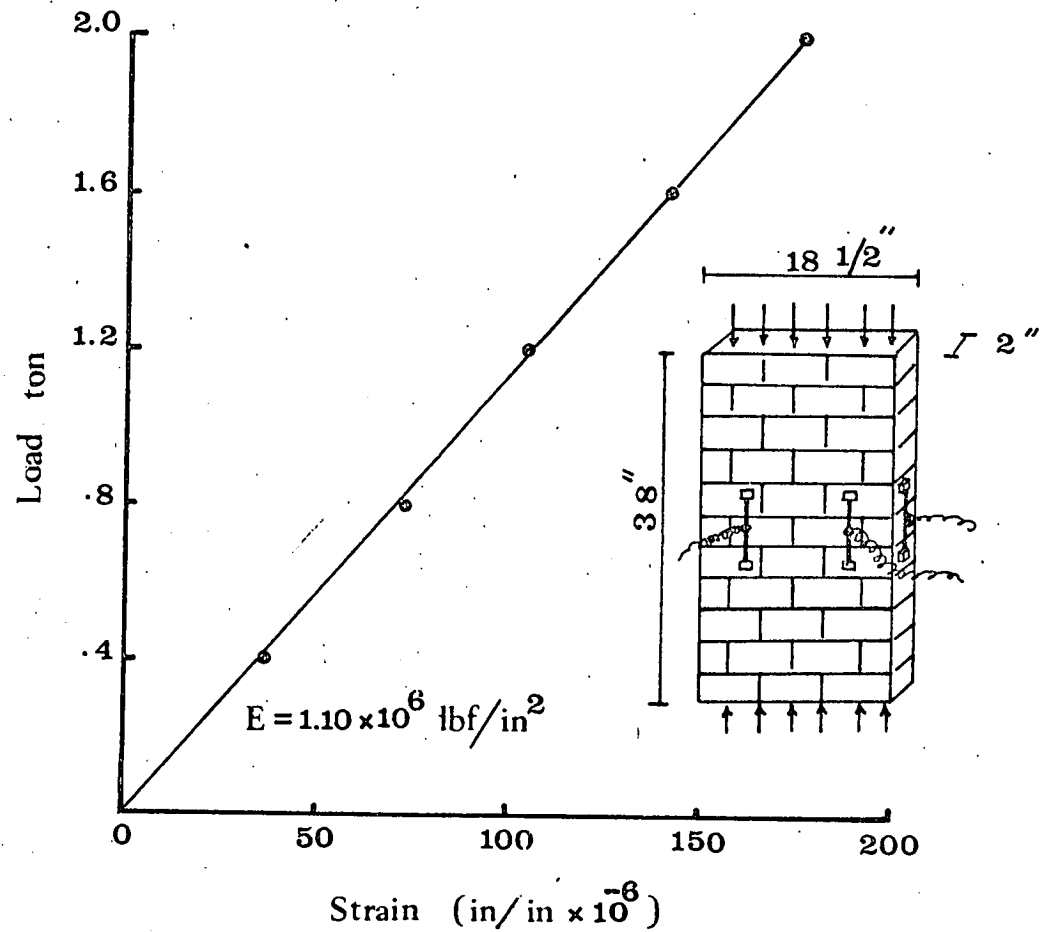


Fig. 4.15 Masonry wall under compression

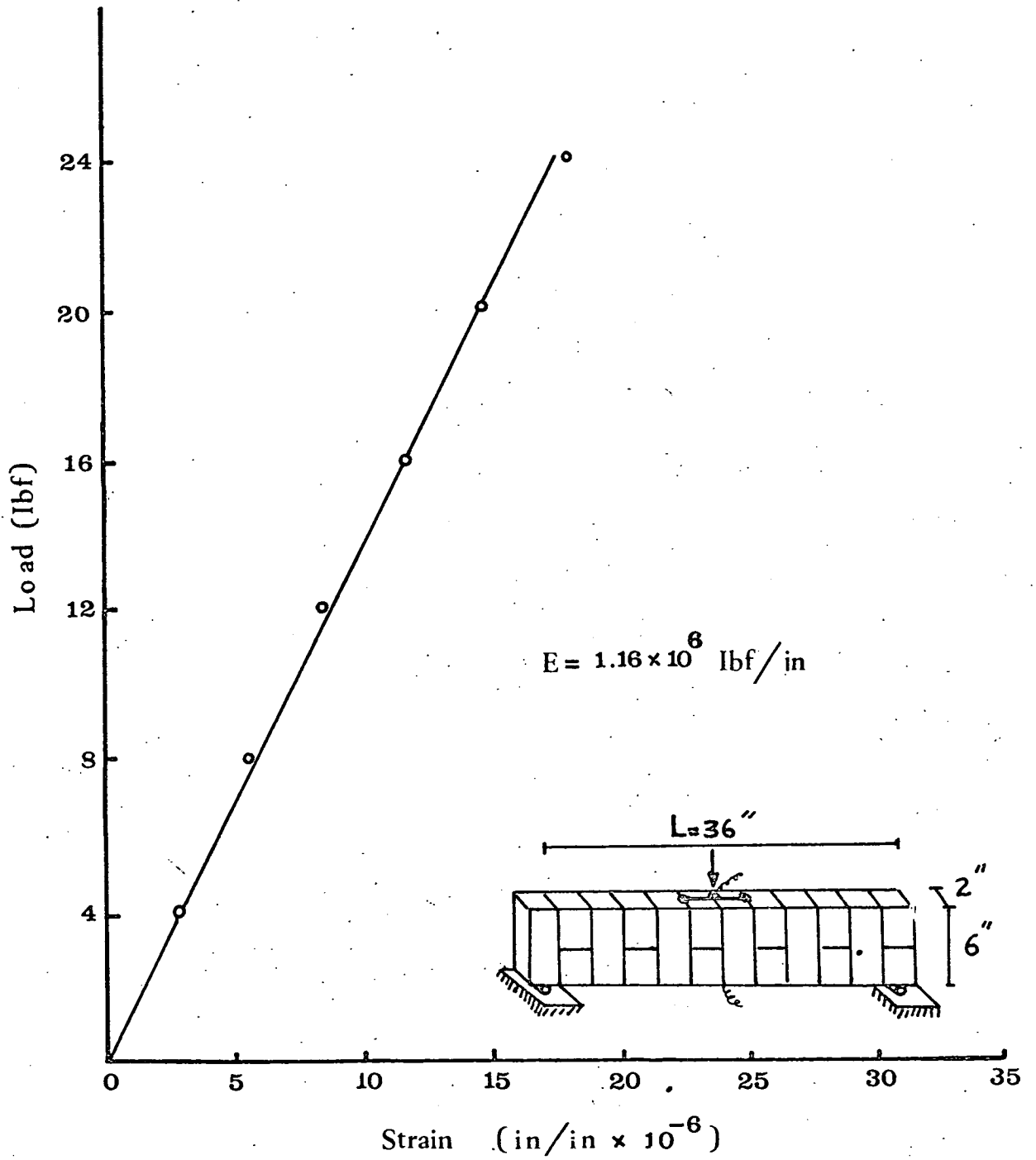


Fig.4.16 Load-Strain Curve for Blockwork Beam

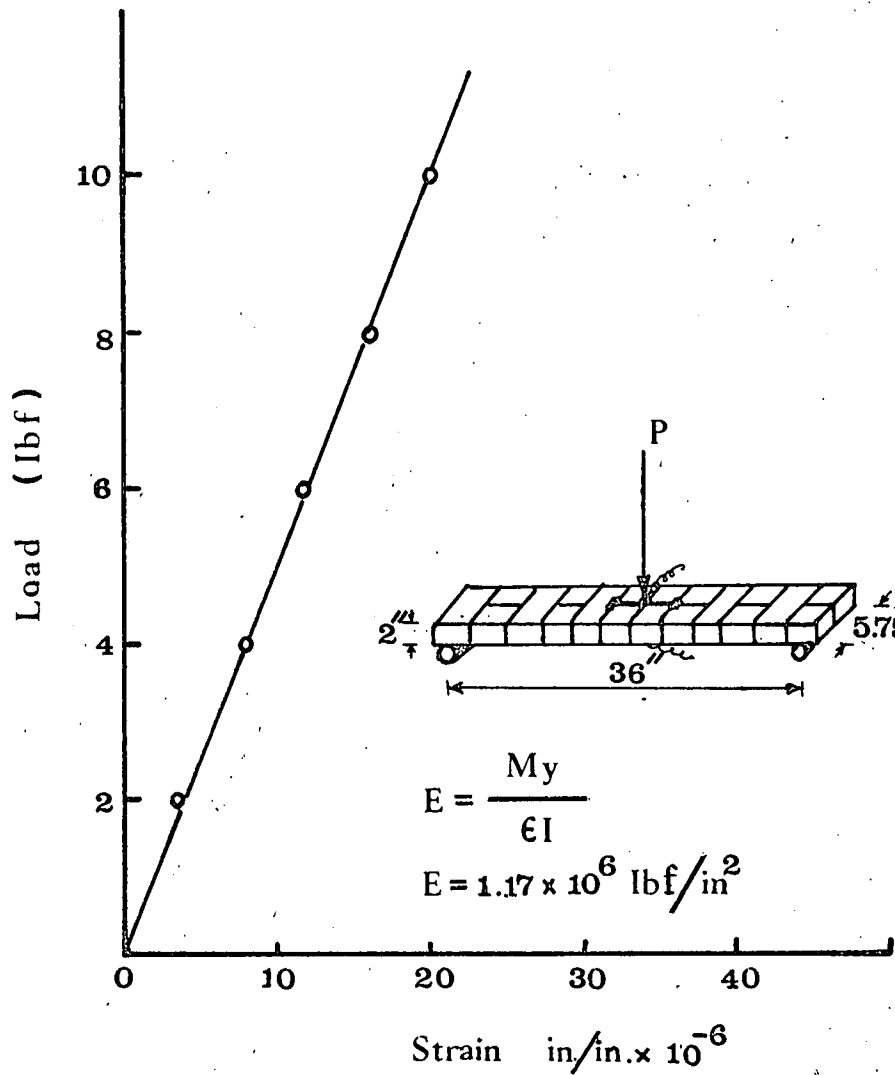


Fig. 4.17 Masonry Beam in Flexure

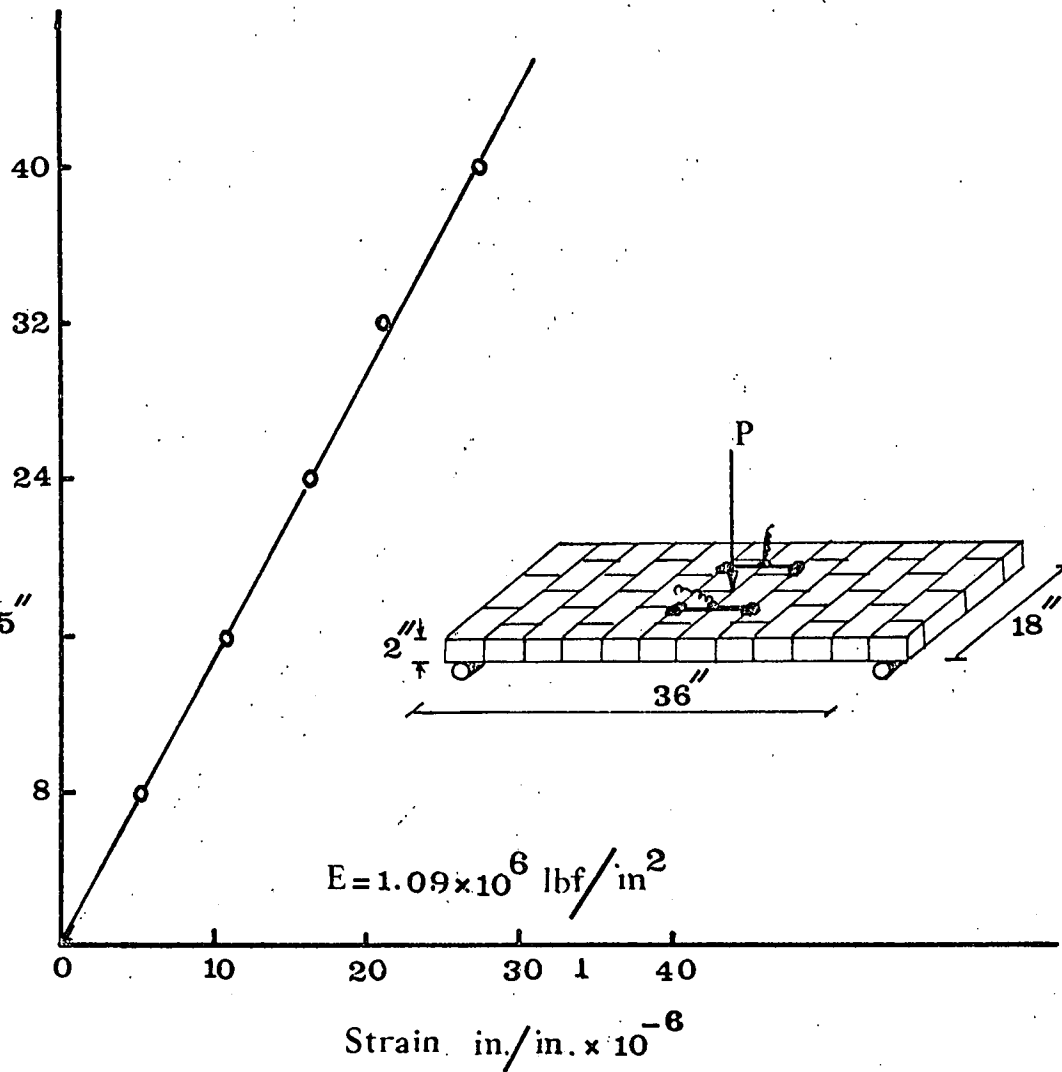


Fig. 4.18 Masonry Wall in Flexure

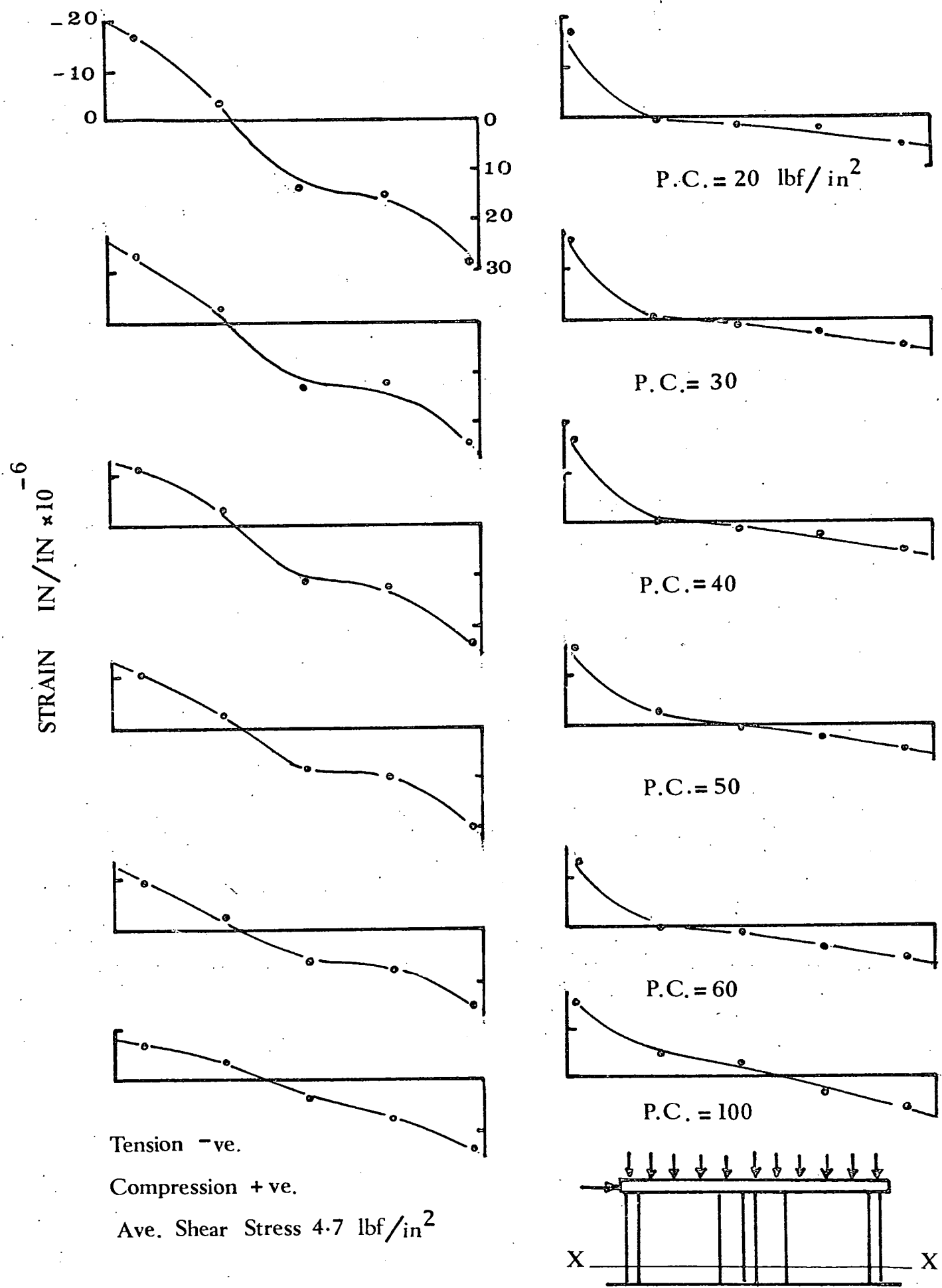
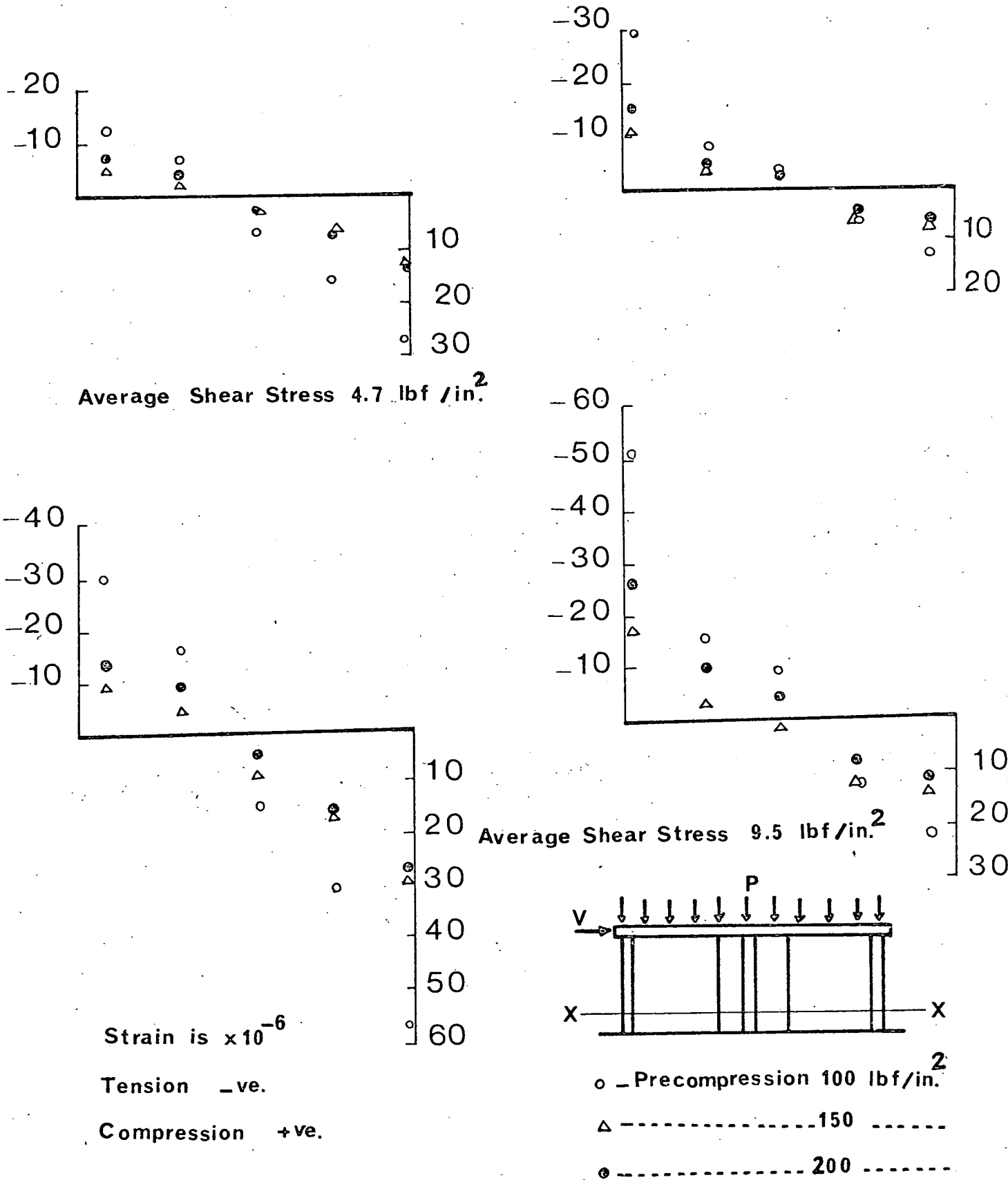
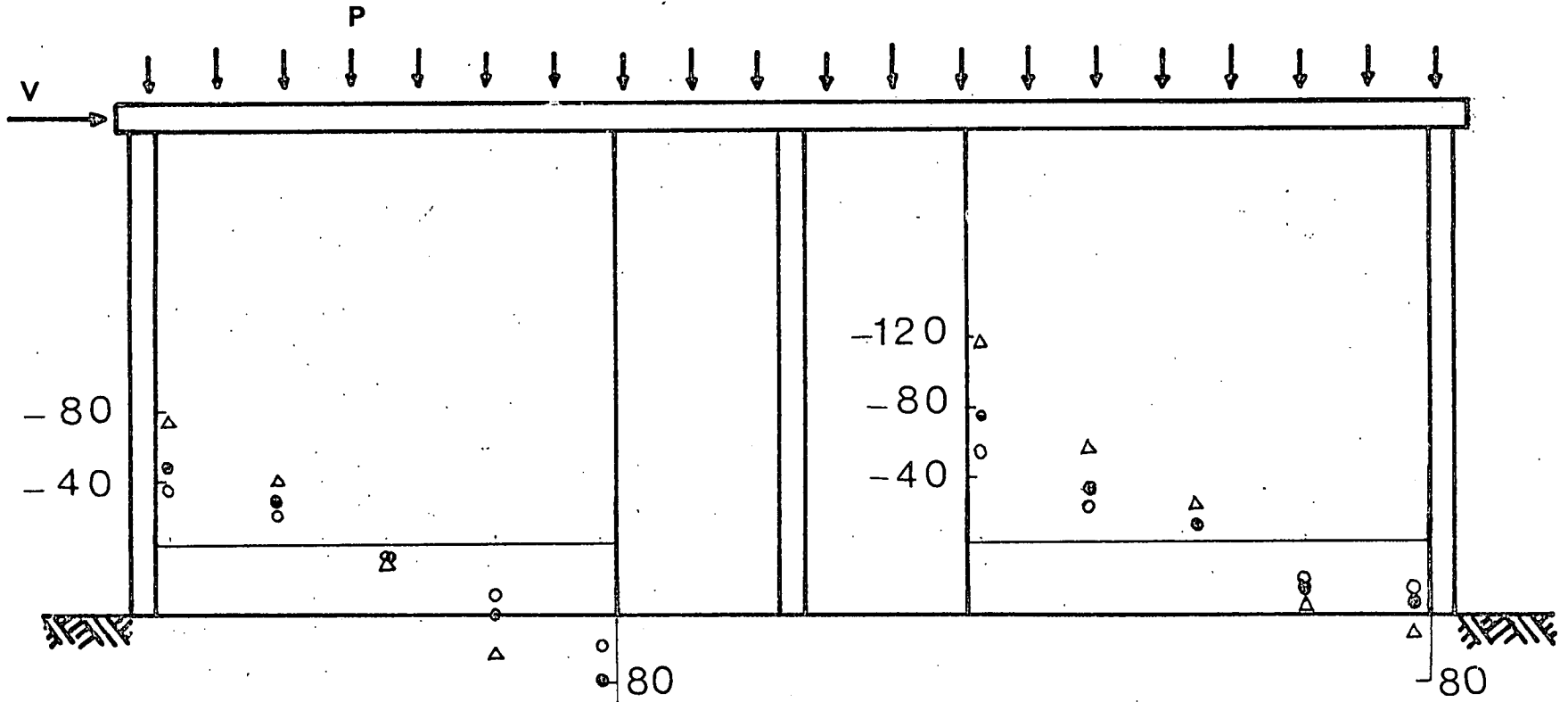


Fig. 4.19 Strain Distribution at Various Pre-Compression
 Section X-X

VERTICAL STRAIN IN THE SHEAR WALLS



STRAIN IN SHEAR WALL STRUCTURE



STRAIN IS $\times 10^{-6}$

Tension - ve.

Compression + ve.

- Precompression = 2.00 lbf/in.²
- Average Shear Stress 23 lbf/in.²
- ----- 31 -----
- △ ----- 47 -----

Fig.4.21

VERTICAL STRAIN IN THE SHEAR WALLS

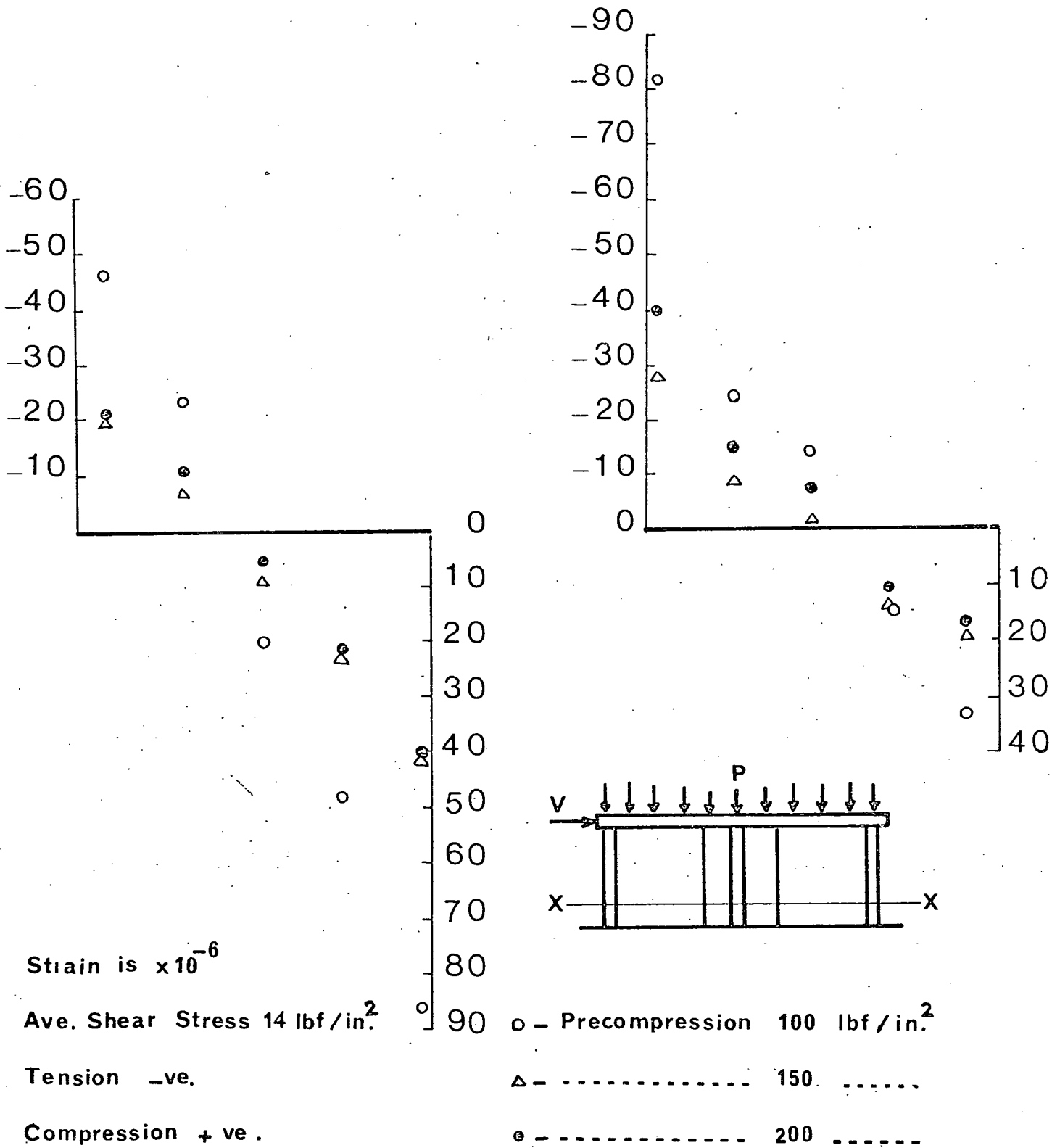
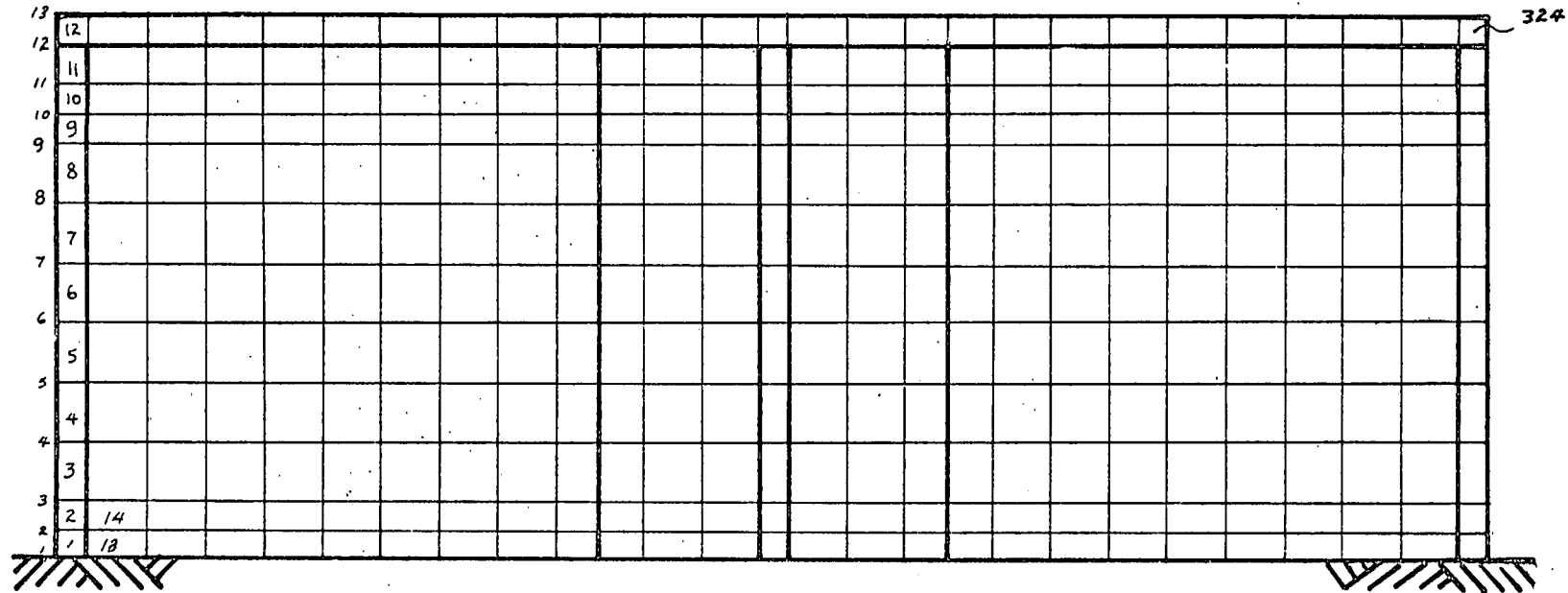


Fig. 4.22



Finite Element Representation of Single Shear Wall Structure.

$$\begin{aligned}
 E_{\text{blockwork}} &= 1.13 \text{ lbf/in}^2 \times 10^6 \\
 \nu &= 0.18 \\
 E_{\text{concrete}} &= 4.13 \text{ lbf/in}^2 \times 10^6 \\
 \nu &= 0.15
 \end{aligned}$$

FIG. 4.23

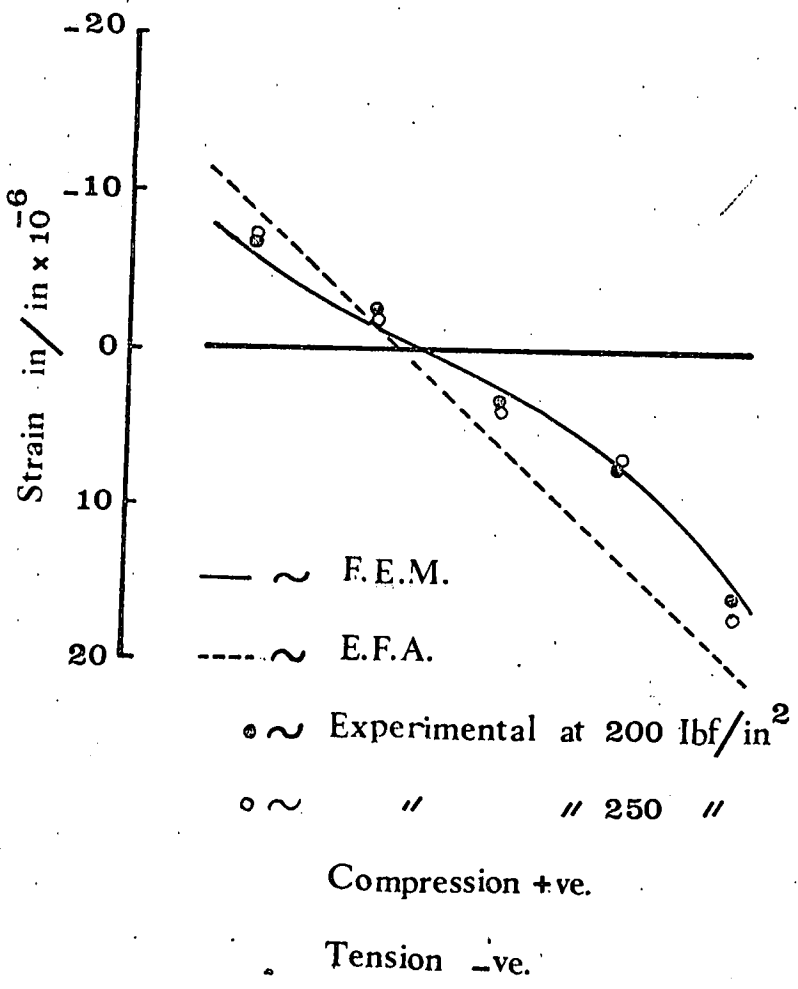


Fig. 4.24. a Strain distribution in the shear walls across section X—X.

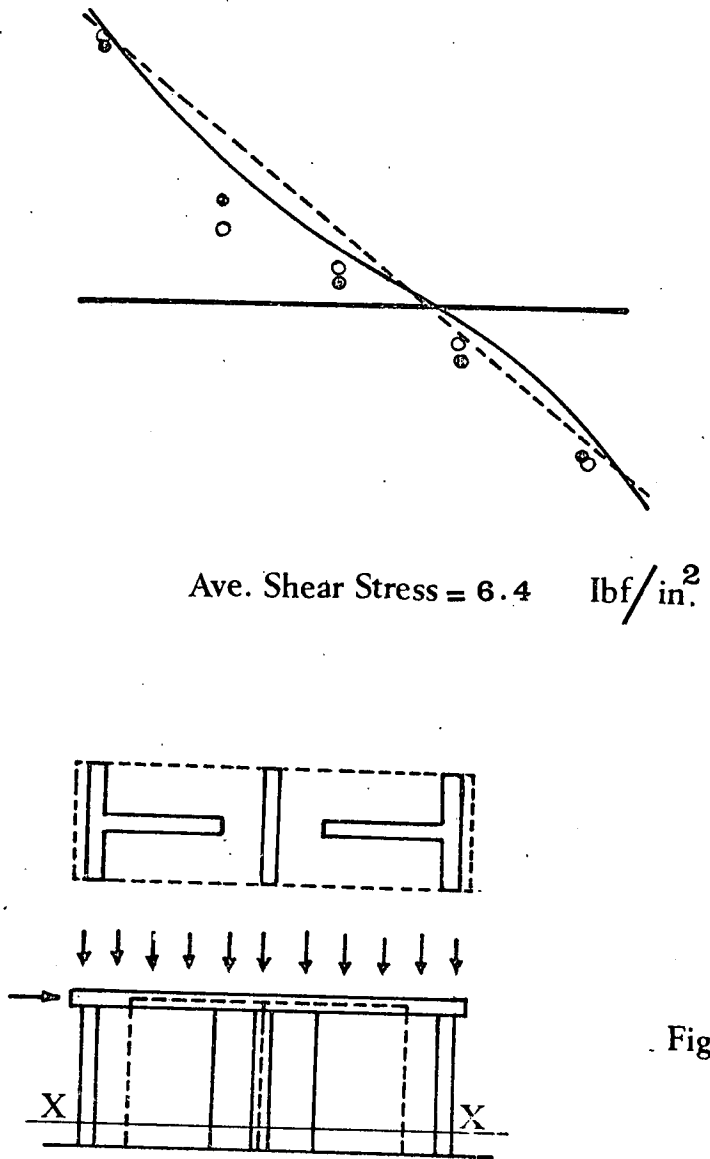


Fig. 4.24. c E. F. Representation

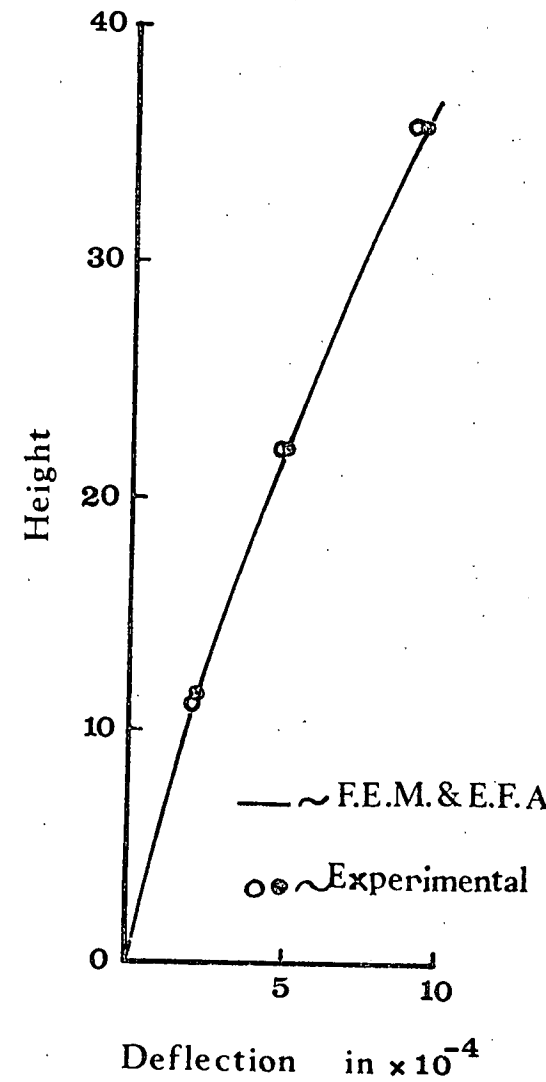
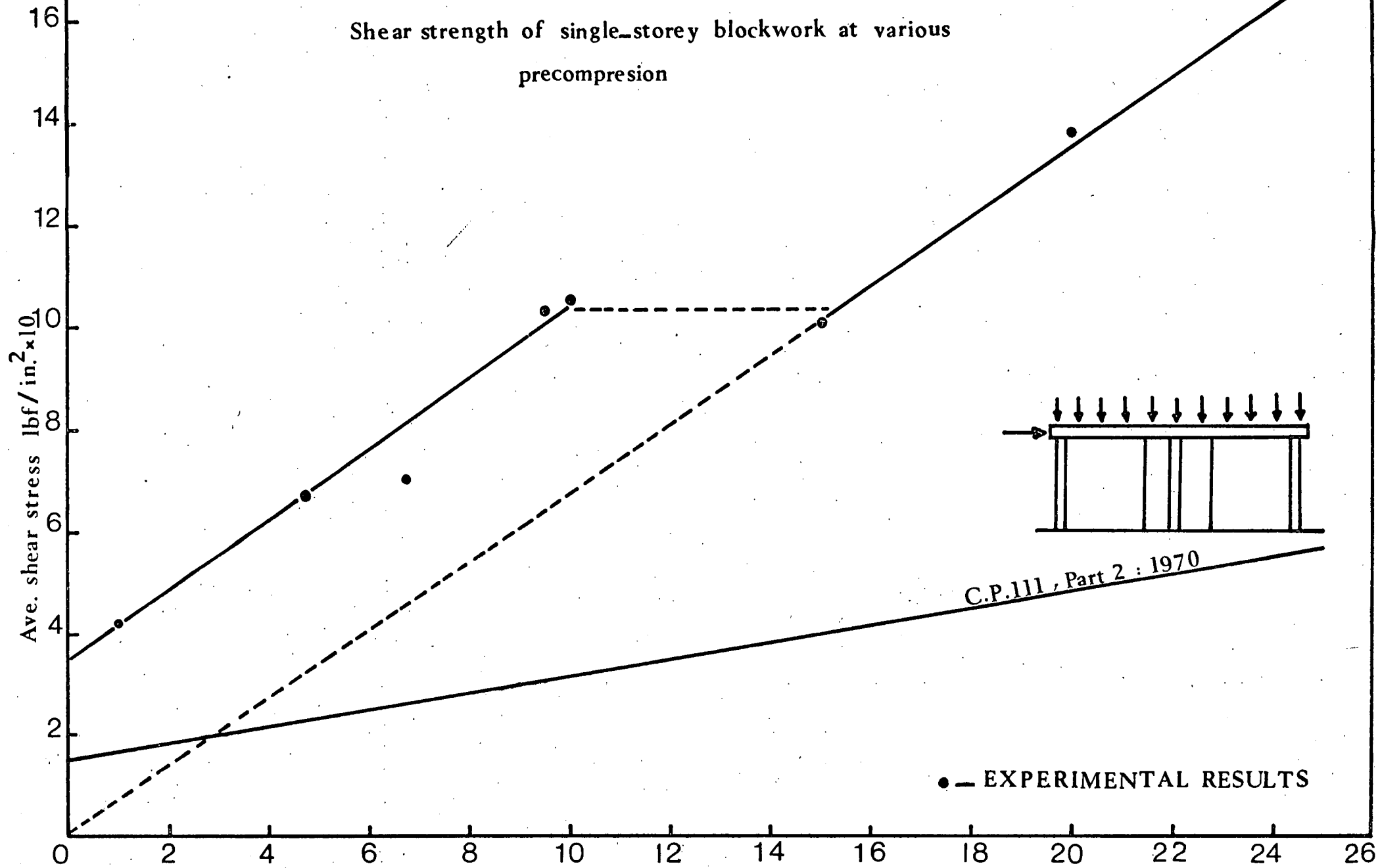


Fig. 4.24. b. Deflection of single-storey structure

Shear strength of single-storey blockwork at various precompression



C.P.111, Part 2 : 1970

● — EXPERIMENTAL RESULTS

Fig. 4.25 Precompression × 10 lbf/in.²

- 2). Wall/Slab joint behaviour.
- 3). Shrinkage cracks.

The degree of fixity at the base of the structure and at the Wall/Slab joints may be affected by precompression. At low precompression only partial fixity exists. The fixity at the base and wall/slab joints increases with increasing precompression tending to full fixity at high precompression.

The analysis is based on the following assumptions:

- a). Materials used in construction are assumed to be elastic and homogeneous.
- b). Full fixity is assumed at the base of the structure at high precompression.

Fig. (4.24) compares the experimental and theoretical results obtained by the finite element method (F.E.M.) and the equivalent frame method (E.F.M) (See Chapter 5). Fig.(4.23) shows the finite element representation of the single storey structure; a total of 325 rectangular elements were used. Results are in a good agreement. At the lower precompression, for the reasons explained earlier, the behaviour of the structure can not be predicted by these methods and further work is needed to ascertain the behaviour of the structure at lower precompression.

4.20 CONCLUSIONS

- 1) Failure of all structures under precompression initiate by formation of hair cracks at about 60 to 70 % of ultimate racking load.
2. At lower precompressions i.e. less than 100 lbf/in^2 the failure of storey/

of storey-height shear wall structure under racking load is due to breakdown of shear and frictional bond in the interface of block and mortar joints. Cracks stepping down through vertical and horizontal mortar joints.

3. At higher precompressions ranging approximately from 100 to 150 lbf/in², the failure is due to diagonal tension leading to diagonal cracks passing through mortar and blocks and occasionally mortar joints. Ultimate shear strength of the structure being approximately constant in this range. More test results are needed to confirm the present results.
4. At high precompression the shear strength of blockwork increases linearly with precompression. Cracks pass through the blocks and across the main diagonals and failure may depend on diagonal tensile strength of blocks.
5. The vertical strain distribution across various horizontal sections in the shear walls were non-linear.
6. The deflections at the openings of the structure under racking load were greater than the outside deflections by about 25%.
7. Assuming a value of 0.73 for the coefficient of friction the initial bond shear was estimated to be 35 lbf/in².
8. The rigidity of the structure increases with the increase in precompression up to a certain limit; the limit of precompression appears to be about 150 lbf/in².
9. Comparison of ultimate shear strength obtained from the test results with the permissible stresses according to the CP111:Part 2:1970 and its 1971 amendment, gives safety factors for all structures ranging from 2.54 to 3.35.
10. The value/

10. The value of the modulus of elasticity obtained from strain measurement in piers and walls under compressive and flexural loading was found to be 1.13×10^6 lbf/in².
11. Finite element method and equivalent frame analogy both gave good agreement with the experimental results at high precompression.

CHAPTER 5 - 'HIGH-RISE' MASONRY SHEAR-WALL STRUCTURES

5.1 INTRODUCTION

An indication that the 'multi-storey masonry building' has established itself as an adequate answer to certain specific problems and demands, at the same time proving itself to be economically viable, can be seen in the increasing rate of construction of multi-storey buildings in this medium all over the world.

In Great Britain it is only during the last two decades that increasing land value and the density of populations surrounding expanding industrial areas, have encouraged the erection of high buildings as a solution to these problems. Therefore, the design and constructional techniques of multi-storey shear wall buildings are relatively new. A minimum of useful floor area for letting to tenants is necessary if an adequate return of the capital invested is expected. The method of construction will have a marked effect on the time of completion, the total cost and the actual structural behaviour of the building.

Depending on the structural behaviour, economical factors and the design and the use of the building, modern buildings may be divided into three basic types:

- (1) Skeleton Framed Structure
- (2) Shear Wall Structure
- (3) Composite structure i.e. a combination of the above two types.

The most economical structural system for a multi-storey building is not always produced by the use of skeleton frame infilled with substantial non load-bearing wall areas. Where floor and roof loads are carried via beams, girders and columns, to the ground, unnecessary severe/

unnecessary severe concentration of gravity stresses may result from bending, torsion and compression of relatively slender prismatic members. Steel and reinforced concrete with their physical properties have been extremely successful in meeting the structural requirements of many framed structures. However, it is apparent that there exists a substantial class of buildings in which the desirability of fixed repetitive partitions makes it possible to build the structures without needlessly concentrating gravity loads. This is achieved by utilising the potential structural participation of all masonry wall areas acting as thin vertical plates. Such plate elements when designed to act jointly with floors and roofs, participating as horizontal diaphragms are effective in resisting not only gravity loads but also lateral forces arising from wind, earthquake and blast.

The masonry walls are designed to resist compressive, shear and bending stresses. Floors and walls act together in a composite manner. Lateral forces are transmitted, by diaphragm action, from the exterior walls via floor slabs to shear walls which in many cases are bearing walls. In order to serve as a diaphragm the floor system must be sufficiently rigid and connections must be adequate to transfer these forces to the shear wall.

In this case use of the shear wall elements to resist the lateral forces avoids severe structural requirements at girder column connection and in the columns themselves, due to horizontal loading. The amount of vertical loading occurring in the walls of buildings depends on a number of factors which are associated to a large extent with/

extent with the design and the use of the building.

A basic factor affecting the use of loadbearing masonry is that the higher the building the more important it is to have relatively short floor spans and light floor loadings.

The ultimate height of these buildings depends on a number of factors:-

- (1) The shear and compressive strengths of the masonry walls.
- (2) The spacing of intersecting walls and floors.
- (3) The connection of the above to each other.
- (4) The member stiffness in all directions.
- (5) The layout of the structures.

In the composite type of construction which utilizes both shear walls and frame, the two components act in conjunction with each other. Shear-resistant component in this system take up the horizontal loads, and the gravity loads (dead + live loads) are carried jointly by both shear walls and the frame.

5.2 Advantages of 'high-rise' Concrete Masonry Shear-Wall Structures

The principal advantage of 'high-rise' load bearing concrete masonry structures is its economic viability as a result of the speed and simplicity of construction. Construction of this type of structure involves the erection of a series of single-storeys, one on top of the other, each having the same floor plan throughout the entire height of the structure.

A six to ten-storey building can be completed easily in six to eight months (61,101). Maximum job efficiency is achieved using this system, large scaffoldings and their 'off and on' problem is eliminated. There are no columns or beams to project into rooms to create a problem. Elimination of beams may permit a reduction in floor to floor height/

Floor height of a foot or more. This could add up to the reduced

building height of 10 feet in an eight to ten storey building.

A further example of the advantage of shear walls is that they

function to transfer the loads from a building superstructure

into the supporting soil avoiding any load concentration. Thus,

the structural requirements are less demanding in relation to the

flow of floor and roof loads into the ground in the frameless type

of building as opposed to skeleton frame structures.

Floor slabs may be precast at ground level and lifted into place.

The slab functions as a ceiling for one storey and a floor for the

storey above. In this system each precast floor provides a work

platform for the next storey. Fewer work trades are required, and

finishing operation can be completed quickly and easily in the

enclosed building.

For lateral stability, the 'high-rise' load bearing masonry structure

is dependent primarily on two factors; on the composite behaviour

of all members, e.g. walls and floors, and on the plan of the structure.

Hence this system is particularly suited to multi-cellular types of

structure such as apartments, hotels, offices, halls of residence

and other similar types of construction designed to accommodate a

large number of people comfortably and economically.

5.3 Review of Some Theoretical and Experimental Work on Shear-Wall Structures

In recent years, reinforced or unreinforced masonry shear wall

structures have been recognized to be very satisfactory in resisting

large lateral forces produced by the action of wind, earthquakes

and blasts. The exact analysis of this type of structure is

difficult due/

difficult due to the three dimensional nature of the problem. In addition, masonry is a composite material, being anisotropic, non-homogeneous and perhaps cracked. Thus, the structural behaviour of masonry shear wall structure may not be similar to the structures constructed with elastic, homogeneous and isotropic material.

Further, stress distribution and deflections may not necessarily be linear and their prediction with any degree of certainty is difficult. However, by neglecting the orthogonal interaction between the various components and idealizing the system as two-dimensional structure, the complexity could be reduced and a solution may be obtained for the design.

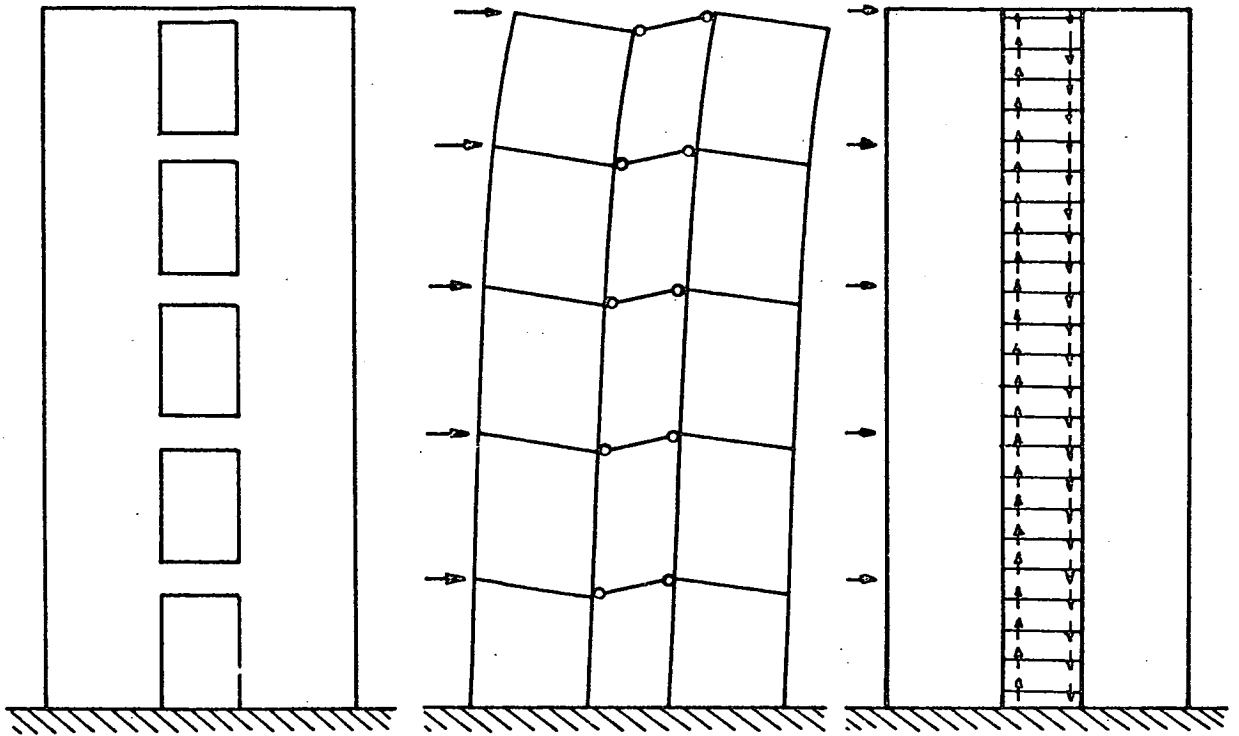
The following methods have generally been used for the analysis of the idealized shear wall structure containing openings.

5.3.1 Simple Cantilevers

Generally the stiffness of the slab or the connecting beams is very small in comparison to that of the wall, hence the slab or the connecting beam is assumed to act as pin-connected strut, Fig. (5b). Thus the walls are treated as simple cantilevers which resist the wind loading in proportion to their flexural rigidities. This procedure ignores the coupling effect of the connecting beams and underestimates the stiffness of the structure.

5.3.2 Continuum Approach

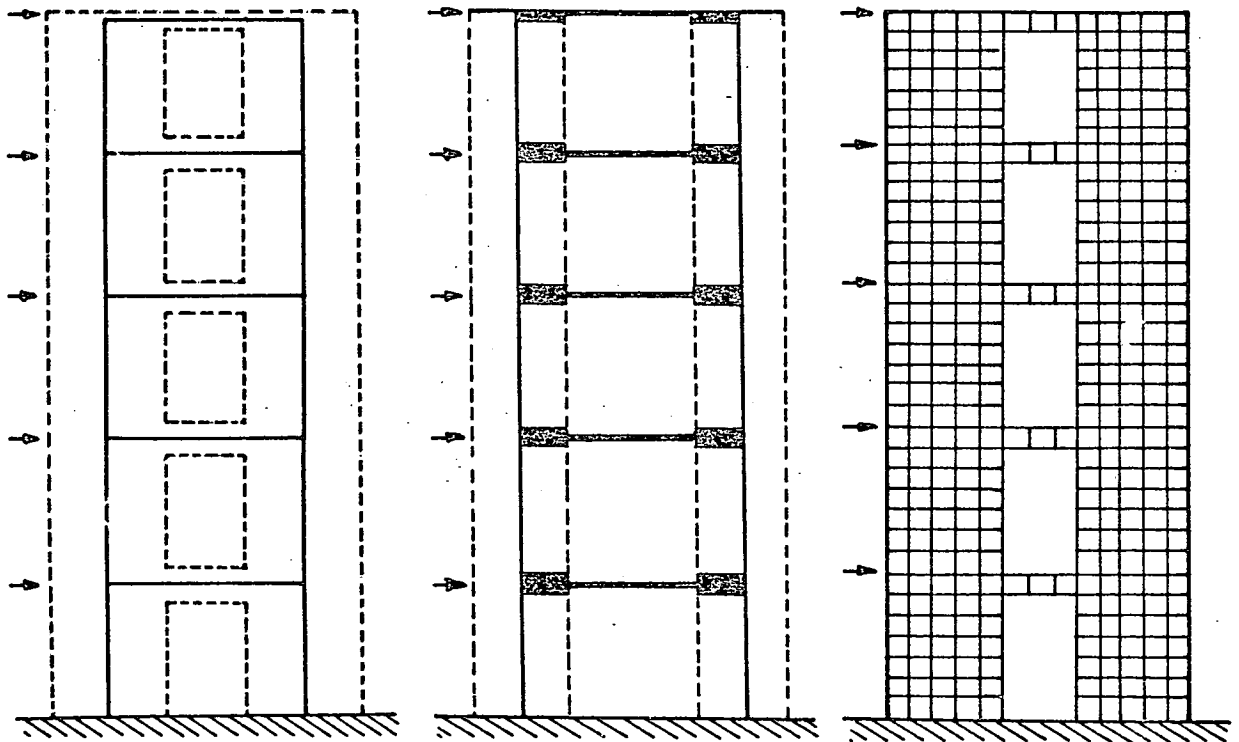
This technique was originally suggested by Pippard and was applied by Chithey and Wan^(20,21) for the analysis of multi-storey building frames. Using this method, the analysis assumes that the elastic structural properties of the system remains constant throughout. The discrete system of connections formed by lintel beams or floor/



a: Shear wall with openings

b: Cantilever method.

c: Shear Connection



d - Frame Analogy

e - W.C.F.A.

f - Finite Element

Fig. 5 Theoretical methods for the estimation of wind stresses and deflection.

or floor slabs are replaced by equivalent continuous medium, Fig. (5c). Further, assumptions are made that the beams have a point of contraflexure in the centre and their axial and shear deformations are negligible. Under lateral loading, the walls deflect and induce shear forces in the laminae. The system can then be expressed as a single second order differential equation and is solved to give shears, moments and deformations throughout the walls. Several papers use this approach with differing choice of variables, all yielding essentially the same results.

An approach based on the theory of composite beams with elastic connections was used by Schulz⁽⁹²⁾; Axial forces in the two walls were considered as a redundant function. Magnus's⁽⁶⁴⁾ redundant function was the variable strain in the walls. Eriksson⁽³⁴⁾, Beck⁽⁴⁾, and Rosman⁽⁸⁶⁾ considered the shear forces in the connecting media as the statically indeterminate function and obtained solution. Soane⁽⁹⁸⁾ used analogue simulation to solve the differential equation suggested by Eriksson. Various authors have further modified the method to take account of relative movement between walls due to non rigid foundation, asymmetrical and variable wall cross-section. Coull⁽²⁷⁾, Burns⁽¹⁸⁾, and Rosman^(87,88) have given solutions for varying wall thicknesses. Accounts of variation in lower storey and foundation conditions have been taken by Rosman and Magnus. For most cases of loading solutions are available and design charts have been produced for rapid computation of wall bending moments, deflections, and shear forces in the connecting beams⁽²⁸⁾.

Coull and Puri⁽²⁹⁾ extended the analysis, taking into consideration the deformation/

the deformation contributed by the shear force in the wall. They concluded that the effect of shearing deformation was very small and can generally be neglected.

Barnard and Schwaighofer⁽³⁾ conducted model studies on a 1/64 scale coupled-shear wall model made of epoxy sheets and demonstrated that the entire slab width is to be considered as effective in coupling the shear walls. Inelastic analysis of coupled-shear walls was undertaken by Winokur and Gluck⁽¹⁰⁵⁾. Michael⁽⁷¹⁾ considered local wall deformations and the effects of these deformations were calculated as reduction factors for the beam stiffnesses, and the variations of the reduction factors with the geometric proportions of the beam were presented as graphs.

So far, the continuum approach has received a great deal of attention and would seem to have reached a stage where further developments in analytical methods, while of theoretical interest, are not likely to result in greatly improved design techniques.

5.3.3 Lattice Analogy

Hrennikoff⁽⁴⁸⁾ and McHenry⁽⁷⁰⁾ independently derived a system of plain stress analysis based on a simple pin-jointed framework. This method which is termed the grid or lattice analogy entails replacing the wall by an analogous patterned grid or framework whose members are proportioned so that under load the grid or frame deform similarly to the original wall. McCormick⁽⁶⁶⁾ introduced a similar analysis in which the bending stiffness of the external members of the grid is included as an extra parameter.

Grinter's grid analogy method⁽⁴¹⁾ idealises the actual structure as a square grid framework consisting of members with rigid joints.

Kazimi⁽⁵⁴⁾/

Kazimi⁽⁵⁴⁾ has applied Grinter's approach to shear wall structures with openings to predict the stress distribution in the shear walls under lateral loading.

5.3.4 Frame Analogies

Structures have been idealized as:

(a) 'Equivalent Frame' or (b) Wide Column Frame.

Green⁽⁴⁰⁾ treated the multi-storey coupled shear walls with deep members as an 'equivalent frame' by assuming a point of contraflexure at the mid-point of the piers and spandrels. He used the 'portal method' of analysis with a modified element stiffness to take account of shear, as well as bending deformations. Lateral forces were distributed among the shear walls by assuming the floor slabs were rigid in their own plane, and a method was developed to give the redistribution of forces which must occur if all walls are to have the same deflection throughout the building, irrespective of stiffness variations. Green, in his analysis, neglected the effects of axial forces in the walls.

Amaratunga⁽¹⁾ used the 'equivalent frame' approach and flexibility method of analysis. In this approach the shear-wall structure is replaced by an 'equivalent frame' (Fig. 5d) with columns having the same flexural rigidities as the walls. The beam lengths are equal to the distance between the centroidal axis of the adjacent columns and having the same stiffness as the actual beam or interconnecting slab. This system may take into account axial and shear deformations in the walls. Amaratunga compared his analytical results with those obtained from photoelastic investigations, using araldite models, and found good agreement between stresses for sections away from zones of stress concentration and re-entrant corners. His deflection results were obtained from/

obtained from perspex model tests and when compared with the theoretical results it was generally found that in using the 'equivalent frame' method deflections were overestimated.

Candy⁽¹⁹⁾ analysed the coupled shear-wall structure using the 'equivalent frame' technique and stiffness analysis, taking into account the bending, axial, and shear deformations of the walls.

Frischmann, Prabhu and Toppler⁽³⁶⁾ used two different approaches, both based on methods of analysis for rigidly jointed wide-column frame-work', to deal with coupled shear walls. In the first approach, known as the 'equivalent column method', the structure is replaced by an equivalent single column having the same stiffness as the summation of all column stiffnesses, and with a single beam restraining moment applied at each floor level. The problem is reduced to a single second order differential equation; by using a continuous distribution of restraining moments and loads, the column bending moments in the continuous system could then be evaluated. The basic requirement of this method is that throughout the height of the building, the moments inertia of all beams in each bay, and throughout each column, should be uniform. Their second approach employed the wide column frame analogy; the method of 'influence coefficients'⁽⁷³⁾ was used to obtain a solution. In this approach, which is basically a modification of the equivalent frame method, the beam length is taken to be the clear distance between the adjacent walls Fig. (53).

It is assumed that between the geometric centre line and the face of the shear wall, the moment of inertia of the connecting member is infinity. Account is taken of the effect of finite deflection at the/

at the ends of the beam due to the wall rotation. The analysis neglects the deformations due to axial and shear forces and therefore underestimates deflections. The calculation in both frame analogies can take into account changes in beam and wall dimensions, storey height and concrete strength.

Hussein⁽⁵⁰⁾ using photo-elastic technique investigated the deflection and stresses in the multi-storey shear wall structures. He used shear connection and wide column frame methods to analyse the experimental results.

Macleod⁽⁶²⁾ using 'wide column frame' technique, analysed coupled-shear walls by considering shear, axial and bending deformations in the members and assuming rigid connections at beam-wall joints.

He showed that the axial deformation in the beams had insignificant and negligible effect on the lateral deflection of the walls however, the corresponding effect of the wall was significant.

Macleod⁽⁶³⁾ also used the 'Finite Element' method (2.2) for the analysis of coupled shear walls with relatively stiff beams. This approach provides a powerful analytical solution and could deal with most structures. In this method shear walls, beams or connecting slabs are idealised as a system of two or three dimensional elements, figure (5f), being connected at their nodes and compatible along their edges. Application of equilibrium equations to the forces acting at these nodal points will lead to similar procedure for that of the stiffness matrix analysis of a frame. Generally the accuracy of the method depends on the types of the elements and the fineness of the mesh used which in turn affects the computer running cost. Finite element method/

element method can take into account variation in the materials properties, thickness of shear walls, geometry of openings and irregularities in loading.

For the analysis of shear walls with slender connecting slabs or lintel beams Macleod (63,49) idealized the structure^{as} combination of shear wall (divided into rectangular finite elements having two degrees of translational freedom at each node), and the lintel beams as a line element in bending (with an extra degree of freedom which corresponds to a rotation at a node point).

Momet et al⁽⁷²⁾ also solved shear wall structures, using a similar procedure to that of Macleod.

Girjavallabhan⁽³⁸⁾ used both triangular and rectangular elements for the analysis of shear wall structures. He found that the rectangular elements gave more accurate solution and were also more convenient to use. He further analysed a six-storey building⁽³⁹⁾, and demonstrated that the stiffness of the floor slabs or the lintel beams significantly influenced the distribution of stresses and deformation in the shear walls. In this analysis, however, the depth of lintel beams varied from one foot to six feet and the exaggerated stiffness of the beams slightly modified the overall behaviour of the wall.

5.4 Review of Experimental Work on Masonry Shear-Wall Structures

A survey of the literature related to the behaviour of isolated reinforced and nonreinforced concrete masonry wall panels under racking load is given in Chapter two. In the following section research work on the single and multi-storey concrete and brickwork masonry shear wall structures with and without openings is reviewed.

In order/

In order to investigate the effect of various types of reinforcement round the openings and their influence on the strength and stiffness of shear walls, Benjamin and Williams⁽⁵⁻⁷⁾ tested a number of models and full-scale brick and reinforced concrete shear walls. Based on their work and from their mathematical analysis in the elastic range, using lattice analogy and an approximate approach, they arrived at two methods of solution for the analysis of shear walls with and without openings. The first method is tedious and the second considers shear and bending deformations. It is assumed that the perforated wall is built up of pier and spandrel elements, the former undergoing bending, shear and axial deformations, while the latter only bending and shear. By assuming further that the piers bend with a point of contraflexure at their mid-height, the stress distribution throughout the entire structure may be determined.

Murthy⁽⁷⁴⁾ conducted a series of racking tests on 1/6 - scale single storey and on a 3-bay, 3 storey high brickwork to investigate the rigidity of the structure under lateral loading. The shear walls were incorporated at different stages of the test and it was found that the rigidity increased to 10.4 times the initial rigidity in the absence of the shear walls.

Sinha⁽⁹⁶⁾ following Murthy's work conducted tests on storey-height shear walls with openings, and on a five-storey 1/6-scale cross-wall structure. Under lateral load, the failure of multi-storey brick cross-wall structures takes place in the lower most storey by cracking through vertical and horizontal mortar joints or by failure at the slab wall joint. Continuum and wide column frame methods were used to predict/

to predict the deflection of the structure. Both methods underestimated the deflection. Sinha concluded that the existing shear wall theories based on idealized structure, as they stand, cannot be applied with reasonable accuracy to calculate the rigidity of brickwork structure. A simple approach was suggested which considers the multi-storey structure as a discontinuous stack of single-storey structures held together by precompression. The method takes into account bending and shear deformation by assuming floor slabs are rigid in their own plane.

Following Sinha's work, Kalita⁽⁵²⁾ conducted tests on a simple 5-storey structure built in 1/6 scale brickwork. The purpose of the work was to investigate whether existing shear wall theories could be used to analyse multi-storey brickwork structures. Finite element technique and a simplified method based on Benjamin's method⁽⁶⁾ originally suggested by Sinha, were used. By assuming the rigidity of each storey changes with precompression, both methods gave good agreement between measured and calculated deflections. The finite element method also gave a reasonable estimate of the stresses in the shear walls. It is however, doubtful if the elastic properties of brickwork would change with precompression.

Maurenbrecher, Sinha and Hendry⁽⁶⁵⁾ report results of a series of tests conducted on a full-scale 5-story brickwork shear-wall structure subjected to lateral loading. Strains and lateral deflections were measured. The deflection results at low shear stresses up to 10 lbf/in^2 showed good agreement with the previous work on 1/6 scale brickwork⁽⁹⁷⁾. Load deflection results were non-linear. Opening deflections at the ground level were about 50% greater than/

greater than the outside deflections. Strain distribution in vertical direction and across a horizontal section in the shear walls were non-linear.

CHAPTER 6 - EXPERIMENTAL AND THEORETICAL INVESTIGATIONS OF THE
BEHAVIOUR OF A MULTI-STOREY CROSS-WALL STRUCTURE IN
BLOCKWORK SUBJECTED TO LATERAL LOADING

6.1 INTRODUCTION

For the design of multi-storey shear wall structures, methods such as those mentioned in Chapter Five may be adopted. In these approximate methods lateral moments are resisted by the shear walls in proportion to their flexural rigidities. More refined methods take into account the interaction between shear walls and the interconnecting floor slabs or beams, with the assumption of fully rigid connections between the various structural elements. The actual behaviour of a masonry structure is likely to lie between these two extremes.

The economical design of these structures, when subjected to combined lateral and vertical loads, requires a knowledge of the rigidity and stress distribution in the elements comprising the structure.

The purpose of the work presented in this Chapter is to study the behaviour of a five-storey one-third scale masonry shear wall structure. (Plate 6.1). The investigation examines the suitability of the existing shear wall theories. Analytical results from methods such as finite element, wide column frame analogy, shear connection, equivalent frame analogy and cantilever are presented and their suitability is assessed by correlating the experimental data with the theoretical results, representing the behaviour of idealized structures.

The testing programme covers building height of up to five storeys. Due to the nature of work, and other limitations, important factors such as the stiffness of the connecting beams and the effective coupling width of/

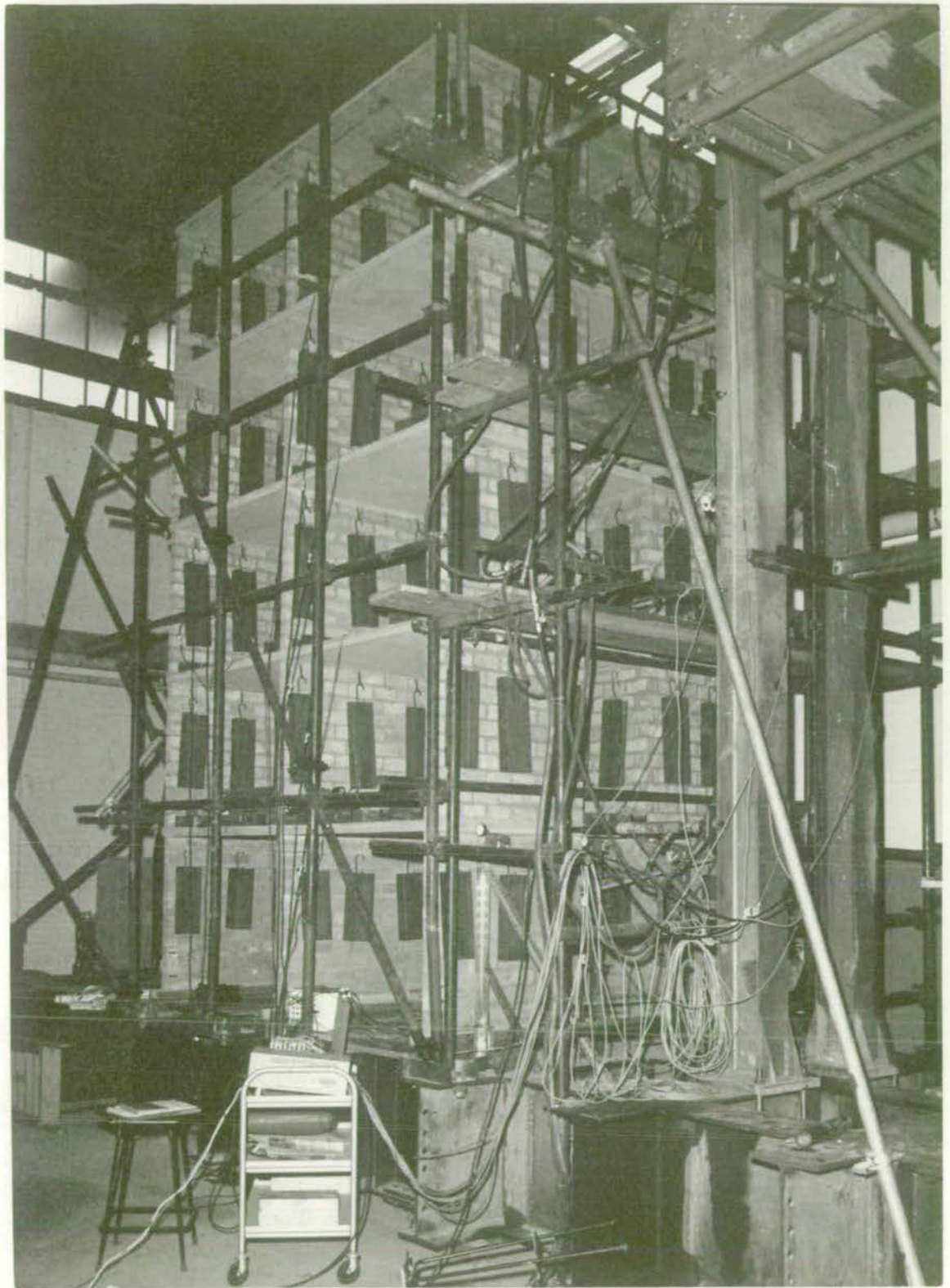
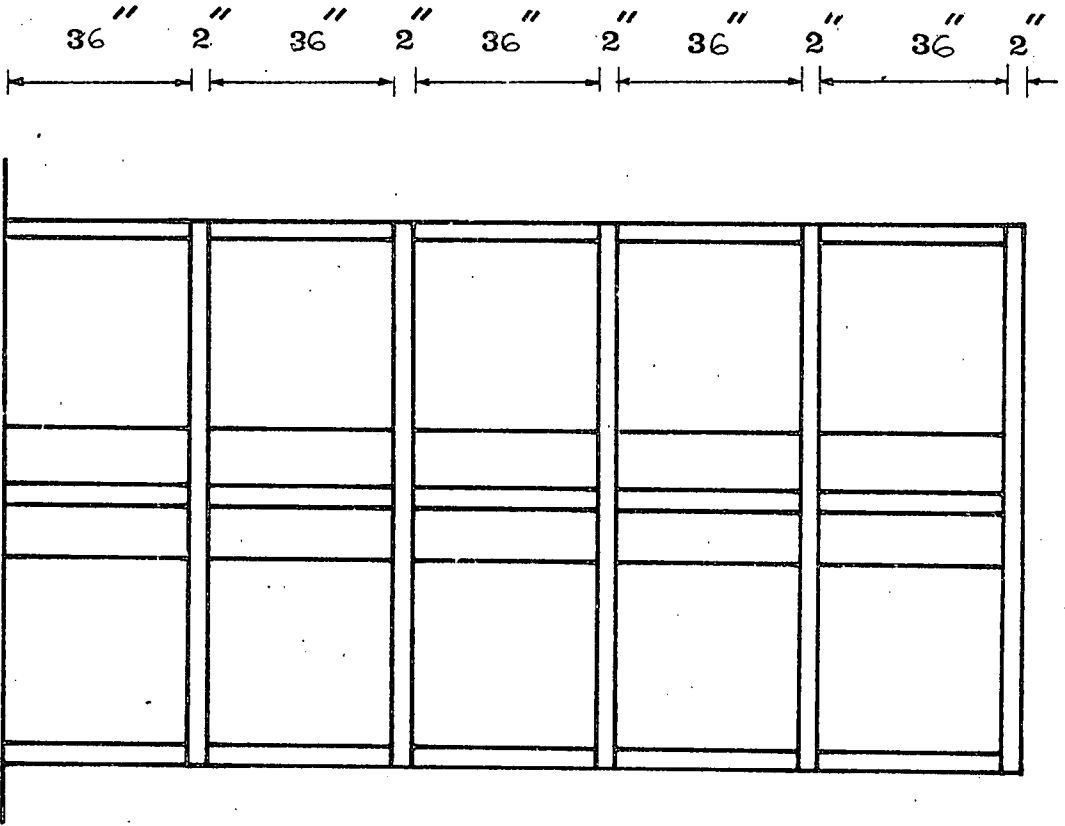
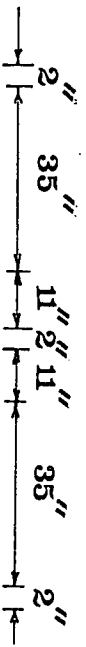
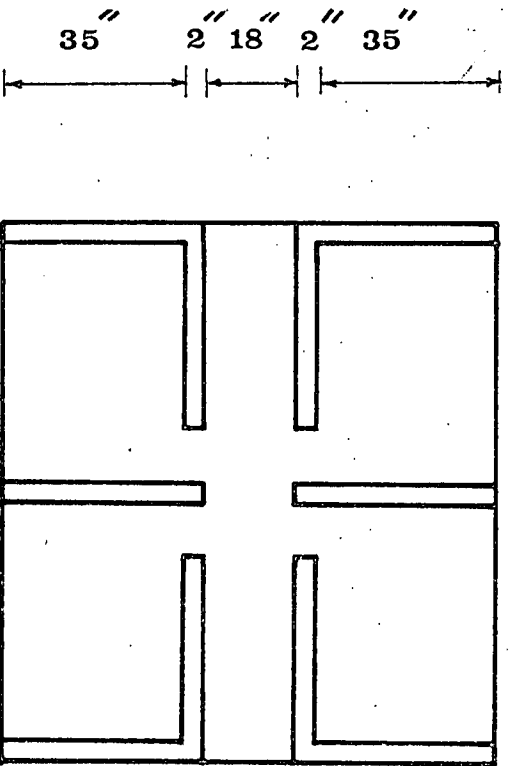


Plate 6.1 ONE-THIRD SCALE BLOCKWORK STRUCTURE

Fig. 6.1 Plan & Elevation of the Model Structure



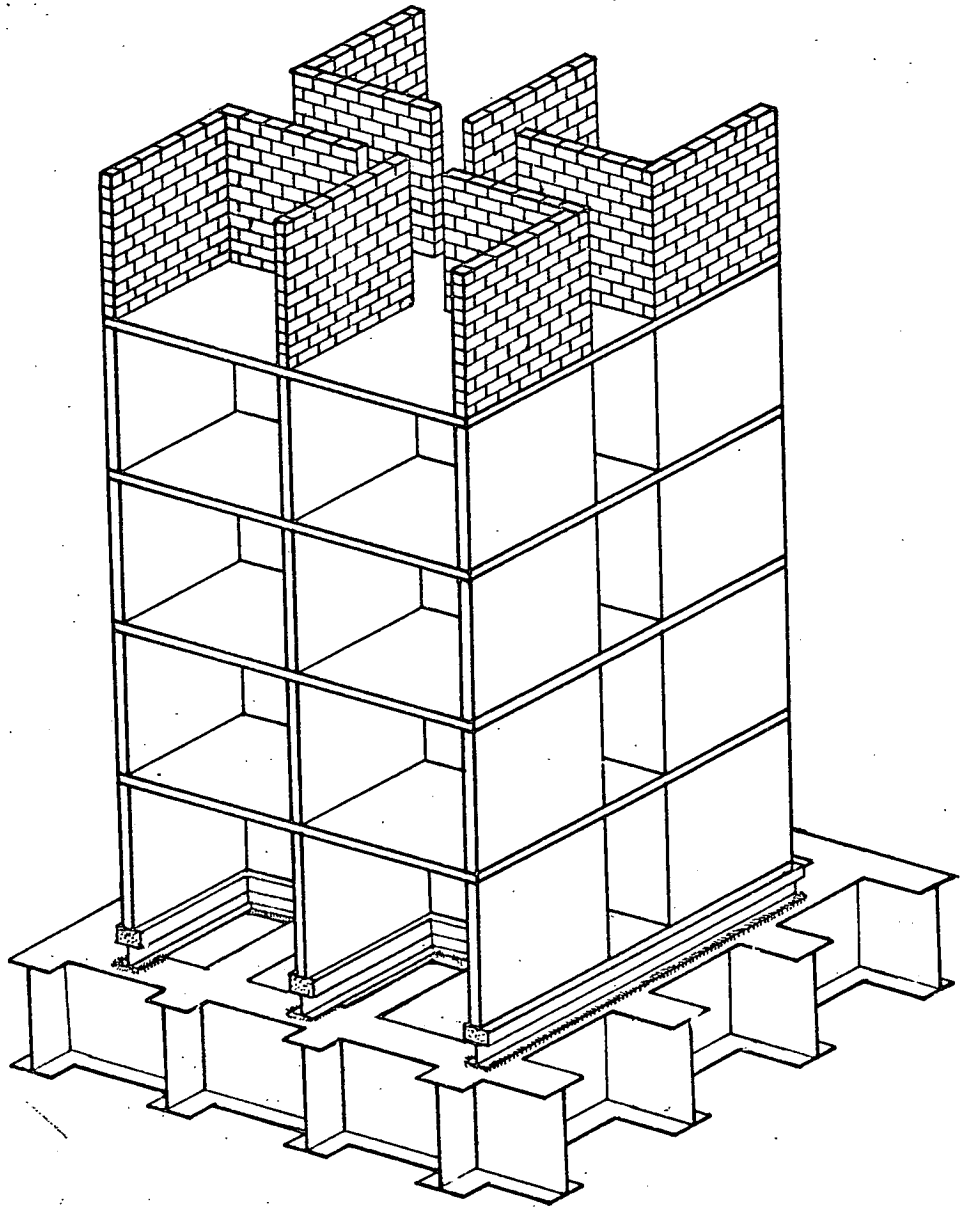


Fig- 6.2 Isometric View of the 5- Storey Shear-Wall Structure.

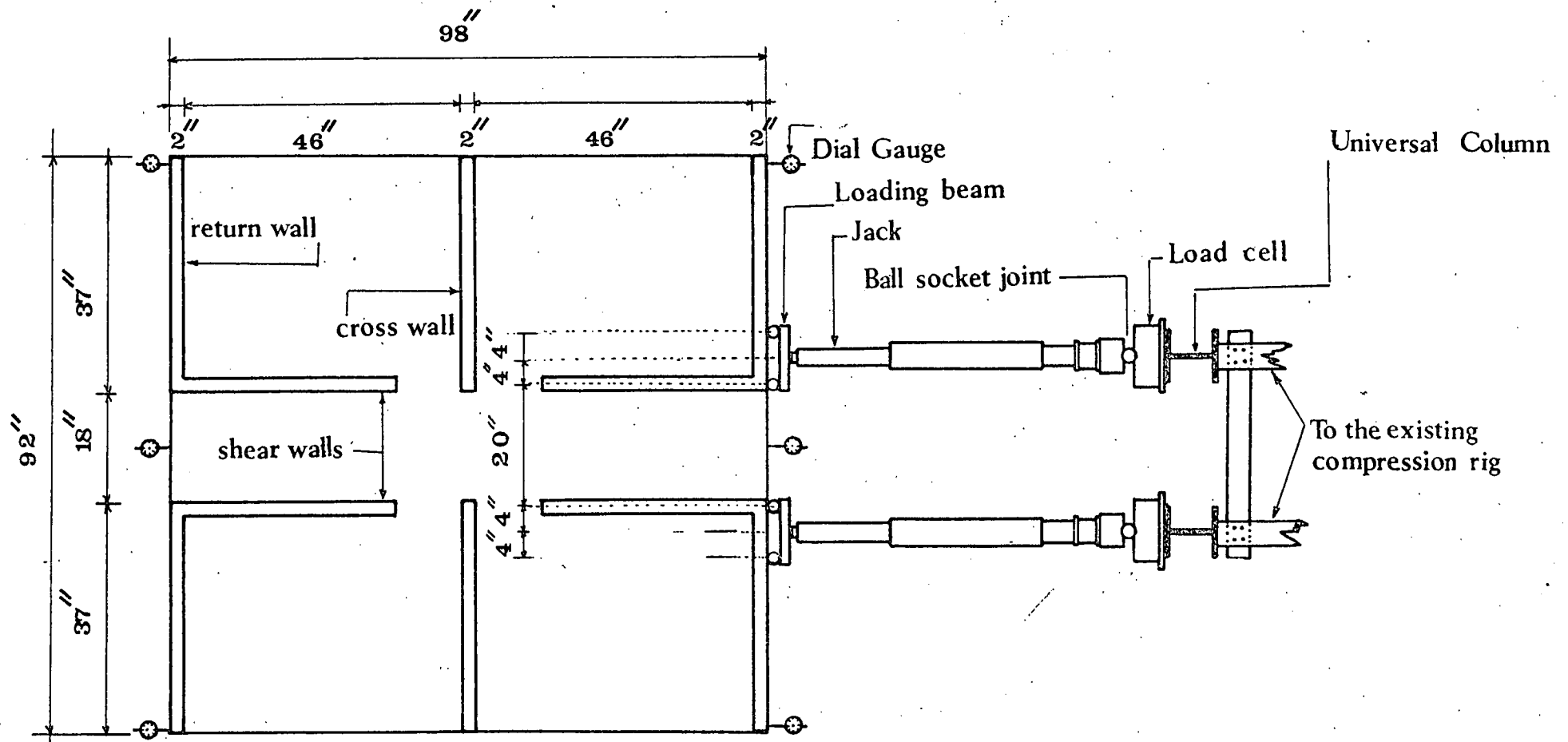


Fig. 6.3 Details of the loading arrangements.

width of the connecting slabs were not investigated.

6.2 MATERIALS

6.2.1 Blocks

One-third scale 'Aglite' blocks were used for the construction of the walls. The blocks came from two different batches. Their crushing strength is given in the following Table (6.1).

Table 6.1 - Mean crushing strength (lbf/in²) of one-third scale 'Aglite' blocks. BS 2028, 1968.

Description	Batch 1	Batch 2
Mean crushing strength	1436	1888
Standard Deviation	114	208
Coefficient of variation	8% ^{0.007}	11%

6.2.2 Sand

As described in Chapter 4, section 4.3.2.

6.2.3. Cement

As described in Chapter 4, section 4.3.3.

6.2.4 Mortar

The mortar used for the construction of all walls and for the wall/slab joints was 1: $\frac{1}{4}$:3 cement:lime:sand by volume, see Chapter 4, section 4.3.5. The mean crushing strength of 3-in mortar cubes cured alongside the walls and tested after 28 days were as follows:

Table 6.2/

TABLE 6.2 - CRUSHING STRENGTH OF 3 INCH MORTAR* CUBES

Description	No. of samples	Mean crushing strength (lbf/in ²)	standard deviation	Coefficient of variation
1st Floor	10	2235	268	12%
2nd Floor	12	2150	236	11%
3rd Floor	10	2268	227	10%
4th Floor	10	1995	160	8%
5th Floor	12	2060	103	5%

* 1:¼:3 Cement:lime:sand (by volume)

6.3 CONSTRUCTIONAL DETAILS

Figs. (6.1 & 6.2) show the plan, the elevation and an isometric view of the model structure. Each storey consists of four shear walls, two cross-walls and four external walls (flanges), all having the same dimensions.

6.3.1 Method of Construction

All walls were constructed in situ. The first course was laid in 4 x 2 inch steel channels welded to the steel base grillage through a 4 inch x 4 inch x 9 lb I-beam. Concrete was then poured round the first course forming a rigid foundation. In order to strengthen the joint between the shear wall and the external wall, these were bonded to each other at every other course. Following the construction of all walls and after 7 to 10 days curing period, a precast floor slab was lifted up by crane, and placed on uniformly laid fresh mortar bed. All gaps were filled where necessary, and the whole mortar joint was tooled.

6.3.2 Reinforced Concrete Slab

All floors consisted of 2-inch thick precast reinforced concrete. The concrete mix was 1:1:2 by weight, having a maximum aggregate size of $\frac{3}{16}$ inch. All slabs were reinforced at the top and bottom, by one inch square steel mesh of $\frac{1}{8}$ inch diameter. Four symmetrically positioned steel hooks were connected to the reinforcement to enable the slab to be lifted. The average crushing strength of 4-inch cubes for the concrete mix are listed in the following table:

Table 6.3/

Table 6.3 - Crushing Strength of 4 inch concrete cubes

Description	Crushing strength (lbf/in ²)	
1st Floor	7850	All specimens cured alongside the slab under damp sack and tested after 15 dys. Figs. are averages for six cubes
2nd Floor	7400	
3rd Floor	7920	
4th Floor	7260	
5th Floor	7735	

6.4 TESTING EQUIPMENT

6.4.1 Loading Frame

A multi-purpose testing rig (9) was used to provide a rigid base for the model structure. Two steel columns (8 in. x 8 in. x 35 lbs) were bolted to the base grillage and secured in position by joining them with I-beam (6 in. x 4½ in. x 20 lb) to the existing compression rig. Loading was applied symmetrically by two loading beams through two half-inch diameter rollers 8 inches apart, at each floor level (Fig. 6.3). The beams were attached to 10-ton hydraulic jacks which were reacting against 3-ton load cells via universal joint, the load cells being fixed to the steel columns Plate 6.3.

6.4.2 Load Measuring Apparatus

Top floor jacks were operated by a hydraulic hand-pump while the remaining floors were loaded by a 'Losenhausen' Testing Machine. The top floor load was half that of the other floors. Readings of the load cells previously calibrated were taken from a digital voltmeter reading to an accuracy of 10 microvolts equivalent to five pounds when using/

when using three ton load-cell.

6.5 SIMULATION OF DEAD LOAD STRESSES

To simulate the effect of gravity load in the prototype structure, the dead weight stresses in the model were increased by a factor of two. This was achieved by hanging weights on hooks, incorporated during construction from all walls (see plate 6.1). Weights were also placed on the floor slabs. The average precompression at the base of each storey from bottom to the top floor was then: 50, 40, 30, 20 and 10 lbf/in² respectively.

6.6 EXPERIMENTAL INVESTIGATIONS

After the completion of each storey and 28 days of curing, lateral loading tests were carried out on the following structures.

1. Single-storey.
2. Two-storey.
3. Three-storey.
4. Four-storey.
5. Five-storey.

In all tests lateral loads were limited to $\frac{1}{4}$ of the expected ultimate value.

6.6.1 Deflection Measurement

6.6.1.1 Overall Deflection

Scaffolding poles were bolted to the base of the structure and braced by horizontal and diagonal members, Plate (6.1). This provided stiff and rigid support for the dial gauges. Rotation of the base and the scaffolding was checked by 'Electrolevels' (see plate 6.2 and Appendix 1). No rotation was observed during the tests.

Using 0.0001 in. dial gauges at each load increment, deflections were measured at/

measured at the corners of the structure at each slab level. Distant dial gauges were read by telescopes. No torsion was developed during the tests. For each structure the testing procedure was repeated three times and no appreciable difference in the measured deflections were observed. Figures (6.4 to 6.7) illustrate the deflection pattern under the given racking loads for various structures. The variation of lateral deflection at each floor level with increasing horizontal load for 5-storey structure is shown in Fig. (6.8).

6.6.1.2 Deflection of the Shear Walls at Ground Floor

The lateral deflection of the shear walls at the openings and their ends were measured by three equally spaced dial gauges (Plates 6.4 and 6.5). The gauges were fixed to Dexion frames which were welded to the steel channel at the base of the structure. Figure 6.9 shows the deflection of the ground floor shear walls.

6.6.2 Strain Measurement

Vibrating wire strain gauges of $5\frac{1}{2}$ inches gauge length were used to measure the strain distribution ' ϵ_y ' in the vertical direction in both shear walls and cross walls. Due to a limited number of gauges being available advantage was taken of the symmetry of the structure and loading to reduce the number of strain measurements. The position of the gauges are shown in Plates 6.4 and 6.5 and Fig. 6.10. Figures 6.11 and 6.12 show the strain distribution ' ϵ_y ' across a horizontal section in the bottom of the single, two, three and four storey models.

Fig. 6.14 gives the strain distribution across a horizontal section in the ground/

the ground floor shear walls for 5-storey structure at lateral loads producing 6 and 12 lbf/in² average shear stress.

The strain variation in the direction of the applied lateral load along two selected sections in the floor slab is shown in Fig. 6.15. 2½-in and 5½-in. gauges were used to measure strains in the slab along the openings and along its edge, (plate 6.6).

The magnitude and the direction of the principal strains in the ground floor shear walls are presented in Fig. 6.16. Using these values the estimated principal strains trajectories are also shown as a set of orthogonal curves, one being tangential to the direction of major principal strain and the other at right angles to it.

Figures 6.17 to 6.20 illustrate the variation of strains along the height of the 5-storey structure at the selected vertical sections in the shear walls, 2 inch, 29 inch, and 34 inch from the opening respectively.

In all the above tests, the experimental data were average values of 3 sets of readings. The maximum variation between two individual sets of readings was about 4%. The following are possible sources of experimental error.

6.6.3 Sources of Experimental Error

<u>Source</u>	<u>Accuracy</u>
Measurement of Deflection	<u>± 1 x 10⁻⁴ in.</u>
Measurement of Load	<u>± 7 lbf</u>
Measurement of Strains	<u>± 3 microstrains</u>

The incomplete fixity at foundation is undetermined.

6.7/

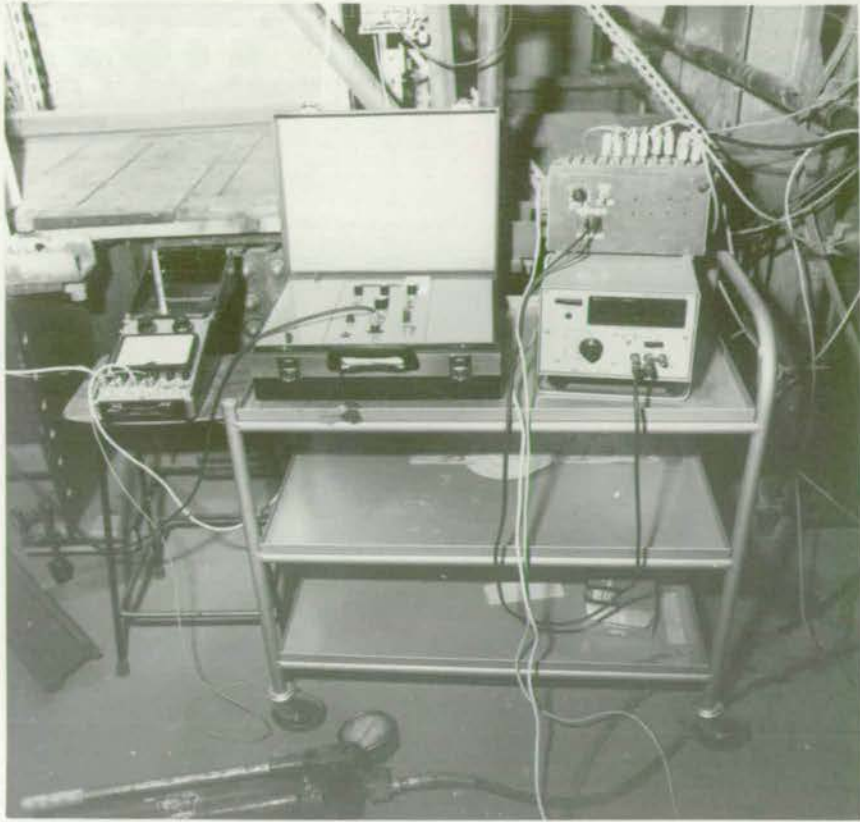


Plate 6.2 EQUIPMENTS MEASURING ROTATION, STRAIN
AND LOAD

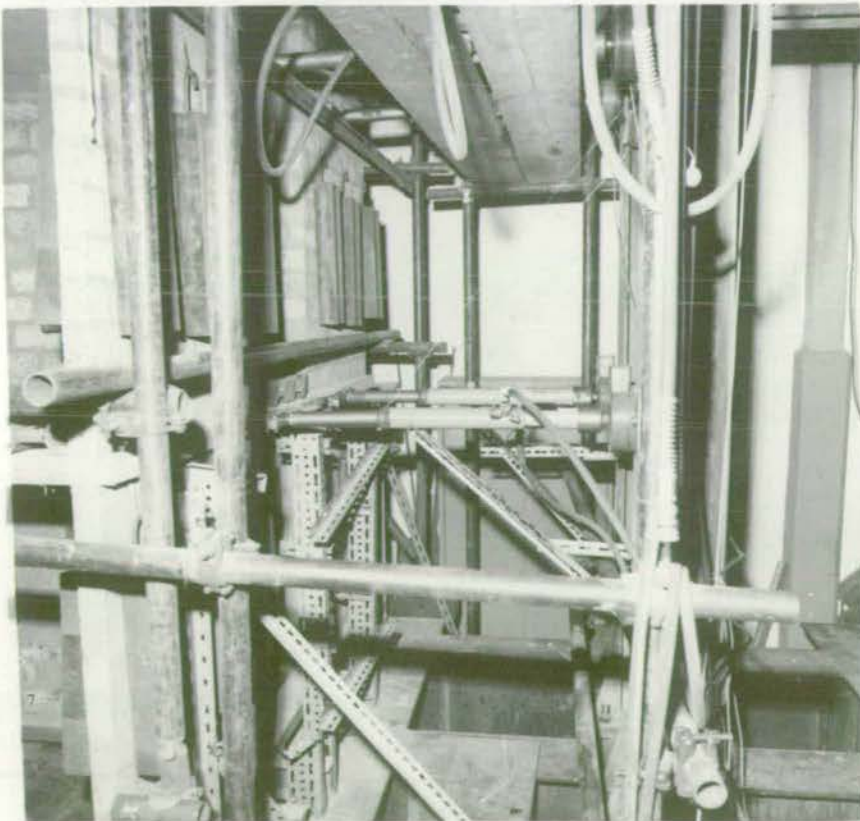


Plate 6.3 APPLICATION OF LATERAL LOAD

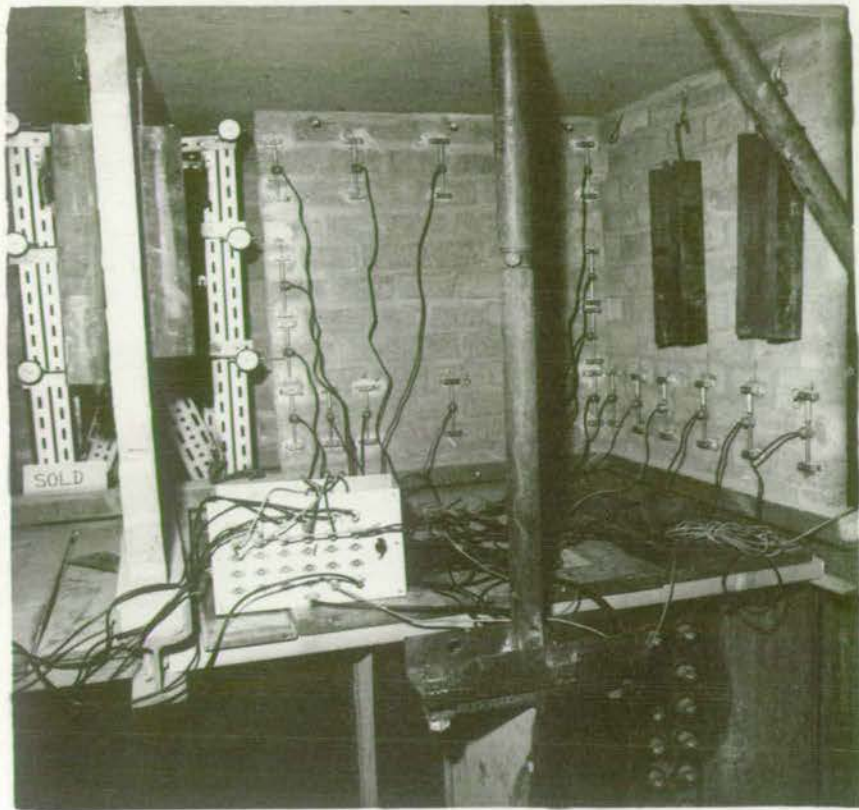


Plate 6.4 DIAL GAUGE MEASURING DEFLECTION OF
GROUND FLOOR SHEAR WALLS

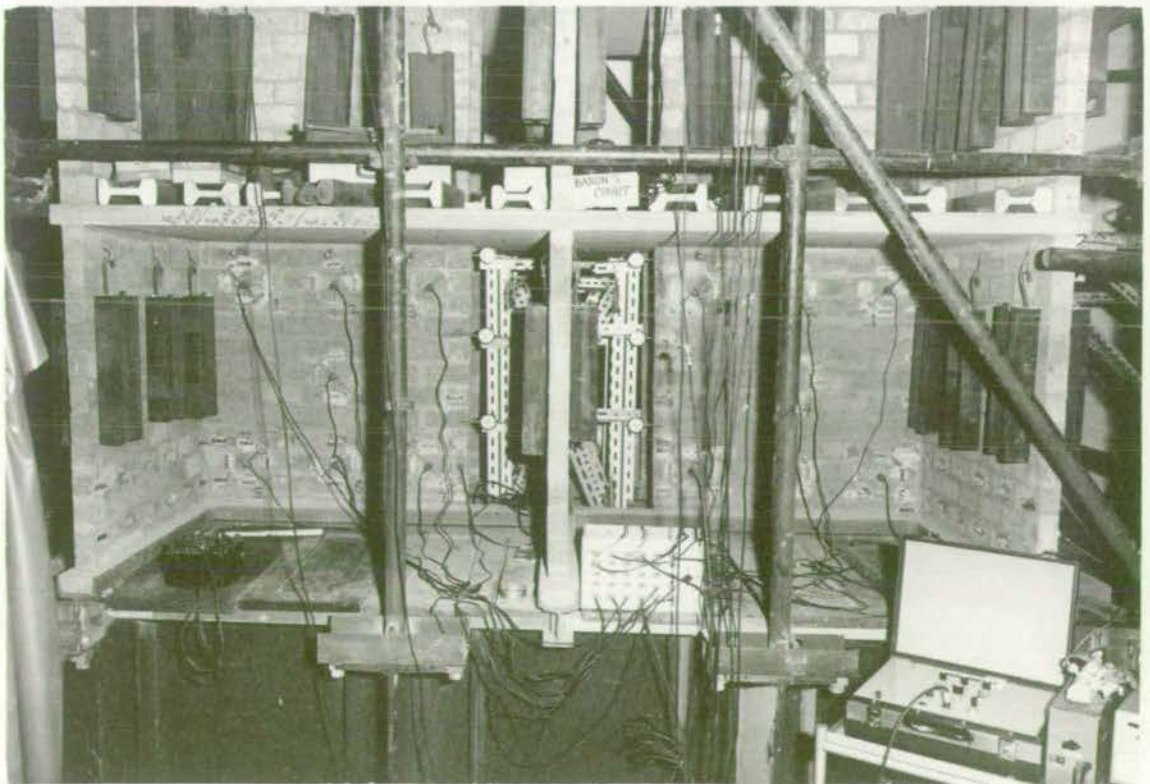


Plate 6.5 VIBRATING WIRE GAUGE MEASURING THE
PRINCIPAL STRAINS IN THE SHEAR WALLS

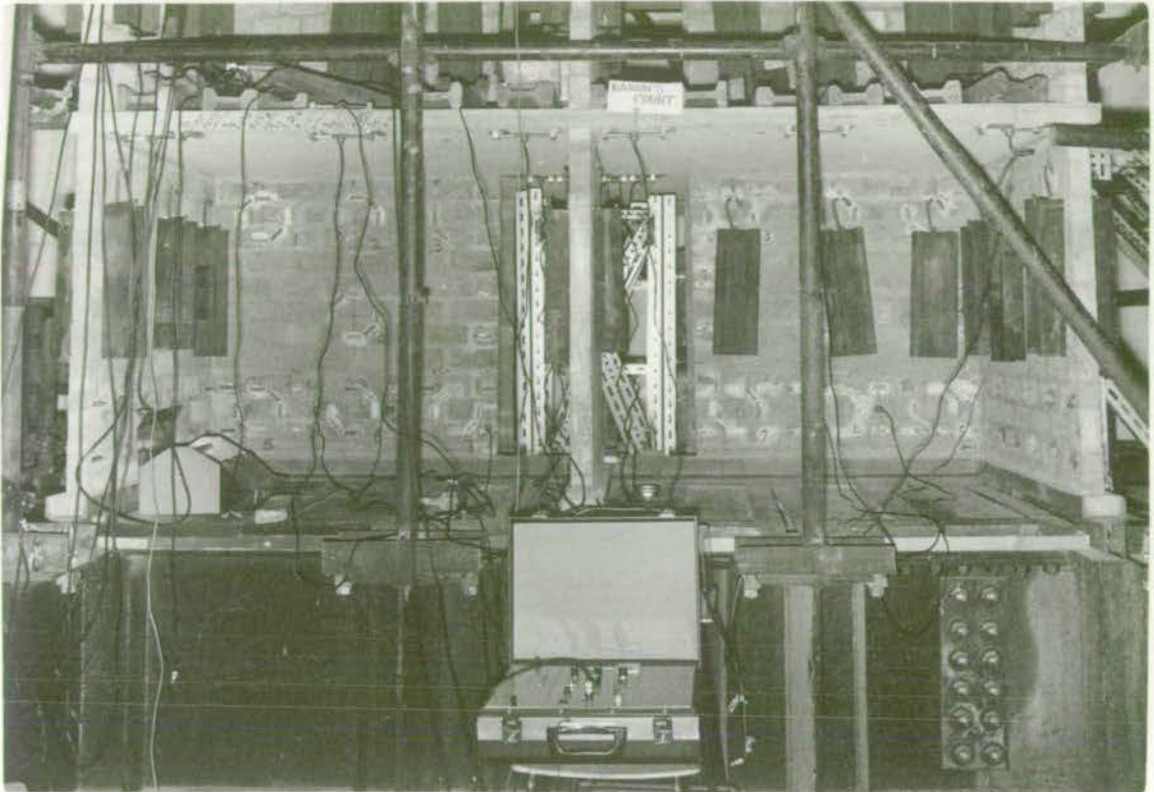


Plate 6.6 MEASUREMENT OF STRAINS IN THE FIRST FLOOR SLAB

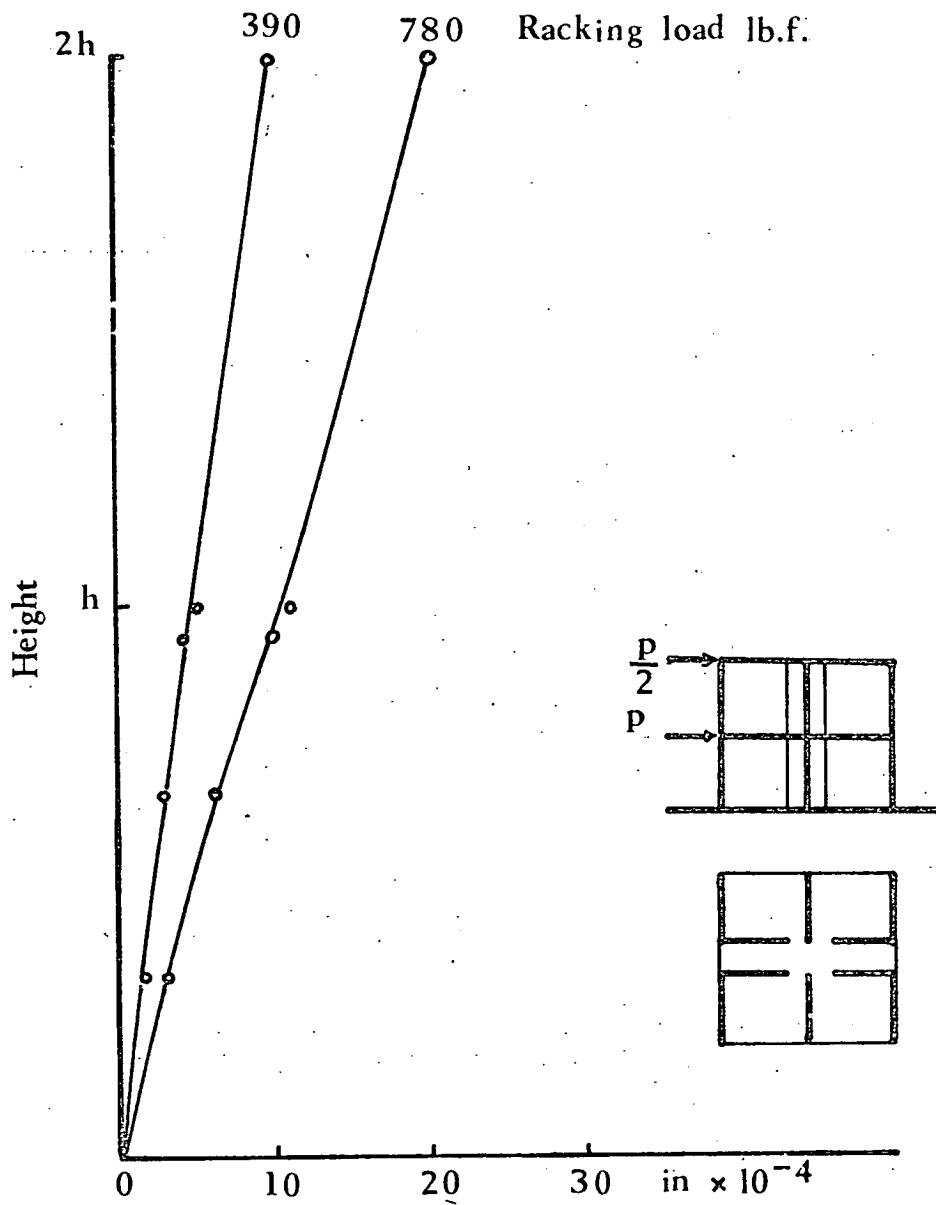


Fig. 6.4 Lateral Deflection of 2-Storey Structure.

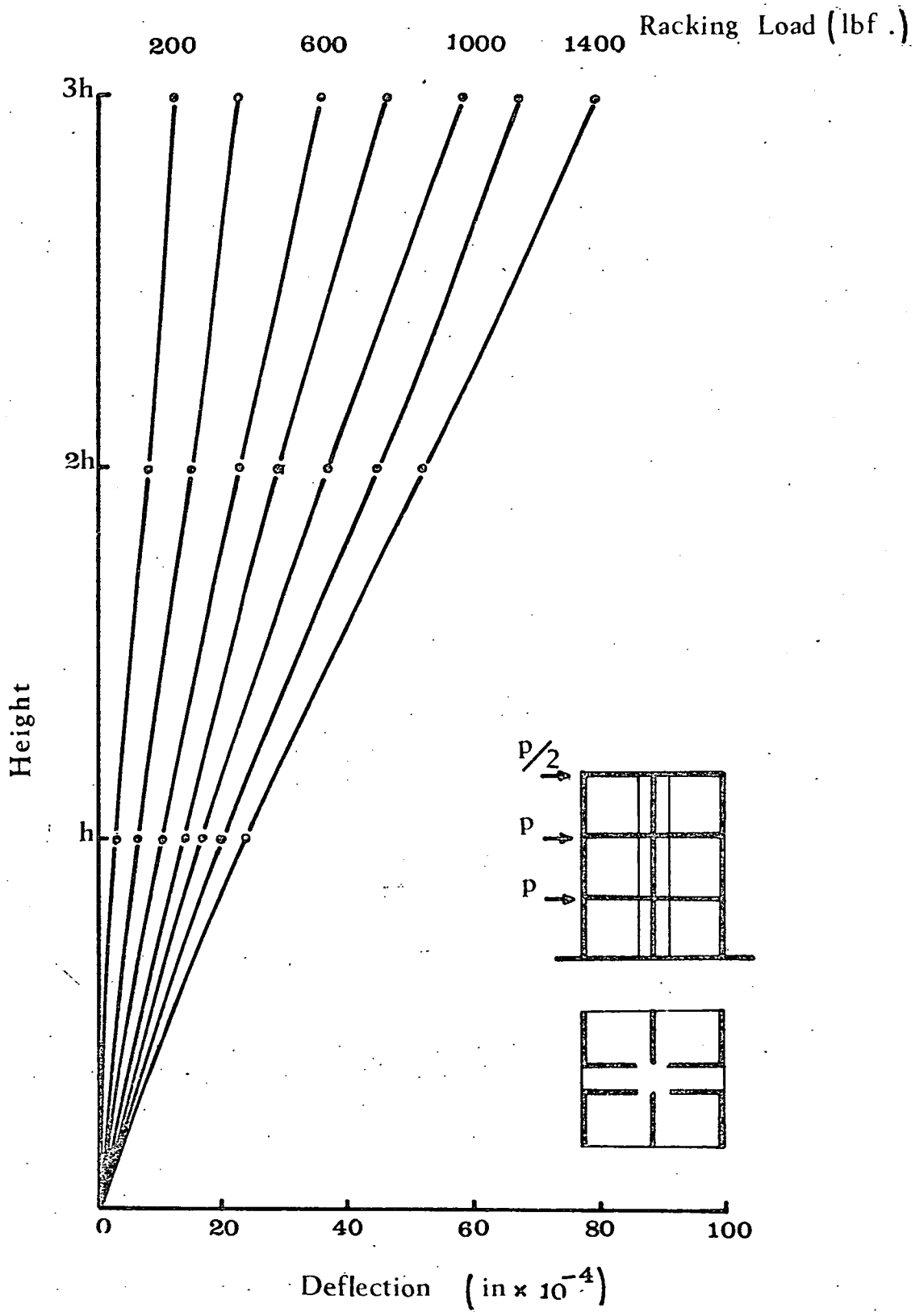


Fig. 6.5 Deflection of a 3-storey structure at various stages of loading.

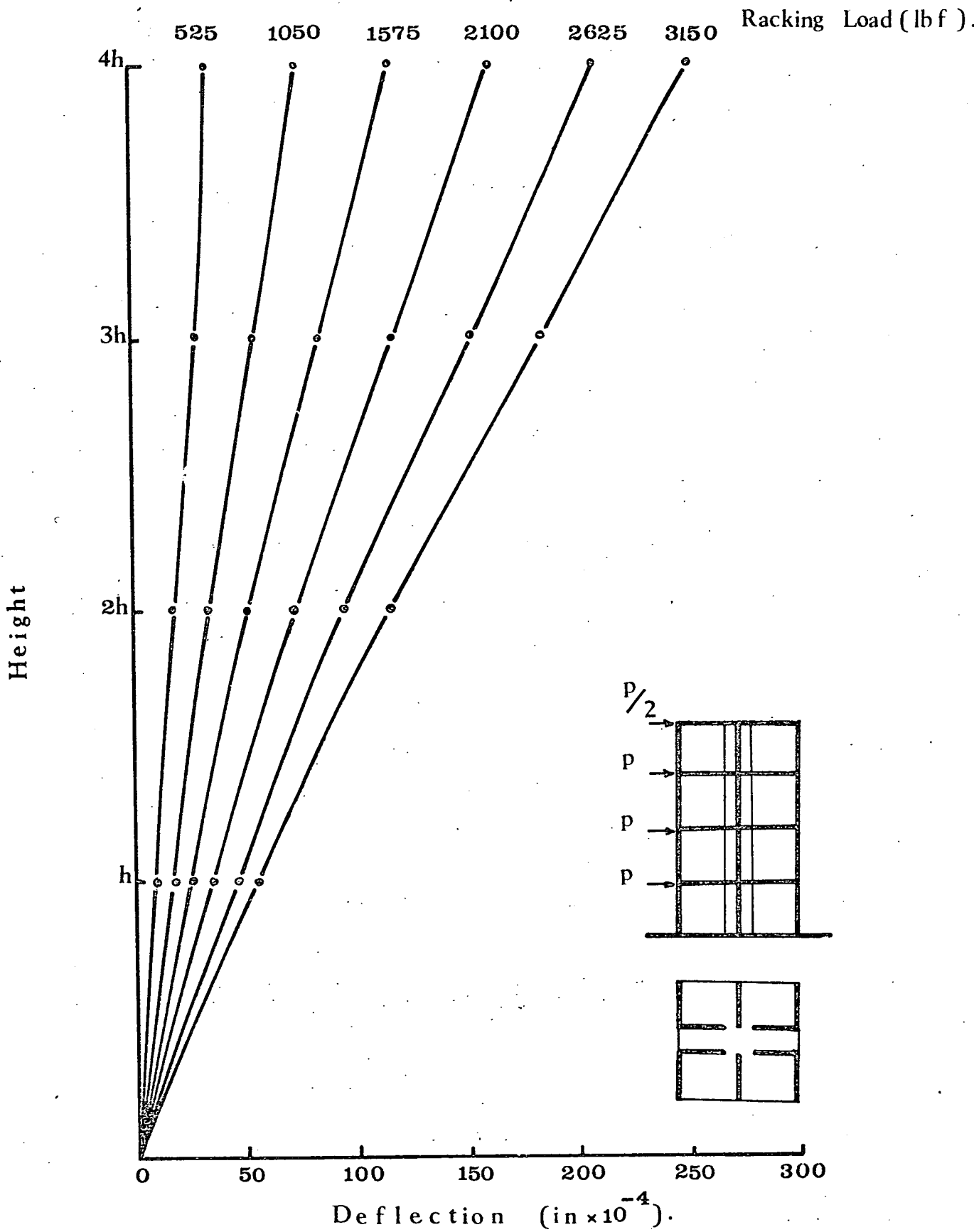


Fig. 6.6 Deflection of a 4-storey structure at various stages of loading.

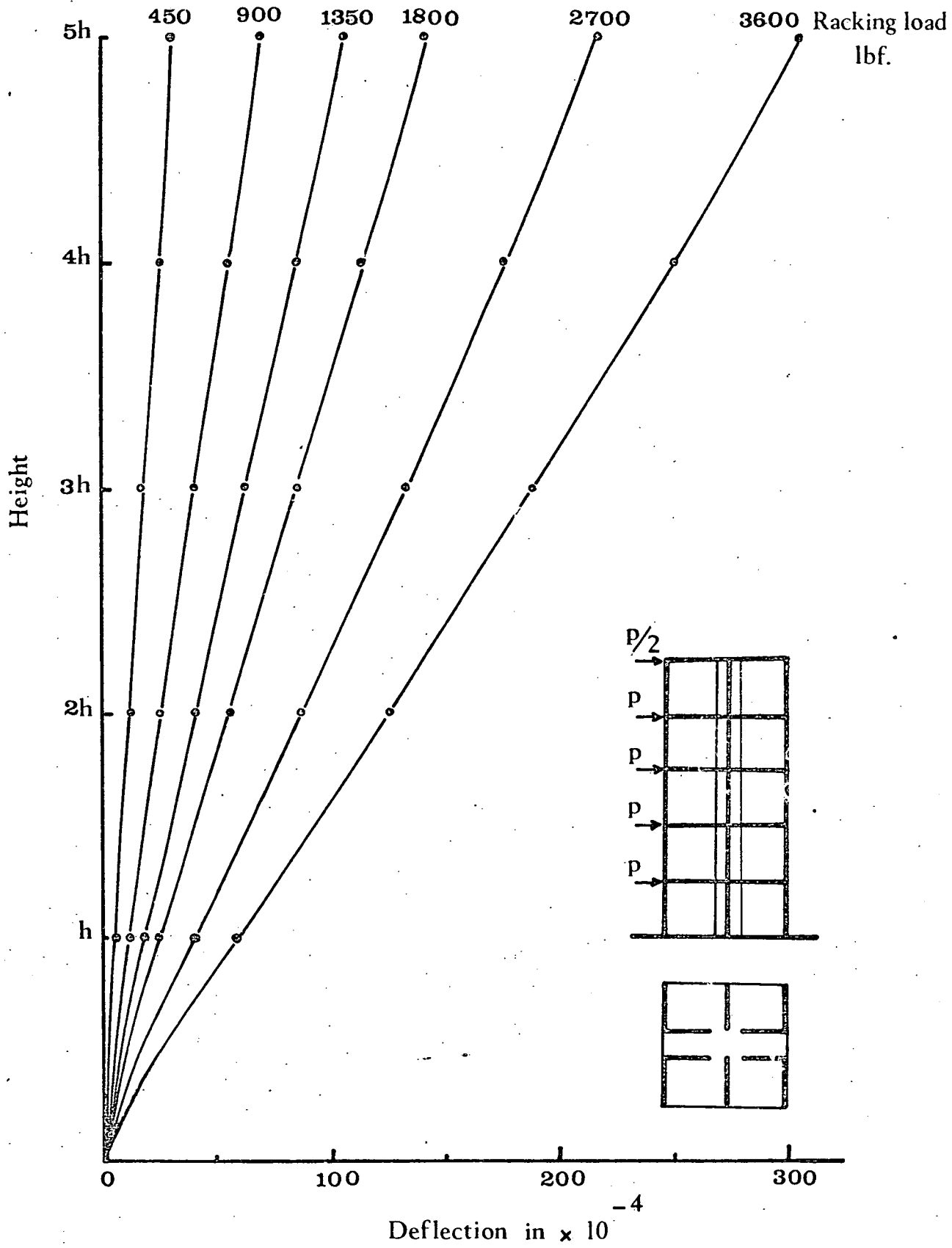


Fig. 6.7 Lateral Deflection of 5-Storey Structure

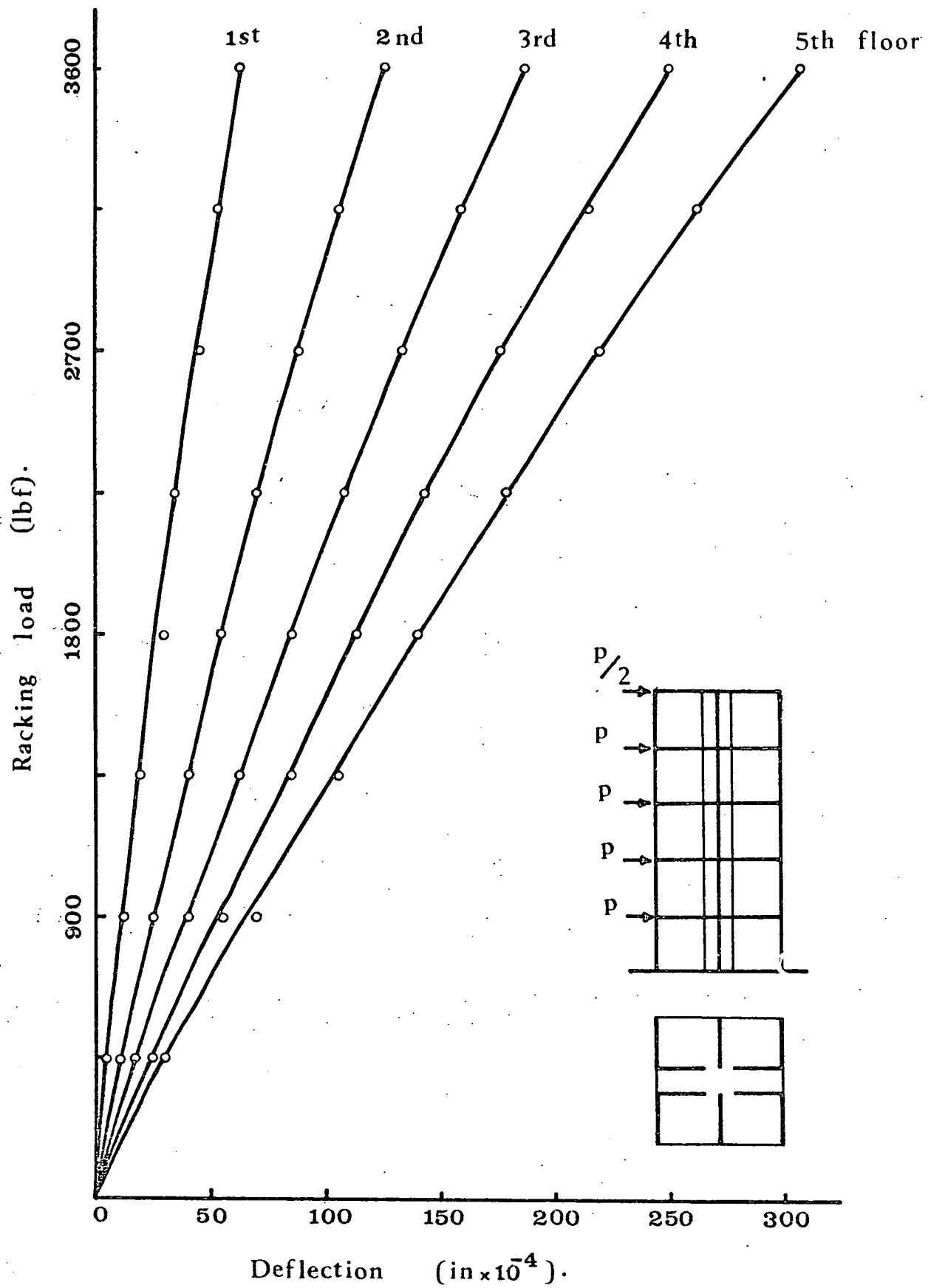


Fig. 6.8 Load / Deflection curves at various floor level.

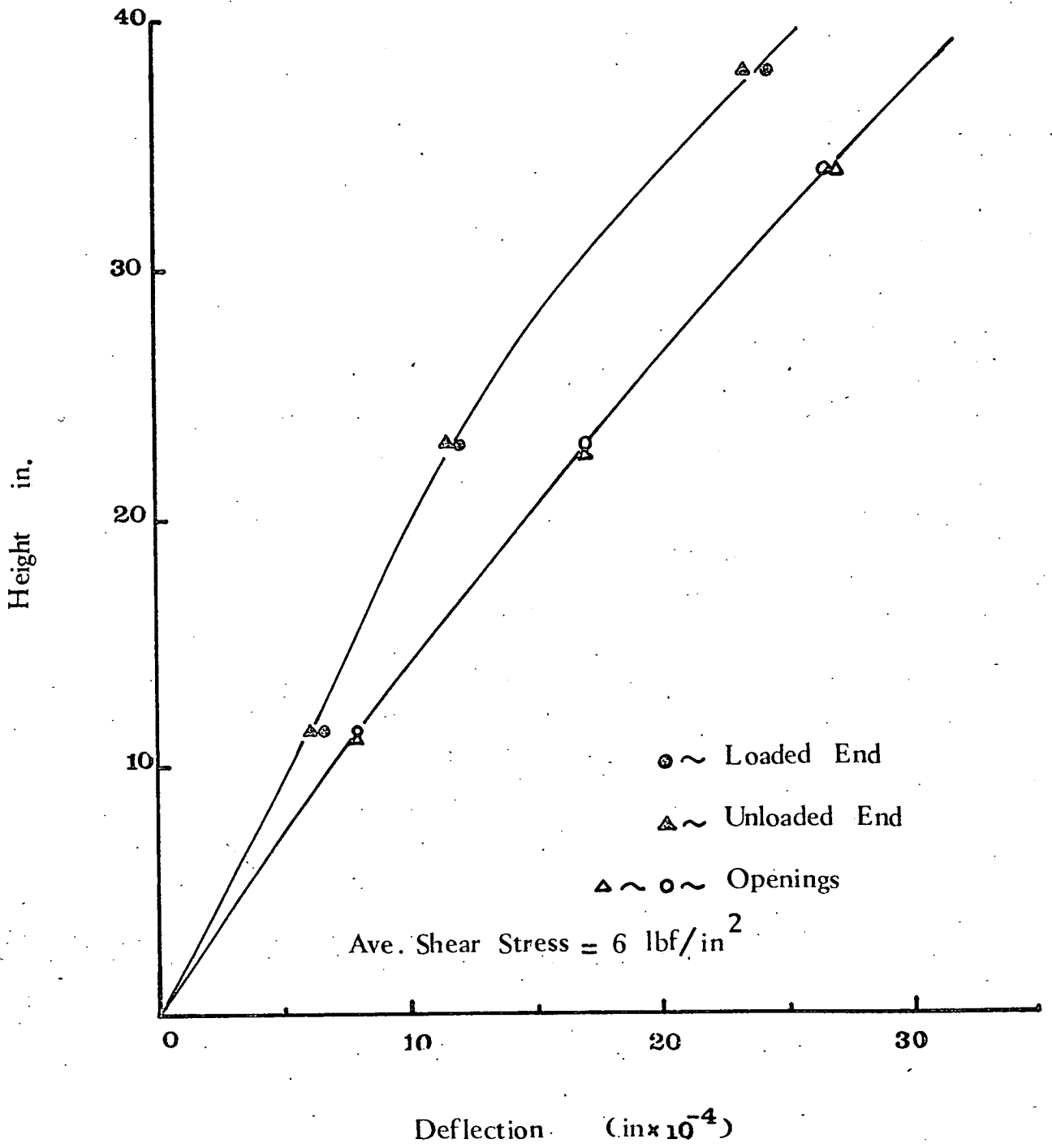


Fig. 6.9 Ground floor deflection of 5-storey structure.

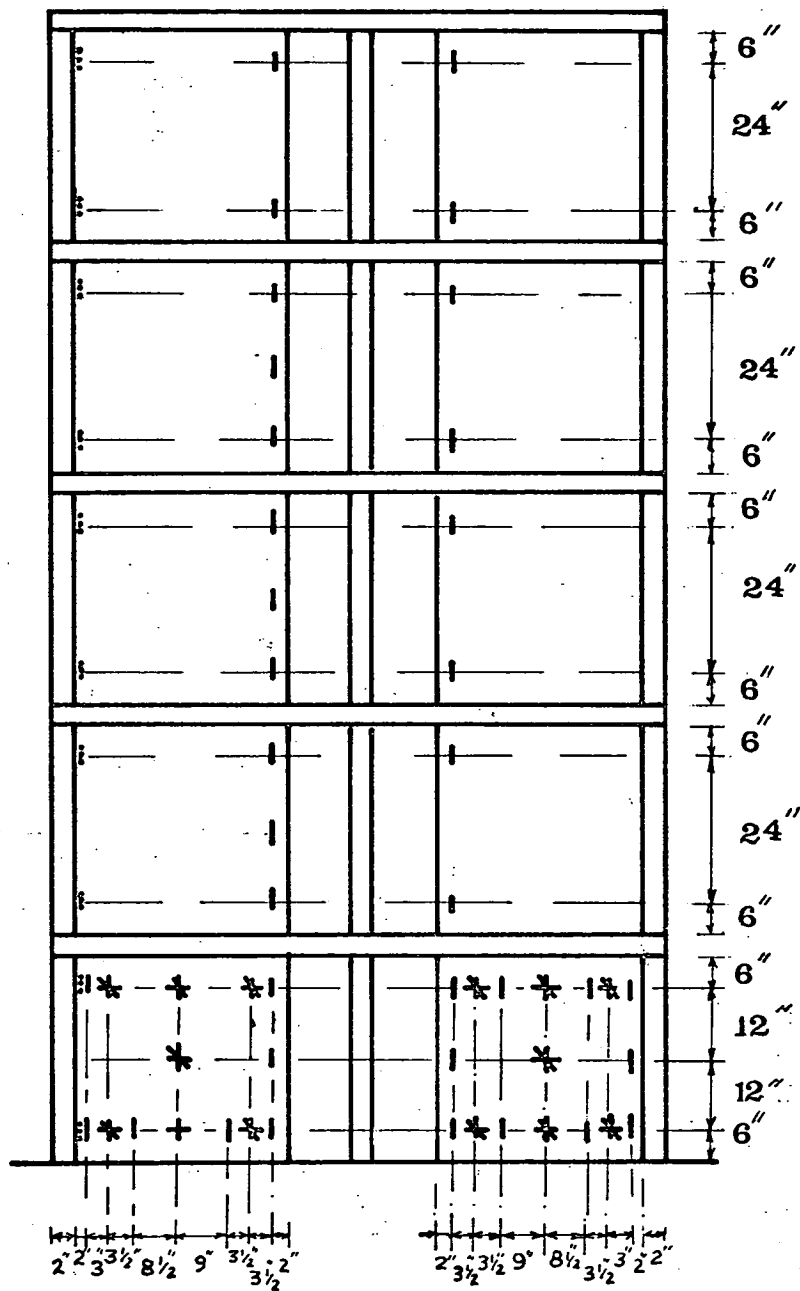
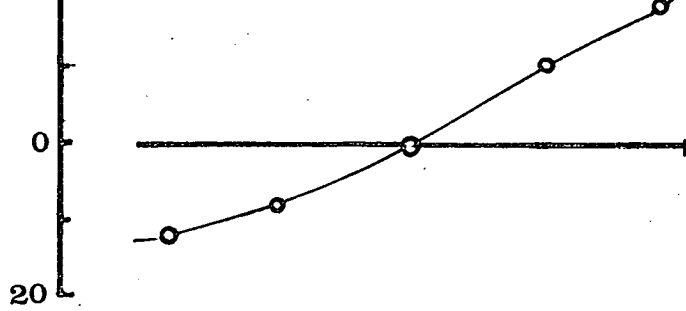
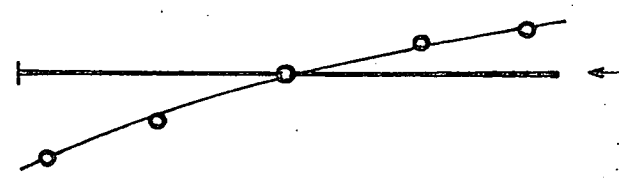


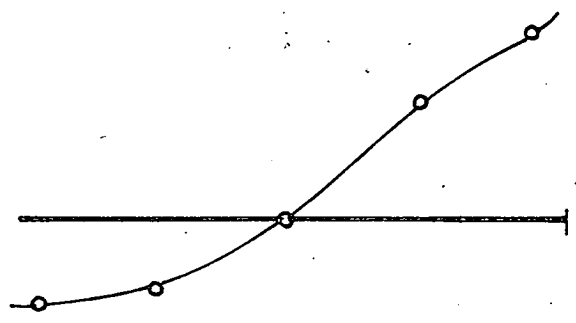
Fig.6.10 Position of vibrating wire strain gauges in 5-storey structure



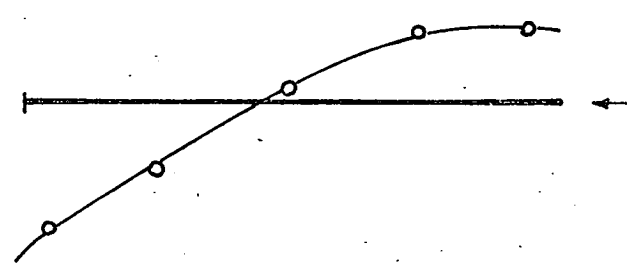
Single-Storey Structure



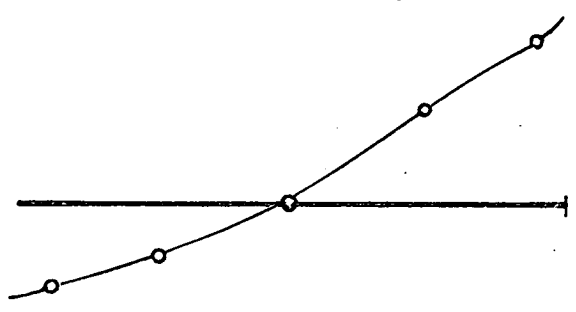
Ave. Shear Stress = 9.3 lbf/in^2



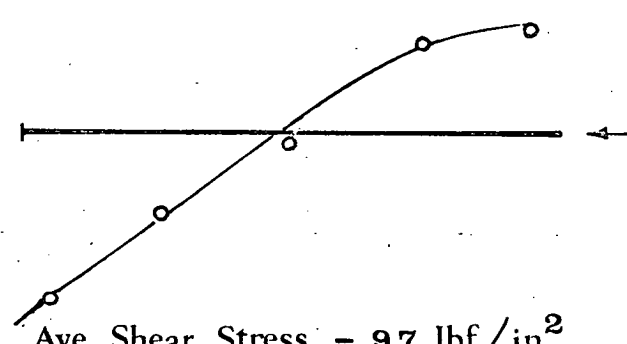
2-Storey Structure



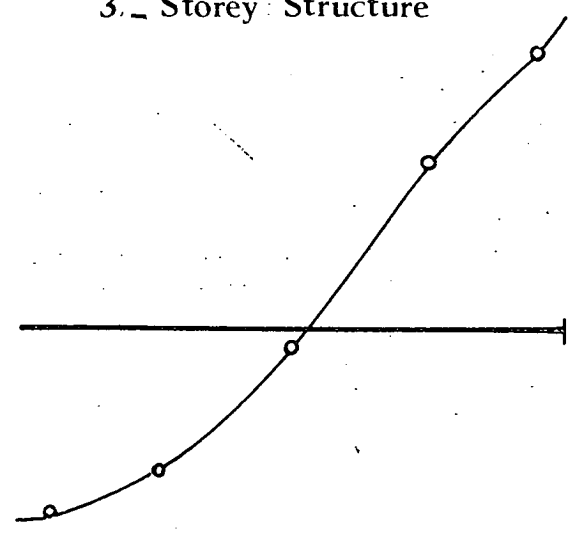
Ave. Shear Stress = 10.8 lbf/in^2



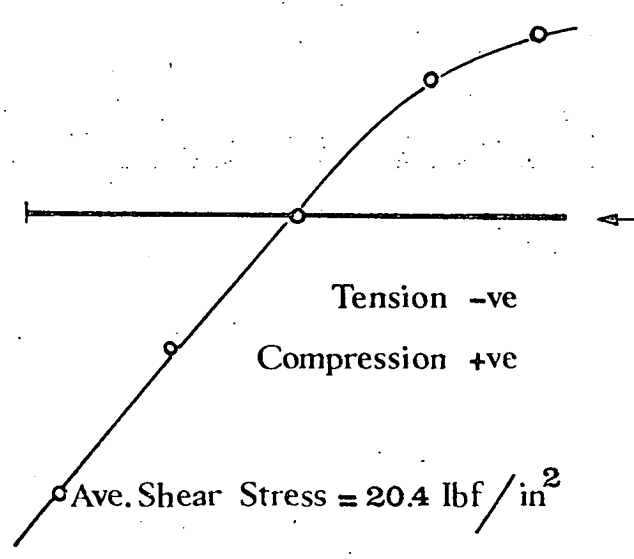
3-Storey Structure



Ave. Shear Stress = 9.7 lbf/in^2



4-Storey Structure



Ave. Shear Stress = 20.4 lbf/in^2

Fig. 6.11 Strain (ϵ_y) distribution across a horizontal section in the shear wall

6 in. above ground level

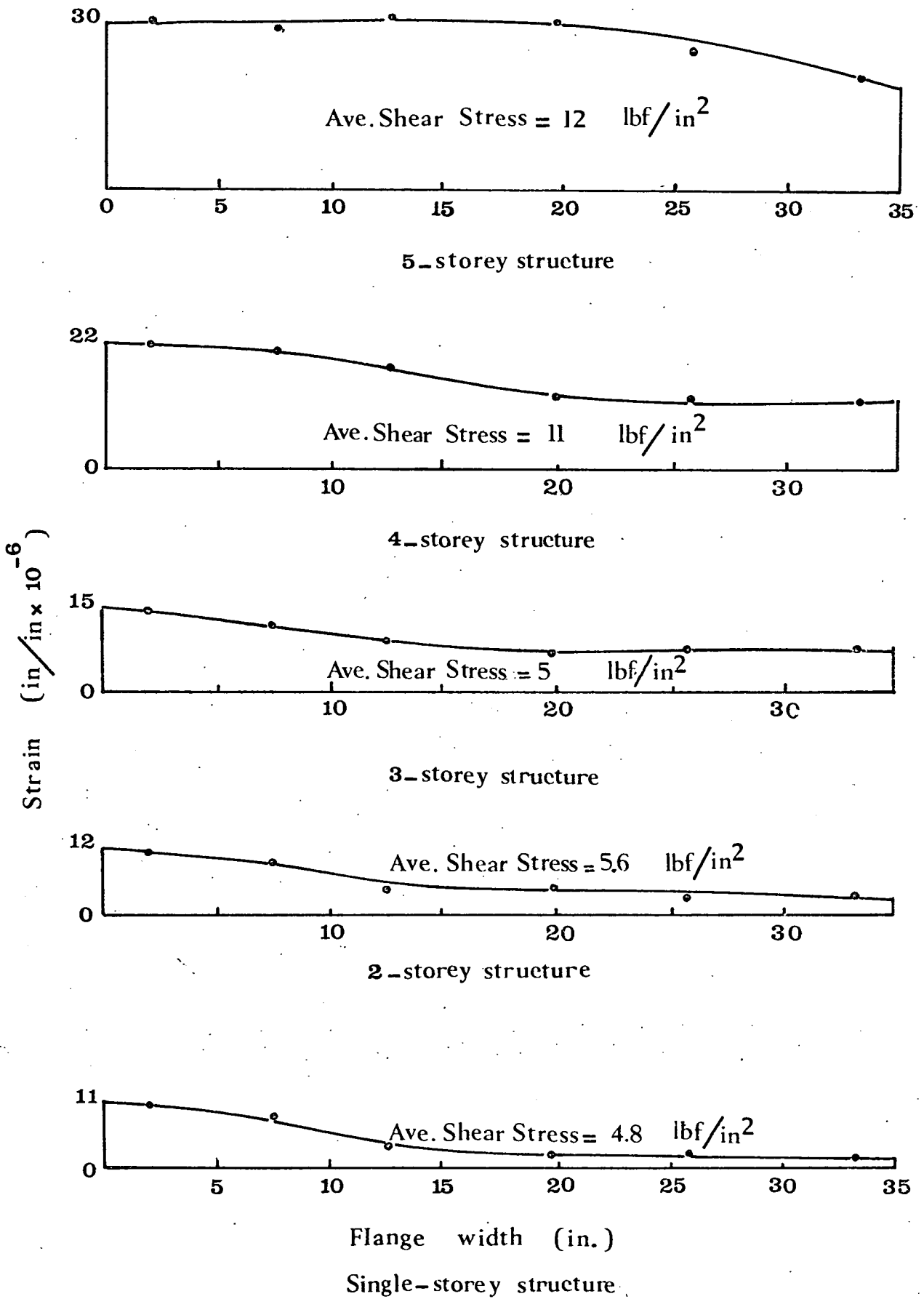


Fig. 6.12 Ave. Strain distribution across a horizontal section in the base of the return walls in various structures

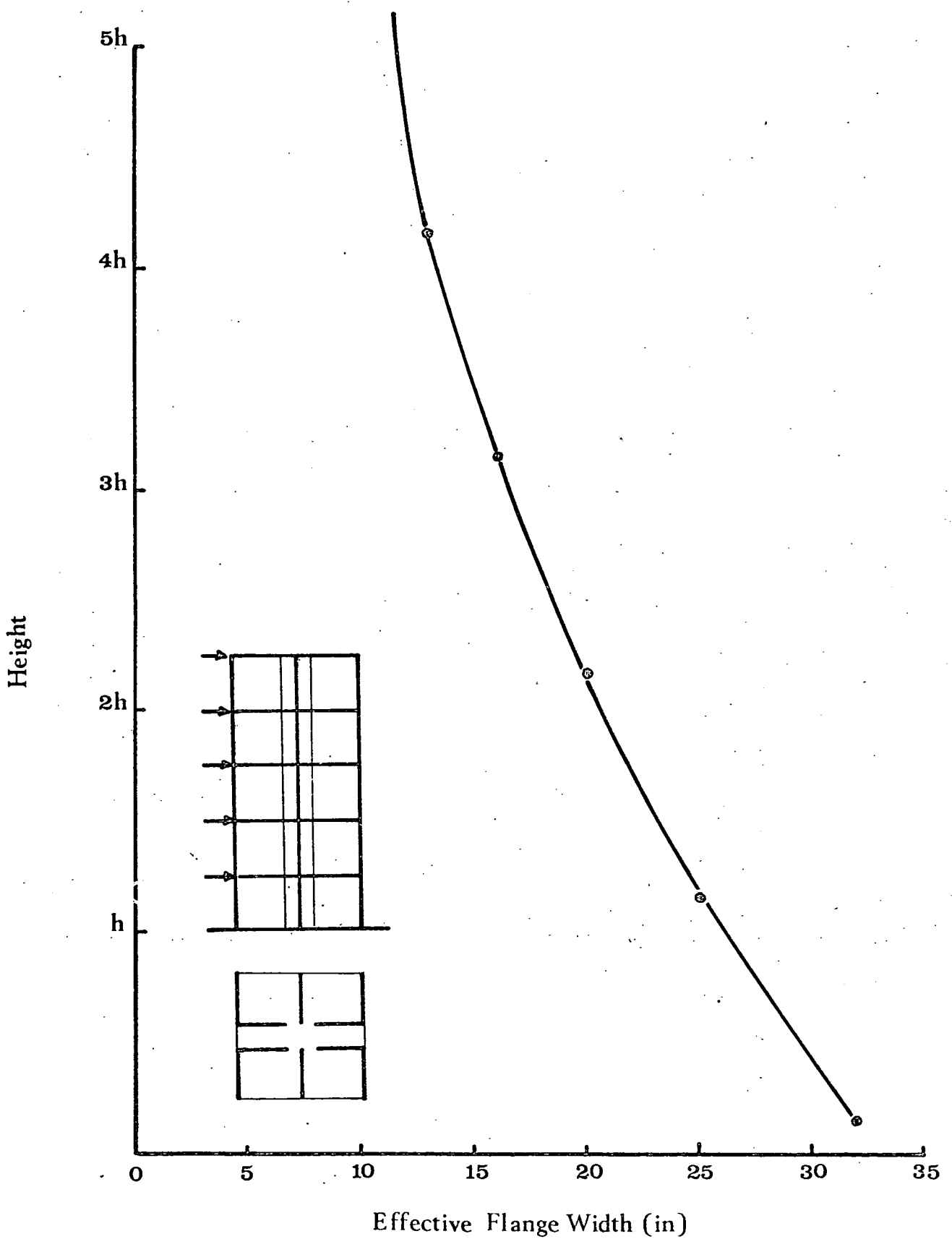
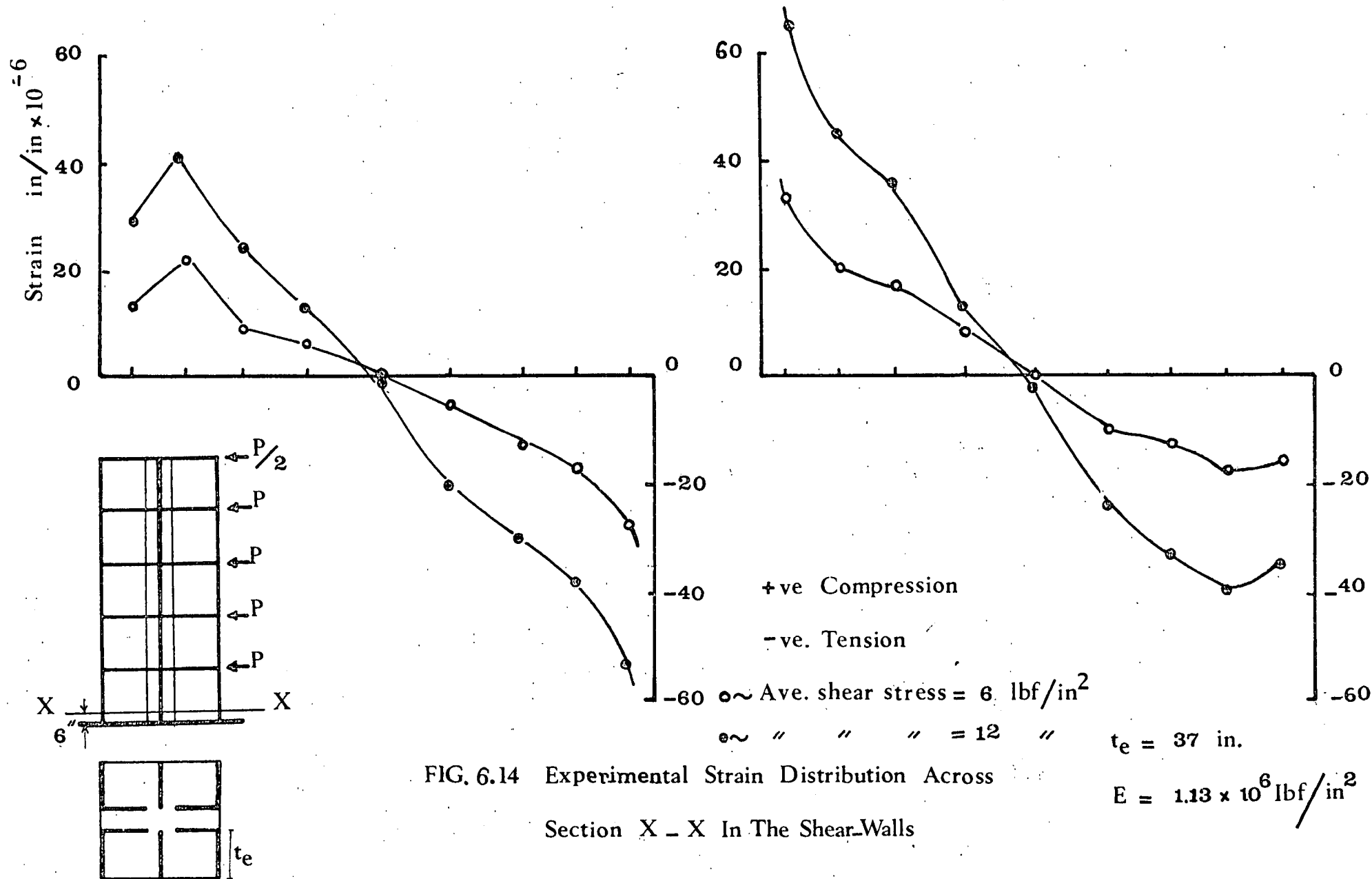


Fig. 6.13 Variation of the effective flange width at each storey level.



Principal Strain Distribution in the Ground Floor Shear Walls.

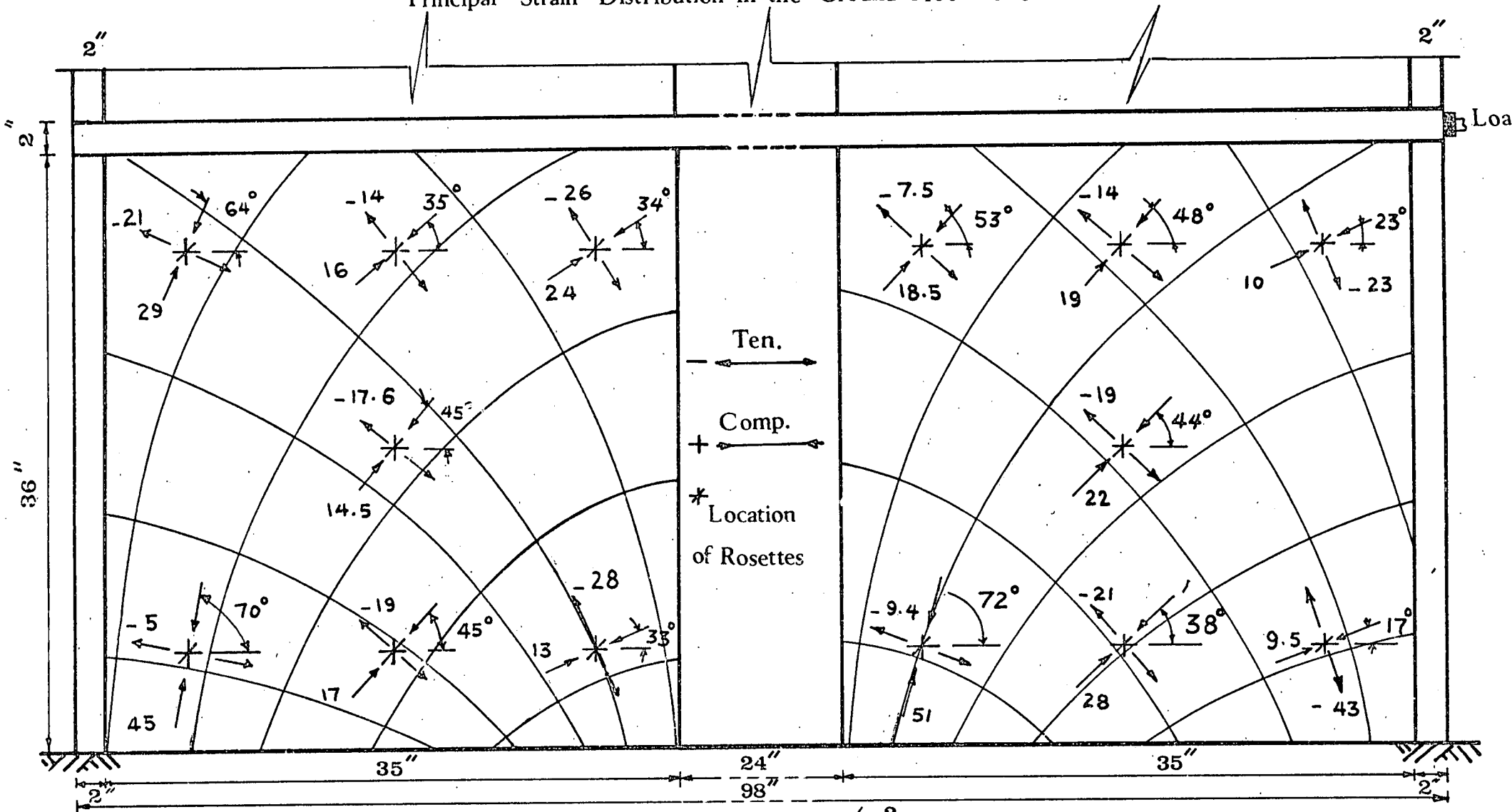


Fig. 6.16

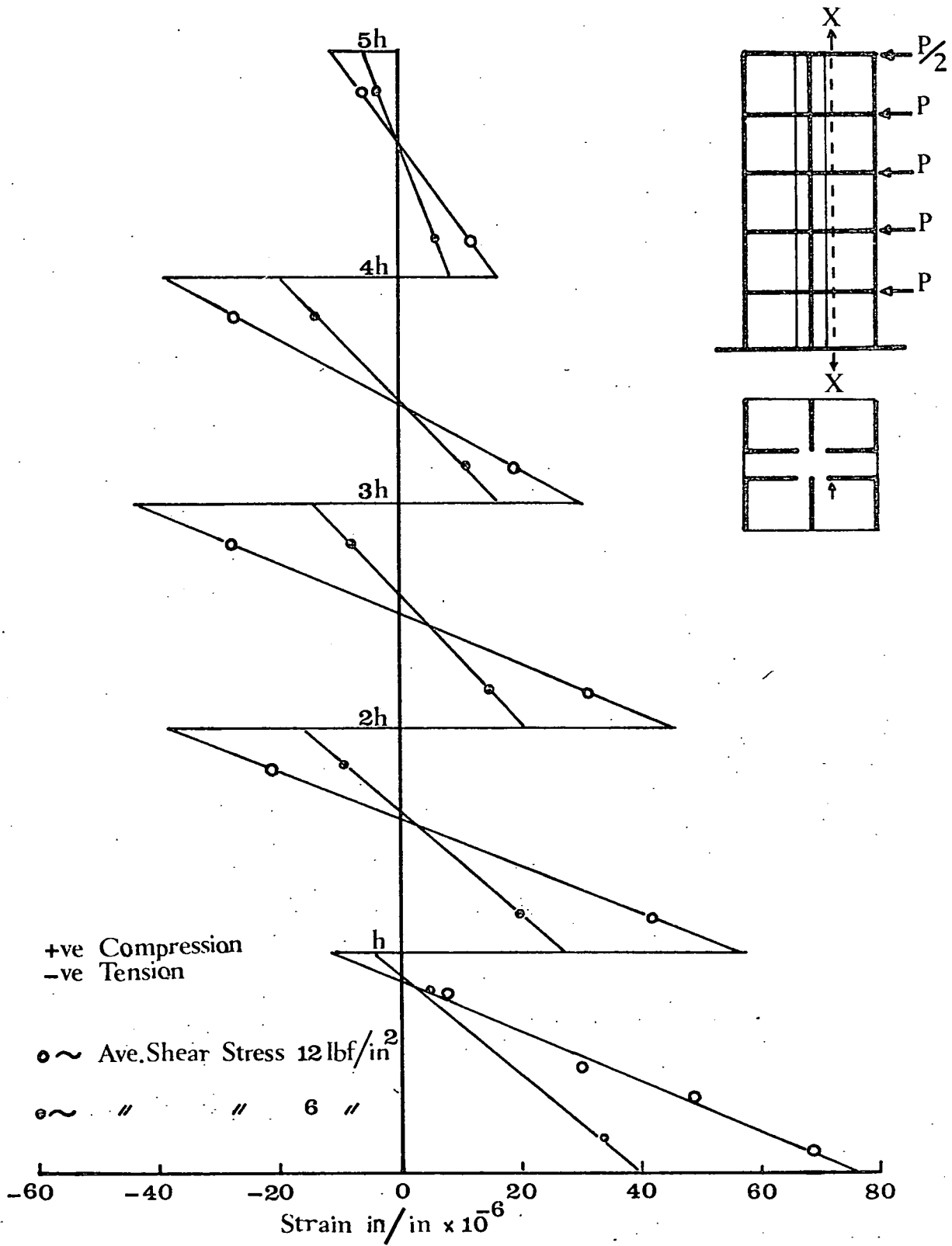


Fig. 6.17 Strain distribution ' ϵ_y ' along section X-X

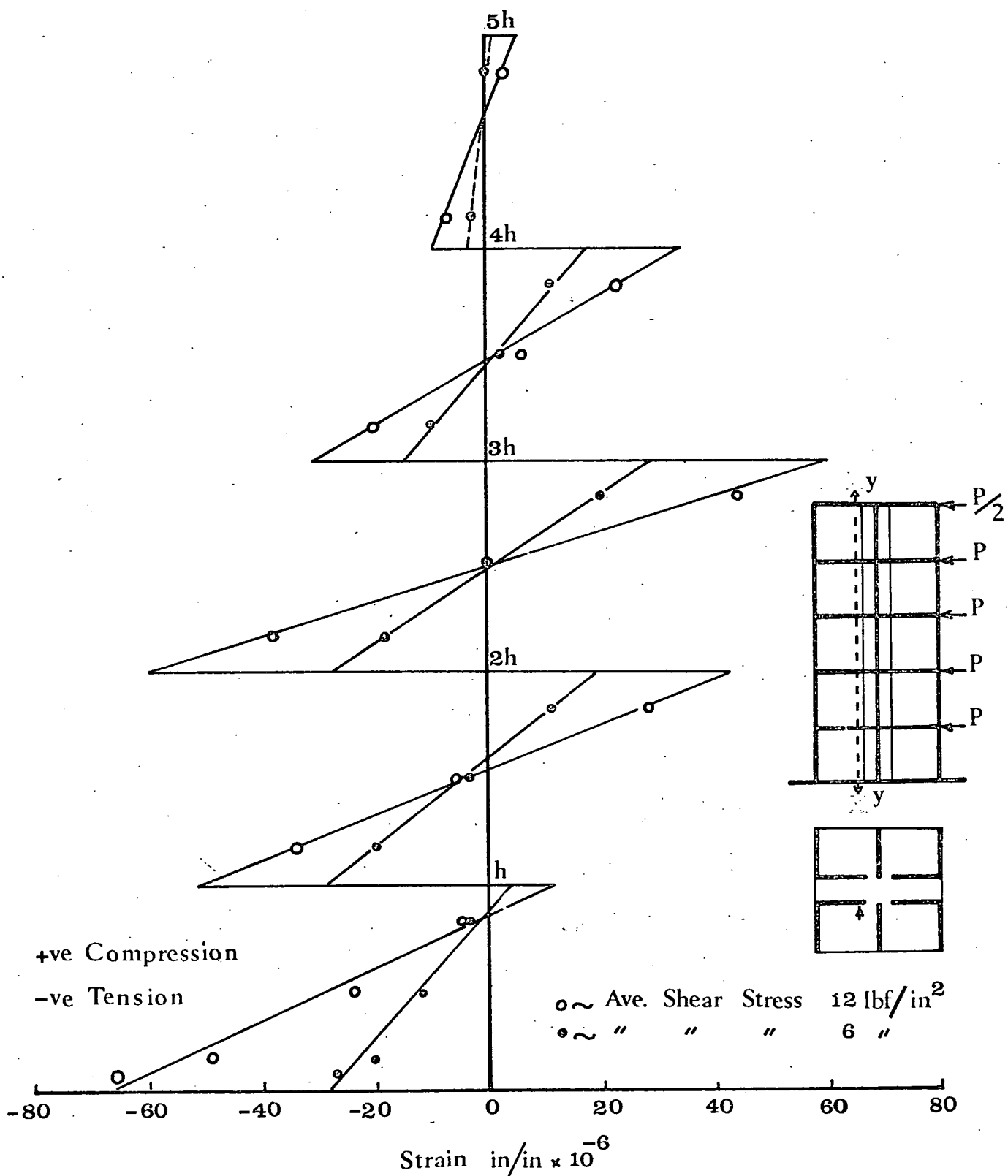


Fig. 6.18 Strain distribution ϵ_y along section $y-y$

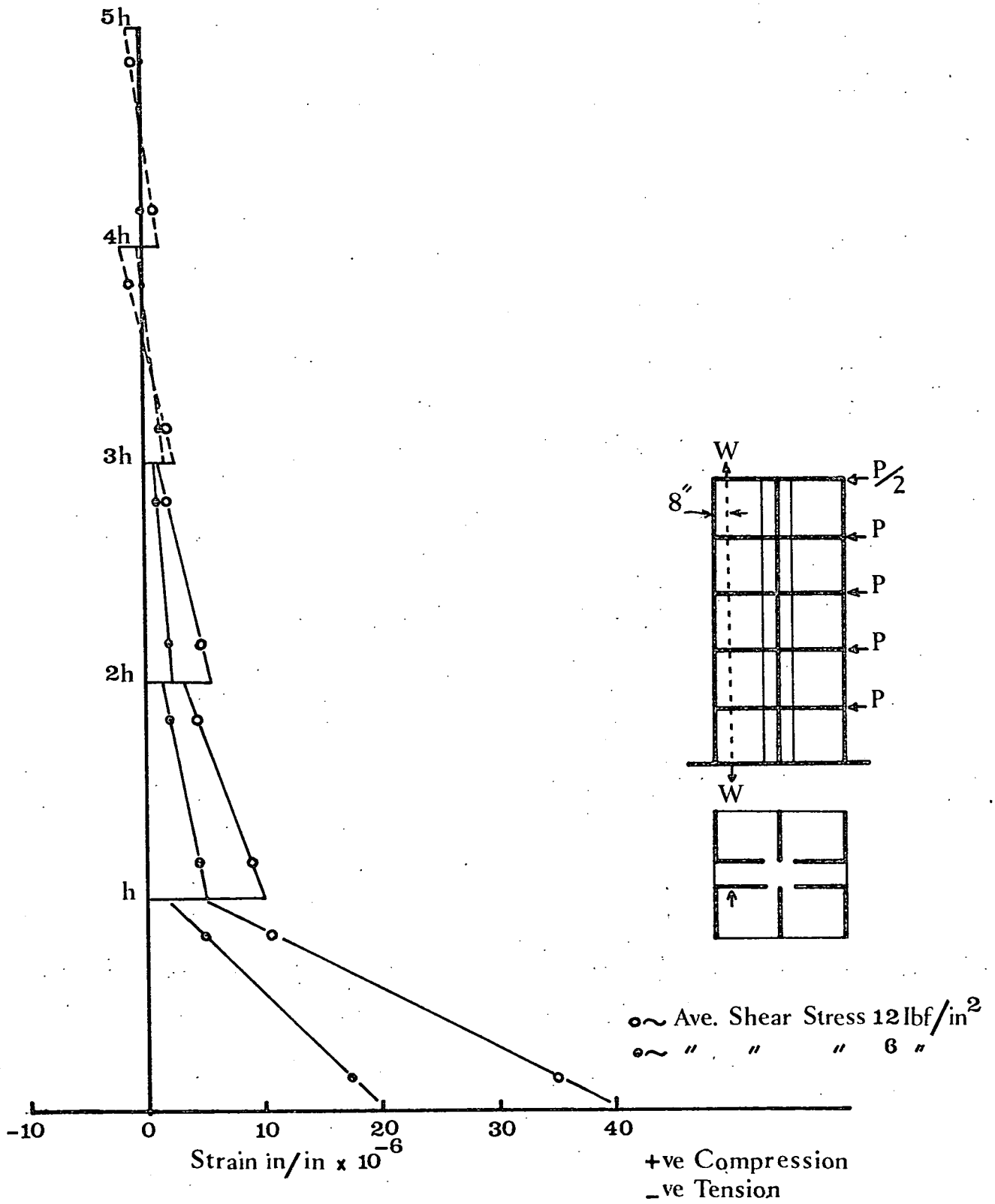


Fig. 6.19 Strain distribution ϵ_y along section W-W

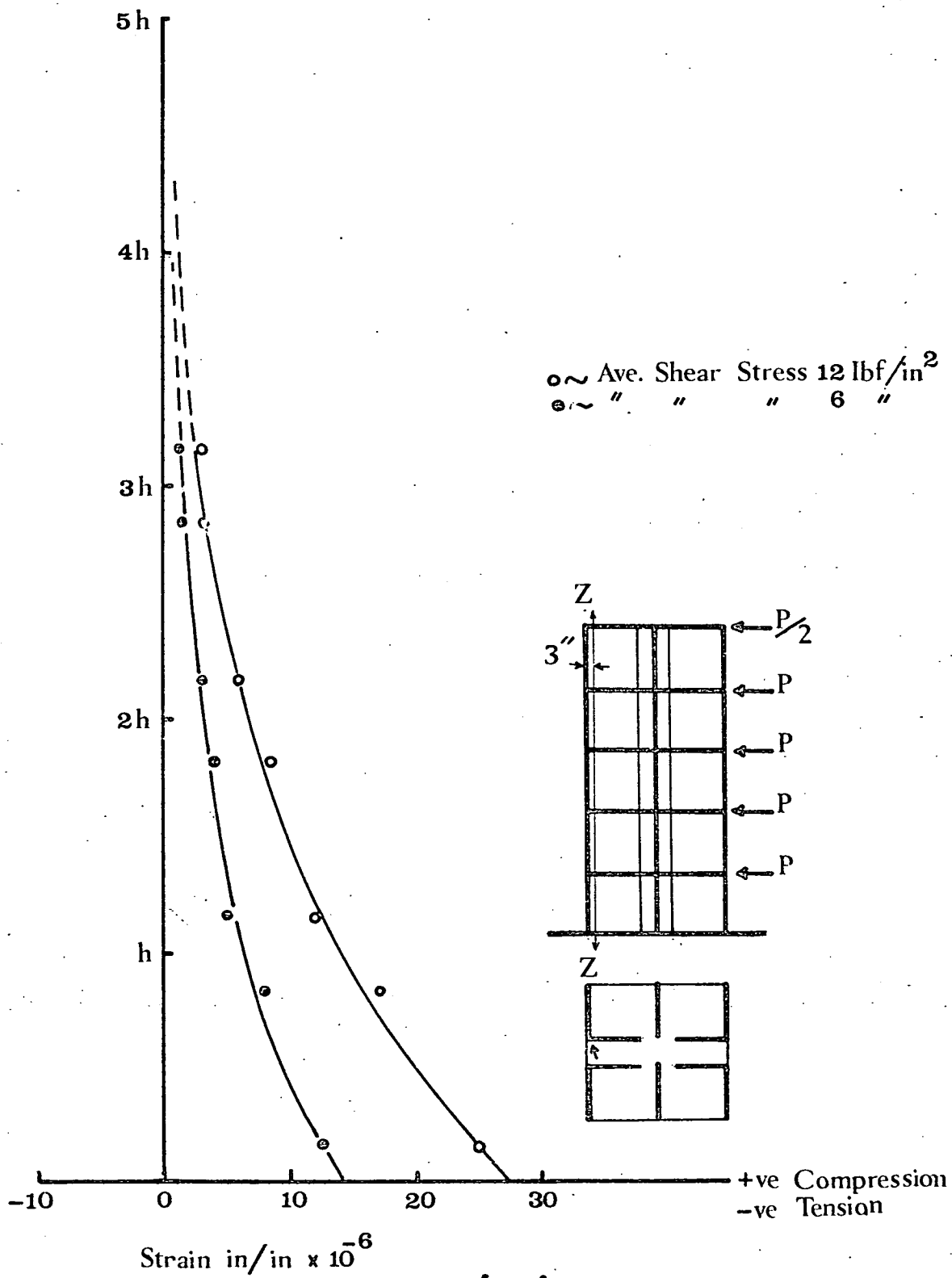


Fig. 6.20 Strain distribution ϵ_y along section Z-Z

6.7 DESIGN LOADING AND FACTOR OF SAFETY

According to CP3, Chapter 5, part 2, 1972⁽¹²⁾ the basic design speed is the 3-second gust speed to be exceeded on average once in 50 years. For the Edinburgh area this is about 112 m.p.h. (50 m/s). The design wind speed is obtained by multiplying the basic wind speed by the following factors:

1. Topography factor S1, takes account of the variation of the ground surface, which is taken as unity.
2. Factor S2 considers the combined effects of ground roughness and building size. A value of 1.03 is assumed, representing small towns and the outskirts of large cities.
3. Building life factor S3 is based on statistical concepts. This was assumed to be equal to 1.0 for a building life of 50 years and a probability level of 0.63.

Thus, the design speed on a 47.5 ft. prototype building will be 115 m.p.h., equivalent to a uniform pressure of 34.0 lbf/ft². The structure must therefore, be designed to resist a lateral load of 34.0 lbf/ft². Based on single storey test results (Table 4.3) and previous work on cross-wall structures in brickwork^(52,74,96), an ultimate shear strength of 50 lbf/in² (equivalent to 122 lbf/ft² on the exposed area) is estimated for the structure, giving a factor of safety of 3.6 against wind loading. However, based on the CP111 recommendation, the structure must be designed for lateral loads producing an allowable shear stress equal to 23.3 lbf/in². Hence, factor of safety against shear will be equal to 2.1 (not taking into account live loads; the lateral resistance of the structure increases with the precompression). The shear wall area in this case could be reduced by/

reduced by 16.9% for economical reasons, without causing any change in the area exposed to the wind provided that the deflections are not excessive. In such case the shear stress produced at the ground floor shear walls will be 16.7 lbf/in^2 which is still below the allowable shear stress according to the CP 111, 1970 and gives a factor of safety of 3 without live loading.

6.8. ANALYSIS OF THE STRUCTURE

Outlines of the analytical methods for the solution of multistorey shear wall buildings and the assumptions concerning their formulation have already been mentioned in Chapter 5. The following methods were used to analyse the structure:

- 1) The Cantilever Method
- 2) The Shear Connection Method (S.C.M.)
- 3) The Wide Column Frame Analogy (W.C.F.A.)
- 4) The Finite Element Method (F.E.M)
- 5) The Equivalent Frame Analogy (E.F.A.)

Due to symmetry, half the structure was considered in the analysis. Young's Modulus of Elasticity for blockwork and concrete was found experimentally (Section 4.17) to be $1.13 \times 10^6 \text{ lbf/in}^2$ and $4.3 \times 10^6 \text{ lbf/in}^2$ respectively. Poisson's ratio for blockwork was found to be 0.18 and a value of 0.15 was assumed for concrete.

Research on shear walls with openings indicates that the overall behaviour of the structure mainly depends on:

- 1) The stiffness of the connecting beams
- 2) The correct effective width of the return walls
- 3) The width of the openings.
- 4) The type of loading

There is/

There is contradictory evidence regarding the assessment of the effective width of the floor slab. Values greater than full width⁽¹¹⁶⁾ equal to full width⁽³⁾ and less than full width⁽¹¹⁷⁾ have all been suggested and shown to be valid under particular circumstances.

In the calculation of the stiffness of floor slabs or lintel beams, the whole width of slab was assumed to be effective. For comparison analyses are also made using W.C.F.A. and assuming an effective slab width equal to one-half of the structures bay width.

For the assessment of correct flange width, similar work has been published^(52,98,120) suggesting different effective flange widths for the calculation of area and moment of inertia. Two cases were considered:

(i) Fully effective flange width.

(ii) Using the CP 114⁽¹²⁰⁾ recommendation, the active flange width was taken as:

$$4 t_f + t_w$$

where t_f = thickness of cross wall

and t_w = thickness of shear wall

The width of the openings can also modify the behaviour of the structure. Coupled-shear walls with large opening tend to behave as separate cantilevers, while with a small opening their behaviour will be similar to a composite structure. The clear distance between the shear walls was used in the S.C.M. and F.E.M., while frame analogies assumed the distance between the centroids of the walls.

6.9 DISCUSSION OF THE RESULTS

Tables 6.4a and 6.4b give the values of the maximum shear force in the connecting beam (Frame Analysis), or the equivalent media (S.C.M.) and compares/

TABLE 6.4a - ANALYSIS OF THE STRUCTURE USING VARIOUS METHODS ASSUMING FULL FLANGE WIDTH

Method of Analysis	Deflection in. x 10 ⁻⁴ shear deformation neglected	% from experimental results	Max. shear force in the connecting beam(lbf)	Deflection in x 10 ⁻⁴ shear deformation considered	% from experimental	Max. shear force in the connecting beam (lbf)
Cantilever	200	+40	-	-	-	-
W.C.F.A.	39	-73	197	52	63	197
E.F.A.	150	+5	45	162.5	14	44.5
S.C.M.	38.5	-73	5.9 lbf/in	-	-	-
F.E.M.	-	-	-	40	-72	-

E for blockwork = 1.13×10^6 lbf/in²

E for concrete = 4.3×10^6 lbf/in²

Poisson's Ratio for concrete = 0.15

Poisson's Ratio for blockwork = 0.18

TABLE 6.4 b - ANALYSIS OF THE STRUCTURE USING VARIOUS METHODS ASSUMING A FLANGE WIDTH OF 10 INCHES

Method of Analysis	Deflection in. x 10 ⁻⁴ shear deformation neglected	% from experimental	Max. shear force in the connecting beam (lbf).	Deflection in. x 10 ⁻⁴ shear deformation considered	% from experimental	Max. shear force in the connect- ing beam (lbf)
Cantilever	312	+118	-	-	-	-
W.C.F.A.	65	-54	233	78	-46	233
E.F.A.	203	+42	72	-	-	-
S.C.M.	63	-56	6 (lbf/in)	-	-	-
F.E.M.	-	-	-	58	-60	-
W.C.F.A.*	-	-	-	92	-36	200

E for blockwork = 1.13×10^6 lbf/in²

E for concrete = 4.3×10^6 lbf/in²

Poisson's Ratio for blockwork = 0.18

Poisson's Ratio for concrete = 0.15

* Effective slab width = $\frac{1}{2}$ structure bay width

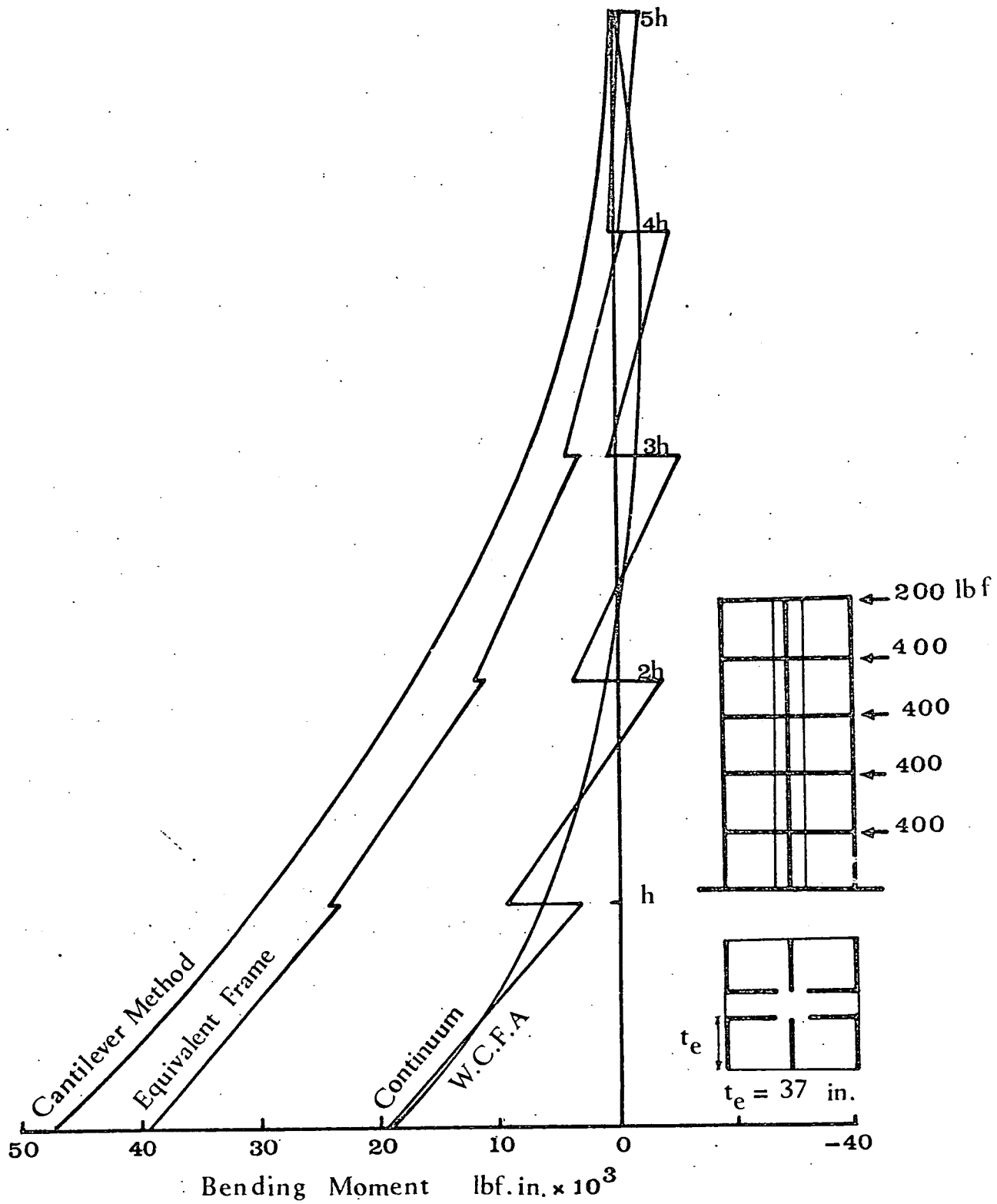


Fig. 6.21

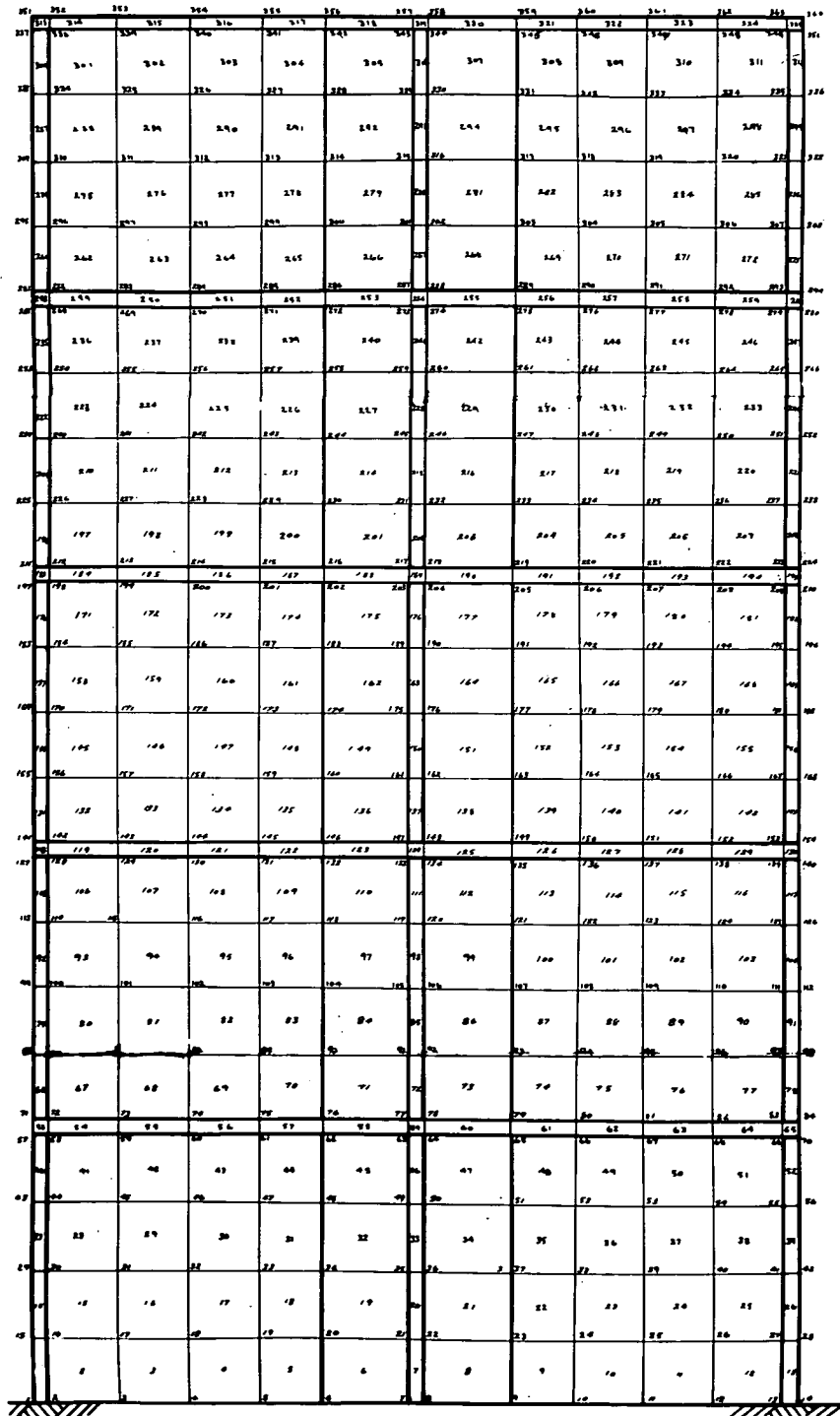


Fig. (6.22). FINITE ELEMENT REPRESENTATION
OF THE FIVE-STOREY STRUCTURE

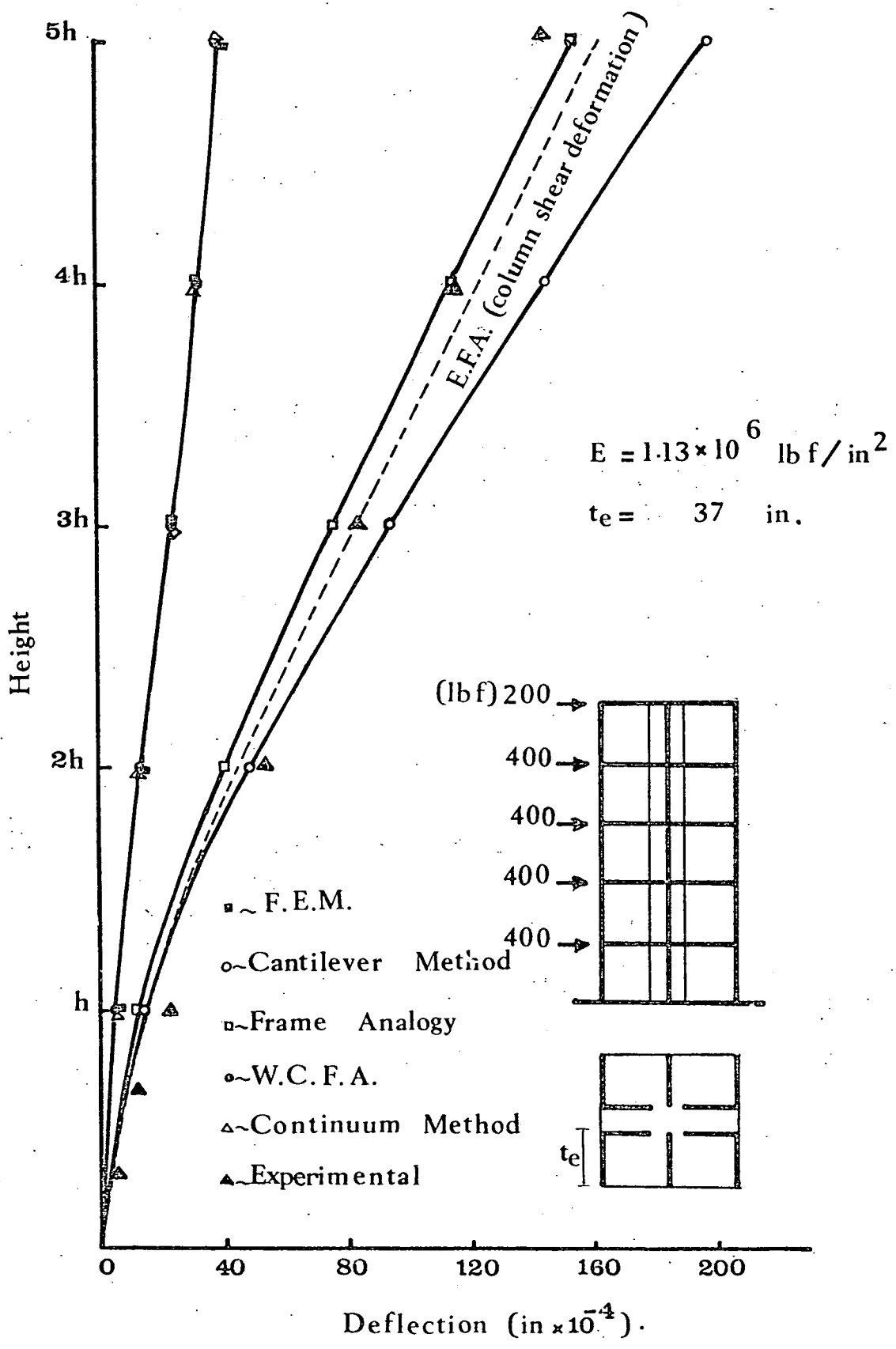


Fig. 6.23 Experimental & theoretical deflection of 5-storey shear wall structure.

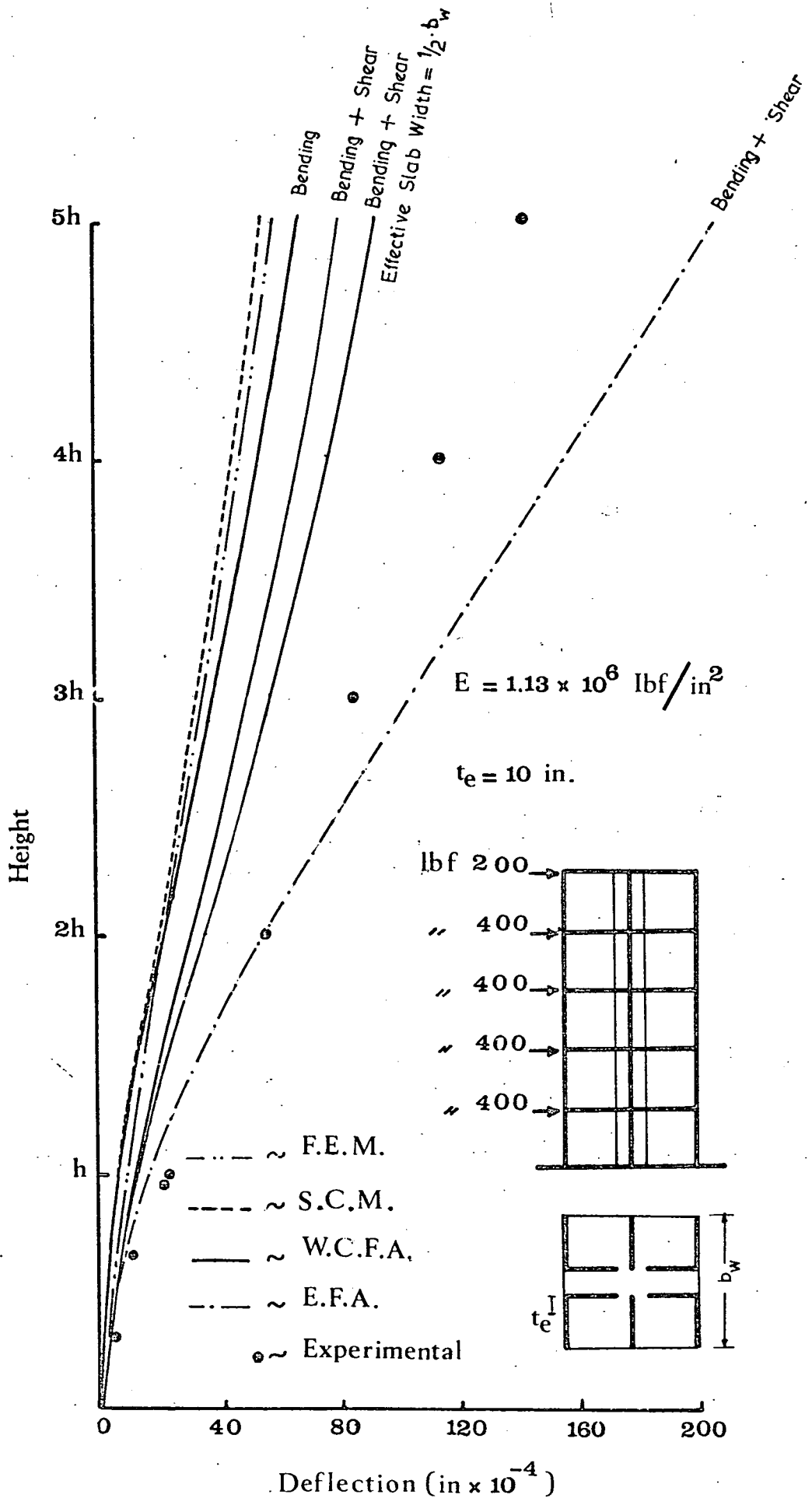


Fig. 6.24 Experimental & theoretical deflection of 5-storey structure

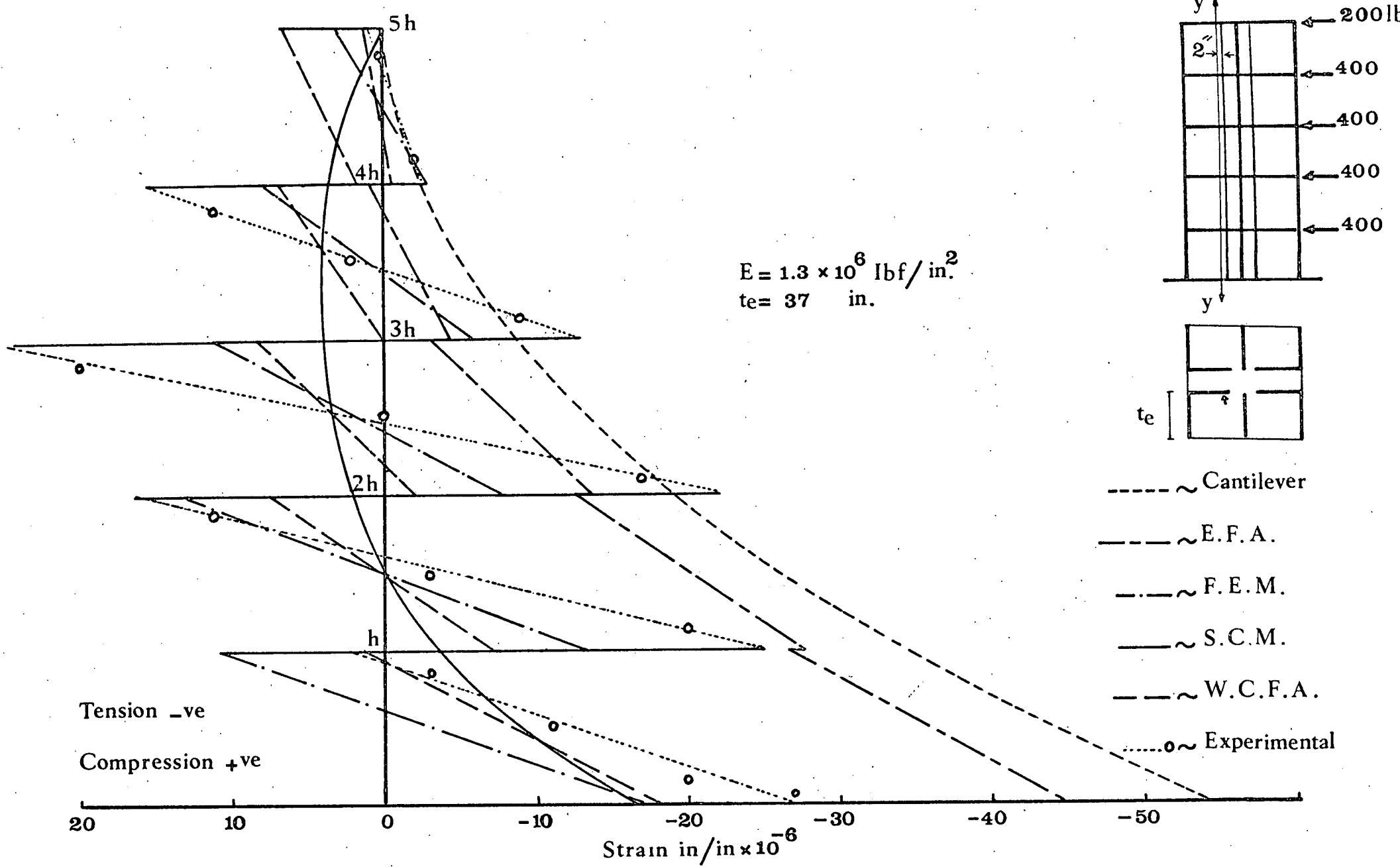


Fig. 6.25 Experimental & theoretical strain ' ϵ_y ' distribution along section y - y

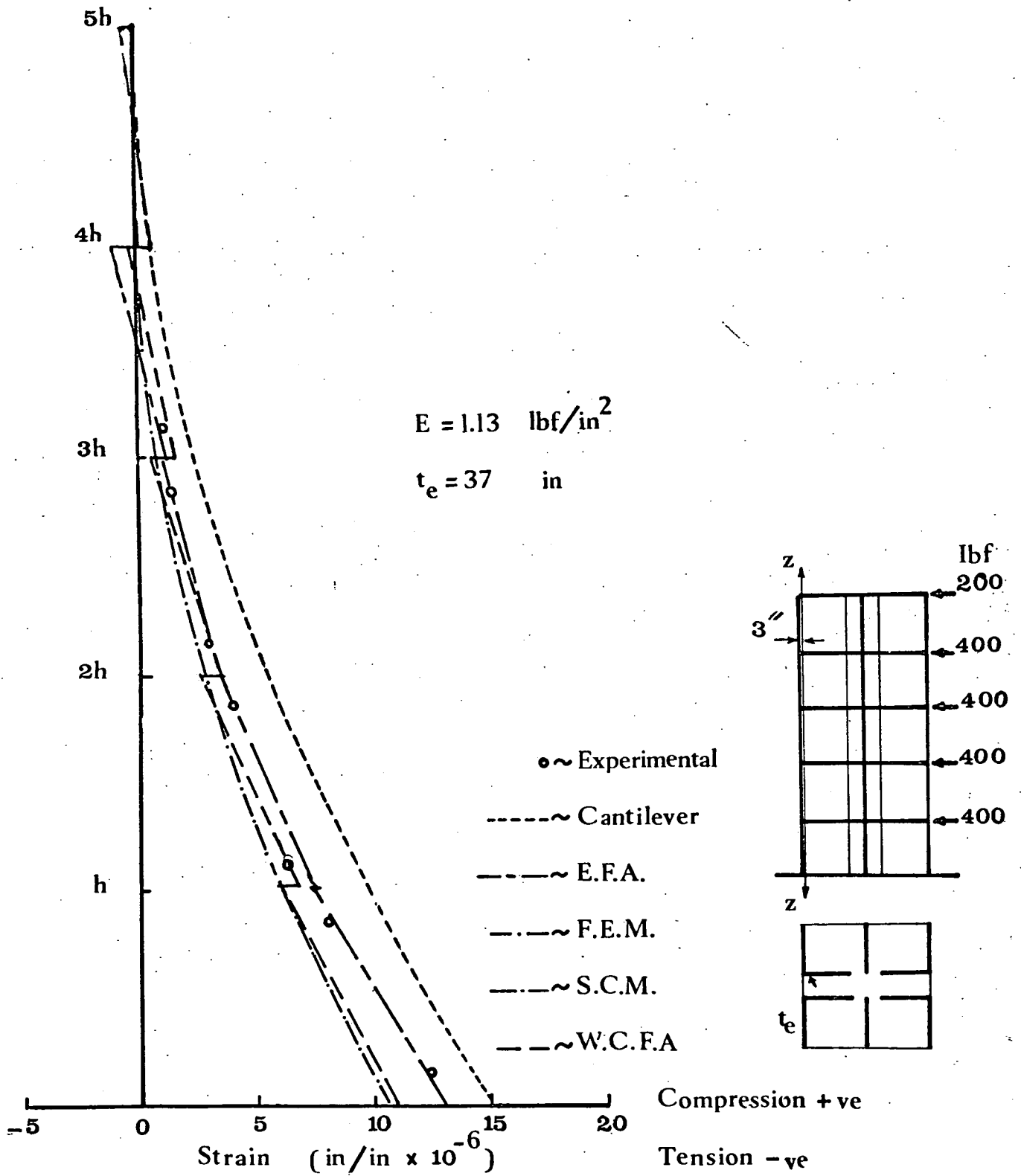


Fig. 6.26 Experimental & theoretical strain ' ϵ_y ' distribution along section Z-Z

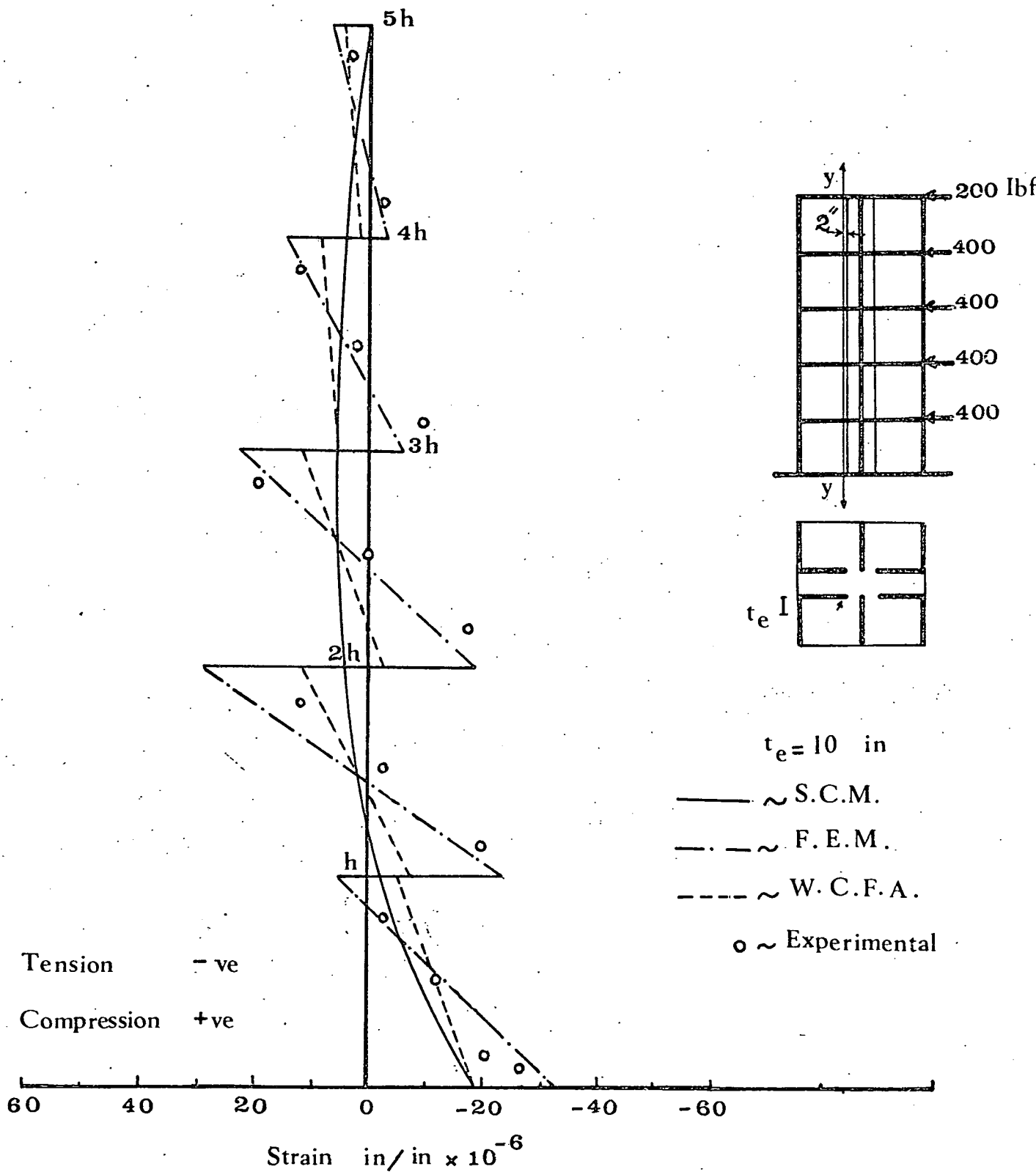


Fig 6.27 Experimental & Theoretical Strain ϵ_y Distribution

Along Section y-y

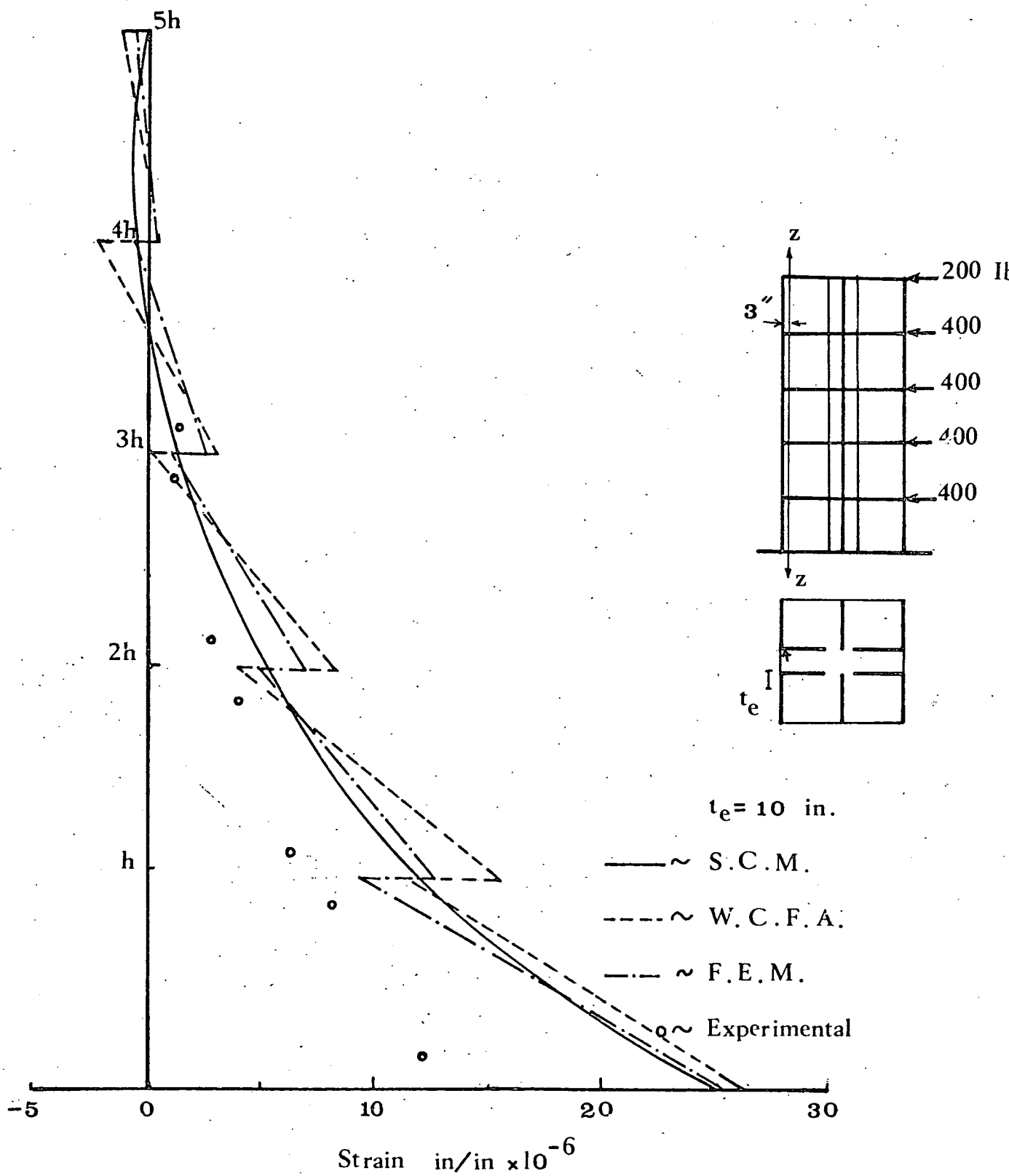
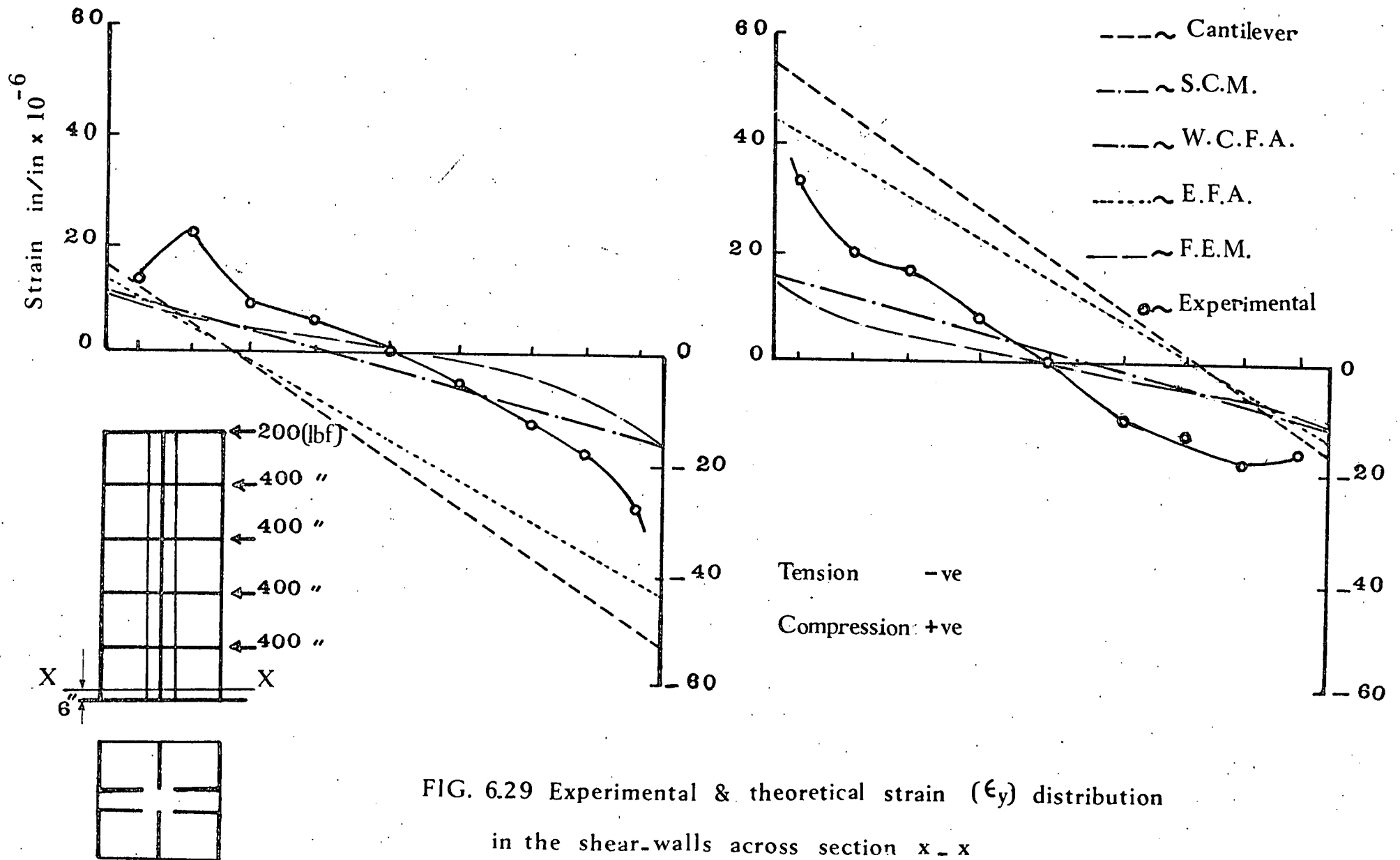


Fig 6.28 Experimental & Theoretical Strain ' ϵ_y ' Distribution along Section Z-Z



and compares the experimental and theoretical tip deflection of the structure. Figures 6.23 and 6.24 compare the experimental deflection of the five-storey model structure (at a lateral load producing 6 lbf/in^2 average shear stress at the ground floor) with results obtained from the analytical methods.

Experimental strain variation along the height of the structure and the corresponding theoretical values assuming a fully effective flange width are compared in figures 6.25 and 6.26.

The Cantilever Method, ignores the coupling action of the floor slabs and thus the bending moments and the deflections are overestimated (figure 6.21). The discrepancy between calculated and measured deflections using this method was 40%. The E.F.A. gave the closest result, overestimating the deflection by 5%, but when the shear deformation in the columns was considered, the overestimation was 14%. It was, however, found that the shear deformation in the connecting beams or the floor slabs had insignificant effect on the overall deflection of the structure.

The Cantilever Method and E.F.A. both overestimated the strain distribution along section y-y (figure 6.25). The discrepancy between these two methods is due to shear forces generated in the connecting beams. Since the stiffness of the connecting beams is comparatively smaller than the stiffness of the columns (a ratio of 1 to 635) the shear forces developed in the beam will be small, and hence have insignificant effect on the overall bending behaviour of the structure. Thus, in this particular case the analogy is somewhat similar to the cantilever method.

Referring to figure 6.23, S.C.M., W.C.F.A. and F.E.M. (see figure 6.22) all underestimated/

all underestimated the experimental deflection by approximately 73%.

The strain distribution along section y-y (figure 6.25) as obtained by F.E.M. and W.C.F.A. appear to follow the same pattern as the measured values, with the F.E.M. giving closer results at the middle of each storey and away from the region of stress concentration. The discrepancies between the results of the above two methods and the experimental values are high near the foundation and close to wall slab junctions which is attributed to the effect of stress concentration. In the upper region of the structure where strains are low, the deviation of the measured values when compared with the theoretical values, although they may be high, are quite tolerable when the design is based on maximum stress at the base of the structure. As shown in Figure 6.29 the vertical strain distribution across a horizontal section in the bottom of the shear walls is non-linear. With the exception of the F.E.M., the theoretical methods used assume a linear strain variation across the horizontal section and thus do not give accurate results.

Also referring to Figure 6.14 the existence of stress concentration in the shear walls near the foundation, openings, and close to wall slab junction is apparent. Photoelastic investigations^(118,119) reveal that the maximum stresses are always produced in the shear walls at re-entrant corners near the openings. From Figures 6.14 however, the stress concentration in the shear walls close to the flanges are suppressed due to the stiffening effect of the flanges.

Referring to Figure 6.26 the close agreement between the experimental strain and E.F.A. results along section z-z may be coincidental. The stress along/

stress along this section are small owing to the stiffening effect of the flange.

The S.C.M. does not present the correct bending moment pattern and hence, its use is not recommended to predict the stresses and the deflection of masonry structures.

The theoretical deflections and strains using different approaches and assuming an effective flange width of ten inches i.e. $4t_f + t_w$, are given in Figures 6.24, 6.27 and 6.28.

Figure 6.24 also compares deflections obtained using W.C.F.A. taking into account:

- (i) Bending deformation
- (ii) Deformation due to bending and shear
- (iii) Assuming effective slab width equal to one half of the structures bay width, and taking into account shear deformations in the beams and columns. (Shear deformation in the beams may be neglected).

The errors with respect to the experimental results in the above three cases were: 54%, 46% and 36% respectively, showing that the reduction in the flange and the slab effective width will not close the gap between the experimental and theoretical idealized structures.

The experimental results shown in Figure 6.15 clearly indicates the existence of flexural strain in the connecting slab, suggesting that the assumption of infinitely stiff ends in the connecting beams is invalid. This illustrates that the application of W.C.F.A. to multistorey masonry crosswall structure would lead to an erroneous results.

In view of the low tensile/compressive strength ratio of concrete masonry and/

masonry and non-linear behaviour of the vertical strains in the shear walls a useful result has been the location and determination of magnitude and direction of the principal strains (see Figure 6.16). Experimental and theoretical investigations carried out on similar types of structures in 1/6 scale and full-scale load bearing brickwork (52,74,96) confirm the results of the present investigation. When theoretical approaches such as cantilever method, W.C.F.A. and S.C.M. were applied to brickwork structure (96) they neither gave close estimate of the deflections nor did they determine its correct profile. Also, strain measurements were limited to the ground floor walls revealing a non-linear relationship and having high magnitude at the corners near the openings. When the E.F.A. was applied to the above structures although it gave a close estimate of the tip deflection, nevertheless, it overestimated the stresses at the base.

From the deflected shape of both the brickwork, and the blockwork structures, it is likely that under lateral load rotation takes place at foundation level. This has already been discussed for the single storey tests (Section 4.19).

6.10 CONCLUSIONS

- (1) The stresses and the deflections of one-third scale blockwork structures can be studied with reasonable accuracy by means of model testing.
- (2) The load factor, based on the assumption of 50 lbf/in² ultimate shear strength of the ground floor shear walls (Section 4.18) was 3.6 on the basis of the Code of Practice CP3 Chapter V. and 2.1 on the basis of CP 111. Live loads have not been taken into account.

(3)/

- (3) Vertical strain distribution across a horizontal section in the ground floor shear walls is non-linear.
- (4) There are region of stress concentration at the corners near the openings.
- (5) Existing methods of analysis of shear wall structures do not apply to a multi-storey blockwork building.

The cantilever method overestimates deflections and stresses.

The equivalent frame analogy gives good estimate of the tip deflection while overestimating the stresses.

In wide column frame analogy the assumption of infinitely stiff end segments in the connecting beams does not hold good.

The finite element method gives the same pattern of the stresses with a close estimate at the mid-height of the shear walls in each storey but it underestimates the stresses close to the corners near the openings.

- (6) For design the cantilever method may be used to calculate the stresses and the lateral deflections; results are conservative. The equivalent frame analogy is suggested if more accurate results are required.

CHAPTER 7

GENERAL CONCLUSIONS

The following conclusions have been reached as a result of the investigations presented in this thesis.

7.1 MODEL TESTING AND LOAD BEARING CAPACITY OF LIGHT-WEIGHT AGGREGATE WALL PANELS

As it would have been uneconomical and difficult to conduct the investigation on full scale structures a series of comparative tests were conducted on one-third and full-scale blockwork which indicates that the strength of blockwork for a given block and mortar strength can be reproduced by means of model tests.

Depending on the relative strength of the block and the mortar under axial compressive loads, concrete blockwork exhibits two distinct types of failure:

- (1) Shear failure when the block and the mortar approximately possess the same strength
- (2) Tensile splitting failure when there is an appreciable difference between the strengths of the two components.

The tensile splitting test (Brazilian) can successfully represent the tensile strength of the blocks and can be satisfactorily adopted. For the blocks tested, there was a linear relationship between the corresponding compressive and tensile strength. This relationship is useful as the strength of blockwork under axial compression may be related to the tensile strength of the blocks.

The relationship between block and blockwork strength is non-linear.

Hermann's⁽⁴⁷⁾ equation is used to predict the blockwork strength for 1:1:6 mortar when/

mortar when block/mortar strength ratio is larger than 1.7.

The variation between the secant modulus of elasticity and the blockwork strength was found to be non-linear.

The load factor based on the draft revision of CP 111⁽¹⁶⁾ although lower than the values suggested by the code, seems to be adequate for the present design purposes till results of further tests are available.

7.2 PROPERTIES OF BLOCKWORK ASSEMBLIES

Tensile bond strength of "Aglite" block couplet built with $1:\frac{1}{4}:3$ mortar is 0.78 times their initial bond shear; precompression increased the shear strength of the triplets. The triplet tests do not represent the true behaviour of blockwork structures, because no shear stresses are developed in the blocks. Due to the lateral loading shear stresses are developed in masonry structure causing failure either by diagonal shear or diagonal tension.

7.3 SINGLE STOREY SHEAR WALL STRUCTURES WITH OPENINGS SUBJECTED TO PRECOMPRESSION AND LATERAL LOADING

Under combined compressive and lateral loads concrete block masonry shear wall exhibit two distinct types of failure:

- (1) Breakdown of bond at the interface of block and mortar producing diagonal cracks stepping through the vertical and horizontal mortar joints. The shear strength of blockwork is then the summation of bond shear and the frictional resistance between block and mortar.
- (2) Diagonal cracks pass through blocks and a few mortar joints. This type of failure only occurring above a certain range of precompression is/

precompression is caused by diagonal tension governed by the maximum tensile stress or strain. The shear strength in this case is directly proportional to the frictional resistance.

Furthermore, for a given lateral load precompression increases the rigidity of the structure. Above certain value of precompression, the rigidity remains constant.

The model shear walls have an average load factor of 2.9 against failure when designed on basis of permissible shear stresses quoted in CP 111. This would suggest that the present provisions given by the relevant clause in the Code is adequate for the design of non-reinforced blockwork shear walls. At high precompression analytical methods such as the equivalent frame analogy and the finite element method gave good estimates of the deflection and the stresses in the shear walls.

7.4 MULTI-STOREY SHEAR WALL STRUCTURE

The analytical methods such as the continuum approach and frame analogies all assume linear elastic behaviour. The methods when applied to multi-storey masonry shear wall buildings do not predict the actual behaviour of the structure. The gap between experimental and theoretical results may be due to idealization and assumptions regarding the interaction between the various structural elements, non-homogeneity of the material, shrinkage cracks, method of construction and workmanship.

The shear connection method is not recommended since it does not appear to be satisfactory for the analysis of blockwork structures. Nor would it/

would it seem that the wide column frame analogy could be widely applied to blockwork structure, since the method, similar to the shear continuum approach, greatly overestimates the stiffness of the structure.

Under special circumstances and in the light of more experimental data one might expect that a rigorous analysis such as the finite element method would give more accurate results. However, for design purposes the following procedure is suggested:

From the layout of the structure and vertical loading the thickness and strength of blockwork can be determined. Using the cantilever method the stresses and deflections due to lateral loading are calculated - the result is conservative as it overestimates the stresses and lateral deflections. However, the equivalent frame analogy is suggested if more accurate results are desired.

7.5 SUGGESTIONS FOR FURTHER RESEARCH

The work presented in this thesis has shown that there is a need to investigate some aspects in details.

Lateral loading tests may be carried out on simple single storey or multi-storey structures to study the following factors:-

- 1) The effective flange width and its variation with precompression and the overall height of the building.
- 2) The effective width of the floor slab. The stiffness of the floor slab has a significant influence on the behaviour of coupled shear walls.
- 3) Influence of the size of the openings.
- 4) Assessment of the degree of fixity at the foundation level and its variation with precompression.
- 5) Development of a more refined theoretical analysis. One possibility is the finite element technique applied to three dimensional structures.

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APPENDIX 1A1.1 THE 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 £57 to £128.

Stainless steel demec studs 112 at £1.75.

Calibration factor by Cement and Concrete Association.

2 inch gauge	2.48×10^{-5}	strain per division			
8 " "	1.01×10^{-6}	" " "	"	"	"
12 " "	6.6×10^{-6}	" " "	"	"	"
24 " "	3.33×10^{-6}	" " "	"	"	"

Description

The "Demec" gauge is a demountable strain gauge (Plate 4.8) developed by Cement and Concrete Association^(1B), being obtainable at various gauge lengths. Its main components consists of a 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} 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 3×10^{-6} can be obtained. A small temperature correction could be applied to the gauge.

A1.2 VIBRATING WIRE STRAIN GAUGE

Gauge type:

Tyler Recoverable Surface Mounting (Plates 4.17 and 4.18), manufactured by Gage Technique at a cost of £11.00 each. (114).

Gauge length used:

2.5 inch and 5.5 inch.

Gauge factors:

$.54 \times 10^{-9}$ for 2.5 inch gauge

3.00×10^{-9} for 5.5 inch gauge.

Plucking Voltage:

24 volts for the 5.5 inch gauge

60 to 120 volts for 2.5 inch gauge

Accuracy: 1.5 to 3×10^{-6} strain for equipment reading to 1HZ.

Mounting of Gauges

$\frac{1}{4}$ -inch thick brackets were fixed onto the test specimen (Plate 4.16) using an epoxy resin commercially known as Cataloy paste. To ensure that the brackets were in the same plane and the correct distance apart they were bolted to a steel plate which kept them in position while the paste hardened. The plate was then removed and the gauge was fixed in its position.

Measurement of Frequency

A portable digital strain measuring instrument, type P.S.M. used to measure the period over 100 or 1000 cycles. The instrument was manufactured by Deakin Instrumentation at approximate cost of £600.

A1.3 DIAL GAUGES

Specifications: Baty Dial Gauge costing approximately £8 each.

1 division = 0.0001 inch.

Range 0.2 inch or 0.5 inch.

A1.4 HYDRAULIC JACK

Enerpac single-acting cylinder with spring resisted return and self sealing valve used for loading single and 5-storey shear wall structures.

Specifications:

Type	Stroke (in)	Collapsed height (in)	Cylinder effective area (in ²)	Plunger dia. (in)	Internal pressure at capacity (lbf/in ²)
RC1010 10 Ton	10 $\frac{1}{4}$	13 $\frac{1}{4}$	2.236	1 $\frac{1}{2}$	10,000
RC224C 23 Ton	13 $\frac{1}{2}$	20 $\frac{1}{4}$	5.157	2 $\frac{1}{4}$	10,000

A1.5 THE STRAIN GAUGE LOADCELL

Davey-United toroidal loadcells with self aligning loading cap were used to measure the load. Details of load cells used are as follows:

Type	Normal max. loading (tons)	Max. input potential (volts)	Nominal output (mV)	Impedance
A	3	10	13	200 ohms
A	5	10	10	"
A	10	10	8	"
B	20	10	10	300 ohms
C	100	10	7	350 ohms

10 volt. D.C. stabilised power was used throughout.

A1.6 DIGITAL VOLTMETER

The output from the loadcell was measured by a "Dynamco" Digital Voltmeter Type DM 2022 having resolution of 10 μ Volts. Later a Solartron digital voltmeter type LM 1450 was used. The operational range of 0 - 1000 volts is covered in six ranges, with a maximum sensitivity of 10 μ Volts, and an accuracy of $\pm 0.05\%$ of reading $\pm 0.05\%$ of range full scale.

A1.7 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 inch

Width : 7 inch

Height : $5\frac{1}{2}$ inches

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.

APPENDIX 2A2.1 FINITE ELEMENT PROGRAM

The ICES STRUDL⁽¹¹²⁾ program was used for the finite element analysis. Using this method the structure may be sub-divided into two-dimensional elements of triangular or rectangular shape, connected at a finite number of nodal points, or joints. This idealization is then analogous to that of framed structures where members are a particular type of element. The finite element capability of this program applied to two dimensional problem is quite extensive, and provided a variety of element types for the solution of plane stress or plane strain problems. The element chosen was rectangular (Type PSR) with four nodes giving eight degrees of freedom, the displacement function being of the form:

$$u(v) = a + bx + cy + d xy$$

where: a, b, c and d are constants. Values of elastic modulus and Poisson's ratio are specified for each element and the plane strain analysis was used. The program was run on a I.B.M. 370/155 computer. The running time for the specified structure composed of 364 nodal points and 324 elements was 6 minutes and 42 seconds and the cost was approximately £30.00

A2.2 FRAME PROGRAM

A standard program provided by I.B.M. known as "The Structural Design Language" Strudl⁽¹¹²⁾ was used. This program is capable of performing a linear elastic, static analysis of a framed structure. Such structures may be two or three dimensional and are composed of slender linear members of constant or variable section which can be represented by properties defined along a centroidal axis. A variety of force conditions on member ends and at support joints may be specified implicitly by means of structural type and/

type and orientation commands or explicitly for a member or joint.

Both equivalent frame and wide column frame solutions were obtained by use of the above program. In the W.C.F. analysis beams were assumed to have variable cross-section, the first segment having infinite sectional area and moment of inertia and the second segment having the actual properties of the beam.

The running time for the above program was 12 seconds and the cost was £0.87p.

A2.3 SHEAR CONNECTION PROGRAM

The computer program used was based on Eriksson's paper⁽³⁴⁾ and written in Imp. (atlas autocode)⁽⁵⁰⁾. The running time on a 4-75 computer was 8 seconds and the cost was £0.25p.

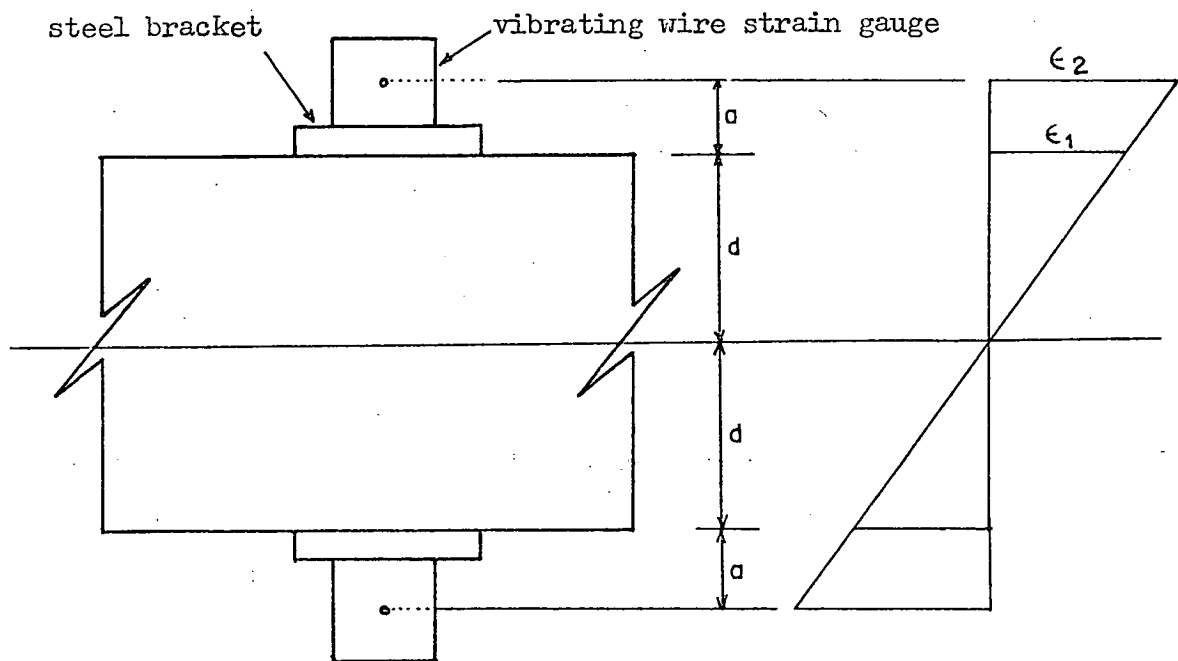
The program concluded with the required analysis, bending, shear and deflection for two interconnected walls. It takes into account axial deformation of the columns and in the calculation of the deflection it neglects the shear deformation of the columns.

A2.4 THE APPROX ROUTINE

Statistical analysis of the test results was based on "approx" a library routine program. The routine was written in Imp. (Atlas Autocode) and run on a 4-75 computer at £0.18p.

APPENDIX 3CORRECTION FACTOR TO BE APPLIED TO THE VIBRATING WIRE GAUGE READINGS
WHEN MEASURING FLEXURAL STRAIN.

If the distance of the vibrating wire from the surface of the specimen is appreciable in comparison to the specimens depth, the vibrating wire gauges indicates strain considerably higher than the true value. Therefore it is necessary to apply a correction factor to all readings.



specimen under flexural loading (strain diagrams)

$$\epsilon_1 = k \epsilon_2$$

where : $k = \frac{d}{a + d}$ = correction factor

$$\epsilon_1 = \text{true strain}$$

$$\epsilon_2 = \text{measured strain}$$

$2d$ = depth of the section

a = distance of vibrating wire from the specimens surface