BRICKWORK PANELS UNDER LATERAL LOADING

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by

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CONTENTS

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ABSTRACT

The object of the work presented in this thesis was to investigate the strength of unreinforced brickwork panel walls under lateral loading with zero axial load. One-sixth scale model walls, with different aspect ratios, were tested simply supported on three and four sides. With each wall companion specimens were built and tested to investigate the modulus of elasticity and the modulus of rupture of brickwork.

Theoretical investigations and analyses were carried out using the yield line, the elastic theory and the strip method of design. Results were discussed and compared with the experimental work, and conclusions were drawn at the end of the thesis.

CHAPTER 1

INTRODUCTION

INTRODUCTION

1.1 GENERAL:-

1.1.1 Brickwork Research:-

Brickwork design, until recently, was largely empirical in approach. Historical example and the rule of thumb were the most common justification for the brickwork systems being used. Despite the use of brickwork over thousands of years, its potentials hardly have been utilized and our knowledge of its structural behaviour is still quite limited.

The last two decades have seen the revival of brickwork as a major structural material. Theory and research have produced a lot of information on brickwork, some of it is still controversial, but it is well realised that recent developments in this subject can be of great value in the economical and efficient design of structures of all kinds.

A significant feature of modern brickwork construction is that it is designed on sound scientific principles. Not surprisingly, the new design methods have made it more necessary to know and understand the behaviour of brickwork under load. This knowledge has grown tremendously over the last few years but there are still many important aspects of which we know very little, among them is the behaviour of brickwork under lateral load.

1.1.2 The Lateral Stability of Non-loadbearing Brickwork:-

There is now growing interest in the stability of brickwork wall panels under lateral loading with no in-plane forces. This kind of wall can be easily damaged by wind, impact loads, or domestic gas explosion. Such stability problems usually arise in in-fill panels between steel or concrete columns, gable end walls or in boundary walls. Also in load-bearing construction the problem can be met with in the top storey walls where the vertical load is comparatively small, or even in the lower most storeys where filling panels and partition walls can span between the bearing cross walls of the system.

The problem of lateral stability is gaining considerable interest with the new developments in brickwork construction and the general trend towards more slender structures with greater spacing of building frames and columns. Moreover, the increased design wind pressures specified in CP3 Chapter 5, Part 2 (1970) (1) and the Fifth Amendment to the Building Regulations (2), guarding against progressive collapse, made the problem more urgent and a particularly pressing one.

Designers have always assumed that no tensile stresses can be taken by brickwork. This approach is necessitated partly because of the low flexural and bond strengths of brickwork and mainly because of the lack of better knowledge.

1.1.3 Design Specifications and Safety Factor:-

Most national building codes are restricting the use of masonry walls by allowing no tensile stresses to occur in such panels and by introducing limiting factors governing the height to thickness relation and the interval of the vertical or the horizontal supports.

In Britain there is at present no code of practice which deals with the design of non-loadbearing walls called upon to resist lateral loading. One difficulty in preparing a design specification for laterally loaded brickwork panels is the great variability that can occur both in the strength of the materials and the quality of the workmanship.

When there are many factors influencing the strength of a structure and the effect of each factor is known only within wide limits, there is sometimes a tendency for engineers to adopt limiting safe assumptions for each factor. The result may be that the overall load factor is unnecessarily high for most practical examples and subsequently the result is an inefficiently and an uneconomically designed brickwork structure.

1.1.4 The Modulus of Rupture of Brickwork:-

The maximum tensile stress in brickwork at failure is usually termed the modulus of rupture of brickwork. As the bending strength of brickwork spanning horizontally is greater than the bending strength spanning vertically, then there are two main values for the modulus of rupture. To determine the load carrying capacity of a panel the moduli of rupture values should already be known.

Brickwork is a composite material so both the brick and mortar properties affect the modulus of rupture value. In general the modulus of rupture is affected by the compressive strength, the suction (IRA) of the bricks and the mortar tensile bond strength, which in its self is influenced by many other different variables. Other factors such as workmanship, curing, loading and testing methods can also have an appreciable influence on the resulting modulus of rupture values.

CP 111: Structural Recommendations for Loadbearing Walls[3] gives figures for the modulus of rupture of 0.07 MN/m^2 for bending perpendicular (normal) to the bed joints and 0.14 MN/m^2 for bending parallel to the bed joints. The difference arises from the fact that the first is almost pure tensile bond stress whereas the latter is part shear.

The code gives only recommendations and general guidance as the transverse strength of brickwork panels is small and highly variable in character.

1.1.5 Theoretical Design:-

Elastic theory and the yield line theory have been extensively applied for the analysis of slabs and plates. It is quite reasonable to try such analytical approaches for the design of brickwork panels subjected to lateral loading.

Elastic methods of analysis usually involve a considerable task of numerical calculation, and the most rational treatment of this is by computer. A computer programme based on the finite element method is used in this work. The yield line method of analysis is also proposed for the design of brickwork panels. It is easier and simpler to apply, and it does not involve any complicated numerical calculation.

1.2 The use of model bricks in brickwork research:-

The use of models is a common practice in engineering research. In most civil engineering projects it is practically impossible to construct a full scale model for the purpose of testing and investigation.

In high rise brickwork buildings, to study the relative behaviour of different members of the structure and the behaviour of the structure as a whole is not an easy job as testing a prism or a single wall. Considerable work has been done on model testing, partly to investigate the strength of brickwork structues and partly to establish the validity of a suitable and reliable technique.

Perhaps the most important work in this subject was that carried out by Hendry and Murthy[4, 5] in 1965. To ascertain the suitability of model bricks, tests which had been done on full scale brick piers were repeated in one-third and one-sixth scale models. These tests were concerned with the relationship between the principle properties of brickwork and mortars.

Furthermore some full scale tests, investigating the interaction between storey height walls and floor slabs, were repeated in one-sixth scale model[5]. It was concluded that it is possible to reproduce the strength of full scale brickwork by means of model tests, thus confirming the suitability of model bricks for structural investigation.

All the work done in this programme was carried out using one-sixth scale model bricks. Test beams and prisms were also built to obtain the properties of the materials used.

CHAPTER 2

REVIEW OF PREVIOUS WORK ON LATERALLY LOADED BRICKWORK PANELS.

Davey

Thomas

Monk

Bradshaw and Entwisle

Hallquist

Losberg and Johanson

Nilsson and Losberg

Haseltine and Thomas

Satti

Baker

West, Hodgkinson and Webb

Haseltine and Hodgkinson

REVIEW OF PREVIOUS WORK

Brickwork has been used for thousands of years but the scientific investigation of its strength is comparatively recent. In about 1888 a committee of the American Society of Civil Engineers conducted what appears to be the first series of tests on this subject, and a few years later a committee of the Royal Institute of British Architects carried out a similar series of tests, on piers and small walls, mainly to investigate the compressive strength.

From that date onward a considerable amount of data has been steadily obtained on the compressive strength of such piers and walls, but little or no knowledge has been gained on the lateral strength of brickwork panels and walls.

Some early research, carried out at the Building Research Station, to investigate the behaviour of brickwork panels, subjected to lateral loading, was reported by Davey (6). Walls with and without precompression were tested. The walls without precompression were approximately 2.7 meters high by 2.1 meters long with a solid thickness of 23 cm and 34 cm. The lateral load was applied to the wall by means of hydraulic jacks, using a grid of steel joists to distribute the load to sixteen points on the wall surface. Later, air pressure from a rubber bag was used to give uniform loading on the walls. Some of the walls were freely supported on four sides with no in-plane forces. These walls failed in a yield line pattern of cracking and no increase in load was possible after initial cracking. When edge restraint was provided by building the walls into a steel channel surround, a considerable increase in the wall capacity was observed even after the wall had cracked. Internal arching of the brickwork, and its thrusting upon the surrounding frame caused it to act as a flat dome.

Thomas (7) also reported some tests carried out at the B.R. Station. In these tests, all the panels with different thicknesses had a length of 3.3 meters and a height of 2.7 meters and were built within concrete cased steel frames. The walls were laterally loaded at 16 points, so spaced that the conditions were not very different from those that would be obtained with uniform lateral pressure. The test results showed that considerable increase of load occurred after the initial cracking of the walls. This work was part of a programme investigating the strength of brickwork and no theoretical calculations were made, but it was concluded that although the tensile strength of brickwork is low and variable, considerable resistance to lateral forces can be developed by wall panels built into a steel framework. Such panels also have a considerable stiffening and strengthening effect on the resistance of the frame to cracking forces.

Transverse tests were performed by Monk (8) to investigate the lateral resistance of 15 cm brick walls: In comparison with other walls, the 15 cm solid wall was found to have a performance equal to or better than a 20 cm brick-block or 25 cm cavity wall.

Also, the laboratory walls tested by the quarter point loads, yielded transverse strength that was lower than that obtained by field tests on model buildings, tested with uniform loading from a bag. This is obvious as long as the boundary conditions and the loading methods were not the same in both cases. However, comparative tests on 1.2 by 2.4 meters panels, showed that a uniformly distributed load from a bag, gives higher mean transverse strength than that given by two line loads applied at the outer quarter points of the span. The work was mainly experimental and no calculations were undertaken.

Bradshaw and Entwisle (9) discussed the case of infill panels with different support conditions. They presented some graphs and notes as an approximate method for determining safe panel sizes and wall thickness for different pressure intensities. In calculating and plotting the graphs, they used nearly the same moment coefficients given in CP 114 table 17 for two way span reinforced concrete slabs with torsional resistance, which are originally based on elastic theory assumptions.

Hallquist (10) reported some tests carried out at the Norwegian Building Research Institute on cavity walls of different support conditions. Based on these tests an elastic analysis design method was proposed. A computer programme was developed based on a finite element procedure for displacement of plate bending. Good agreement was found between the measured and the calculated deflections, also the calculated stresses at first crack loads were found to be close to the average modulus of rupture. The two wythes of the wall were observed to have the same deflections and about half the load was carried by each wythe. It was concluded that masonry walls subjected to uniform lateral load will act as elastic plates in bending and may be designed using calculation method based on the theory of elasticity for thin anisotropic plates in bending.

In support of the yield line method of analysis, came the report of Losberg and Johanson (11). In this report 11 full-scale half-brick walls were tested, all of them were supported at their four edges and acted upon by a distributed pressure. The support frame is supposed to give free rotating along three of the edges and a minor restraint moment at the lower edge where the wall is resting via a mortar joint against the floor.

The test observations showed that the first crack could begin at relatively low load, after which a remarkable increase was observed. The final cracks at failure are very similar to the yield pattern obtained by testing for two-way reinforced concrete slabs. In applying the same yield line theory to the brick panels good agreement between the calculated load and the experimental failure load was claimed. Discussing the suitability of the yield line method to calculate the carrying capacity of brickwork panels, bearing in mind the brittleness of the brickwork and the early formation of cracks, the authors referred to Johanson's (12) work on plane concrete slabs on soil, saying that there is a considerable moment capacity in the cracks due to an arch effect. Free deformation between the cracked parts of slab could be prevented by the surrounding parts of the slab.

In the tests reported by Nilsson and Losberg, [13] prefabricated panels supported on all four sides were tested. The panels were 280 cm high and 196 cm long and consisted of two brick leaves with an overall thickness of 14 cm. One of the walls was kept plain while the rest were reinforced in both directions. In the analysis carried out both the elastic theory and the yield-line theory were proposed. The former to predict the cracking load and the latter to estimate the failure load. All the walls failed in a yield-line pattern of cracks. The unreinforced wall failed suddenly without prior warning and so the cracking load was equal to the failure load. For this wall the cracking pressure was calculated in accordance with the theory of elasticity and showed good agreement with the test results. The presence of the reinforcement in the walls increased the ultimate capacity and safeguarded the panels against sudden collapse. For the walls reinforced with a mesh of 5 mm bars at 150 mm, the load increased considerably after cracking, and failure occurred gradually after large deflection. For these walls the failure pressures were closely estimated by means of the yield-line theory.

Following the collapse of a London block of flats due to a gas explosion in 1968, which in one way or another led to the publication of the Building (Fifth Amendment) Regulations (2), a series of tests has been carried out at the British Ceramic Research Association Lab. (14). Storey height walls spanning vertically were tested in order to establish the relationship between lateral load and precompression, and to obtain data to be used in the design of brickwork panels to the Fifth Amendment. At low compressive load the failure is found to be hinge like, and the height of the wall is increased as the upper and lower halves rotate. Increasing the precompression loads, the mode of failure changes and there is some slight crushing failure of the bricks on the laterally loaded face of the wall. The relationship between lateral load and precompression is almost linear for the range of precompression loads needed for practical purposes. For low pre-loads a deviation in the relation could be possible due to the influence of the tensile bond of the joints. At high pre-loads too, the crushing of the bricks affects the results and the relation is linear no more. To emphasise the effect of end restraints, a wall was tested with end returns at both sides. The effect was very clear. The lateral load carried by this wall was more than twice the load carried by the equivalent wall without returns and the failure lines were typically yield line pattern of cracks.

Satti's[15] work was carried out on one-sixth scale-model brickwork with varying length to height ratios, the lateral load being applied through an air bag. The walls were supported on either three or four sides and some of them with precompression load. It was observed that the walls failed in a yield-line pattern of cracks comparable with that obtained for reinforced concrete under the same loading. Different from the gradual formation of cracks in the case of concrete, the brick walls failed suddenly with no cracks being detected previously. Most of the precompressed walls failed suddenly like those without precompression, and no cracks were observed before failure. Some precompressed walls with all four sides supported experienced cracking and full yield pattern formation before failure. Hence, beside adding to the overall strength of the walls, the vertical precompression contributed to the general stability, thus, producing a change in the mode of failure.

Elastic calculations using finite element analysis were carried out to trace the distribution of maximum tensile principal stresses in the walls. The patterns of failure traced, using this analytical method, were found to be similar to the actual experimental failure modes produced by the walls. In all these failure modes distinct yield-line patterns were shown. Also, the failure load was calculated using as a criterion of failure the attainment of the modulus of rupture at the point of maximum tensile principal stresses. The calculated load was found to be much less than the experimental one.

Calculating the moment of resistance of the wall from the modulus of rupture, taking the ratio of moments of resistance in orthogonal directions as 5, the yield-line formula also underestimated the failure load. However, using the same moment of resistance in both direction (ratio=1:1) more consistent results were obtained. This cannot justify the use of the yield-line as long as the ratio taken is not true and merely assumed.

Using single-leaf one third scale model brickwork panels, Baker[16] presented his work. The panels, with constant height of 680 mm had different aspect ratios and different support conditions. The panels were tested by applying uniform transverse loads from a water bag placed under the horizontally laid panels. The basic bending properties of the brickwork were obtained from bending tests on prisms and beams cut from the panels after being tested. The prisms, as well as the beams, having different spans gave different values for the corresponding strengths. Taking into account results of separately built prisms and beams, adjusted values were adapted for both the strengths of brickwork spanning in both the vertical and horizontal direction respectively. From the experimental results, it was observed that in some panels complete failure did not occur with the beginning of the initial cracks and there was a considerable reserve of strength.

The yield line theory, the elastic theory and an empirical strip action theory were proposed as possible methods of design. Using yield-line theory, taking the bending strength in the vertical direction as the basic strength and using a strength ratio of 3.44 the collapse load was overestimated. However, it was reported that using a strength ratio of 2, the collapse load was closely predicted by this theory. The elastic theory was found to underestimate the failure load for all the panels except the panels with three sides simply supported. The proposed empirical strip theory assumes that the total load capacity of a panel is the sum of the load capacities of two independent strips spanning horizontally and vertically respectively. In comparison with the experimental results good agreement was found with panels supported on four sides and conservative results when supported on three sides.

West, Hodgkinson and Webb,[17] in their paper of 1973, discussed only the experimental work being carried out at the British Ceramic Research Association. All the walls tested were 2.6 meters high and mostly 5.5 meters long. They were built within rectangular frames, constructed of steel channels and acted upon by a uniformly distributed pressure. The object of the tests was to determine the lateral resistance of walls built of various bricks and mortar and having different degrees of peripheral fixity, including the incorporation of several formats of window and door opening. The theoretical investigation of these test results were given by Haseltine and Hodgkinson [18]. As the presented results were only part of an incomplete programme no final conclusions were drawn, but the yield line theory as well as the elastic theory were suggested as a possible method of design. Both theories were found to give a low estimate for the strength of the walls.

This is the review of the existing published work to date on the strength of brickwork panels under lateral loading. It is clear that the work done is still in its very early stages and much research is needed to get results to form sound bases of design. The following chapters describe a programme of tests carried out on one-sixth scale model walls supported on three and four sides.

CHAPTER 3.

MATERIAL PROPERTIES

MATERIAL PROPERTIES

3.1 INTRODUCTION:

Brickwork is a bonded structure of bricks and mortar, the individual properties of which influence one another and together determine the load carrying capacity of the brickwork. So, the type and shape of the bricks, their dimensional tolerences, the type of bond and the quality of the workmanship, altogether have great influence on the resulting strength of brickwork.

The degree of accuracy and exactness of the calculation carried out and their prediction to the actual failure loads depend to great extent on the values of material constants used in the calculations. There is often wide variation in the resulting strength of brickwork but with well prepared materials, good workmanship and careful method of testing, reasonably steady results could be obtained.

In this chapter the properties of bricks, mortars and model brickwork were investigated to establish the basic properties of the model brickwork used in preparing the test walls. The tests described in this chapter were mainly to determine the moduli of elasticity and the moduli of rupture of the walls in the vertical and the horizontal directions. From results of these tests, and similar results from additional prisms and beams, the relation between the moduli of rupture in the two perpendicular directions was plotted and some conclusions were drawn.

3.2 MATERIALS

3.2.1 Bricks:

When brickwork is stressed in compression the compressive strength of the individual bricks usually has great effect on the resulting compressive strength of the brickwork. Although in panels under lateral loading the brick strength is not the decisive factor on the overall strength of brickwork but still it has a direct or indirect role to play.

Tests by SCPRF[19] and Hallquist[20] showed that bending strength increased with the increase of the compressive strength of the bricks. It would seem reasonable to assume a relation between the bond strength, the suction rate and the compressive strength of the bricks. A strong brick with low suction can develop good bond with the mortar thus resulting in a higher value for the lateral strength of the brickwork.

The tensile strength of bricks is usually measured by three different methods of testing. The direct tension test[21], the transverse bending test (21, 22) and the Brazilian or the indirect tension test (21, 22). Like brickwork in tension considerable variation could be observed in tensile strength of bricks, particularly when using different methods of testing.

A type of wire-cut one-sixth scale model bricks from two batches were used in carrying out this work. The bricks were tested in compression according to B.S. 3921, and the results are shown in table (3.1).

3.2.2 Mortar

The sand used for the mortar mix was Leighton Buzzard sand No. 19 of 25/52 grading. "Ferrocrete" rapid hardening cement and hydrated lime in complience with the relevant British Standards were used through out the tests. The ratios were $1:\frac{1}{4}:3$ cement: lime: sand by weight. Sufficient water was added to each mix to produce a workable consistancy.

To test the hardened mortar strength, 1 inch cubes were prepared in the above ratios. The cubes were cured in water for seven days then kept at ordinary lab temperature for the rest of the period. An Instron testing machine was used for testing the cubes, without packing, between moulded faces.

The mean compressive strength for 7, 14 and 21 days is given in table (3.2).

3.3 BRICKWORK PROPERTIES:

3.3.1 Compression Tests:

One of the important tests carried out on brickwork specimens is the compression test. Piers 4 cm square and 3 courses high were built of each type of brick and the same mortar. The piers were tested under the Instron testing machine.

Almost all the piers failed in the typical compression failure. The failure was generally along two vertical planes at right angles to one another through the vertical joints. Table (3.3) shows the mean crushing strength for 7 and 14 days.

3.3.2 Bending Tests:

3.3.2.1 General

Although tensile strength and sheer strength have been regarded as relatively minor properties of brickwork, the increasing interest in brickwork structures has underlined the importance of these properties. The bending strength of brickwork is usually due to tensile bond, or combination of tensile bond and sheer when brickwork is spanning horizontally.

As the bending strength of brickwork spanning horizontally is usually greater than the bending strength spanning vertically, then there are two main values for the modulus of rupture. The first one F_p which is the modulus of rupture of brickwork when stressed in bending parallel to the bedding plane. The other F_n when stressed in bending normal to the bedding plane. When beams are stressed spanning inclined at an angle to the bed joints, an intermediate value of the modulus of rupture between F_n and F_p could be obtained. In order to be able to analyse the failure pressures of the walls, the moments at failure parallel and normal to the horizontal joints should be determined by means of detailed tests. Sahlin in his book (22) discussed the various factors affecting the modulus of rupture of brickwork, reporting work done by SCPRF and others on this subject. Also some work including a review of the literature and existing work was given by Satti and Hendry (15) (23).

One of the important factors affecting the modulus of rupture is the mortar tensile and bond strength. Tests by SCPRF (19) show that the modulus of rupture increases with the increase of the tensile strength of mortar but not in direct proportion. The tensile strength in itself and the bending strength were affected by the brick suction and the mortar composition and water retentivity. In general the bending strength decreases as the suction increases, with high suction bricks the bending strength is approximately proportional to the water retentivity.

Some authors (19) (20) have reported the increase of modulus of rupture with the increase of brick strength. As mentioned in section 3.2 the influence of brick strength is only indrect because of the change of suction with the change in brick strength. The same could be mentioned for the relation between the brickwork and the modulus of rupture as the brickwork strength is influenced by many of the factors which determine the modulus of rupture.

According to results published by SCPRF (19) a series of panels were built using the same type of bricks with varying bed joint thickness. The tests results showed a decrease in the bending strength with the increase in the bed joint thickness.

Beside the previous factors and the workmanship and curing, the modulus of rupture could still be affected by other variables. The span of the specimen, the method of testing and the nature of loading could have a marked effect. There is presently no standard test for flexural strength of brickwork. The 70 mm span prism and the 4 courses 280 mm (8 bricks) span beam which are convenient for testing were adapted as specimens for determining the modulus of rupture of brickwork.

3.3.2.2 The Moduli of Rupture and Elasticity normal to the Bedding plane (F_n).

Prisms and beams with the above dimensions were built and tested to investigate the modulus of rupture of brickwork. Some of the prisms and beams were built with companion walls, two prisms and two beams with each wall. Those specimens were cured under the same conditions and kept until required for testing on the same day as the corresponding wall. The rest of the specimens were built separately to investigate the relation between the bending strength in the two perpendicular directions, but each prism and the companion beam were built and kept under the same conditions.

Building a prism and a companion beam was found more convenient in testing than building one specimen and testing it twice. Firstly for the strength spanning vertically and secondly - the remaining two parts - for strength spanning horizontally. When building them separately each specimen can have dimensions independent of the other specimen dimensions. This allows more freedom in choosing the method of testing and the range of the applied loads.

The prisms were supported on a span of 170 mm by two supports of 12.5 mm diameter. A quick hardening non-stick plaster was laid at the support to take up errors in alignment of the bricks. The specimen was loaded by two line loads at one-third points, the load being applied by adding weights to a suitable hanger. The arrangement of the test is as shown in fig. (3.1).

It is obvious that the prisms when loaded will fail mostly by breaking of the bond between brick and mortar, as the bond strength is the weakest element in a prism when stressed in bending. The calculated values of the modulus of rupture of the prisms are presented in table (3.5), and the values of those built in companion with test walls are shown in table (3.6).

To determine the modulus of elasticity of brickwork, two vibrating wire strain gauges (see 5.4.2)were mounted to the specimen one in each side to measure the compressive and tensile strains. The average of 4 readings were usually taken for each specimen, the stress strain diagrams were plotted and the corresponding elastic modulus calculated. Fig. (3.1) shows the test arrangement and the average elastic moduli are listed in table (3.4). The stress strain relation for a prism is shown in fig. (3.2).

3.3.2.3 The Moduli of Rupture and Elasticity parallel to the Bedding plane F_p.

As mentioned in section 3.4.3.2 each beam was built with a companion prism. The curing and testing arrangements were as mentioned previously. The arrangement is as shown in fig. (3.1). The beams were tested simply supported with two line loads at one-third points. All the failure lines were noticed to be within the middle third of the beam.

Most of the beams failed with a straight fracture line passing through two bricks and two mortar joints, while some of them fractured through one brick only and very few failed in a zigzag line following the mortar joints. The type of failure depends on the bond at the brick-mortar interface with the poorly-bonded beam failing in the zigzag line through the joints. Ryder (24) used similar beams to investigate the bond strength in brickwork, different modes of failure according to the type of bond was also reported.

The calculated values of modulus of rupture of the beams are shown in table (3.5) and (3.6).

The same arrangement as for the prisms was made to measure the strains in the tension and compression sides of the beams at different stresses fig (3.1). Fig. (3.3) shows the stress strain diagram of one of the beams and the average modulus of elasticity values were presented in table (3.3). It was observed that there is no significant difference in the rigidity of brickwork when stressed in bending about the horizontal or the vertical axis. The modulus of elasticity of brickwork could be taken the same in both directions.

3.3.2.4 The relationship between the modulus of rupture values in the vertical and horizontal directions.

Most codes do not allow tensile stresses to develop in brickwork. Those which permit such stresses usually imply that the modulus of rupture in the horizontal direction is twice the modulus of rupture in the vertical direction. Factors varying from 1 to 7 were reported by different authors. Nilsson and Losberg (13) used a factor of 1.19 in calculating the strength of a four sides supported wall. Bradshaw and Entwistle reported a factor of 4, Satti and Hendry observed a factor of 5 for 1 : $\frac{1}{4}$: 3 mortar and 7 for 1 : 1 : 6 mortar. A ratio of 3 to 6 was also observed by Nilsson (25).

Fig. (3.4) shows the relationship between the moduli of rupture in the vertical and horizontal directions. The factor Fp/Fn plotted against the modulus of rupture in the vertical direction F_n is presented in fig (3.5).

From Fig (3.4) it seems that Fp - Fn relation is independant of brick strength. This could be true as the brick strength has no direct effect on the flexural strength, the bond being the important factor. Also illustrating this, one of the highest values of the modulus of rupture was obtained with bricks not of the highest strength. Some tests by Ryder (24) indicate that with water retentive mortars, wetting the bricks to reduce their suction may cause a slight drop in transverse strength. This drop in strength can still be much bigger with low suction bricks.

Sinha and Hendry got the same relation using for Fp test a three courses beam. The results are comparable with a slight difference for the higher values of F_n, Sinha and Hendry results being lower. This could be because in plotting the curve they used Fp results when failure occurred through one brick and two bed joints and did not include higher results when failure occurred through two bricks and one bed joint.

CONCLUSIONS

The bending strength of brickwork is very variable. Beside the properties of the material used it seems to depend to some extent on the dimensions of the specimen tested and the nature of the loading.

The ratio of the modulus of rupture in the horizontal direction to that in the vertical direction is not a constant value. The relation is non-linear as shown in fig. (3.4).

TABLE 3.1 COMPRESSIVE STRENGTH OF MODEL BRICKS.

Batch No.	Number tested	Mean crushing strength N/mm^2	(lb/in^2)
1	15	26.5	(3840)
2	15	34.2	(4950)

TABLE 3.2 COMPRESSIVE STRENGTH OF 1" MORTAR CUBES.

Age of Test	Number tested	Mean Compr. strength N/mm^2	(lb/in^2)
7 days	8	20	(2889)
14 days	6	30.6	(4450)
21 days	6	31	(4492)

TABLE 3.3. COMPRESSIVE STRENGTH OF BRICKWORK PRISMS.

Age of test	Brick strength N/mm^2 (lb/in^2)	Number tested	Mean comp. strength N/mm^2 (lb/in^2)
7	26.5 (3840)	4	19.2 (2770)
	34.2 (4950)	4	25 (3620)
14	26.5 (3840)	3	21.4 (3105)
	34.2 (4950)	3	28 (4060)

TABLE 3.4 AVERAGE ELASTIC MODULUS IN BENDING.

E	No. of Tests	Average N/mm^2	(lb/in^2)
E_{normal}	11	9.5×10^3	1.38×10^6
$E_{parallel}$	17	9.7×10^3	1.41×10^6

TABLE 3.5 RELATIONSHIP BETWEEN THE FLEXURAL STRENGTHS IN THE HORIZONTAL AND VERTICAL DIRECTIONS $F_p : F_n$

MORTAR 1 : $\frac{1}{4}$: 3 BRICK 26.5 N/mm^2

SPECIMEN	F_p N/mm^2	F_p lb/in^2	F_n N/mm^2	F_n lb/in^2	F_p/F_n
T_{12}	1.66	240	0.4	58	4.14
T_{13}	1.52	220	0.37	54	4.06
T_{14}	1.66	240	0.74	108	2.23
T_{15}	1.86	268	0.8	116	2.3
T_{16}	0.95	137	0.375	53	2.6
T_{20}	1.4	204	0.25	36	5.7
T_{21}	1.36	198	0.73	106	1.85
T_{22}	1.52	220	0.71	103	2.13
T_{23}	1.8	260	0.66	96	2.7
T_{24}	1.8	260	0.9	130	2.0
T_{26}	1.96	284	1.32	192	1.49
T_{27}	1.88	272	1.31	190	1.43
T_{28}	1.84	267	1.04	152	1.75
T_{29}	1.76	256	0.83	120	2.2
T_{30}	2.02	292	0.55	80	3.65
T_{36}	1.94	280	1.14	165	1.7
T_{37}	1.76	256	0.72	104	2.43
T_{38}	1.97	285	1.09	158	1.8
T_{40}	1.62	234	0.47	68	3.45
T_{41}	1.9	276	0.66	96	2.9
T_{42}	1.94	280	0.9	130	2.16
T_{43}	1.8	260	0.55	80	3.25

Table 3.6 RELATIONSHIP BETWEEN THE FLEXURAL STRENGTHS IN THE HORIZONTAL AND VERTICAL DIRECTIONS $F_p : F_n$

COMPANION SPECIMENS.

Mortar $1:\frac{1}{4}:3$ Brick 26.5 N/mm^2

Specimen	F_p N/mm^2	F_p lb/in^2	F_n N/mm^2	F_n lb/in^2	F_p/F_n
A_1	1.3	188	0.32	46	4
A_2	1.2	174	0.3	43	4
A_3	1.26	182	0.28	39	4.5
B_1	1.2	174	0.3	43	4
C_3	1.3	188	0.395	57	3.3

Table 3.6 RELATIONSHIP BETWEEN THE FLEXURAL STRENGTHS IN THE HORIZONTAL AND VERTICAL DIRECTION COMPANION SPECIMEN.

Mortar 1:$\frac{1}{4}$:3 Brick 34.2 N/mm^2

Specimen	F_p		F_n		F_p/F_n
	N/mm^2	lb/in^2	N/mm^2	lb/in^2	
A_5	1.56	255	0.56	81	2.8
A_6	1.62	234	0.64	93	2.5
B_7	1.43	207	0.52	75	2.75
B_8	1.49	215	0.46	67	3.25
C_4	1.56	226	0.5	72	3.1
C_5	1.4	203	0.38	55	3.7
G_1	1.7	246	0.59	85	2.88
G_2	1.74	252	0.495	72	3.5
G_3	1.46	211	0.45	65	3.25
F_1	1.35	195	0.46	67	2.9
F_2	1.47	213	0.62	90	2.4
F_3	1.7	246	0.49	71	3.45
H_1	1.46	211	0.475	69	3.05
H_2	1.53	222	0.47	68	3.25

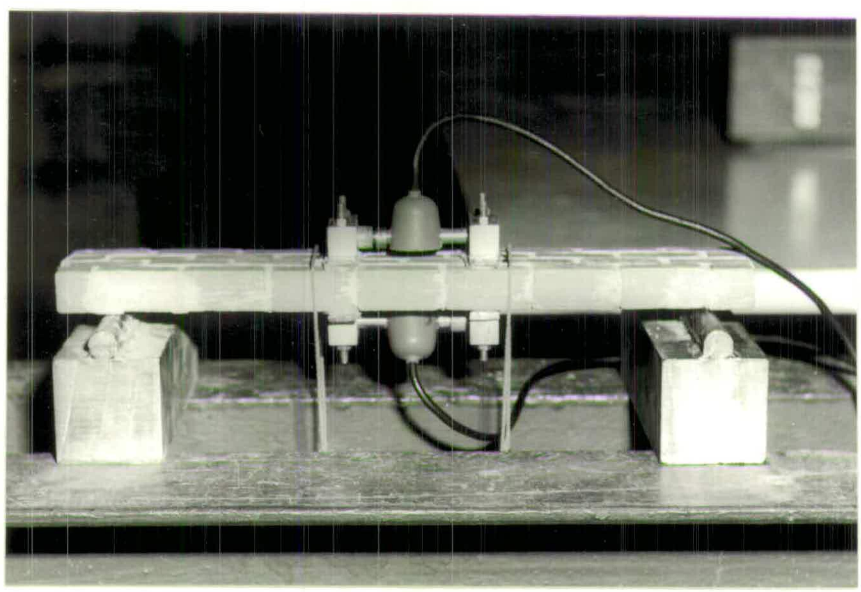


FIG. 3.1

Fig. 3.2
Stress - strain relationship - Prism F_3

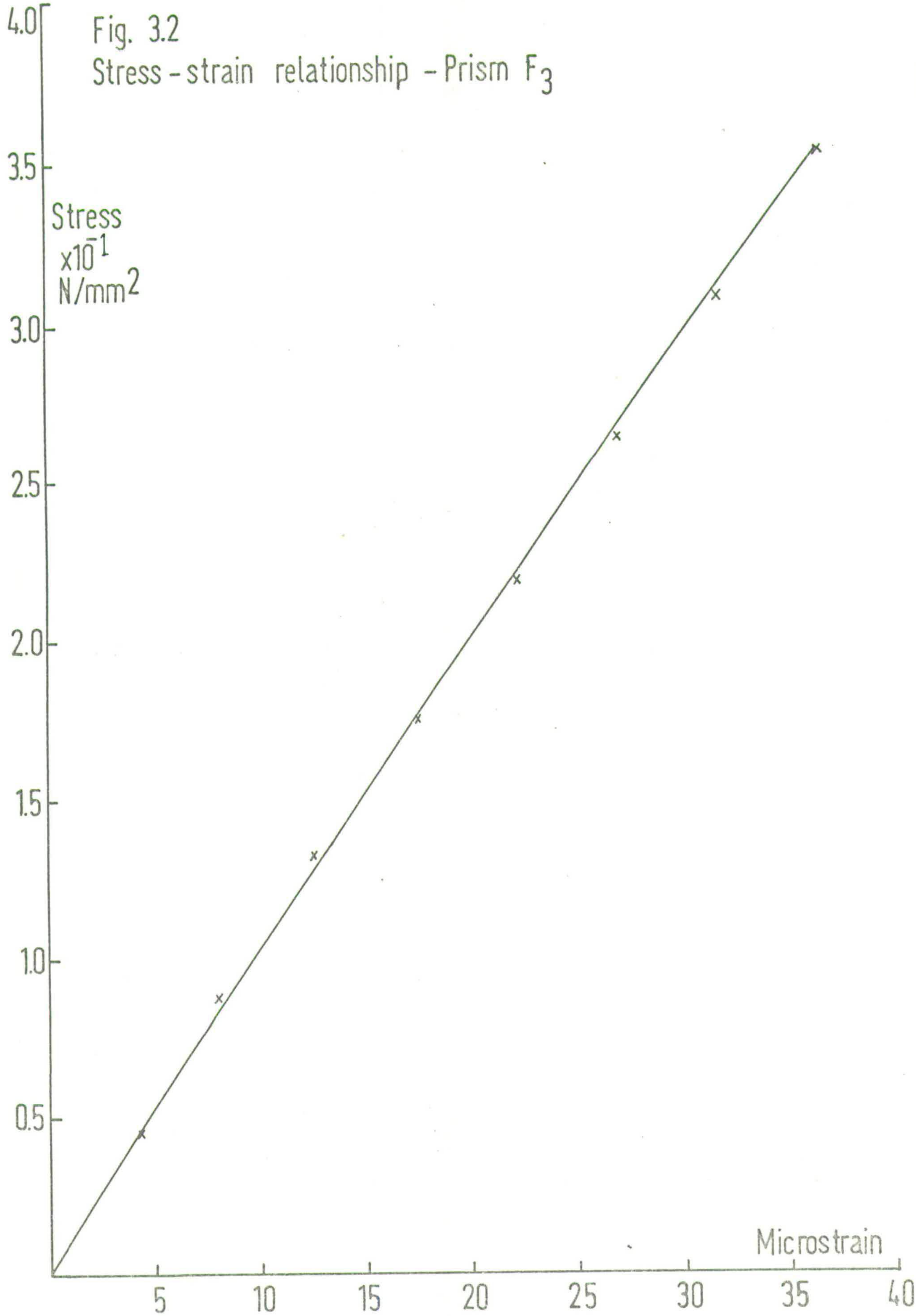


Fig. 3.3
Stress-strain relationship Beam G_1

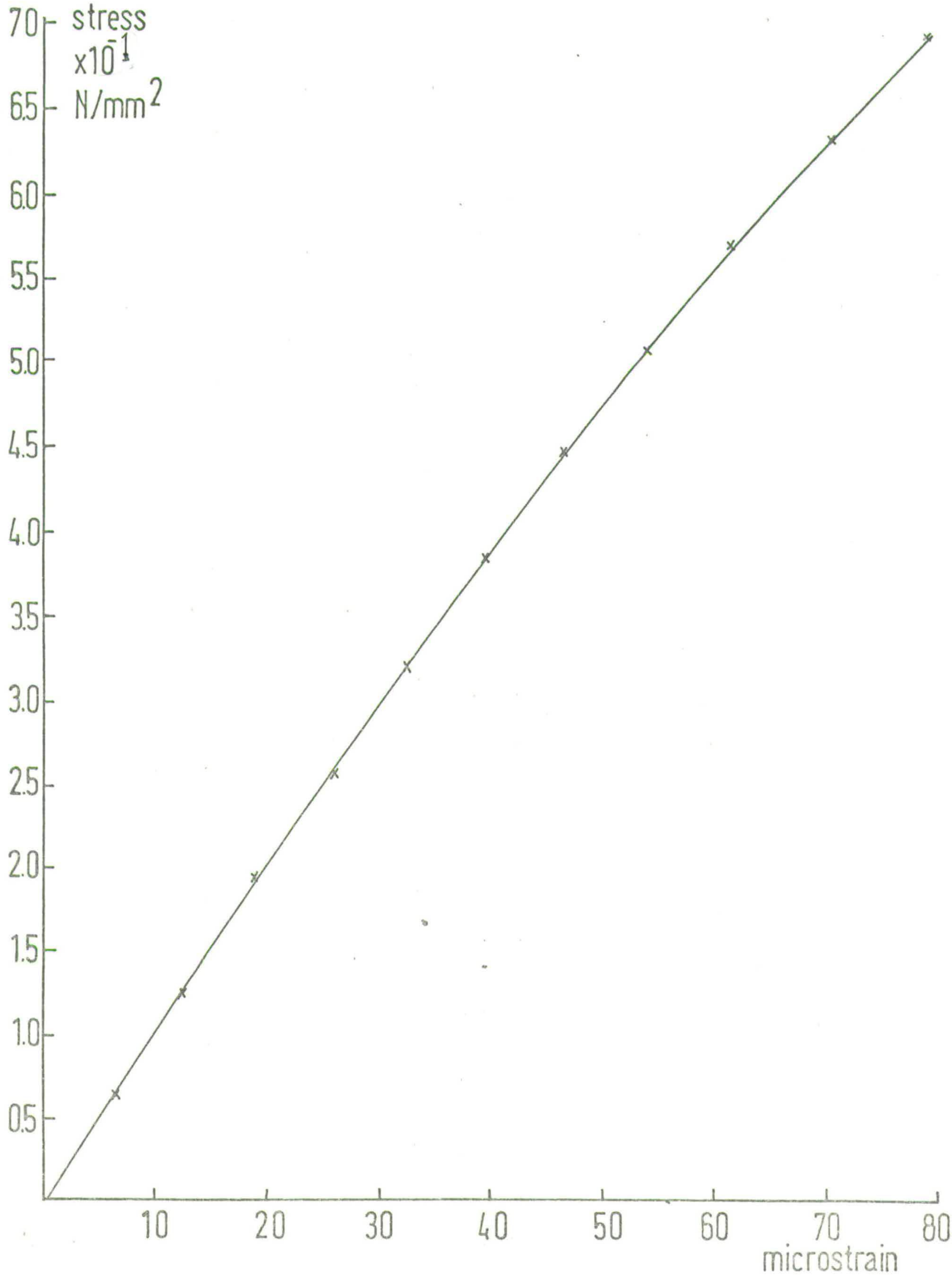
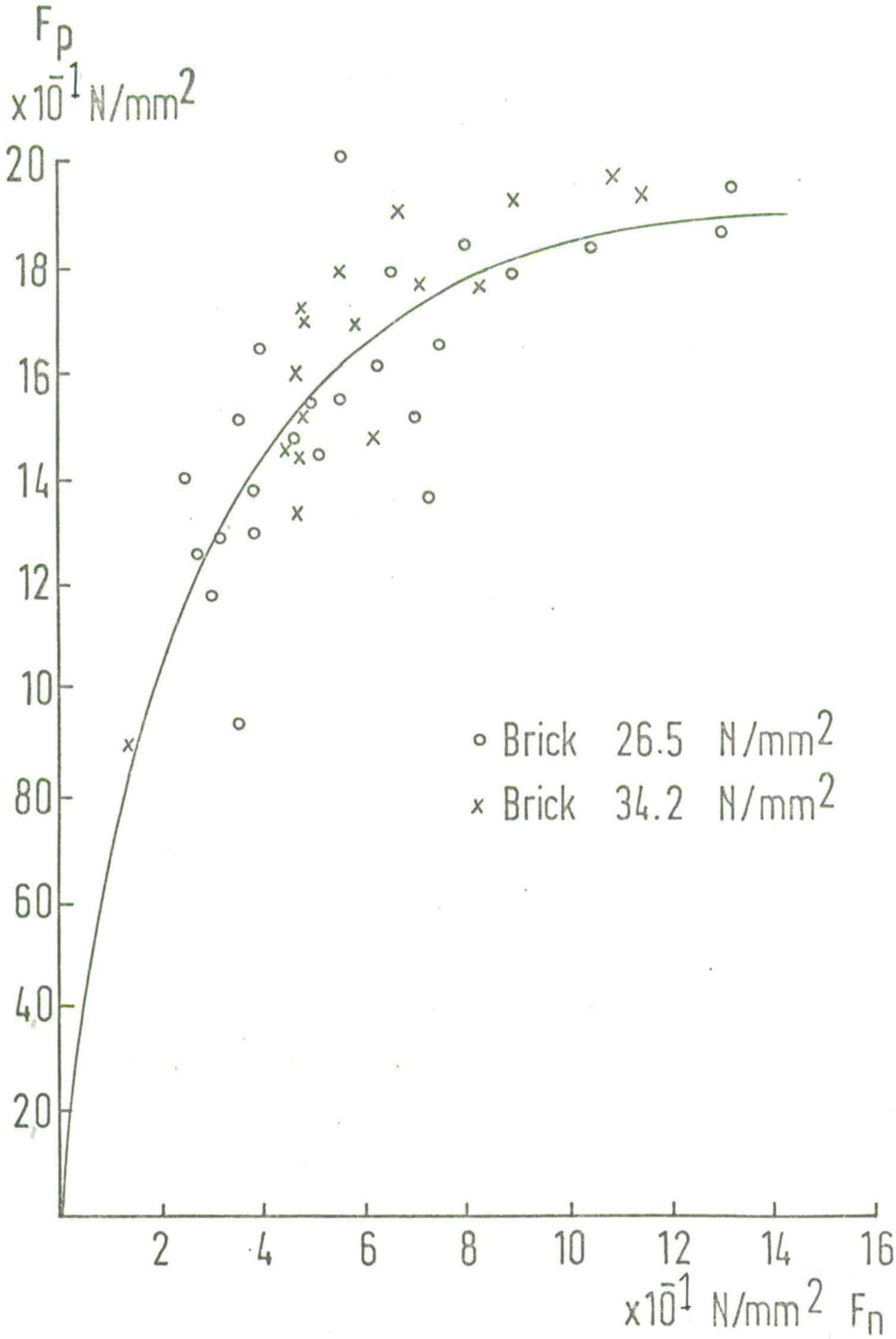
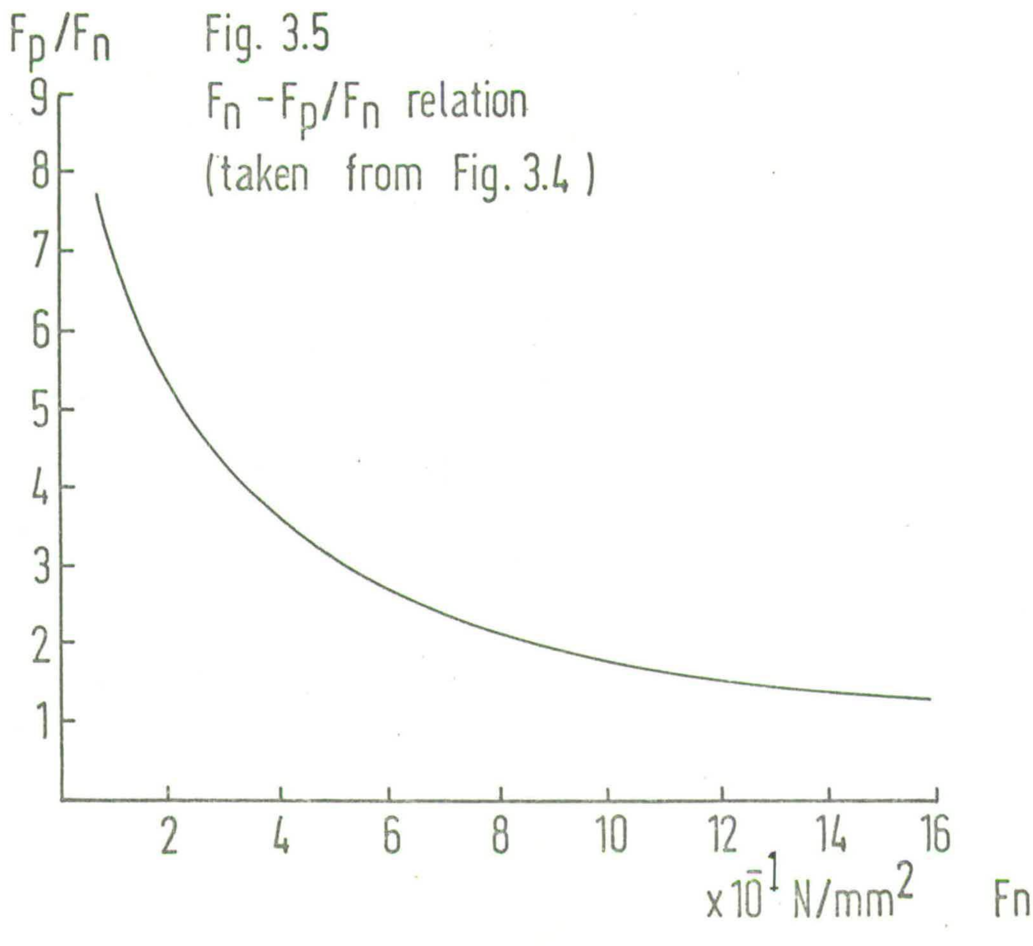


Fig. 3.4
The relationship between the Modulus of Rupture normal to the Bed Joints (F_n) and parallel to the Bed Joints (F_p)




Fp/Fn
Fig. 3.5
Fn -Fp/Fn relation
(taken from Fig. 3.4)
9
8
7
6
5
4
3
2
1
2
4
6
8
10
12
14
16
x10̄1 N/mm2
Fn

CHAPTER 4.

THEORETICAL INVESTIGATION

4.1 INTRODUCTION

A two-way spanning slab is a redundant structure. The distribution of moment throughout the slab can therefore be determined most conveniently from either an elastic or a plastic analysis. An exact solution is difficult and very complicated. Nevertheless, in both cases, test results have confirmed that calculations could be made easier by making certain simplifying assumptions.

Using elastic analysis, a plate problem could be treated by the classical plate theory (26), comprehensively discussed by Timoshenko, or by other numerical methods. The three basic techniques for direct numerical solutions are finite differences (27), finite elements (28) and the grillage analogy method (29).

Elastic theory solutions for slabs suffer from two distinct disadvantages. Theoretical solutions are complicated and often require the use of computers, electrical analogues or similar techniques, while semi-empirical methods are strictly limited in their applicability.

For more complicated shapes of slab, Johansen's yield-line method (30) was found to be more convenient. It is applicable to slabs of any shape, loading and support conditions.

The strip method introduced by Hillerborg (31) is simple and straightforward procedure of design. It lends itself to the solution of problems intractable by the theory of elasticity and those for which yield-line theory is complicated. Furthermore it does not require more than an elementary knowledge of simple beam theory.

4.2 ELASTIC DESIGN OF BRICKWORK.

In elastic analysis the slabs are usually assumed to act as perfectly flat thin plates made of a homogeneous elastic material which has equal strength and stiffness in every direction, that is, an isotropic material.

This assumption, while nearly correct for plates of steel or other metals, has often been made for reinforced concrete. Although brickwork is brittle at failure and is neither like steel nor like concrete, the same assumptions could be made for the purpose of calculation. At least, within the range of loadings likely to be encountered in panel walls properly designed to resist wind loads, brickwork could be assumed to behave with reasonable elasticity.

Most of the currently used design methods, based on elastic analysis, are semi-empirical methods. That is a practical approach using a theoretical analysis which is modified and adjusted according to tests carried out to failure.

Such an approach is much simpler to use. It could even take into account such effects which influence the strength and the behaviour of panels under loading if sufficient experimental data were available.

4.3 ULTIMATE STRENGTH DESIGN OF BRICKWORK.

When considering a design approach to a brickwork panel, due account must be taken of the different strength of the brickwork in the bed and in the perpendicular directions.

If a brick wall is spanning one way in the horizontal or the vertical direction, the stress distribution across the width could be assumed uniform and it could be designed as an ordinary beam. When a panel is supported at three or four sides the load will be distributed in the two directions and two way bending develops across the panel.

From observation of brickwork failure mode, it was seen that the failure of brickwork subjected to lateral pressure, is initiated by cracking at the centre of the panel. In some cases an increase of load is possible after the first cracking followed by a crack distribution with diagonal cracks in direction towards the corners.

The crack pattern obtained at final failure is similar to the yield-line pattern obtained by testing two-way concrete slabs. It is obvious that there are many uncertainties about the application of the yield-line theory. Brickwork is a brittle material and cannot satisfy the yield-line theory conditions. It is also difficult to imagine any form of yield behaviour after the cracks have appeared and once cracked, brickwork can carry little or no bending moment as no moment can be transmitted along a joint that has already cracked. However, there are good reasons to consider this method as a possible means of calculation, not only because of the similarity in crack pattern obtained at failure, but because it is a simple and straightforward method of design. Moreover the method as developed by Johanson (12) was firstly applied on plain concrete slabs on soil where it was mentioned that there is a considerable moment capacity in the cracks, so that surrounding parts of the slab prevent free deformation between the cracked parts.

4.4 FINITE ELEMENT METHOD

In view of the complexity of plate bending problems it is hardly surprising that numerical methods are being applied to their solution. The finite element method has gained wide acceptance in recent years. Comprehensive work was done by Clough (28) and Zienkiewicz [32, 33].

In the finite element analysis the structure is approximated by a finite number of elements interconnected at a finite number of nodes. The structure is a mathematical assembly of physical elements.

There is no approximation required in the mathematical procedures, only in the choice and physical assembly of the elements.

The basic steps in any finite elements analysis are as follows:
The structure is divided into a finite number of elements connected at their nodal points.
The stiffness matrix is computed for each element.
The total stiffness matrix of the structure is assembled, satisfying equilibrium of nodal forces and compatibility of corresponding displacements.

Either of two approaches, force or displacement approach, can be used to derive the element stiffness matrix. If the displacements are considered as the unknowns, these displacements are found in terms of the forces by means of a stiffness matrix and by applying equilibrium conditions at the nodes the unknown displacements can be found. Then the individual stiffness matrices are assembled to form the stiffness matrix of the structure and the resulting equations are solved.

4.5 YIELD LINE ANALYSIS:-

The yield line analysis is a limit design method which involves the location of a failure pattern when a slab is loaded to its ultimate capacity. The theory is applicable to both isotropic and orthotropic slabs and can be used for slabs of any shape, loading and edge conditions.

As with most methods of analysis certain assumptions are made, which are found from tests to be reasonably true. It is assumed that the elastic deformations are negligible in comparison with the plastic deformations and that the slab elements between the yield lines remain as rigid, plane regions. Consequently the yield lines which are the intersection between these plane elements are also straight.

At failure, the slab is assumed to deflect by causing the rigid regions to rotate about their axes of rotation, whilst compatible rotations take place along the yield lines.

The first stage of the ultimate load analysis of any slab is to predict the yield line pattern at failure. The general crack pattern may sometimes be deduced from geometry or be obtained from model or full scale tests. Once a failure pattern has been postulated two alternative techniques of solution are available in order to find the relation between the ultimate resistance moments in the slab and the ultimate load.

The first of these, the work method, is to equate the internal energy of dissipation on plastically yielding 'fracture' lines to the work done by the externally applied loads. The layout of yield lines for the worst mode being found by trial and error to give the minimum collapse load.

The second method is the equilibrium method using 'nodal' forces where yield lines meet, or where they meet edges.

Extensive work about this theory has been published by Wood (34), Jones (35) and Jones and Wood (36).

4.6 <u>THE STRIP METHOD</u>:

The strip method is introduced by Hillerborg in 1956 for the design of reinforced concrete slabs. It gives the designer wide freedom of choice in his design approach. Hence many different solutions for a given slab design are possible. Obviously not all solutions will be of equal economy.

The equilibrium equation (for elastic plate analysis) for any valid solution for the moments in a slab is

$$\frac{\partial^2 Mx}{\partial x^2} + \frac{\partial^2 My}{\partial y^2} - \frac{2\partial^2 Mxy}{\partial x \partial y} = -q$$

where the bending moments Mx and My and the twist moment Mxy follow Timoshenko's notation and q is the load per unit area on the slab. The slab is designed assuming Mxy = 0 and then the load is apportioned to $\frac{\partial^2 Mx}{\partial x^2}$ and to $\frac{\partial^2 My}{\partial y^2}$ in any ratio, i.e.

$$\frac{\partial^2 Mx}{\partial x^2} = -\alpha q \qquad\qquad \frac{\partial^2 My}{\partial y^2} = -(1-\alpha)q$$

usually α is taken as either 1 or 0. Loads in a particular area are assigned to particular slab strips and continuity of the resulting moments and sheers must be carefully maintained. Apparent discontinuity in torque or deflection may be disregarded, but a discontinuity in moment or shear is not permitted. Applying this theory to brickwork the different strength of brickwork in the two perpendicular directions is considered and the panel is regarded to consist of a simplified grid of strips in the two directions. The method of calculation and the discussion of the results are presented in chapter six and seven respectively.

CHAPTER 5.

EXPERIMENTAL WORK AND RESULTS

EXPERIMENTAL WORK AND RESULTS.

5.1 INTRODUCTION.

In this chapter the manufacture of the walls, the test procedure and the test results are described. Twenty seven one-sixth scale model walls were tested under lateral pressure. All of them were "half brick" in thickness and simply supported on three or four sides. About eighty test beams and prisms were also tested to get the material properties of the walls (Chapter 3).

5.2 MATERIALS.

The type of brick used and mortar properties were the same as those discussed in Chapter 3.

5.3 TEST SPECIMENS.

All the walls were tested without precompression and supported on three or four sides. They were built in wooden jigs using wetted bricks after being kept for twenty minutes in water. The bed joint locations were marked on the wooden mould in order to control the thickness of these joints through the entire series of tests. The walls were kept under plastic sheets for seven days and then left to cure under normal laboratory conditions until tested at an average age of three weeks. The first series of the walls were built and cured in the main laboratory which has a nearly constant temperature throughout the year. The rest were built in an Annex to the laboratory which had different curing conditions.

For each wall, companion brick prism and beam specimens were built at the same time as the wall, and kept under the same curing conditions until tested with the wall. These specimens were tested to obtain the modulus of elasticity and the modulus of rupture of brickwork. The latter quantity was used to calculate the moment of resistance of brickwork in the vertical and horizontal directions.

5.4 EQUIPMENT:-

5.4.1 Loading rig:-

A specially designed loading rig for testing one sixth scale model walls was already available. It consists of a vertically standing frame for supporting the walls and a resisting plate. The supporting frame has two fixed and two movable sides to give the required span and height for the wall to be tested.

The test walls were placed vertically against the supporting frame and acted upon by a distributed load from an air filled plastic bag, mounted between the wall and the rigidly connected resisting plate. The pressure in the bag was measured using a water manometer connected to the bag.

5.4.2 Measuring Instruments:-

Dial gauges with 0.001 inches scale divisions and half an inch travel were used to measure the deflections.

2 inches Demec gauges were used in one experiment to measure the strains but they did not give satisfactory results.

Vibrating wire strain gauges with the following characteristics were used for most of the tests:

Length: 2.5 inches, gauge factor = 0.54×10^{-9}, plucking voltage voltage = 60 volts.

The testing arrangement is shown in plate (5.1).

5.5 TEST PROCEDURE.

Walls were taken to the testing rig in their wooden moulds and great care was taken to avoid developing tensile or flexural stresses during the handling and placing of the panels. The bigger wall with a span of 400 mm, a height of 800 mm and a thickness of 19 mm was very difficult to handle and a great effort was needed to place it safely on the rig. Testing walls in the vertical direction has the advantage that it is easier to place such slender walls vertically and remove the mould safely from behind. Also it is easier to place and take the readings of the dial gauges.

After placing the wall a conscious effort was made to bed in the wall on to the supporting frame using mortar. To safeguard against fixity at supports, oil was painted on the steel, and plastic sheets were placed between the supporting frame and the test wall.

Uniform loading of the walls was achieved by pumping air into plastic bag between the test wall and the resisting steel plate. The load was gradually increased by small increments of pressure from the air compressor.

To measure the wall deflections dial gauges were located at different positions on the wall face. The dial gauges were mounted on a rod which rested on the supporting frame in an effort to eliminate any deflections from yielding of supports.

Strain measurements were also taken at some points using vibrating wire gauges.

Readings of the dial gauges, the strain gauges and the difference in height of water in the manometer columns were taken at each load increment.

The bag used for carrying out the tests was bigger than the wall so the end of the bag was folded to give the appropriate sizes. Special care was taken to ensure that the bag did not balloon around its edges, thus giving rise to membrane stresses which might affect the reading of wall pressure. Accordingly, the results of 8 walls were discarded when the bag was noticed to bulge around the free edge of the three sides supported wall.

5.6 TEST PROGRAMME AND RESULTS.

The experimental work carried out in this programme is summarised in tables (5.1) and (5.2). The experimental failure loads of the walls and the modulus of rupture in both directions were given in the last four columns of the tables.

The first series of walls tested were designated by the letters A, B and C. These walls were tested on three sides simply supported, the free edge being one of the vertical sides, where the wall was kept vertically with the bed joints in the horizontal direction. The height of the walls was kept constant at 380 mm and height to span ratios of 0.5, 1 and 2 were taken for the three panel sizes tested.

The second series of walls with the letters G, F and H were tested with four sides simply supported. They were eight in number with the same height to span ratios of 0.5, 1 and 2 but with a constant span of 400 mm.

The measured deflections and strains for the applied pressures are given in appendix A, and figs. 5.4 to 5.10 show the pressure-deflection curves for some of the walls. The effect of the aspect ratio on the failure pressure of the panel is shown in the figs (5.11) and (5.12) for panels simply supported on three and four sides respectively. Fig. (5.11) indicates a reduction in the failure pressure with increasing aspect ratio (the height being kept constant), and the curve approaches the horizontal line which is the failure pressure for a one-way panel. Fig. (5.12), for panels four sides simply supported shows an increase in the failure pressure with the increase in aspect ratio as the span of the panel is kept constant for all panels.

5.7 OBSERVATIONS ON THE TESTS.

It was noticed that the panels built in the Annex were stronger than those built in the main laboratory. This is mainly because of the different curing conditions, since the temperature in the Annex was lower than that in the main laboratory. Also it could be partially because of the improving workmanship.

The failure of the walls was not uniform. Some walls failed suddenly and with a loud report, without any definite visible cracking beforehand. With other walls a hardly visible cracking could be seen across the bed joints before the final failure of the wall. In most of the walls fracture lines occurred in the bed joints being mostly a bond failure at the brick-mortar interface. However in some instances, particularly in walls with length to height ratio of one or less the fracture lines went through the bricks in the vertically running failure lines.

With the exception of wall A_3 all walls series A three sides supported, failed suddenly without warning and cracks could not be detected before failure.

Walls B_7 and B_8 the three sides supported square panels failed in a gradual manner and the initial cracking could be detected before the ultimate capacity of the wall was reached. First a horizontal hair crack appeared at the free edge, and while the load is increasing, diagonal cracks appeared progressing towards the corners with the first horizontal crack extending towards the end of the wall.

Walls C_3 and C_4 behaved the same way as walls B except wall C_5 which failed without any reserve in strength.

All walls of the groups F and H, except wall F_3, failed without prior warning with splitting of the bricks in the vertical direction.

Wall F_3 had the same final pattern of cracks but a horizontal crack appeared before the final failure. Walls G with span to height ratio of 2 did not fail suddenly like the other walls with four sides supported. They showed a reserve of strength after the initial cracking.

It was clearly seen that the final crack formation in most of the walls tested were similar to the yield line pattern obtained for concrete slabs with similar conditions. The different patterns developed at failure are shown in figures 5.13 to 5.20

TABLE 5.1 EXPERIMENTAL PROGRAMME

PANELS WITH 3 SLIDES SIMPLY SUPPORTED.

Wall No.	L X H X t mm	L/H	F_p N/mm^2	F_n N/mm^2	First crack pressure $N/mm^2 x 10^{-3}$	Ultimate pressure $N/mm^2 x 10^{-3}$
A_1	190 x 380 x 19	0.5	1.3	0.32	-	8.4
A_2			1.2	0.3	-	5.8
A_3			1.26	0.28	4.2	6.3
A_5			1.56	0.56	-	9.3
A_6			1.62	0.64	-	10
B_1	380 x 380 x 19	1.0	1.2	0.3	-	3.1
B_7			1.43	0.52	4.0	4.7
B_8			1.49	0.46	3.5	4.6
C_3	760 x 380 x 19	2.0	1.3	0.395	2.00	2.35
C_4			1.56	0.5	2.25	2.9
C_5			1.4	0.38	-	2.8

FIG. 5.2 EXPERIMENTAL PROGRAMME

PANELS WITH 4 SIDES SIMPLY SUPPORTED.

Wall No.	L X H X t mm	L/H	F_p N/mm^2	F_n N/mm^2	First crack pressure $N/mm^2 x10^{-3}$	Failure pressure $N/mm^2 x10^{-3}$
G_1	400 x 200 x 19	2.0	1.7	0.59	-	18.2
G_2			1.74	0.495	14	19
G_3			1.46	0.45	16	18
F_1	400 x 400 x 19	1.0	1.35	0.465	-	8.4
F_2			1.47	0.63	-	10.5
F_3			1.70	0.49	8.0	10.0
H_1	400 x 800 x 19	0.5	1.46	0.475	-	5.6
H_2			1.53	0.47	-	7.0

TABLE 5.3 EXPERIMENTAL MOMENT COEFF. K.

PANELS WITH 3 SIDES SIMPLY SUPPORTED $M_h = Kq\ H^2$

Wall	L/H	Moment Coeff. K
A_1	0.5	0.016
A_2		0.0215
A_3		0.02
A_5		0.025
A_6		0.027
B_1	1.0	0.04
B_7		0.048
B_8		0.0415
C_3	2.0	0.07
C_4		0.073
C_5		0.056

TABLE 5.4 EXPERIMENTAL MOMENT COEFF K.

PANELS WITH 4 SIDES SIMPLY SUPPORTED.

$M_h = Kq\ H^2$

Wall	$L/_H$	Moment Coeff. K.
G_1	2	0.048
G_2		0.39
G_3		0.37
F_1	1.0	0.02
F_2		0.022
F_3		0.0184
H_1	0.5	0.0080
H_2		0.0063

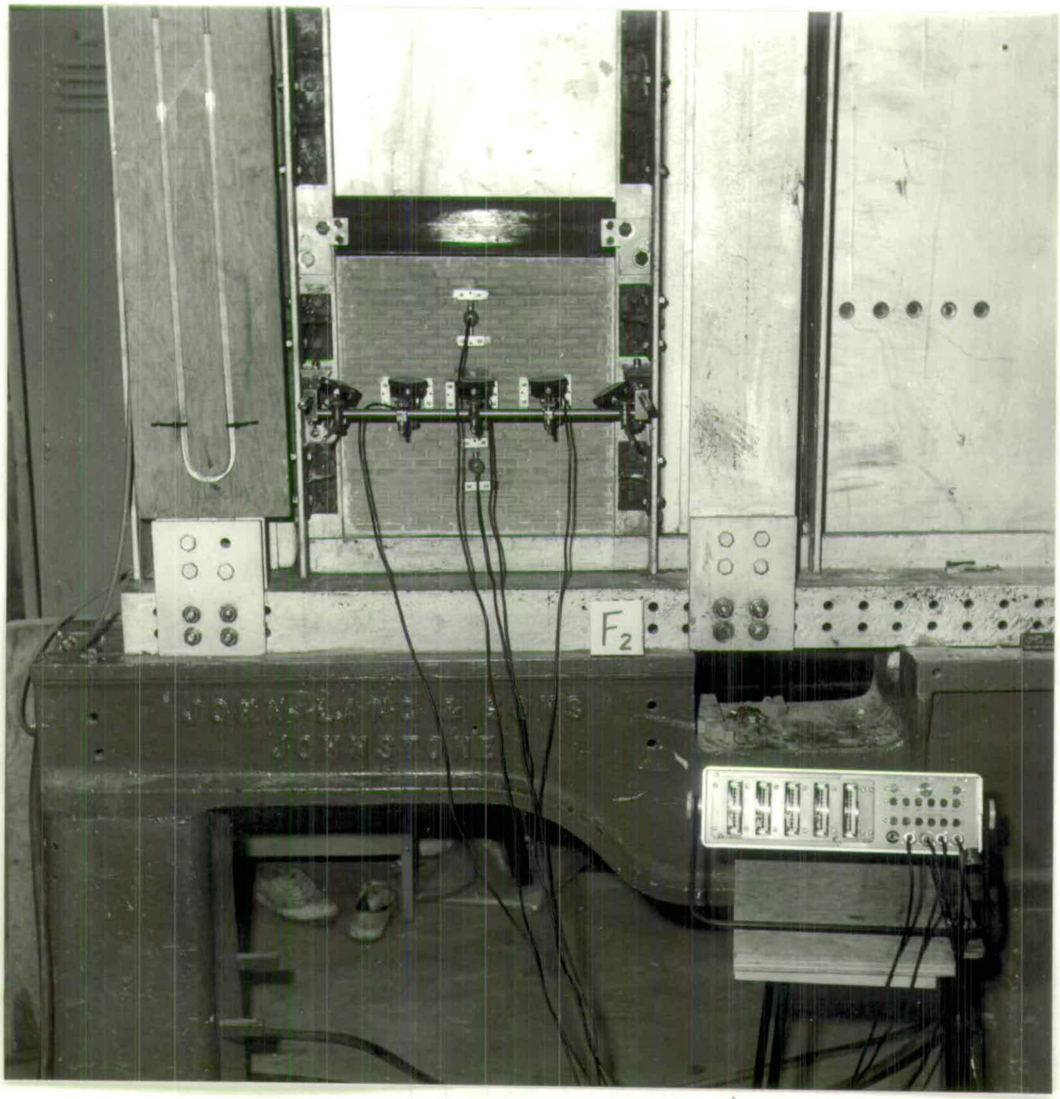
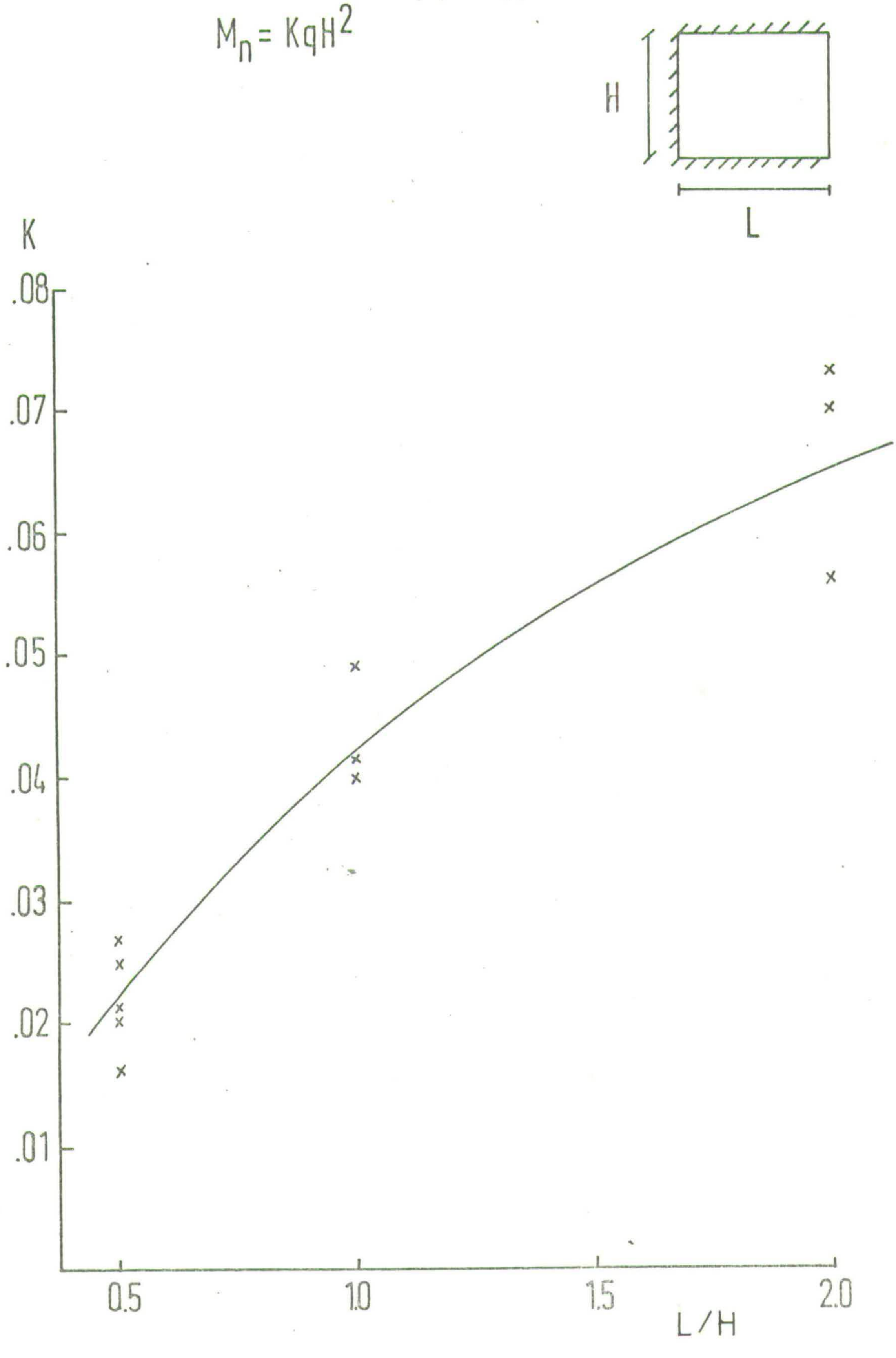


FIG. 5.1

Fig. 5.2
Experimental Moment Coeff. K
Panels with 3 sides simply supported

$$M_n = KqH^2$$



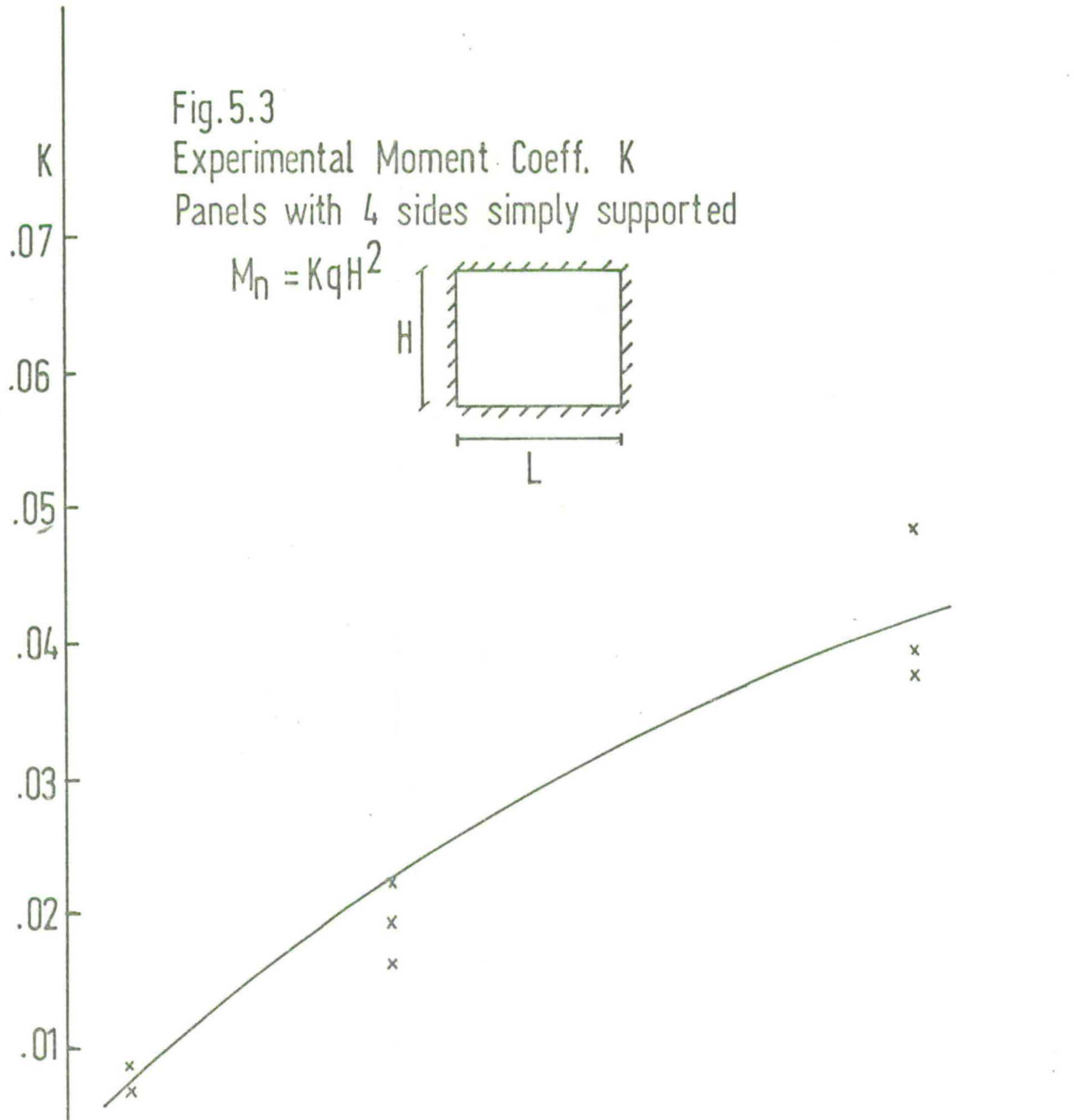


Fig. 5.3
Experimental Moment Coeff. K
Panels with 4 sides simply supported
$M_n = KqH^2$
H
L
K
.07
.06
.05
.04
.03
.02
.01
0.5
1.0
1.5
L/H
2.0

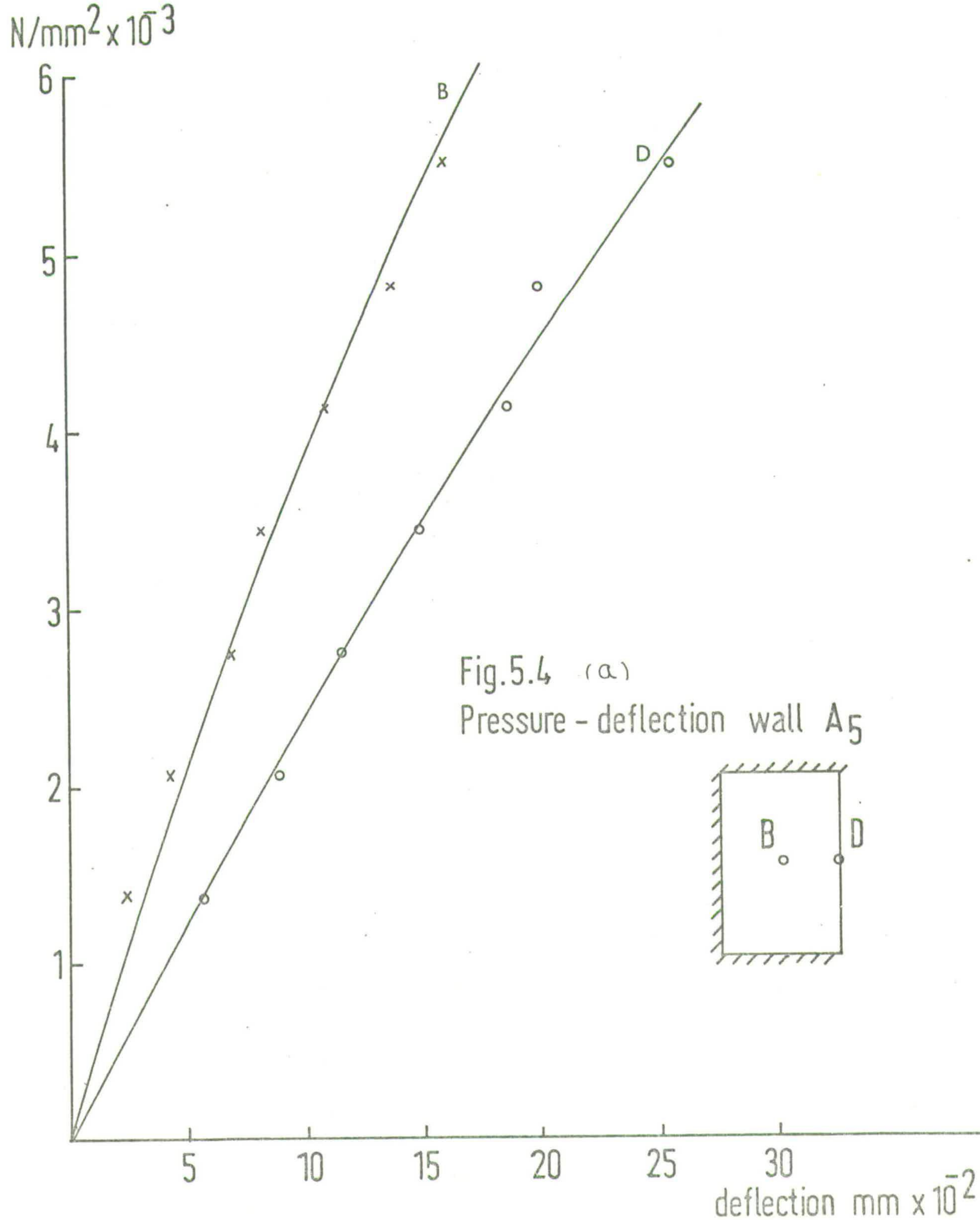


Fig.5.4 (a)
Pressure - deflection wall A_5

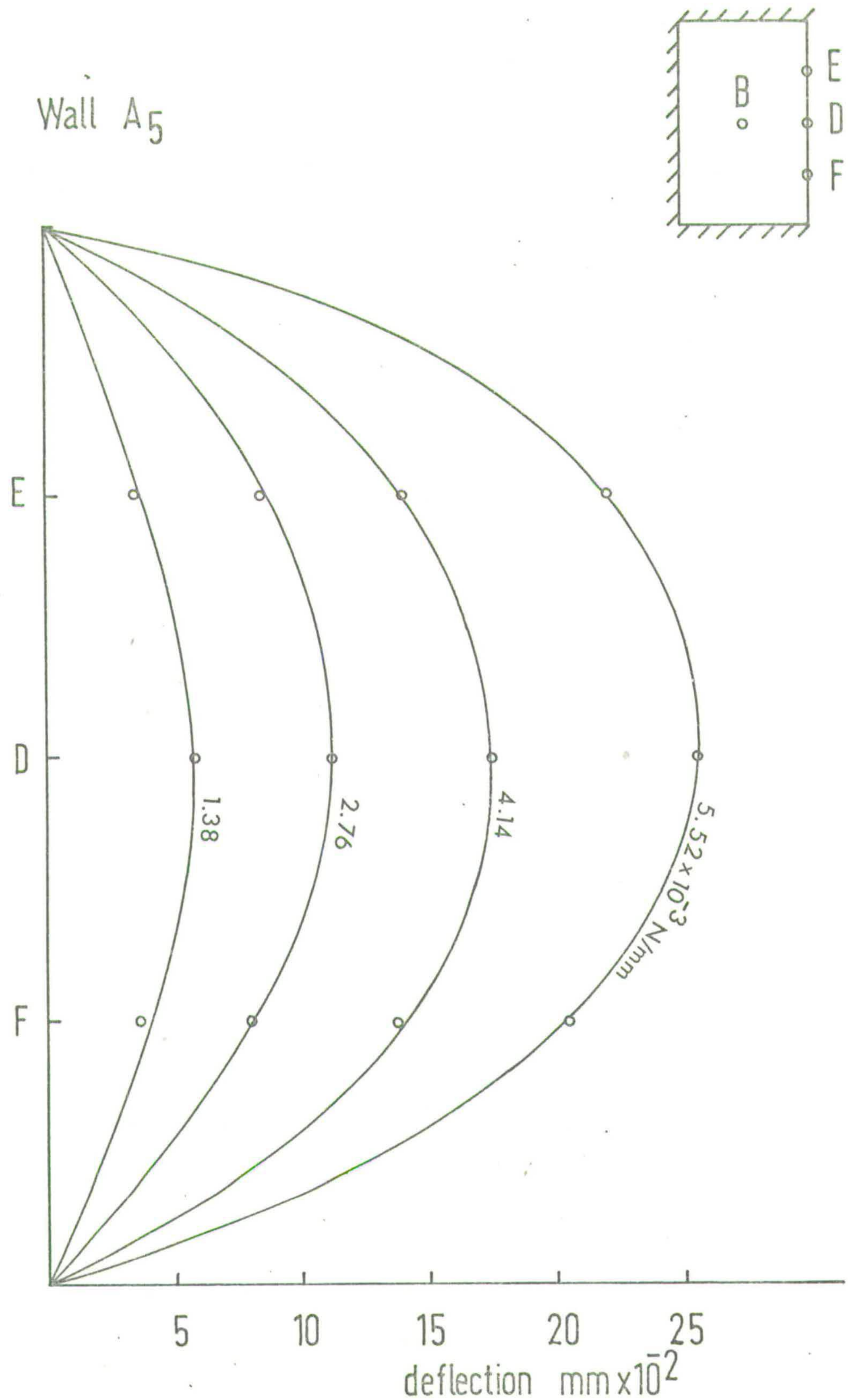

Wall A5
B
E
D
F
E
D
F
1.38
2.76
4.14
5.52 x 10-3 N/mm
5
10
15
20
25
deflection mm x 10-2

Fig 5.4 (b)

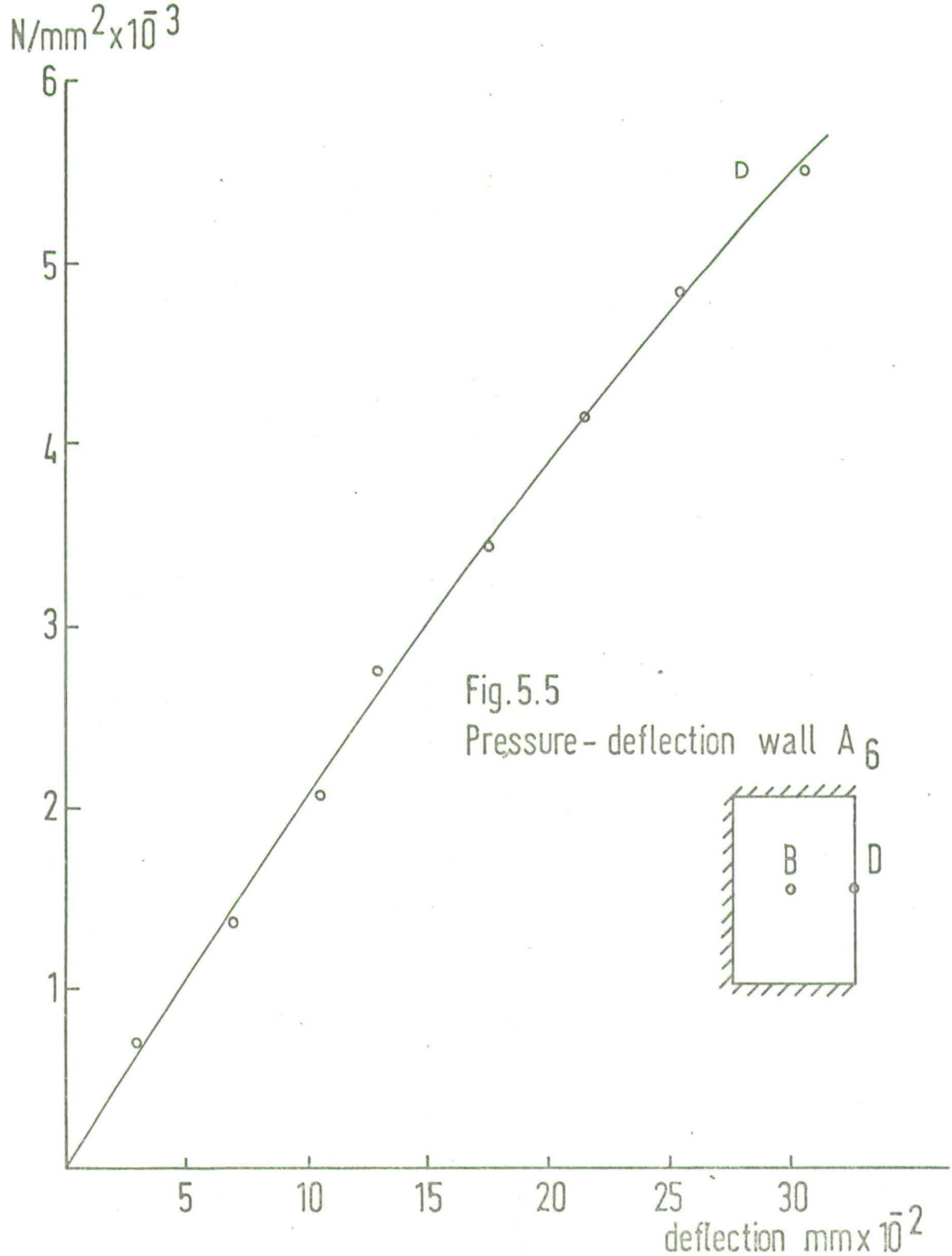


Fig. 5.5
Pressure - deflection wall A_6

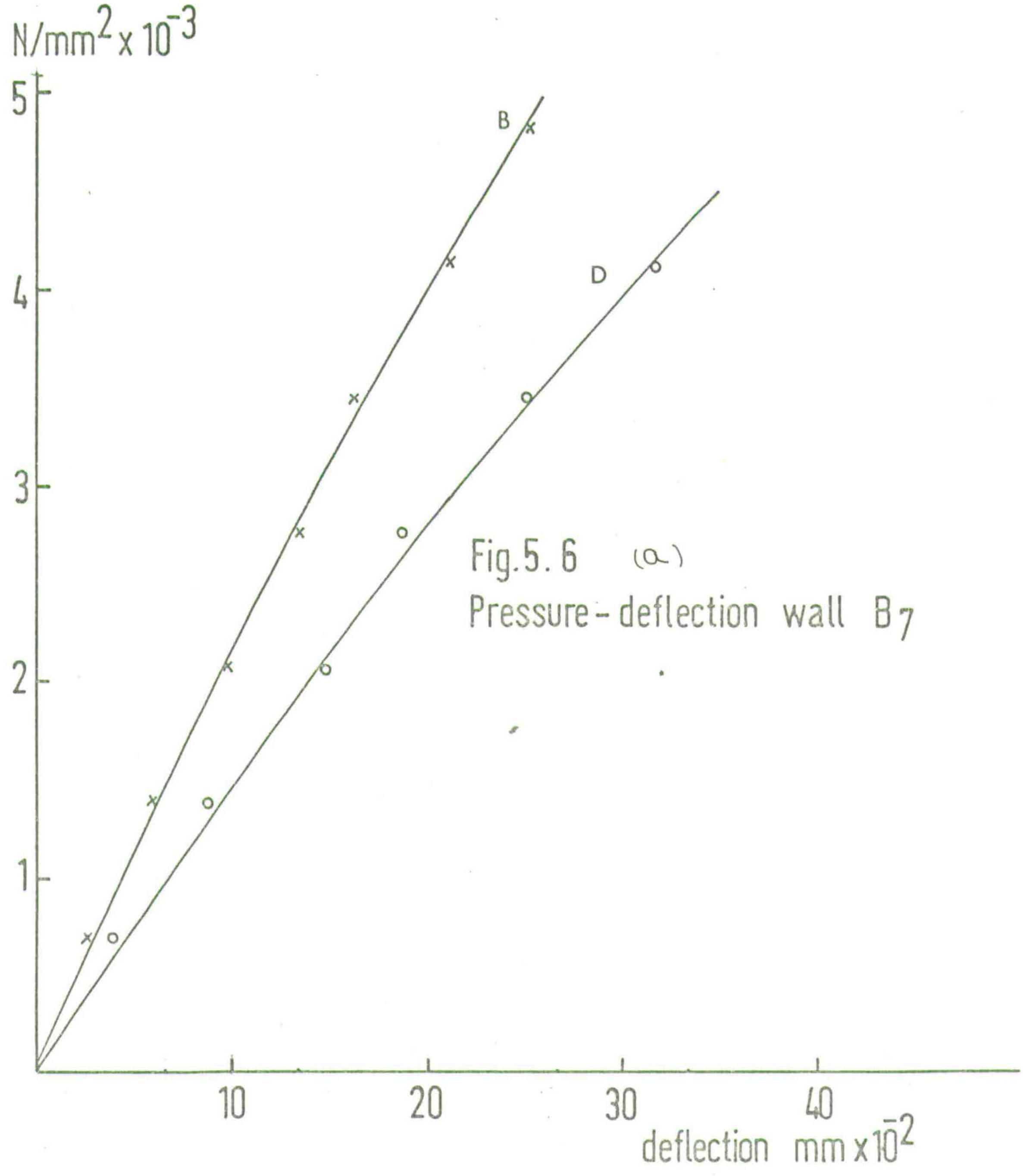


Fig.5.6 (a)
Pressure-deflection wall B_7

B_7

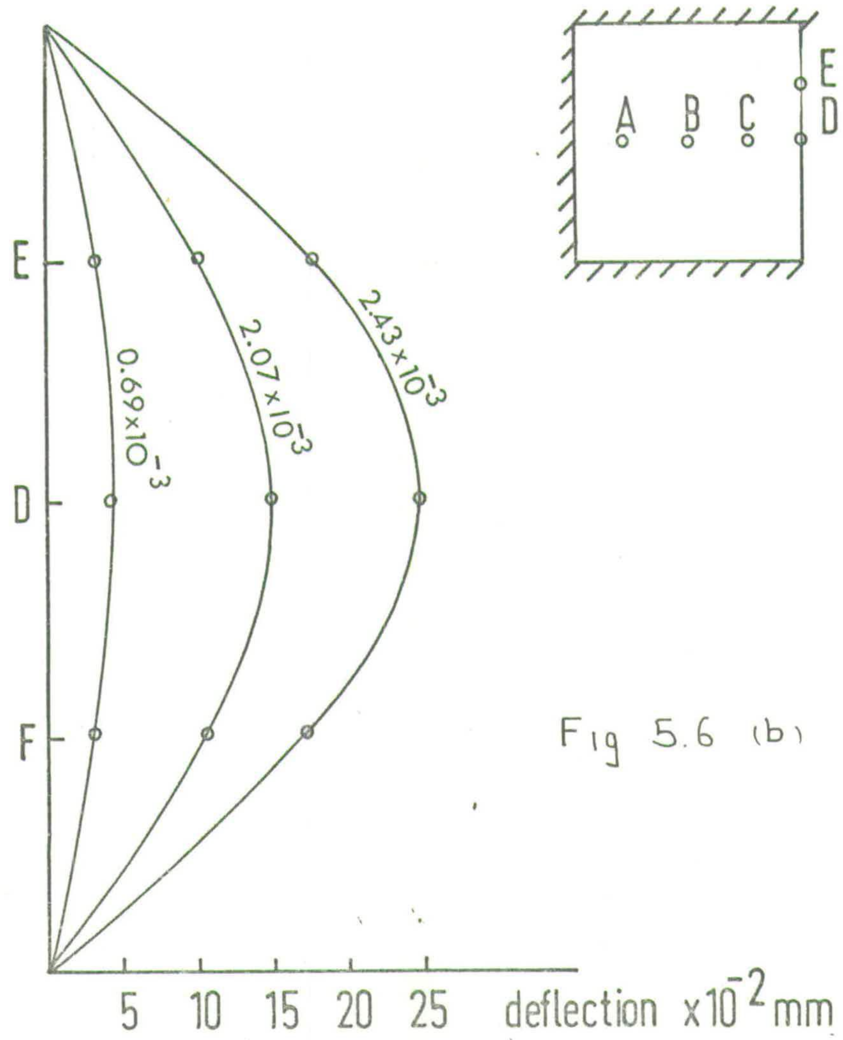


Fig 5.6 (b)

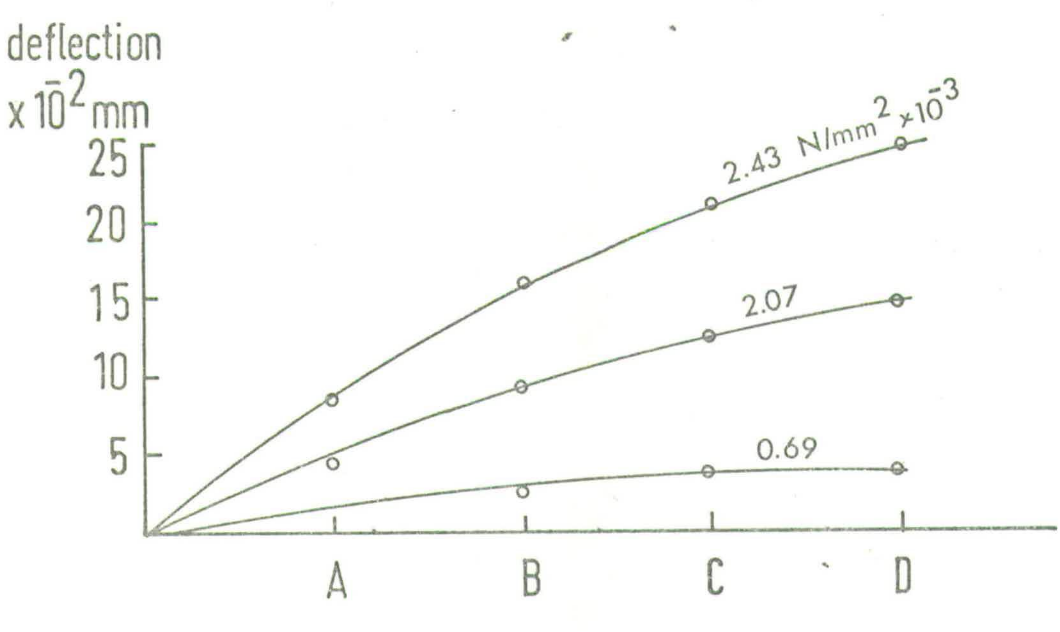


Fig 5.6 (c)

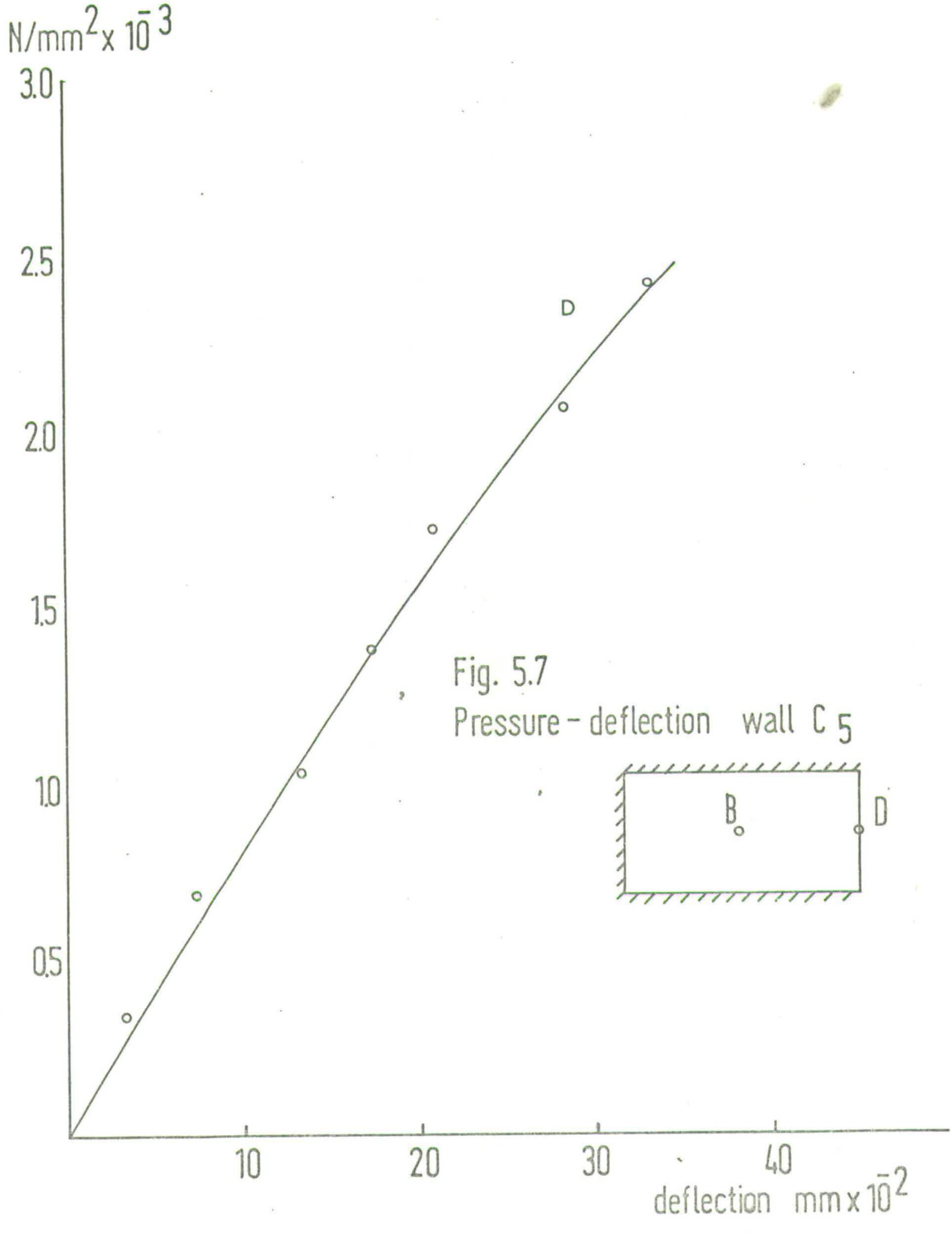


Fig. 5.7
Pressure - deflection wall C_5

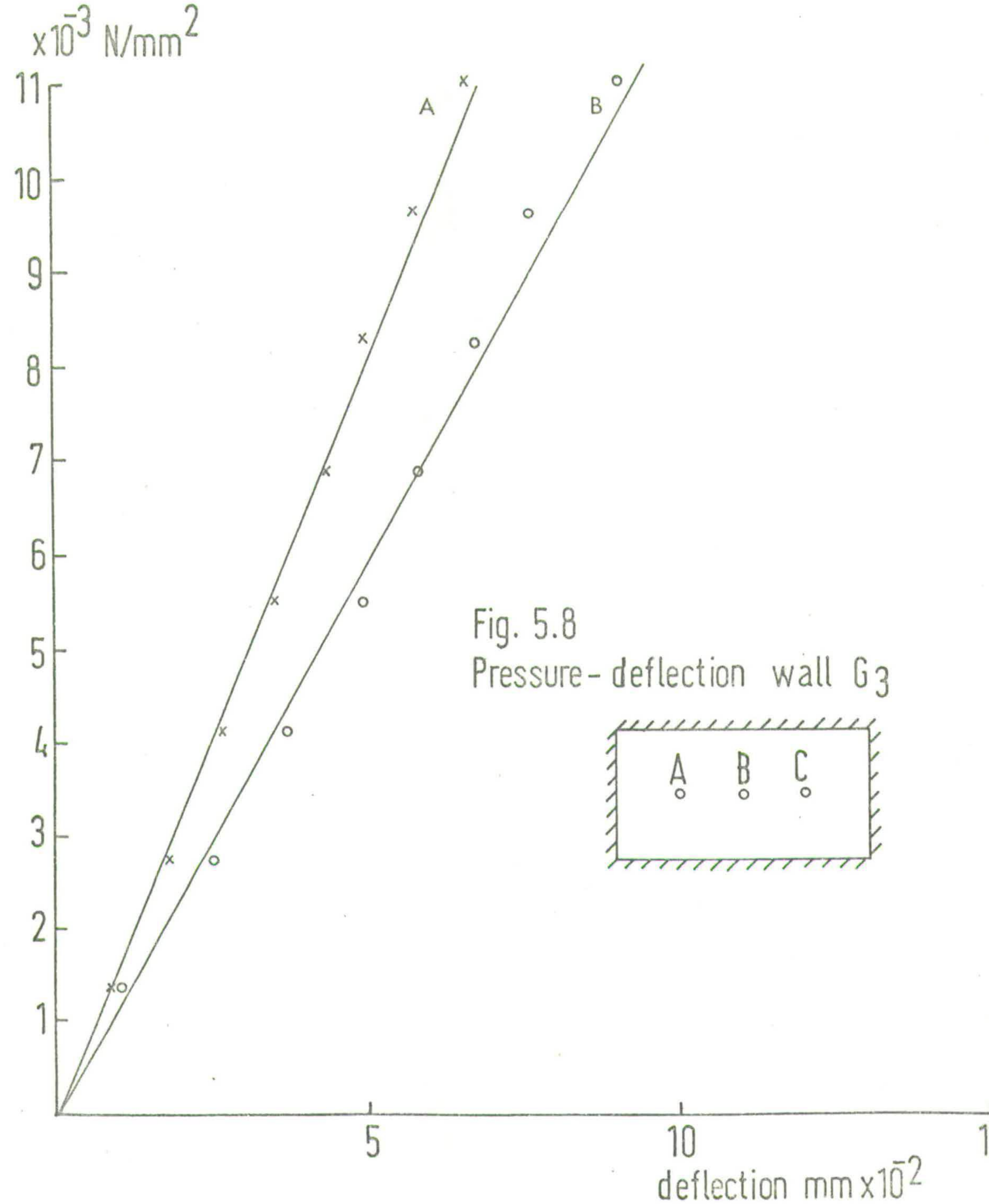


Fig. 5.8
Pressure-deflection wall G_3

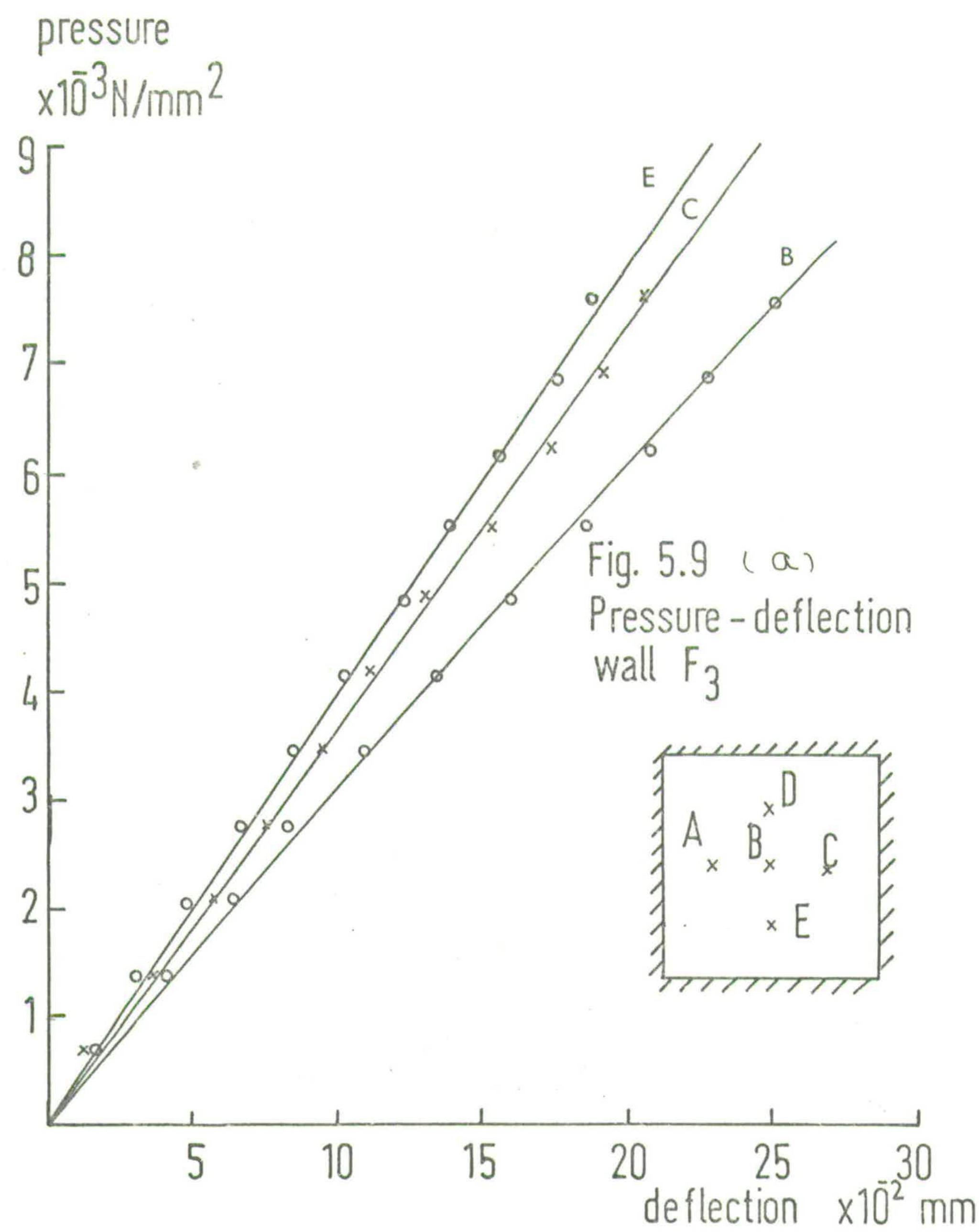


Fig. 5.9 (a)
Pressure - deflection
wall F_3

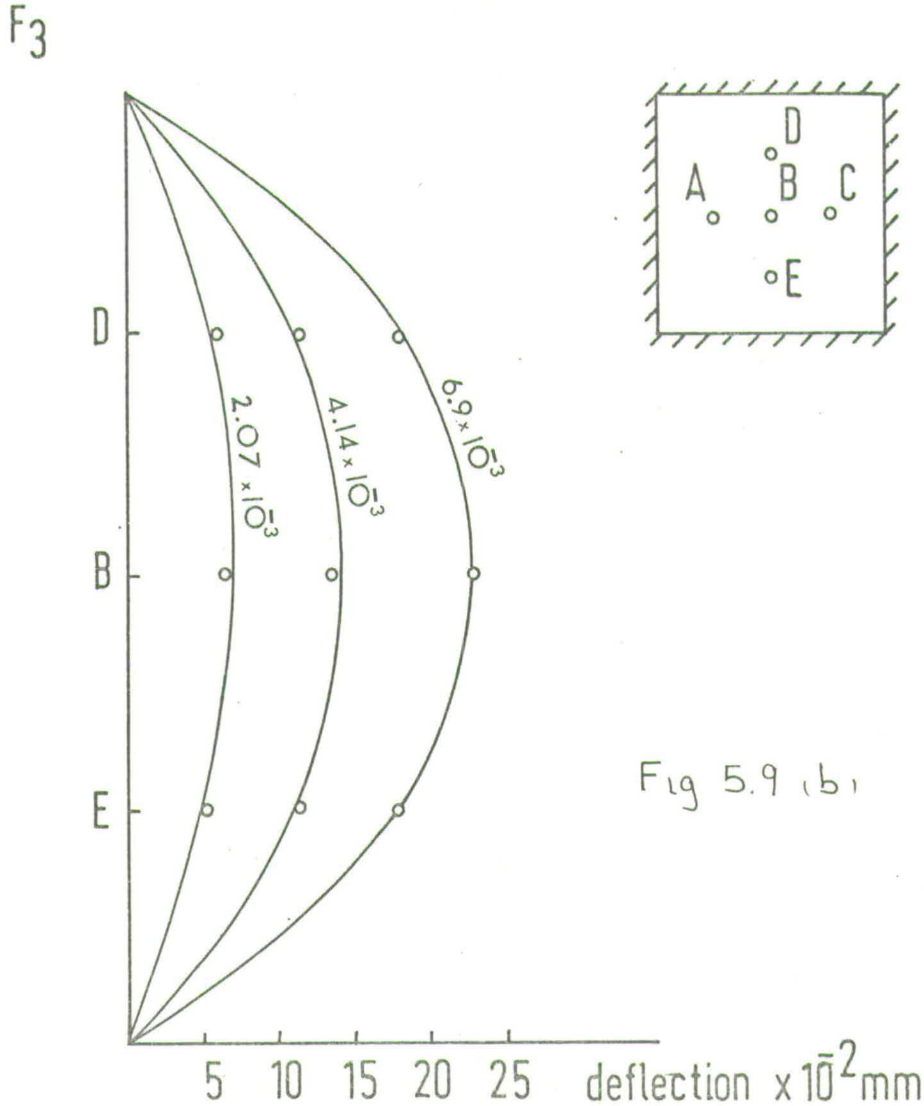


Fig 5.9 (b)

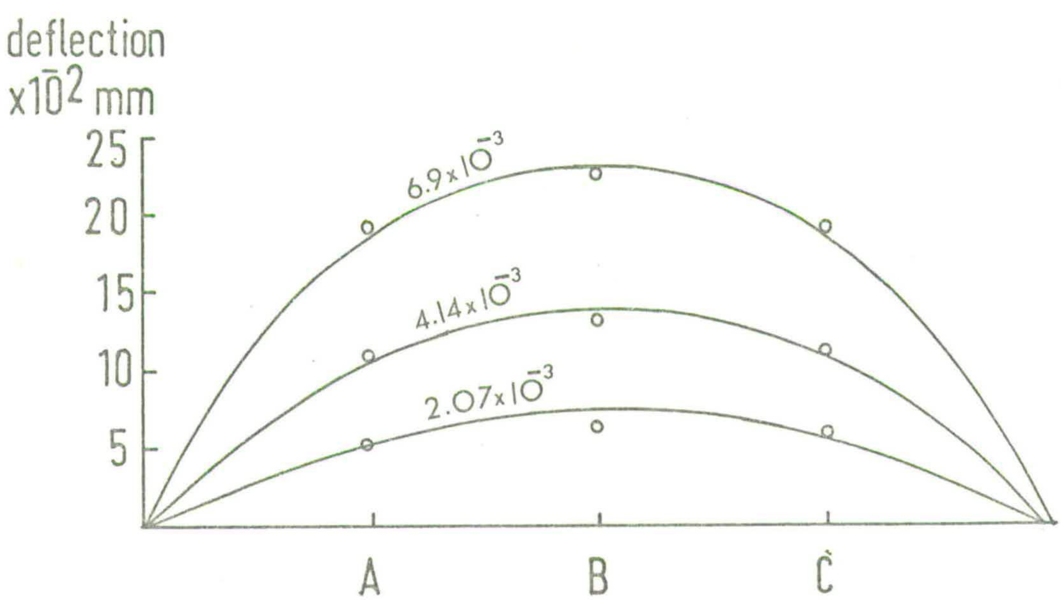


Fig 5.9 (c)

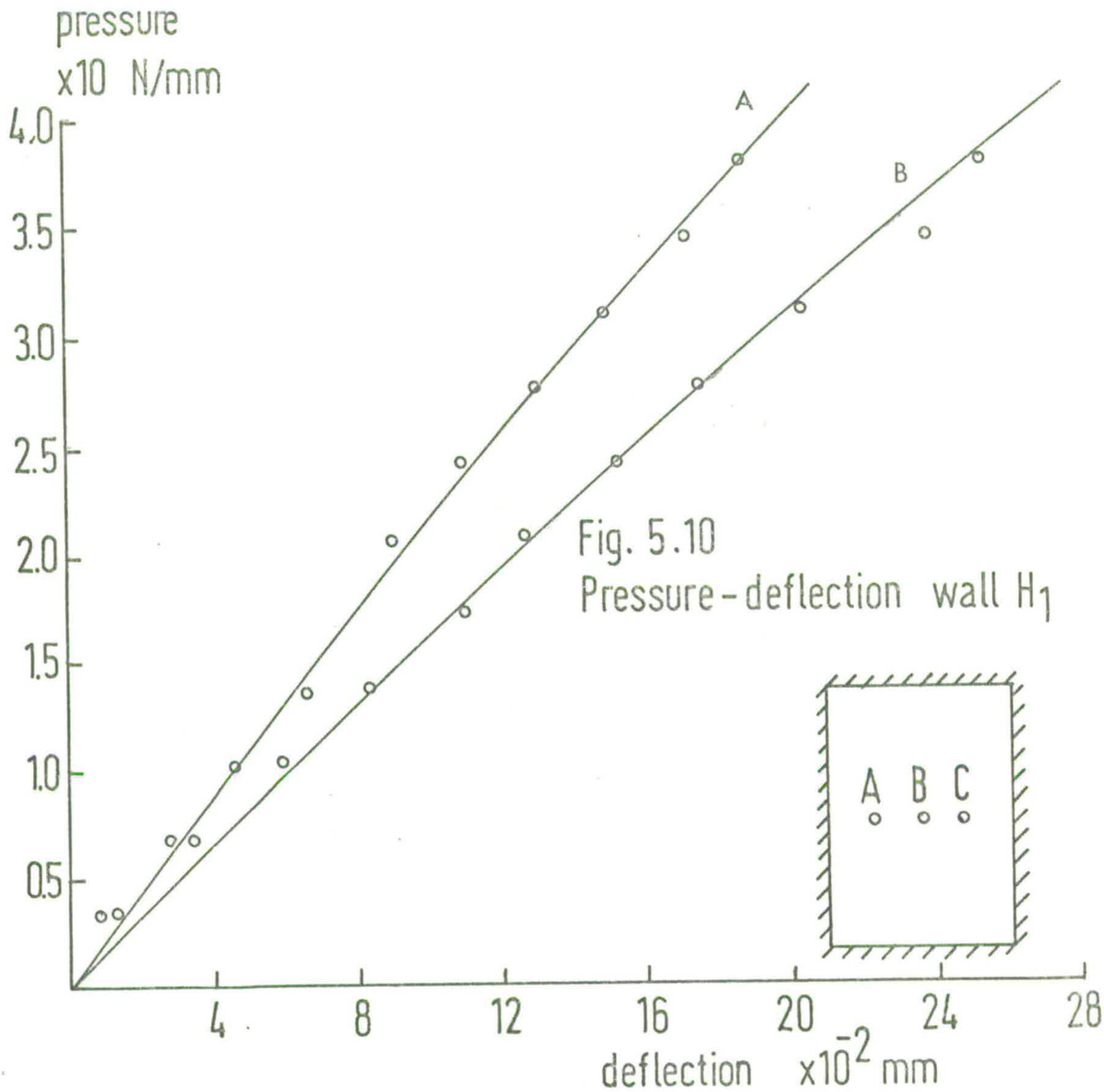


Fig. 5.10
Pressure-deflection wall H_1

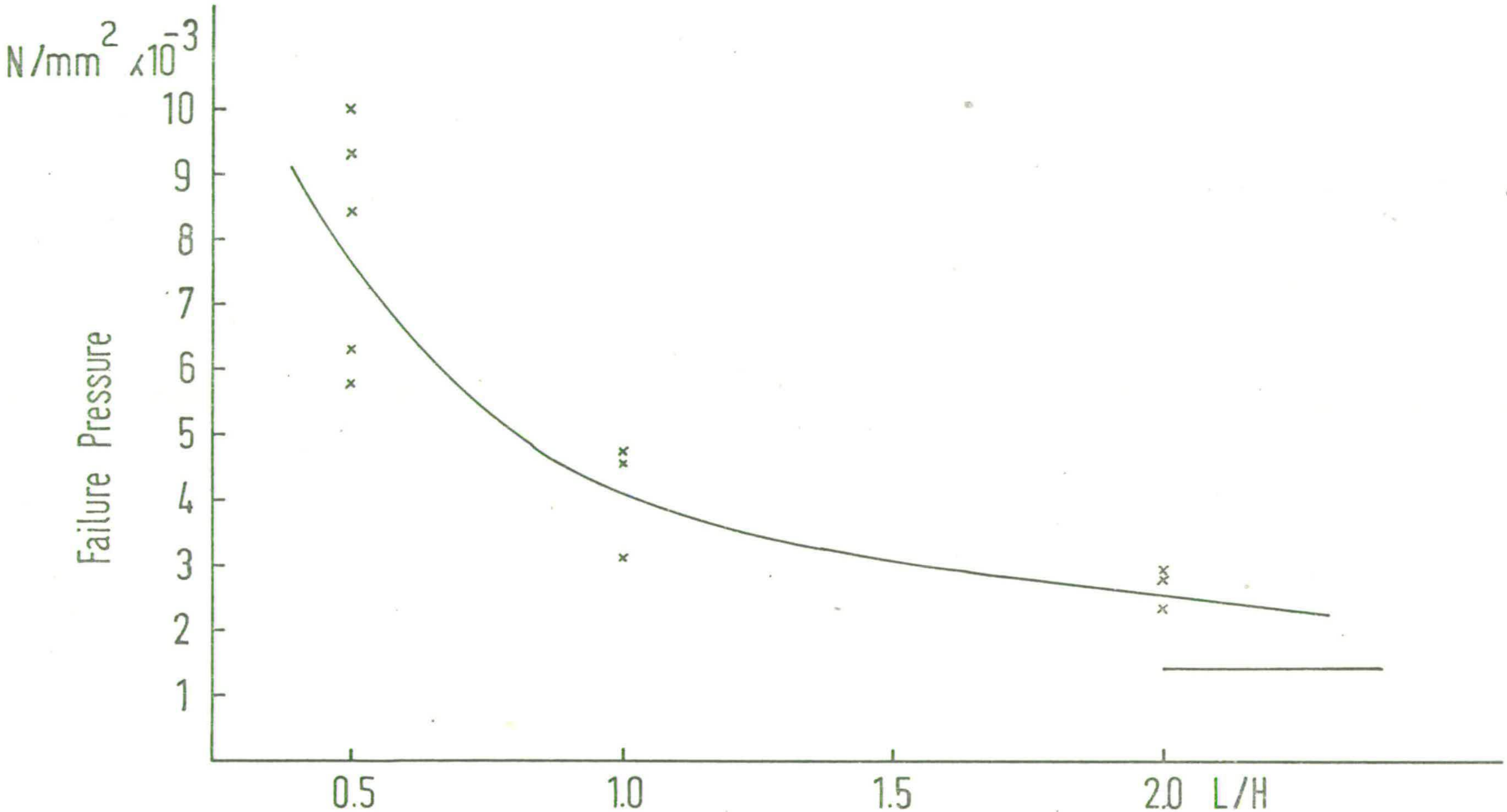


Fig. 5.11 Failure Pressure versus aspect ratio (L/H)
Panels with 3 sides simply supported.

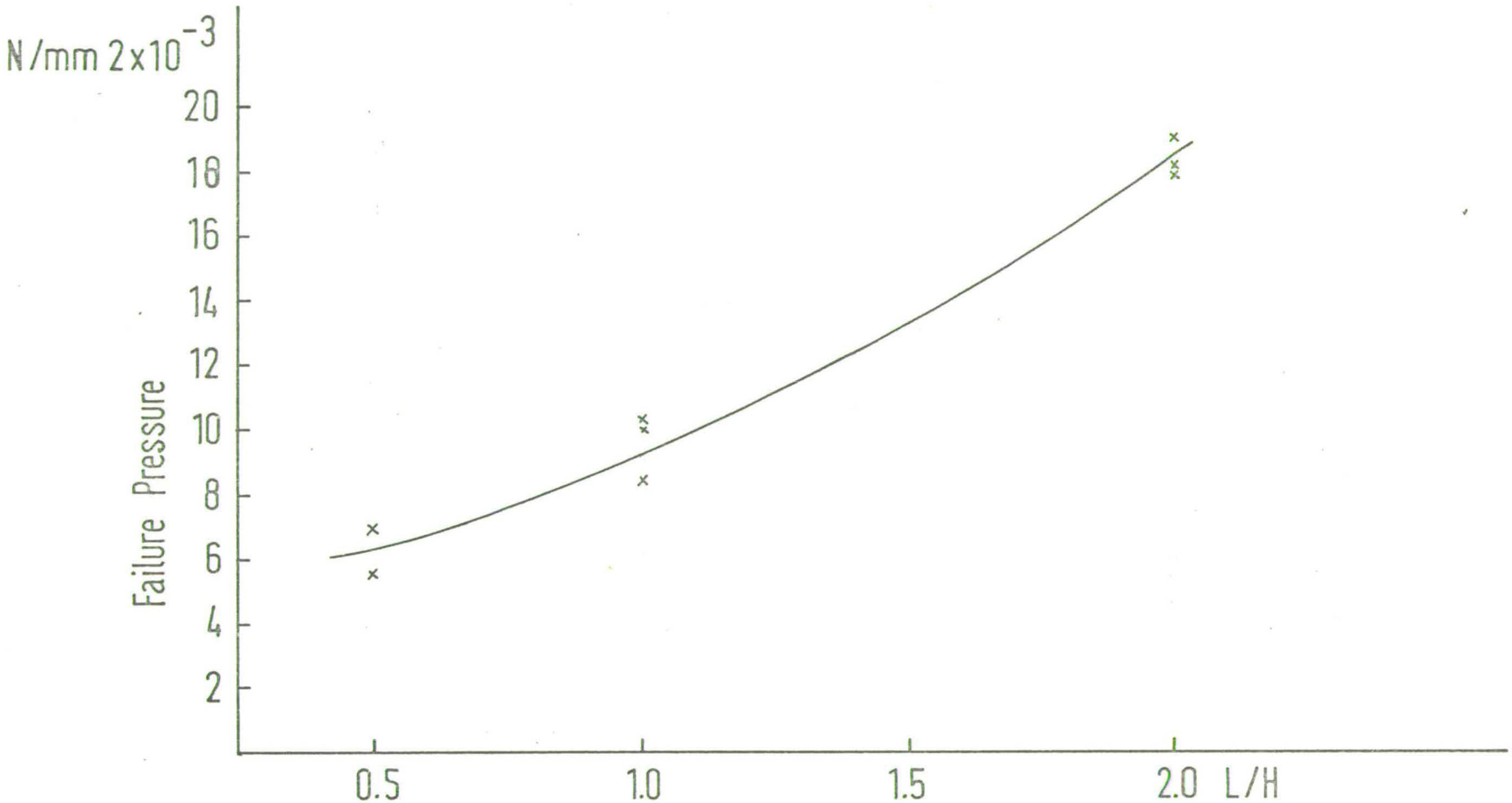


Fig. 5.12 Failure Pressure versus aspect ratio (L/H)
Panels with 4 sides simply supported.

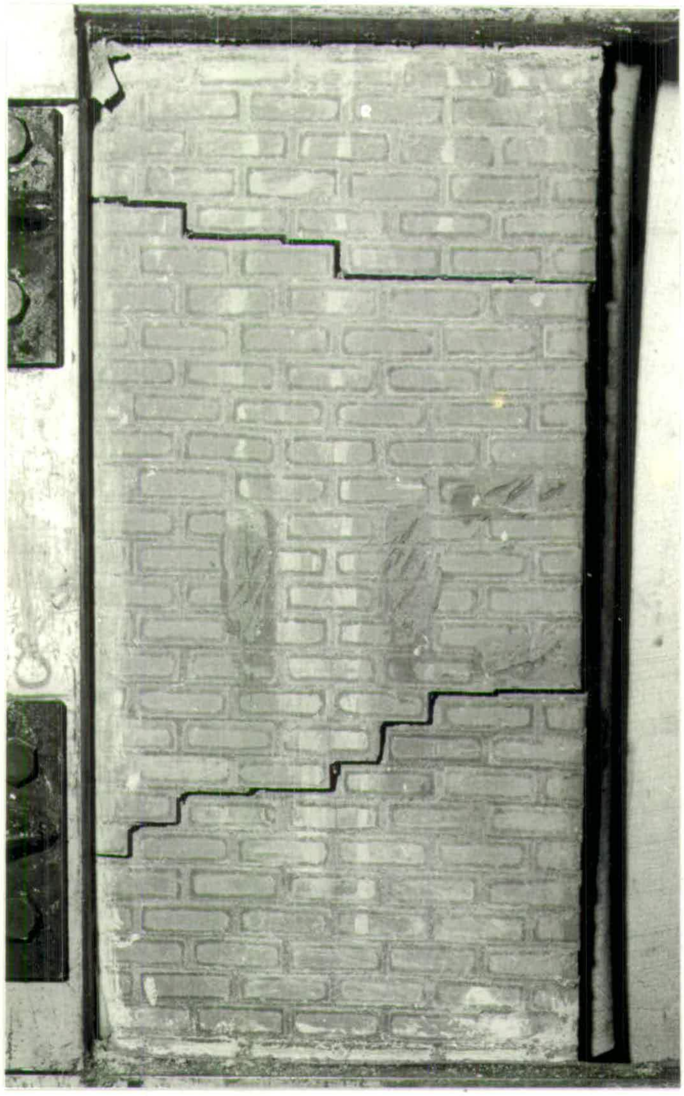


FIG. 5.13

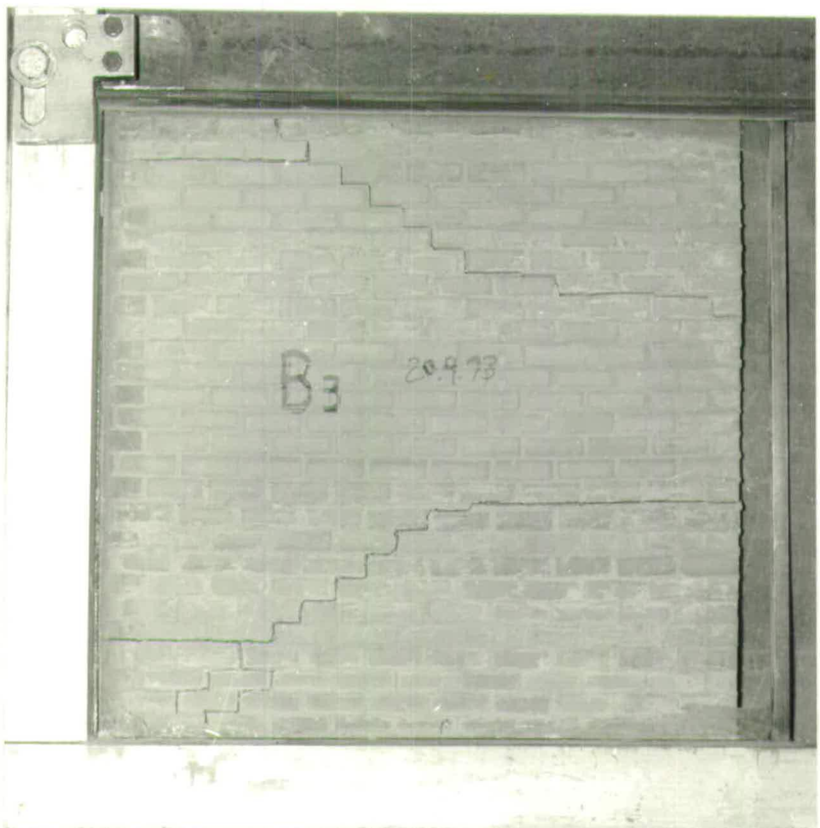


FIG. 5.14

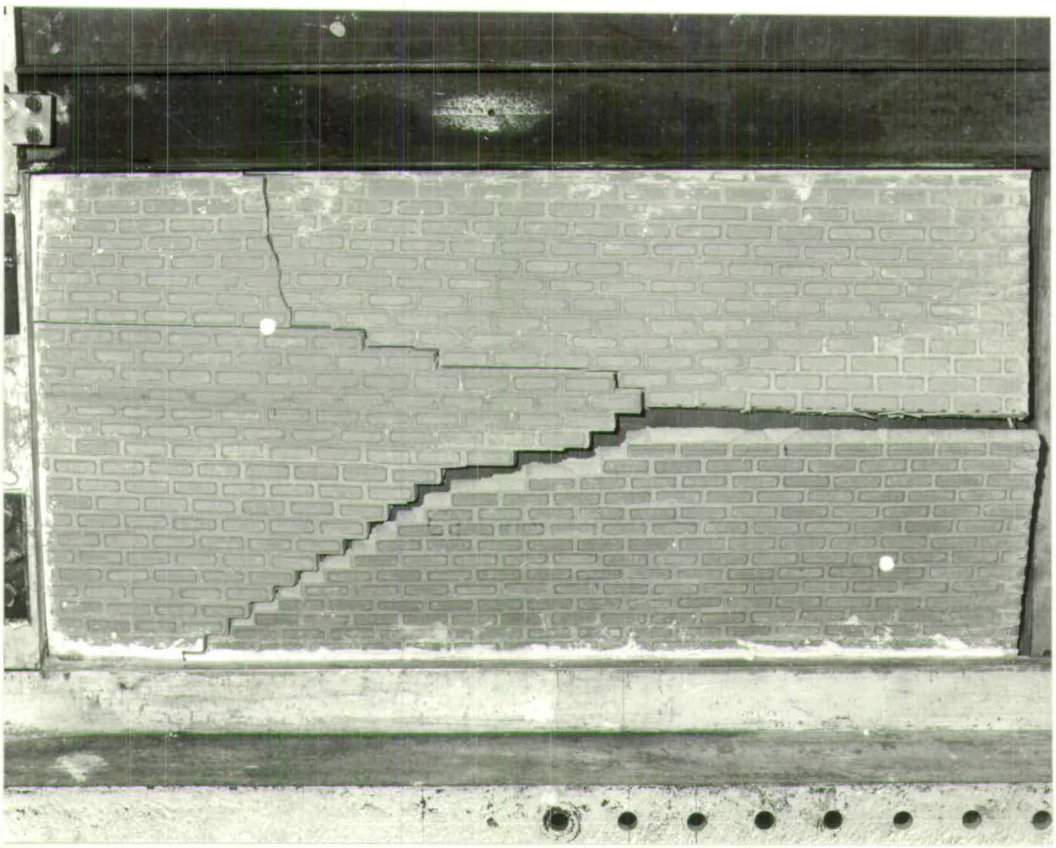


FIG. 5.15

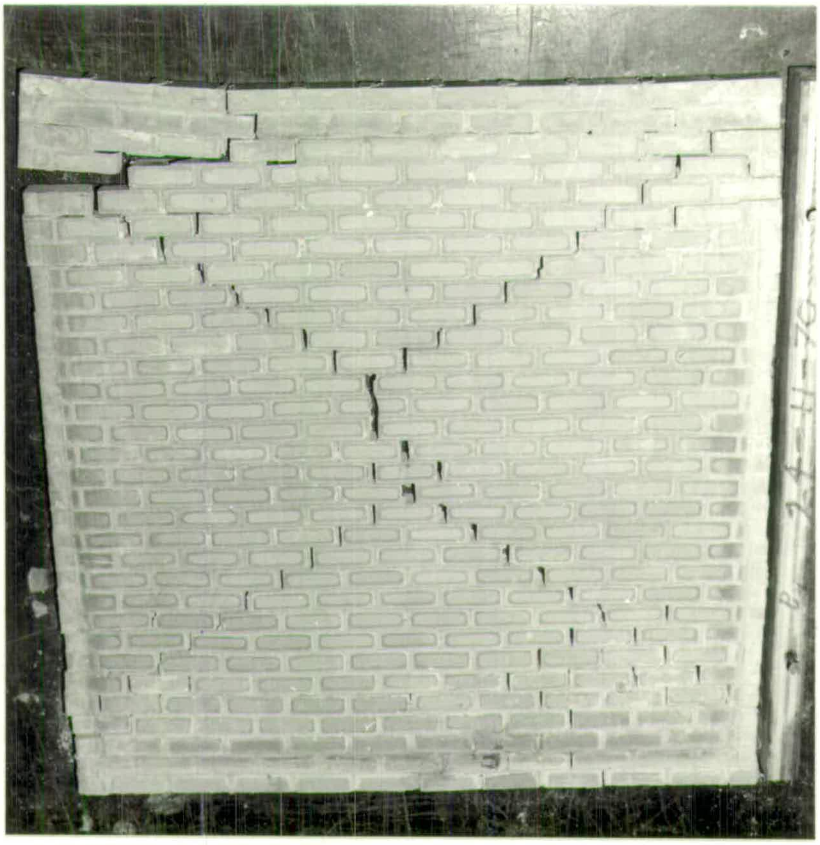


FIG. 5.16

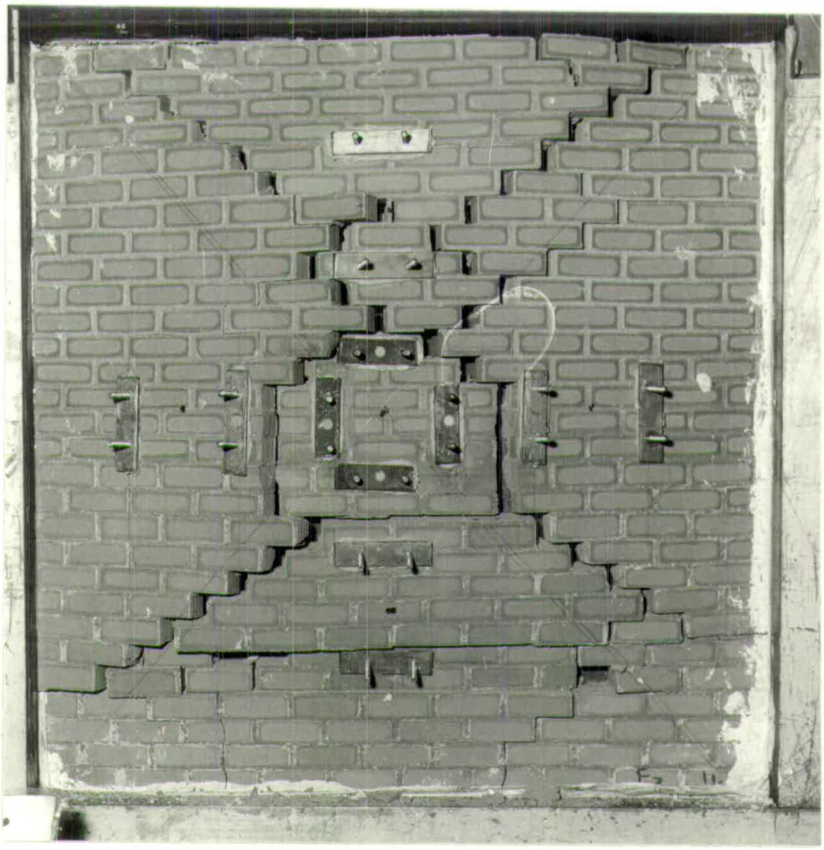


FIG. 5.17

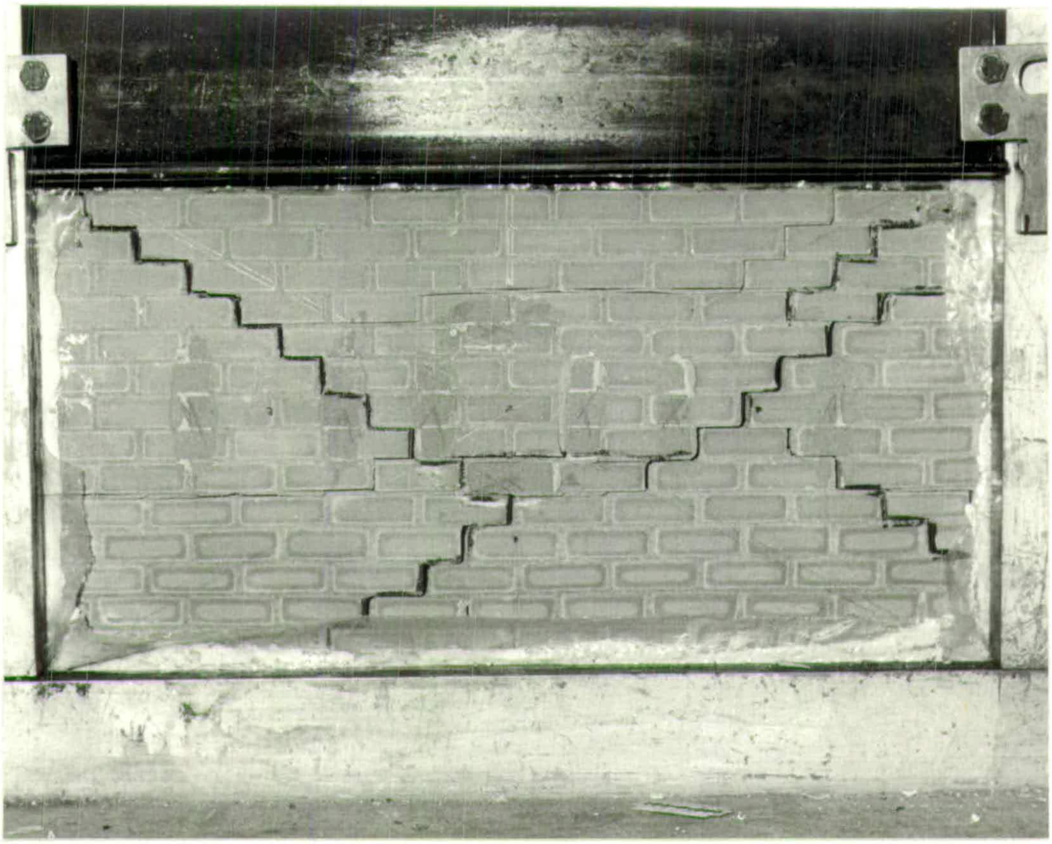


FIG. 5.18

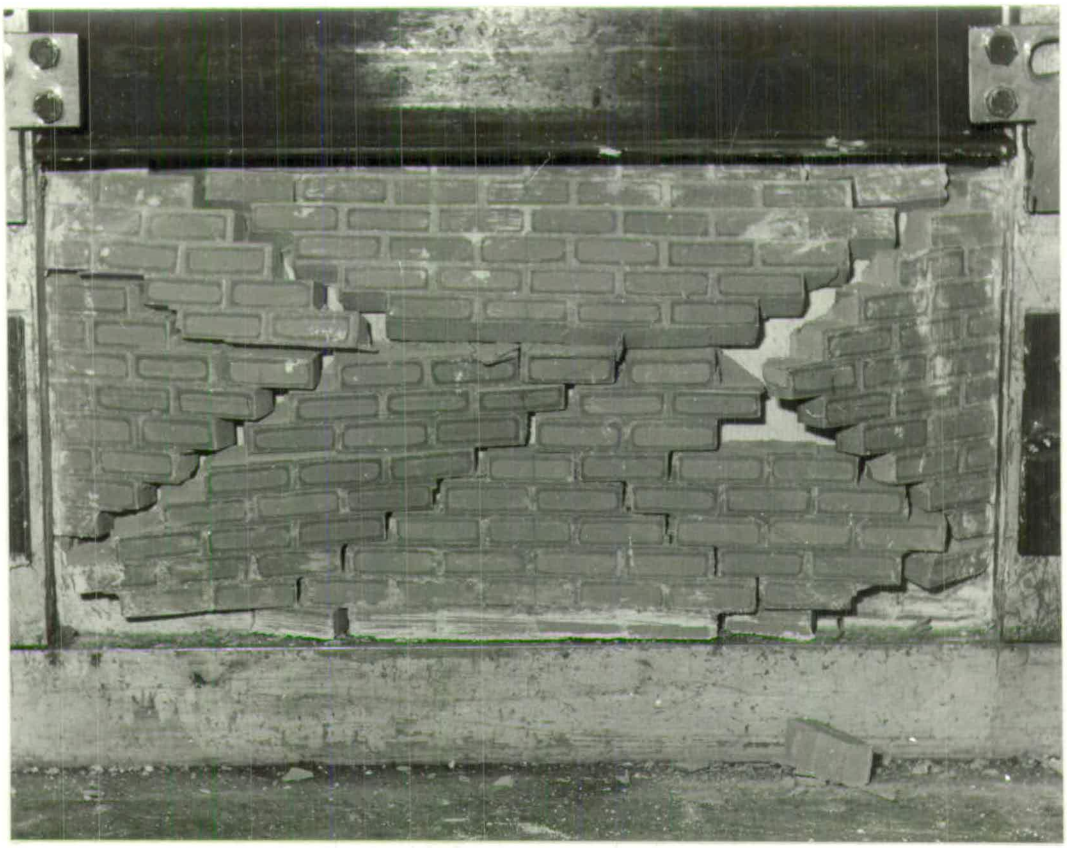


FIG. 5.19

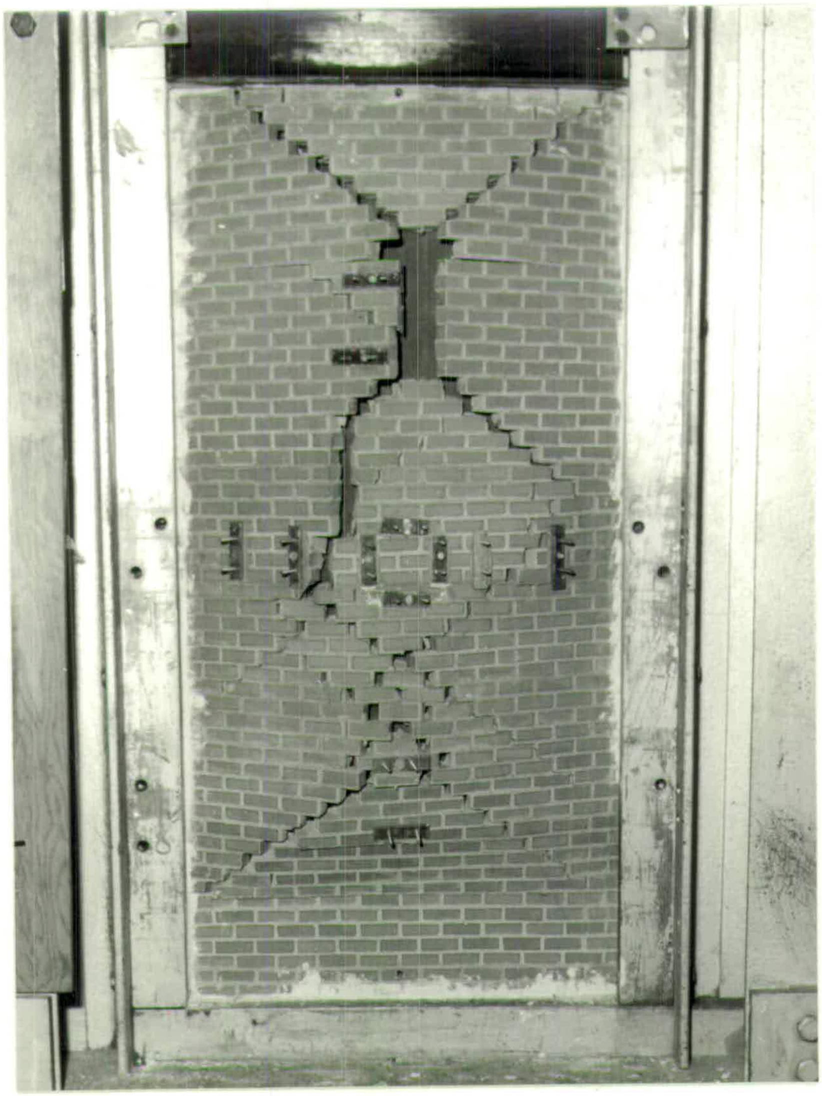


FIG. 5.20

CHAPTER 6.

ANALYSIS OF BRICKWORK PANELS.

ANALYSIS OF BRICKWORK PANELS.

6.1 INTRODUCTION:

In this chapter the application of the finite element, the yield line and the strip methods are considered. The assumptions and simplifications associated with each of the theories are assumed to hold. The different strengths of brickwork when spanning horizontally and vertically are considered in analysis. Failure loads were calculated for each wall by the three considered approaches and results are presented in tables and figures at the end of the chapter.

6.2 ANALYSIS OF BRICKWORK PANELS BY THE FINITE ELEMENT METHOD:

6.2.1 Computer Programme:-

To carry out the numerical calculation a computer programme(37) based on a finite element procedure for displacement analysis of plate bending is used. The programme, the strudl, allows calculation of the displacements, moments and reactions, for different shape of loading and support conditions. The panels were divided into a reasonable number of elements. The element's coordinates and incidents, the thickness, the elastic modulus and Poisson's ratio inserted and boundary conditions imposed. The rectangular element BPR, of the mentioned programme, with sides parallel to the X and Y directions was used in this computation.

For the purpose of calculation, the panels were considered to be of thin linearly elastic materials. In order to analyse the failure load of the panels, the elastic constants and the moment at failure parallel to and normal to the horizontal joints were determined by means of detail tests. The calculations have been made using a modulus of elasticity of 9.66×10^3 N/mm^2 (1.4×10^6 lb/in^2) in the vertical and horizontal directions. Detailed tests on brickwork specimens indicated that values of Young's modulus in both directions were approximately the same (3.3.2). Poisson's ratio was taken as 0.1.

6.2.2 Panels 3 and 4 sides simply supported.

Calculations were carried out assuming that failure occurs with the attainment of the modulus of rupture at the point of maximum tensile principal stress. In other words the panel will fail if either the bending moment in the vertical or the horizontal direction reached the moment of resistance in the corresponding direction respectively. Accordingly the failure load will be the modulus of rupture divided by the stresses due to the application of a unit load.

So the failure load is the least of the following:

$$P = \frac{f_n Z}{Mn} \qquad \text{or} \qquad P = \frac{f_p Z}{Mp}$$

where M_n & Mp are the moment giving maximum tensile principal stresses in directions normal and parallel to the bed joints respectively.

Moment coefficient K and failure pressures were calculated using the above criterion of failure. Results of panels 3 sides supported are listed in table (6.1) and fig. (6.1) shows the moment coefficient versus the aspect ratios.

For panels 4 sides simply supported the moment coefficient as well as the failure pressures are shown in table (6.2). The moment coefficient K for various aspect ratios is presented graphically in Fig. (6.2).

6.3 ANALYSIS OF BRICKWORK PANELS BY THE YIELD-LINE METHOD:-

6.3.1 General:-

The analysis carried out here was to establish whether the failure load of brickwork panels subjected to lateral loading can be estimated by means of the yield line theory.

The principle of virtual work is used in calculating the failure loads of the walls simply supported on three and four sides. The moment of resistance in the vertical and horizontal directions were assumed equal to the modulus of rupture moment, Mn and Mp, normal and parallel to the bed joints respectively. Mn and Mp being obtained from the detailed tests described previously.

There is always, in connection with brickwork panels, a preferential location of the diagonal cracks along the joint pattern. However, the presence of the mortar joints is neglected and failure calculations were carried out according to an idealized yield line pattern as shown on the next page. Figures (a) and (b) for panels three sides simply supported and figure (c) for the panels four sides supported.

Yield-line location at failure were chosen by differentiation of the work equation to give the minimum possible collapse load for each mechanism.

6.3.2 Panels three sides simply supported:-

For the panels three sides simply supported two modes of failure are possible to occur.

6.3.2.1 First mode of failure:

This mode of failure is used for panels (A) with length to height ratio of 0.5. The simplified yield line pattern is shown in fig. (a).

The following expression is given for the ultimate moment per unit length (see appendix B for the work equation).

$$M_n = \frac{qH^2\gamma}{16} \qquad (6.1)$$

or

$$M_n = K \quad qH^2 \qquad (6.2)$$

where

$$\gamma = \frac{4\alpha^2\left(\sqrt{1 + \frac{q\mu}{4\alpha^2}} - 1\right)}{3\mu}$$

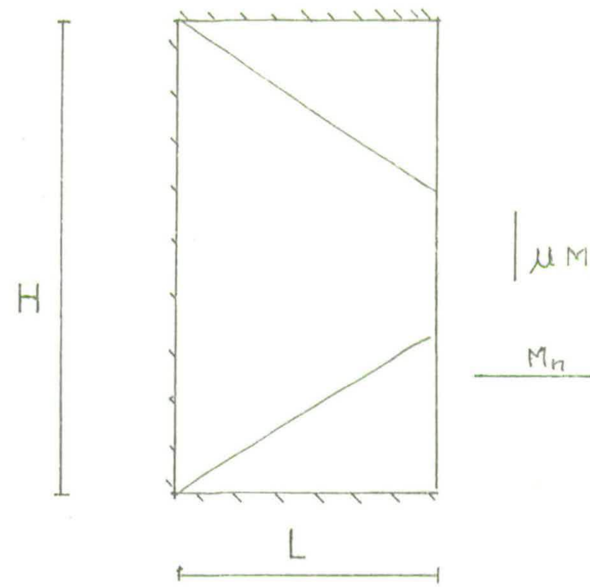


Fig (a)

q = lateral pressure N/mm^2

H = height of the panel mm

L = length of the panel mm

α = length to height ratio

μ = degree of orthotropy $M_p/M_n > 1$

K = dimensionless moment coefficient.

The failure load can be written in the following equation

$$P = \frac{16\ Mn}{H^2} \quad \frac{1}{\gamma} \qquad (6.3)$$

Second mode of failure:-

For this mode of failure the idealized yield line pattern is shown in figure (b). The equation of the ultimate moment per unit length is as follows (appendix B).

$$M_n = \frac{qH^2\gamma^2}{6} \qquad (6.4)$$

where

$$\gamma = \frac{\sqrt{\mu^2 + 12\alpha^2\mu} - \mu}{4\alpha} = \frac{C}{H}$$

$$\alpha = \frac{L}{H}$$

$$\mu = M_p/M_n > 1$$

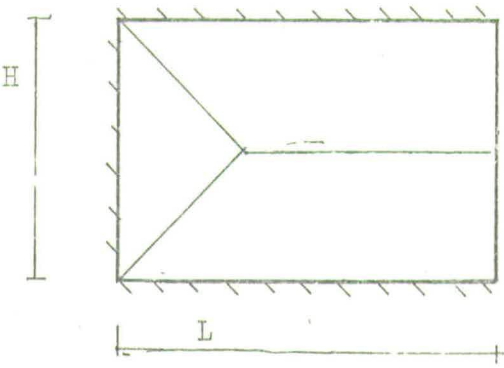
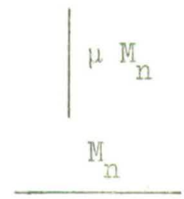


Fig (b)



The failure load will be:-

$$q = \frac{6\,M_n}{H}\quad\frac{\mu}{\gamma^2} \qquad (6.5)$$

Failure pressures has been calculated for all the three sided panels using equations 6.3 and 6.5. Results are listed in table (6.3). The dimensionless moment coefficient versus the aspect ratios is given in figure (6.3).

6.3.3 <u>Panels four sides simply supported</u>:-

Unlike the failure in the case when a panel is supported on three sides with the vertical side free, the failure for a four sides supported panel will vary according to the dimensions of the loaded panel. It can be due to tensile bond, compound tensile shear bond or by breaking through the bricks and the vertical bed joints.

The failure mode used to carry out the calculations is shown in figure (c).

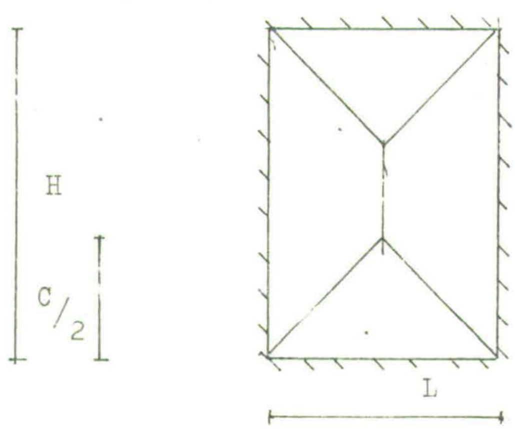
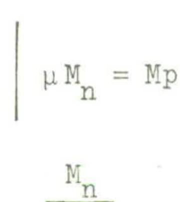


Fig (c)



The same procedure of the virtual work is used here and the moment equation for a panel with length to height ratio one or less is given by the following relation:-

$$M_n = \frac{qL^2\ \gamma^2}{24} \qquad (6.6)$$

$$\gamma = \frac{\sqrt{1 + 3\ \mu\alpha^2} - 1}{\mu\alpha} = \frac{C}{L}$$

$$\mu = M_p/M_n \geqslant 1$$

$$\alpha = H/L$$

For a square panel the failure mode assumes a two crossed diagonal fracture lines. The moment equation is the same as the above equation with $\alpha = 1$

$$M_n = \frac{qL^2}{24}\gamma^2$$

$$\gamma = \frac{C}{L} = \frac{(\sqrt{1 + 3\mu} - 1}{\mu}$$

where

$$\alpha = H/L$$

$$\mu = M_p/M_n \geqslant 1$$

Equation (6.6) will hold for a panel with length to height ratio of 2. The ratio of orthotropy being as follows:

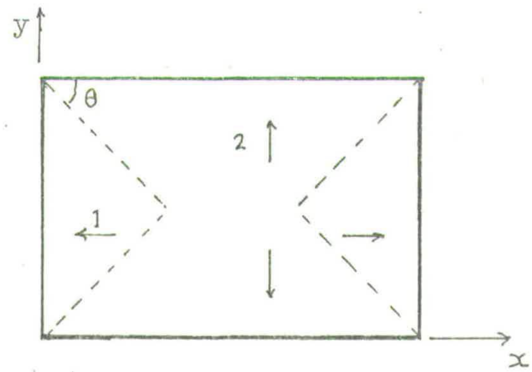
$$\mu = M_n/M_p \leqslant 1$$

The failure pressures calculated on the basis of the yield line patterns shown, were listed in table (6.4) for all the panels four sides simply supported.

The values of the coefficient K has been computed and plotted versus the ratio of panel side lengths. Results are presented in figure (6.4).

6.4 THE STRIP METHOD.

To find the moment coefficient of a panel by the strip method, the moments are calculated for each strip in each direction and the corresponding moment volume is evaluated. Considering a rectangular panel simply supported on four sides, lines of stress discontinuity are introduced as shown in fig. (d) below. These discontinuity lines indicate the designers decision to carry all the load in areas 1 in the x-direction on x strips, and all load in area 2 in the y-direction on Y strips. The discontinuity lines are not yield lines, and the designer is free to choose the angle θ .



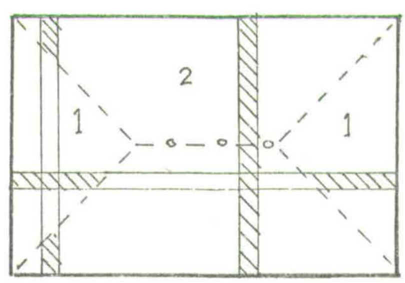
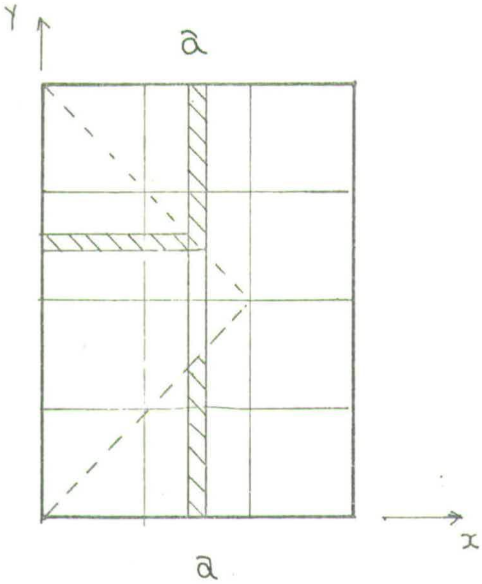


Fig (d) - - - - discontinuity lines
— o — o — zero shear lines

Fig (e)

In fig. (e) above, the central y strips are simple one-way slab strips under a uniform load or such other distribution of load as may exist. The y strips running through an area 1 are unloaded in that area and loaded only in the two area 2 end portions, as indicated by the shaded areas. Likewise x strips are all unloaded except at areas 1 near the supports.

A panel with three sides simply supported is shown in fig (f).



Strip a-a

Fig. (f) Fig (g)

The x strips are loaded in area 1 only and are assumed to be supported at their free edges by the y strips. The reaction on the y strips been treated as concentrated loads as shown in fig. (g).

The average moment coefficient versus the aspect ratios is presented in fig. (6.5) and (6.6) for panels supported on three sides and four sides respectively. A numerical example showing the method of calculation is given in appendix (c).

TABLE 6.1 PANELS 3 SIDES SIMPLY SUPPORTED MOMENT COEFF. K AND FAILURE PRESSURES.

ELASTIC ANALYSIS $M_n = K\ q\ H^2$

Wall	L/H	Moment Coeff.K	Failure Pressure $N/mm^2 \times 10^{-3}$
A_1	0.5		2.5
A_2			2.3
A_3		0.054	2.15
A_5			4.3
A_6			5.0
B_1	1.0		1.2
B_7		0.106	2.0
B_8			1.8
C_3	2.0		1.4
C_4		0.12	1.76
C_5			1.34

TABLE 6.2 PANELS 4 SIDES SIMPLY SUPPORTED MOMENT COEFF. K AND FAILURE PRESSURES.

ELASTIC ANALYSIS $M_n = KqH^2$

Wall No.	L/H	Moment Coeff. K	Failure Pressure $N/mm \times 10^{-3}$
G_1	2	0.098	9.0
G_2			7.6
G_3			6.9
F_1		0.04	4.35
F_2			5.9
F_3			4.55
H_1	0.5	0.025[x]	5.6
H_2			5.9

x stresses parallel to bed joints (F_p) critical.

TABLE 6.3 PANELS 3 SIDES SIMPLY SUPPORTED MOMENT COEFF. K AND FAILURE PRESSURES.

YIELD-LINE ANALYSIS $M_n = Kq\ H^2$

$M_n = KqH^2$

Wall No	$L/_H$	Moment Coeff K	Failure Pressure $N/mm^2 \times 10^{-3}$
A_1	0.5	0.0265	5.0
A_2		0.0265	4.7
A_3		0.025	4.6
A_5		0.031	7.6
A_6		0.032	8.3
B_1	1.0	0.045	2.8
B_7		0.051	4.25
B_8		0.048	4.0
C_3	2.0	0.075	2.2
C_4		0.076	2.7
C_5		0.070	2.2

TABLE 6.4 PANELS 4 SIDES SIMPLY SUPPORTED MOMENT COEFF. K

AND FAILURE PRESSURES.

YIELD-LINE ANALYSIS $M_n = KqH^2$

Wall No.	L/H	Moment Coeff.	Failure Pressure $N/mm^2 \times 10^{-3}$
G_1	2.0	0.0485	18.0
G_2		0.044	16.4
G_3		0.047	14.3
F_1	1.0	0.022	7.9
F_2		0.026	9.2
F_3		0.0198	9.25
H_1	0.5	0.0073	6.1
H_2		0.0070	6.3

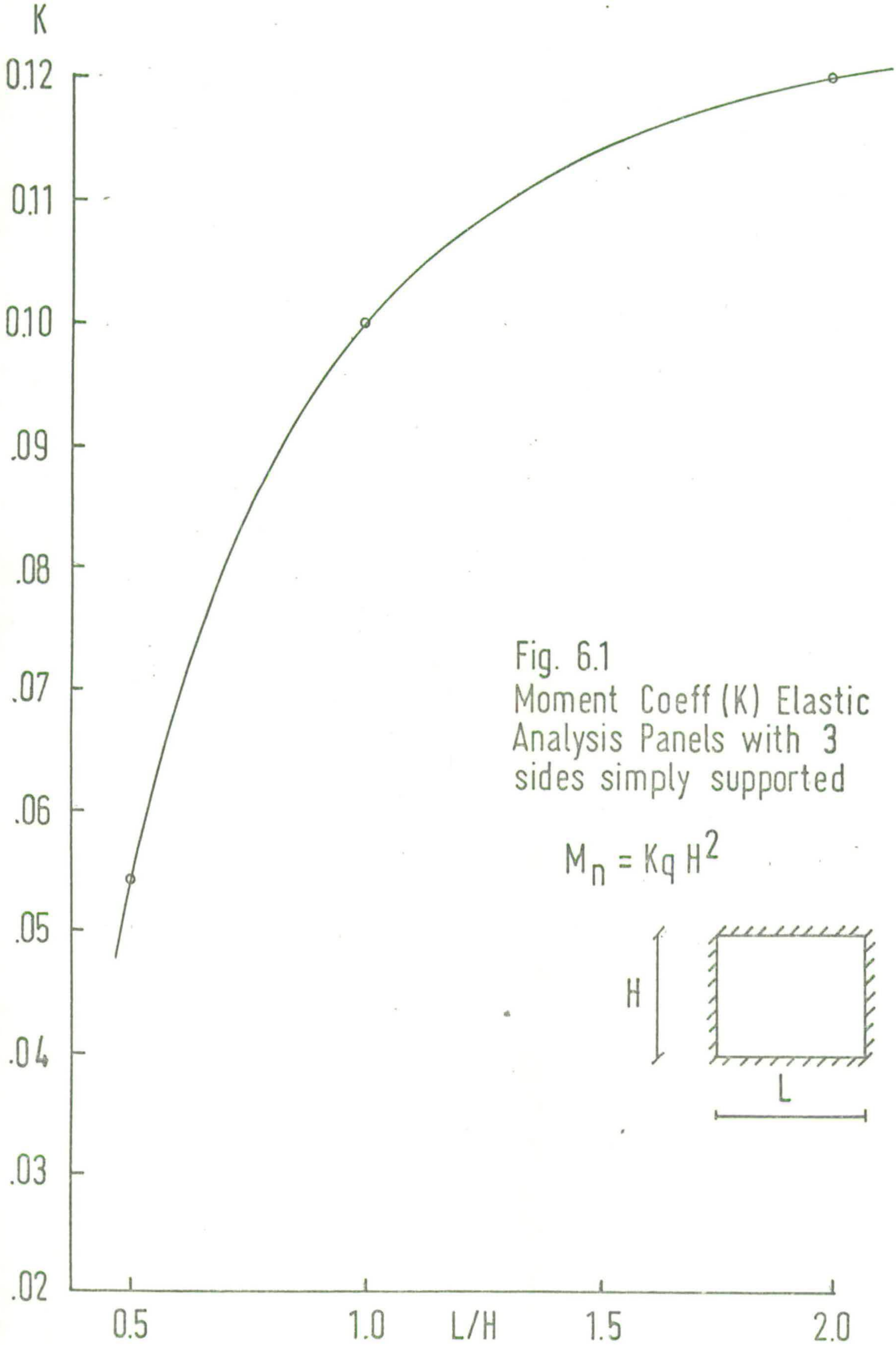


Fig. 6.1
Moment Coeff (K) Elastic Analysis Panels with 3 sides simply supported

$$M_n = Kq\,H^2$$

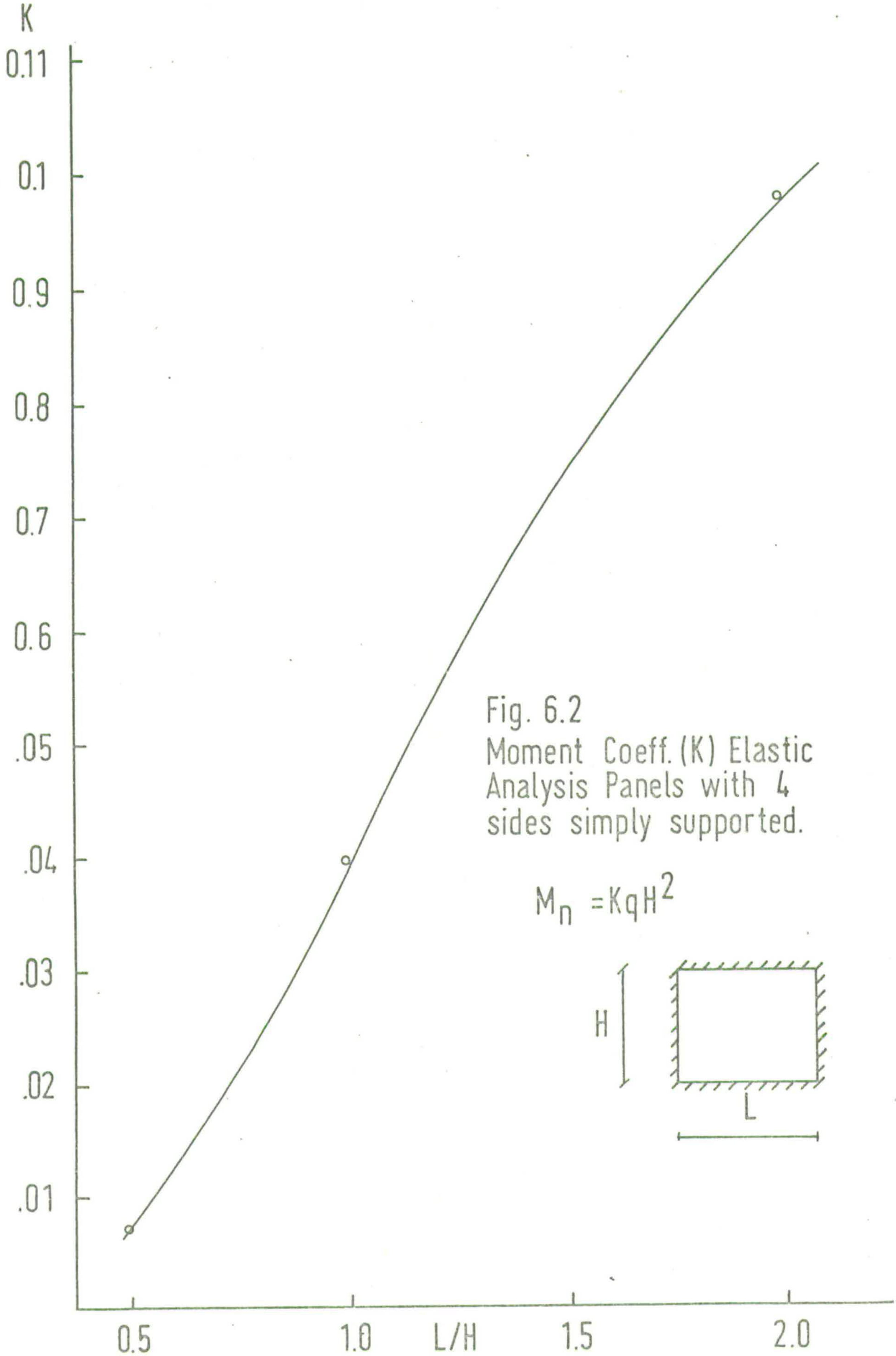


Fig. 6.2
Moment Coeff. (K) Elastic Analysis Panels with 4 sides simply supported.

$M_n = KqH^2$

Fig. 6.3
Moment Coeff (K) Yield-Line Analysis
Panels with 3 sides simply supported

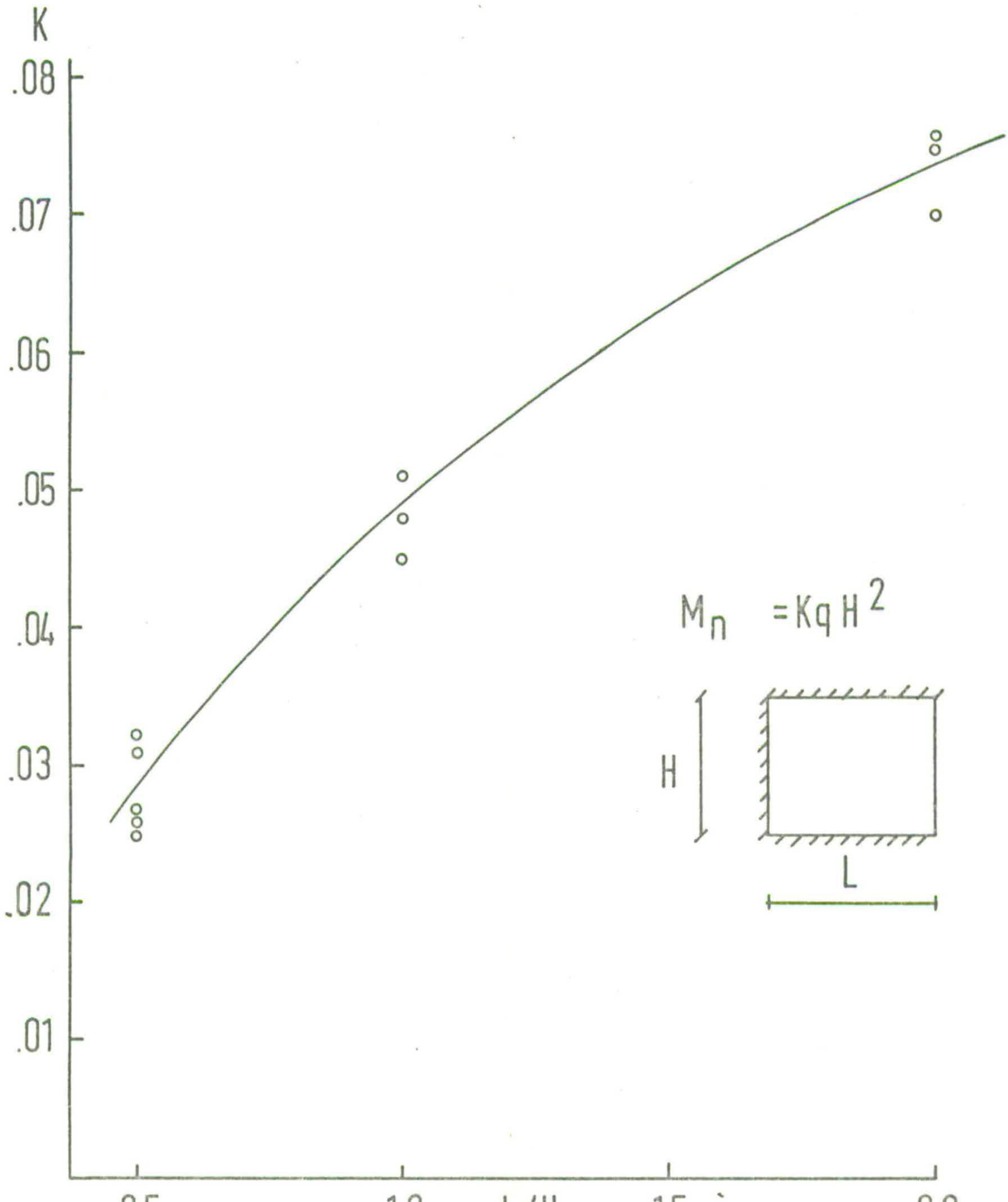


Fig. 6.4
Moment Coeff (K) Yield-Line Analysis
Panels with 4 sides simply supported

$M_n = Kq\ H^2$

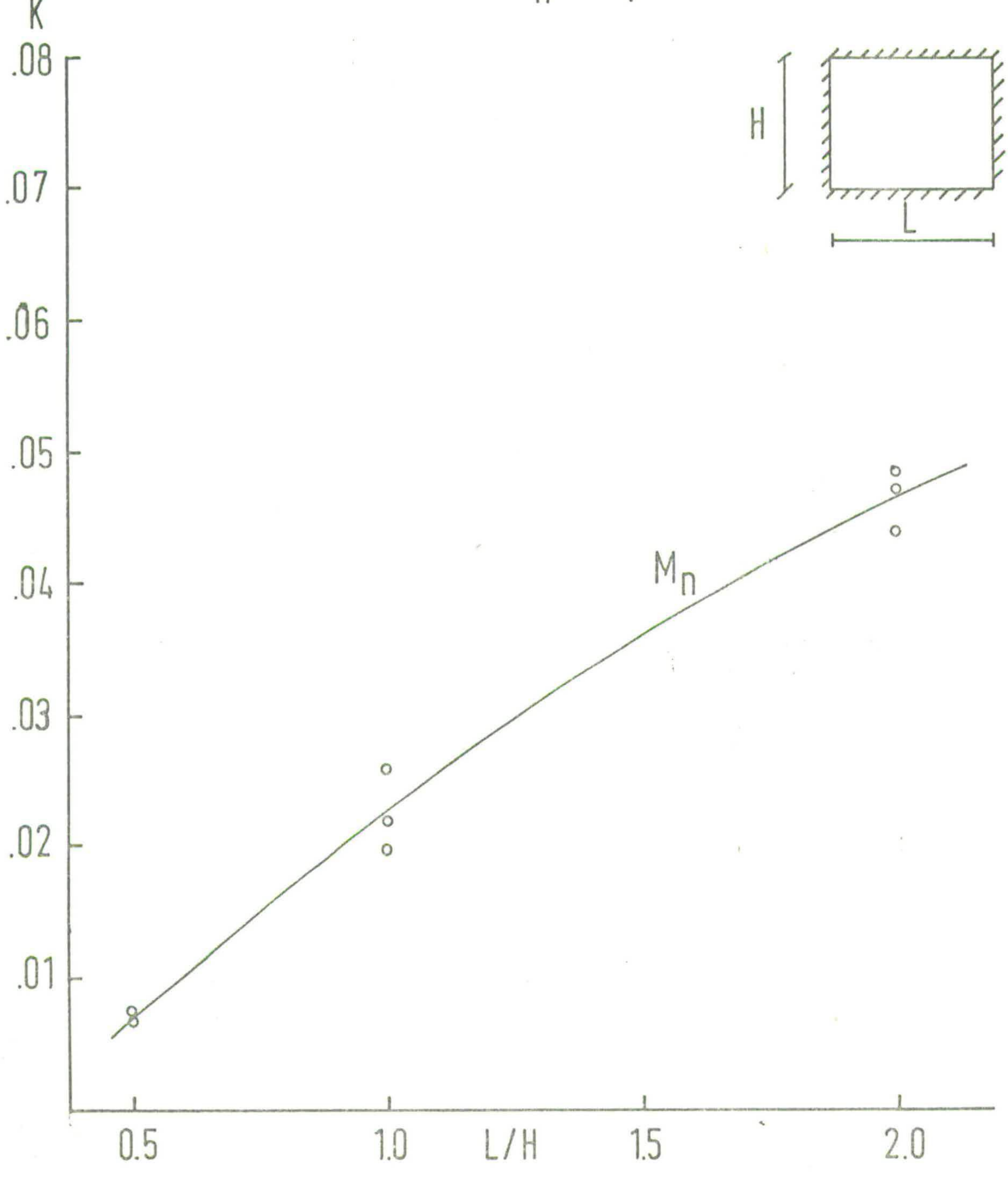


Fig. 6.5
Moment Coeff. (K) strip theory
Panels with 3 sides simply supported

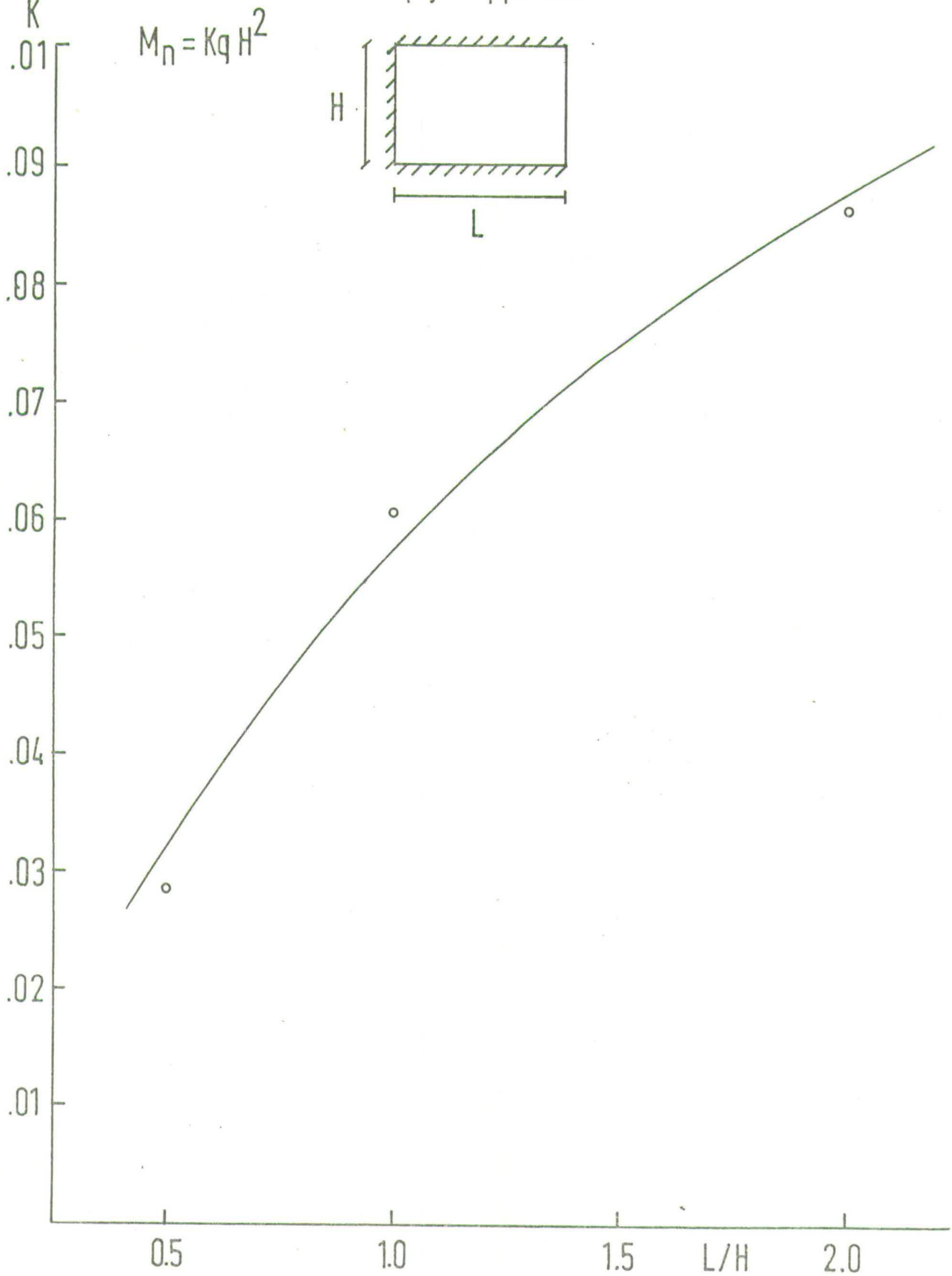
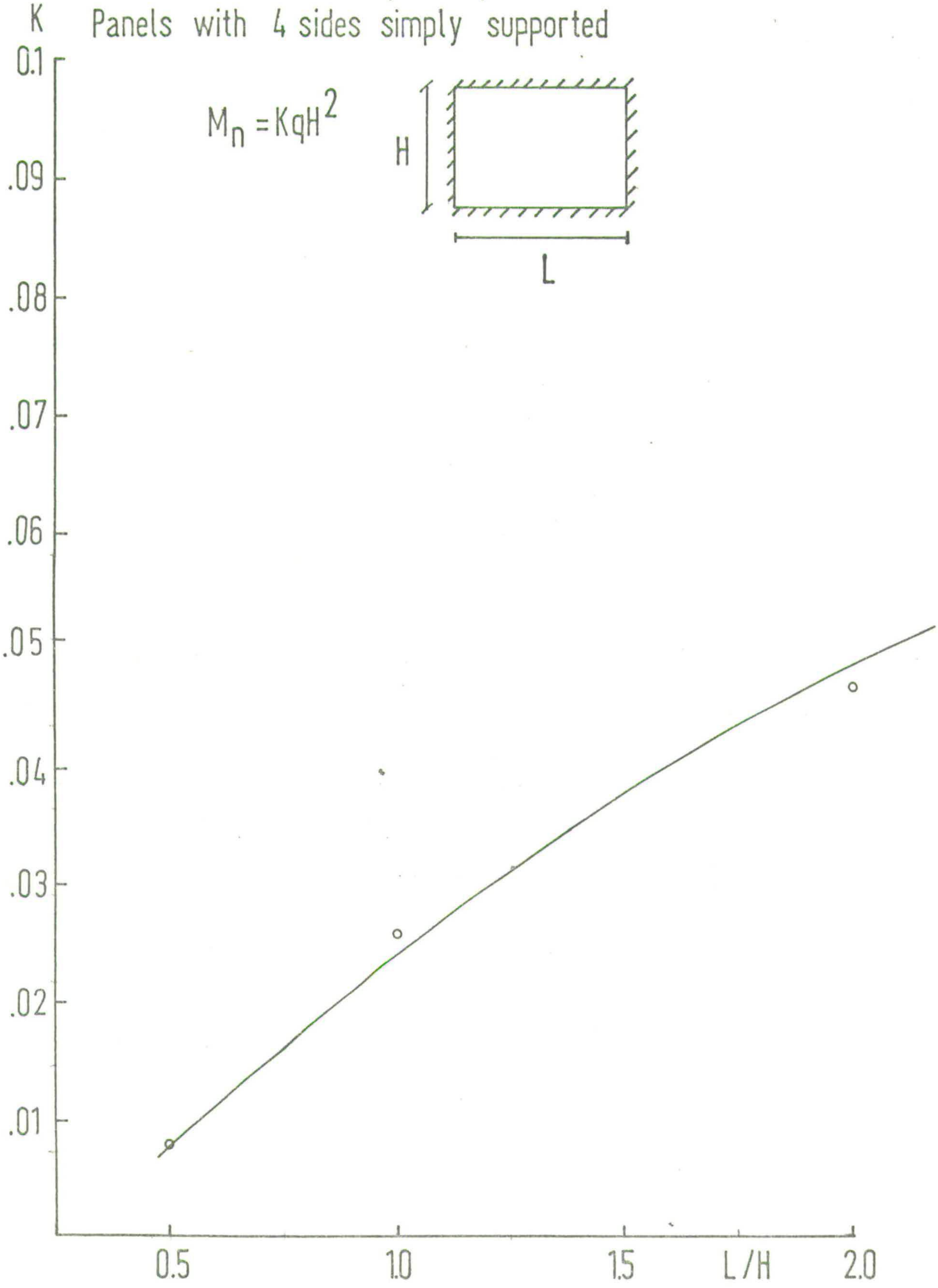


Fig. 6.6
Moment Coeff. (K) strip theory
Panels with 4 sides simply supported



CHAPTER 7

DISCUSSION, COMPARISON AND CONCLUSIONS.

7.1 INTRODUCTION

In this chapter, results of the analysis of model walls by the elastic theory, the yield line and the strip method, presented in the previous chapters, are discussed and compared with the experimental results. The calculated moment coefficients as well as the measured deflections are plotted in comparison with the experimental ones. At the end of the chapter, conclusions are drawn and recommendations for further research are made.

7.2 ELASTIC THEORY

For panels supported on three sides, the unsupported edge was one of the vertical sides. At the free edge the panel will tend to span in the vertical direction between the upper and lower supports and the first crack was always a horizontal one at the free edge regardless of the panel dimensions.

Fig. (7.1) shows the moment coefficient from elastic analysis compared with the experimental moment coefficient. It is clear that elastic theory greatly underestimated the failure load for all the walls tested. A scatter of results was noticed with these walls. This could be because F_n is highly variable and for these walls the stresses on bed joints were critical for all aspect ratios.

For panel G, supported on four sides, with span to height ratio of two, the panel tends to span in the shorter direction which happened to be the direction with the weaker moment of resistance. The failure of these panels usually initiated by a horizontal crack at the centre of the panel. For these walls the failure load was underestimated by the elastic theory. The elastic theory also underestimated the failure load for square panels supported on four sides. It was noticed that the inclination of the cracks at failure was nearly 45 degrees to the horizontal. The modulus of rupture in that direction could be taken as the average of the modulus of rupture in the vertical and the horizontal direction. It is of interest to mention that, assuming failure occurred when the stress at a point on an inclined yield line near the centre exceeded the corresponding modulus of rupture (at 45°), then the failure load could be reasonably estimated by elastic theory. Results are shown in the last column of table 7.2.

The panel (H) with span to height ratio of 0.5 failed suddenly and the maximum load occurred prior to the first crack. In other words the failure load was the cracking load. Failure pressures calculated in accordance with the theory of elasticity showed good agreement with the test results as illustrated in table (7.2) Deflections obtained from elastic analusis are compared with experimental results as shown in fig. (7.3) and (7.4). The measured deflections were always bigger than the theoretical ones.

7.3 <u>The Yield Line Theory</u>

Reasonable agreement has been obtained between tests results and the analysis carried out using the yield line theory. The failure pressure was satisfactorily estimated using this method of analysis. In table (7.1) and (7.2) the resulting failure pressure values are summarized and compared with experimental ones. Also in fig (7.1) and (7.2) the moment coefficient K obtained by the yield line analysis is shown in comparison with the experimental results.

It was observed that, in most of the tests, failure took place along the yield lines simultaneously with no cracks appearing at earlier stages. Therefore all yield lines were assumed to have attained the maximum bending moment and yield stress at the same time. Obviously, as mentioned before, there are many uncertainities about the application of the yield line theory to brickwork. Also the calculations were carried out according to an idealized straight fracture lines while in practice there is always a preferential location of the cracks along the joint pattern, and for the same type of wall, with the same dimensions, the failure pattern could be different from one wall to another. However, the obvious coincidence between test results and calculated pressures may show that the assumptions used for the calculation could be realistic.

7.4 The strip theory

Moment coefficients calculated using the strip theory are shown in comparison with the experimental ones in fig (7.1) and (7.2) for panels with three and four sides supported respectively. For panels supported on three sides the theory took account of two way action by assuming that the horizontally spanning strips are supported at their free edge by the vertically spanning ones. An average ratio of moments in orthogonal directions, for each set of panels of the same aspect ratio, was used in carrying out the calculations. The theory underestimated the failure load but the results obtained were better than those obtained by the elastic theory. As the strip theory is originally introduced for the design of reinforced concrete slabs, it seems of doubtful applicability to the design of brickwork as the basic assumptions do not hold. Moreover the lines of stress discontinuity are assumed to be straight which is not true in the case of brickwork.

Although the strip theory as well as the yield line showed reasonable agreement with the experimental results, there is at the moment no rational justification for their use. Until further work has been done the failure load could be safely estimated using either theory as both tend to underestimate the load carrying capacity of a brickwork panel.

7.5 Experimental moment coefficient from previous work.

It is of interest to calculate the experimental moment coefficient (K) for some reported tests from existing literature. These values are plotted versus the aspect ratios in fig. (7.5). It is clear that there is a wide scatter of results. If this scatter is not due to an experimental or numerical error, then it means that there are still many unknown factors influencing the flexural strength of brickwork panels. This influence could be either on the behaviour of the panel itself or on the assessment of the modulus of rupture from the test beams.

It may also be that the different methods of loading and testing used, as well as the different dimensions of the test beams could also be a reason for the wide scatter observed.

7.6 Conclusions.

The study which has been described led to the following conclusions, some of which are self evident.

a) The flexural strength parallel to the bed joints (F_p) is several times the flexural strength normal to the bed joints (F_n) for lower value of (F_n), and nearly twice for higher values. The relation is a nonlinear one.

b) As the bending strength of brickwork in the horizontal direction is different from that in the vertical direction, then, the wall capacity depends to a large extent on the aspect ratio of the panel.

c) The panels three sides simply supported with the vertical edge free, showed considerable variation in the results obtained. This is consistent with the large coefficient of variation for brickwork prisms tested spanning vertically.

d) Comparisons made using experimental results and the theoretical failure pressures, show that all the theories used tend to underestimate the strength of a wall.

e) Elastic theory gave good results when the brickwork was spanning with the greater strength in the direction of the smaller slab dimension, otherwise it underestimated the failure pressure.

f) The strip theory gave better results than those obtained by elastic theory, but still conservative in comparison with the experimental results.

g) The yield line theory, with all the reservations regarding its applicability, gave good agreement with the experimental results.

h) There is a considerable variation in the experimental moment coefficient calculated from the existing experimental work. This variation suggests that there are still some unknown factors affecting the strength of brickwork under lateral loading.

i) The variation in the experimental moment coefficient, from reported work, is most probably because of the different methods used for assessment of the modulus of rupture from small tests specimens. As there is no standard flexural test for brickwork, the dimensions of the specimen as well as the method of testing and the nature of loading seem to be responsible for the variation.

7.7 Recommendation for future research.

1. Investigation of the use of small test specimen to determine the material properties, their dimension, the method of testing and the relation between the strength of those specimens and the strength of the wall panels.
2. More work is needed to study the load distribution across the panel and its transferance to the supports.
3. Further tests on walls with returns and continuous walls of more than one span.

TABLE 7.1 SUMMARY OF FAILURE PRESSURES PANELS SUPPORTED ON THREE SIDES.

Wall No.	$^L/_H$	Failure Pressure N/mm x 10^{-3}		
		Experimental	Elastic	Yield Line
A_1	0.5	8.4	2.5	5.0
A_2		5.8	2.3	4.7
A_3		6.3	2.15	2.6
A_5		9.3	4.3	7.6
A_6		10	5.0	8.3
B_1	1.0	3.1	1.2	2.8
B_7		4.7	2.0	4.25
B_8		4.4	1.8	4.0
C_3	2.0	2.4	1.4	2.2
C_4		2.9	1.76	2.7
C_5		2.8	1.34	2.2

TABLE 7.2 SUMMARY OF FAILURE PRESSURES PANELS SUPPORTED ON FOUR SIDES.

Wall No.	L/H	Failure Pressure $N/mm^2 \times 10^{-3}$			
		Experimental	Elastic	Yield Line	Elastic modulus of rupture at 45°
G_1	2.0	18.2	9.0	18.0	
G_2		19.0	7.6	16.4	
G_3		18.0	6.9	14.3	
F_1	1.0	8.4	4.35	7.9	8.45
F_2		10.5	5.9	9.2	10.0
F_3		10.0	4.55	9.25	10.3
H_1	0.5	5.6	5.6	6.1	
H_2		7.0	5.9	6.3	

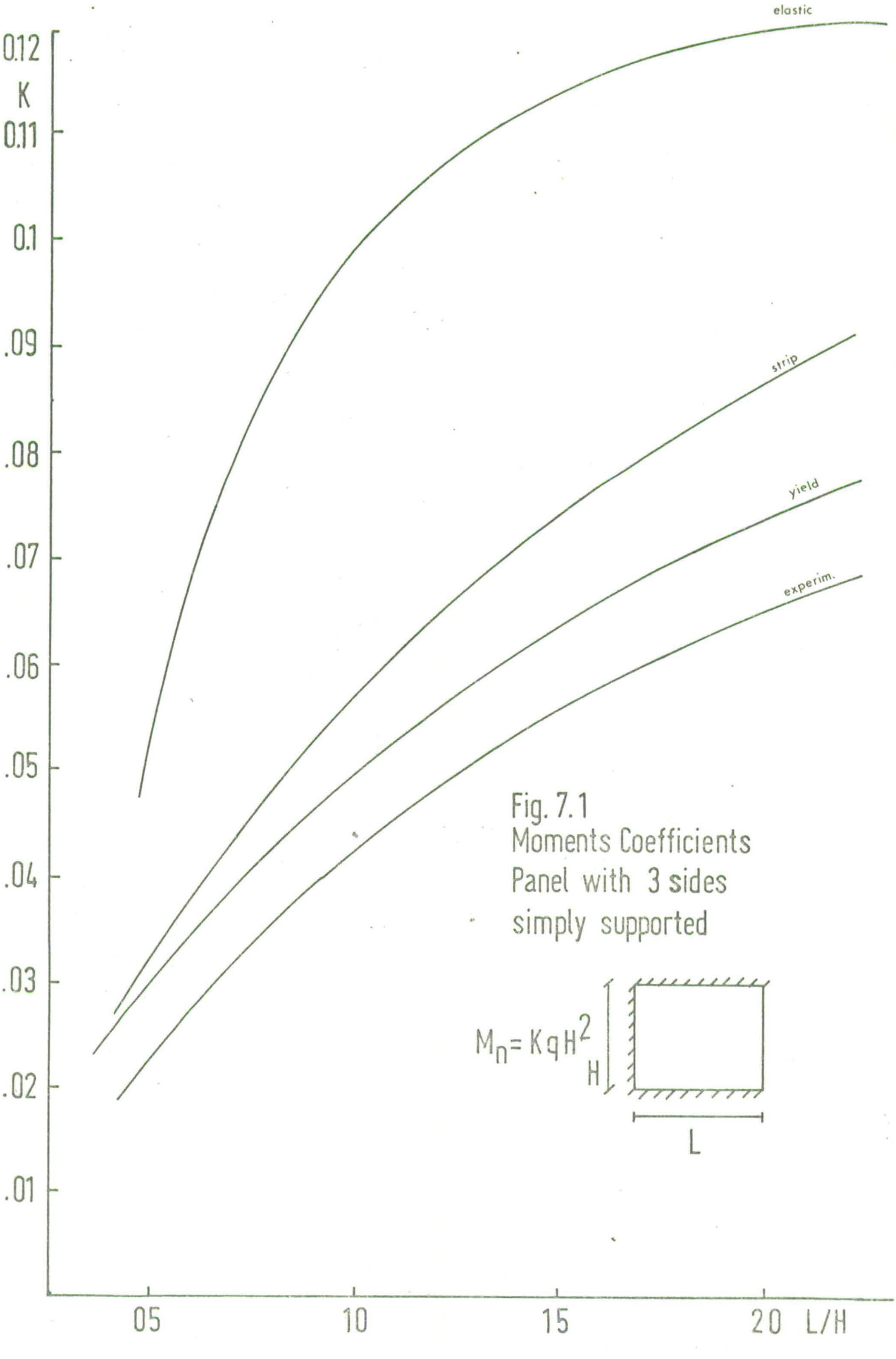


Fig. 7.1
Moments Coefficients
Panel with 3 sides
simply supported

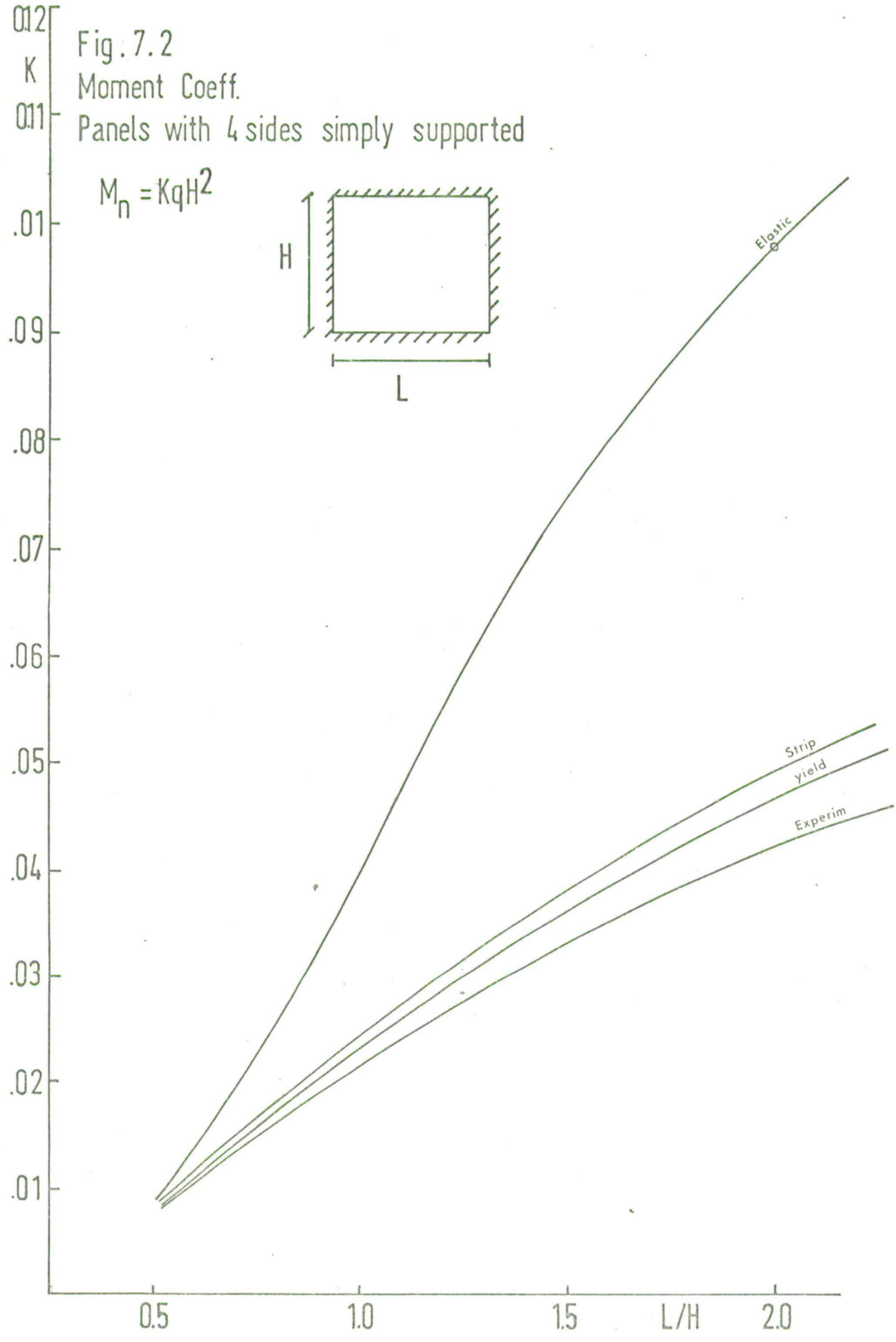


Fig. 7.2
Moment Coeff.
Panels with 4 sides simply supported
$M_n = KqH^2$
H
L
K
0.12
0.11
.01
.09
.08
.07
.06
.05
.04
.03
.02
.01
0.5
1.0
1.5
L/H
2.0
Elastic
Strip
yield
Experim

Fig 7.3
Comparison between theoretical and experimental deflection - wall F_1

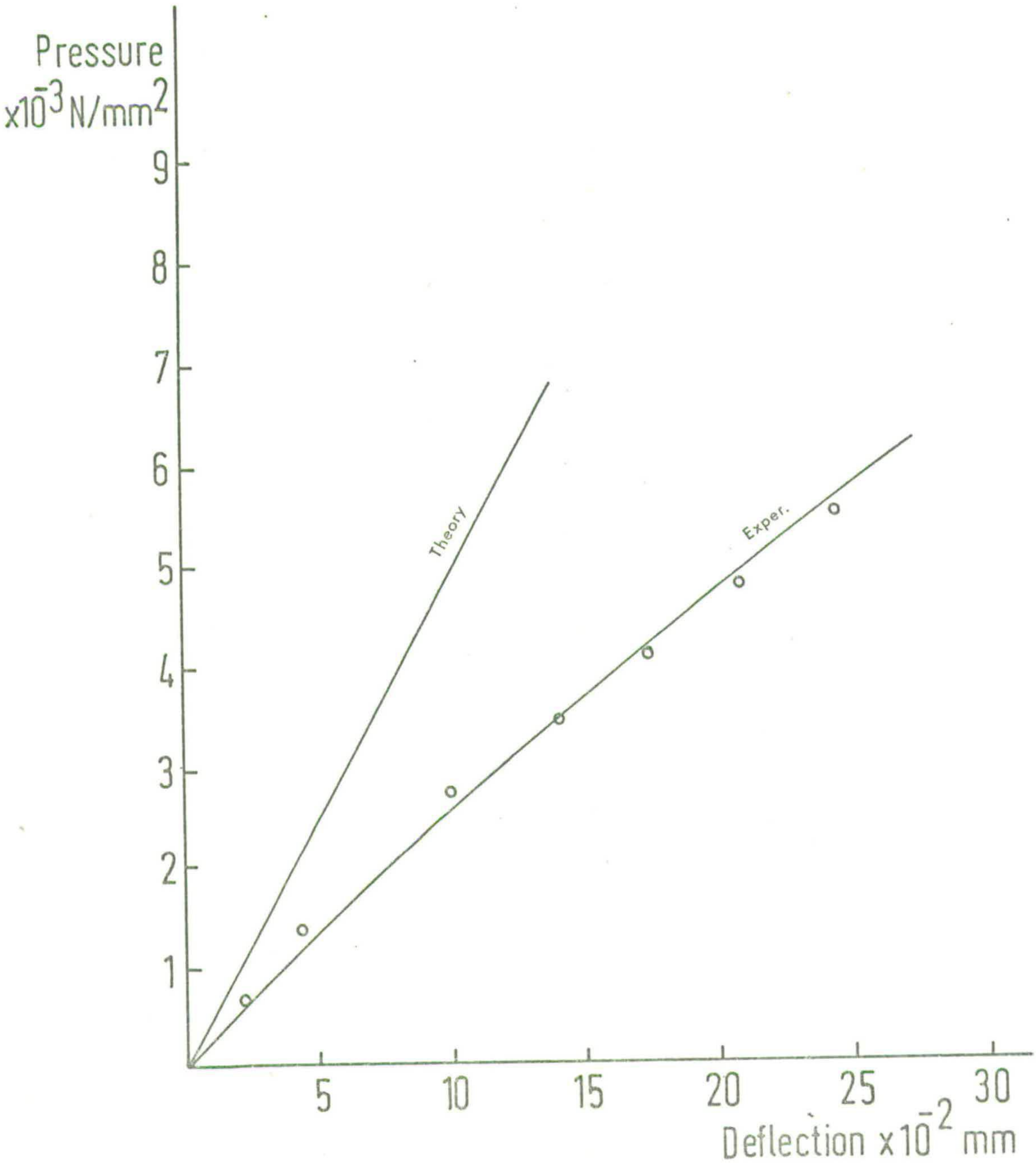
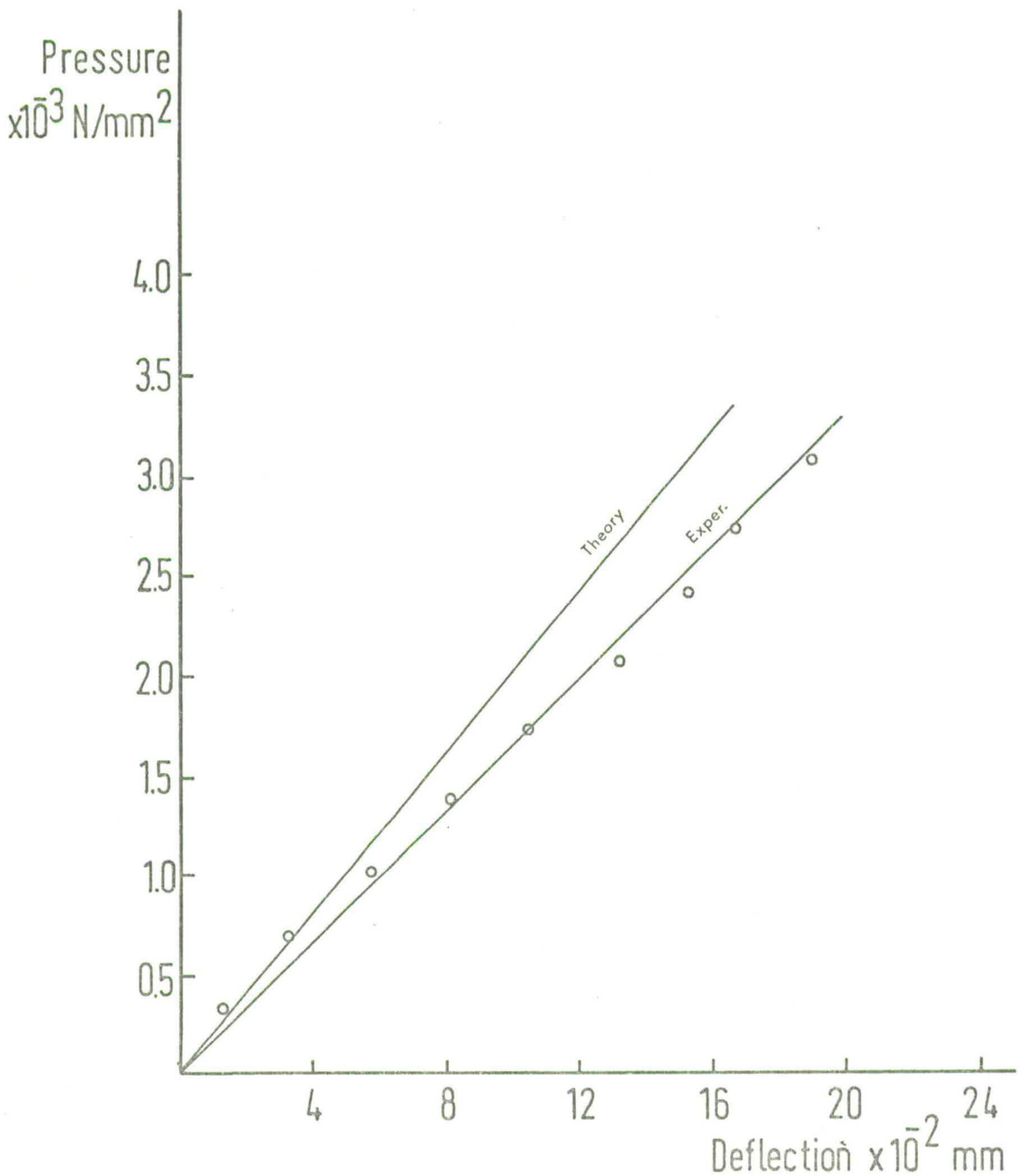


Fig. 7.4
Comparison between theoretical and experimental deflection - wall H_2



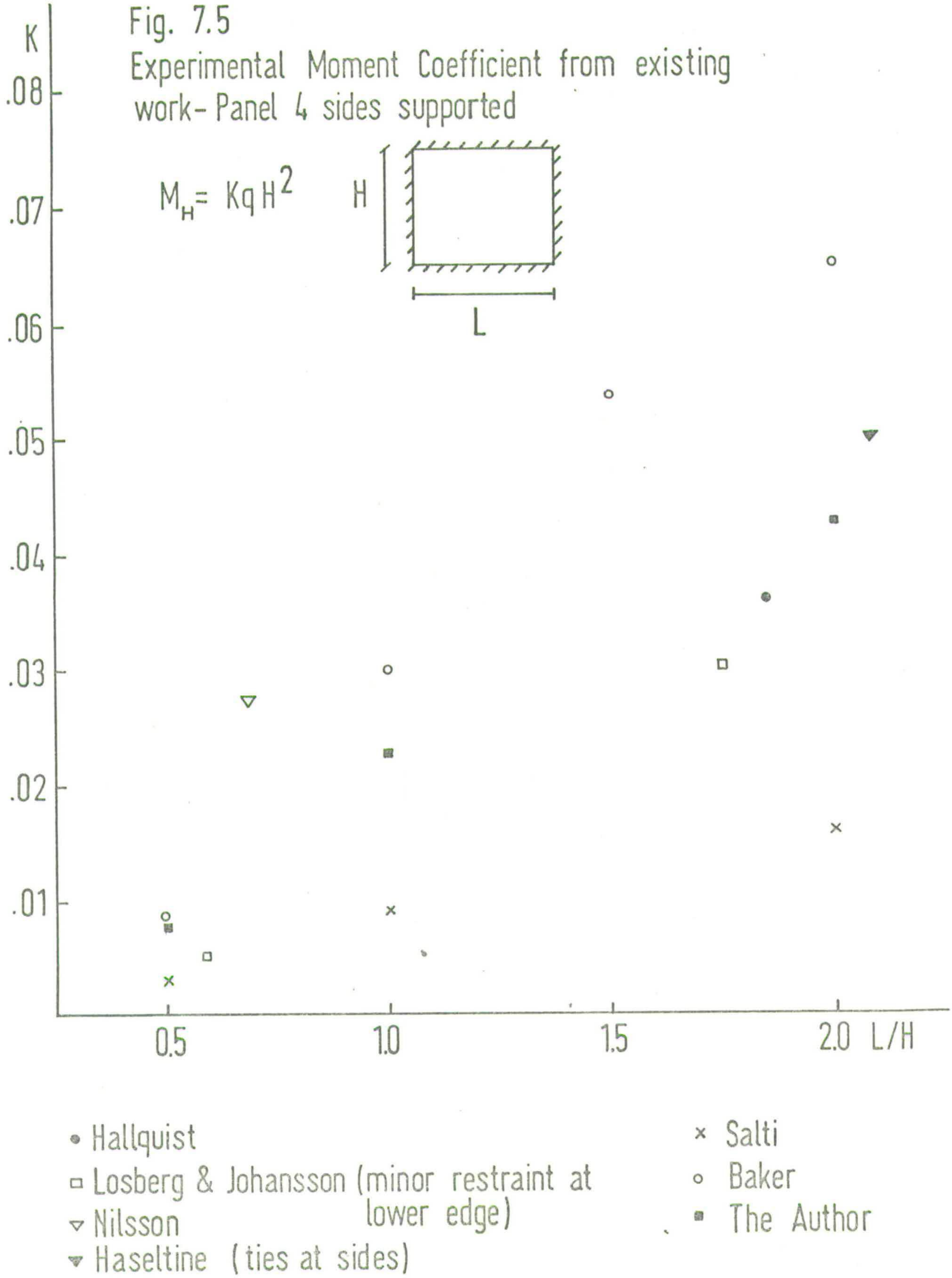
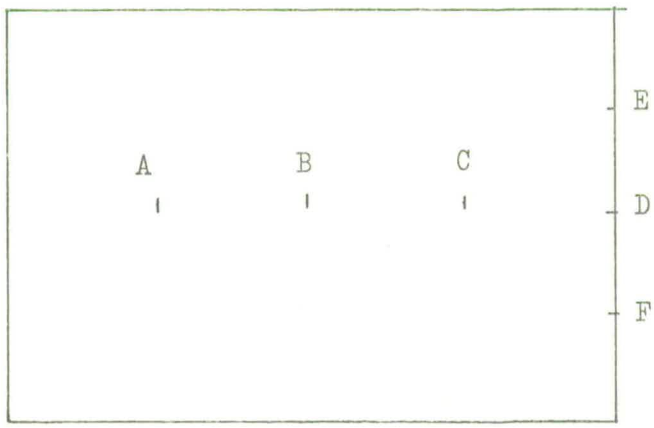


Fig. 7.5
Experimental Moment Coefficient from existing work- Panel 4 sides supported

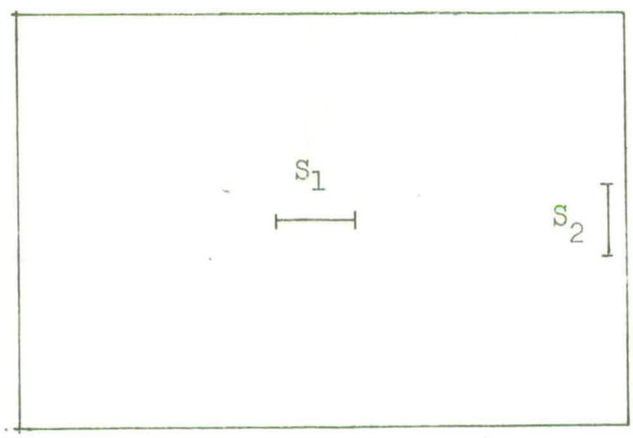
APPENDIX A.

(1) Walls simply supported on three sides.

Deflections and strains were usually measured at quarter points of the span with the following notation.



Dial guages positions



Strain guages positions

WALL A_1 WALL A_2

Pressure N/mm2x10^{-3}	Deflection mm x 10^{-2}		Pressure N/mm^{2}x10^{-3}	Deflection mm x 10^{-2}	
	B	D		B	D
0	0	0	0	0	0
0.63	2.54	2.54	0.69	2	3.5
1.2	4.6	5.0	1.38	4.5	7.5
1.8	6.1	8.3	2.07	9	13.5
2.4	8.9	12.7	2.76	12	18.5
3.0	10.6	15.2	3.45	16	23.5
3.8	12.8	18.5	4.14	21.5	30
4.4	14.8	23	4.83	27.5	37.5
5.0	16.8	25.4	5.52	34.5	47.5
6.2	19.4	34	6.2	42	56.0
8.3	3.0	46			

Pressure $N/mm \times 10^{-3}$	A_3. Deflection mm x 10^{-2}		A_6 Deflection mm x 10^{-2}		Strain	
	B	D	B	D	S_1	S_2
0	0	0	0	0	0	0
0.69	0.69	3.1	1.2	3	1.9	9.5
1.38	4.0	4.8	3.8	7	2.45	18.9
2.07	6.0	6.6	5.5	10.5	3.0	27
2.76	7.3	8.0	7.0	13	5.7	33.3
3.45	10.1	11.4	10.2	17.4	6.4	42
4.14	12	15.5	13.5	21.5	9.0	47.2
4.83			17	25.5	17	40.7
5.52			22.5	30.5	20	

WALL A_5

Pressure $N/mm^2 \times 10^{-3}$	Deflection mm $\times 10^{-2}$			
	B	D	E	F
0	0	0	0	0
0.69	1.5	3	2	2
1.38	2.5	5.8	3.5	3.6
2.07	4.3	9	5.8	5.5
2.76	7.0	11.7	8.5	8
3.45	8.25	15	10.5	10
4.14	11	17.5	14	13.8
4.83	13.8	20	17.5	15.5
5.52	16	25.5	22	20.5

	Wall B_1 Deflection		Wall B_8 Deflection		Wall B_8 strain x 10^{-6} S_2
	B	D	B	D	
0	0	0	0	0	0
0.35	1.25	2.5	1.5	2.5	2.6
0.69	3.0	5.5	3.8	5.5	5.85
1.04	5.0	8.7	6.0	8.1	7.75
1.38	7.2	11.3	9.2	13.8	12.4
1.73	8.5	13	11.5	18.1	
2.07	11.0	18	12.5	22.5	18.2
242	14.5	22.5	16.2	26.5	
267	16.0	28.8	19	30	262
3.10			22	32.5	
4.83					34.2

WALL B_7

	A	B	C	D	E	F
0	0	0	0	0	0	0
0.69	1.25	2.7	4.0	4.0	3.0	3.0
1.38	3.0	6.0	7.8	8.9	6.5	6.6
207	4.5	9.8	13	14.8	10	10.5
276	6.0	13.3	16.3	18.8	13.2	13.5
4.14	9.8	21	28.5	31.5	22	21
4.83	12	25	35	-	27.5	28

Pressure N/mm^2 x 10^{-3}	Wall C_3 B	Wall C_3 D	Wall C_5 B	Wall C_5 D
0	0	0	0	0
0.35	3.5	5.0	3.3	5.0
0.69	6.3	6.3	5.3	10.8
1.04	10	11.7	10	13.3
1.38	17	20	11.8	15.5
1.73	21.7	28.3	14.5	16.8
2.07	26.8	37	19	23.3
2.42			23.8	30.0

WALL C_4

Pressure $N/mm^2 \times 10^{-3}$	Deflection mm x 10^{-2}			
	B	D	E	F
0	0	0	0	0
0.35	3.0	3.3	2.8	2.8
0.69	6.0	7.3	4.9	5.7
1.04	10.5	13.3	8.0	9.0
1.38	13.4	17	9.5	12
1.73	18.3	21	14.8	16
2.07	23	28.4	19	21
2.42	26.7	33	22.8	24

	Wall G_1	Wall G_2		
Pressure $N/mm^2 \times 10^{-3}$	def mm x 10^2	def. mm x 10^{-2}	strain x 10^{-6}	
	B	B	S_3	S_5
0	0	0	0	0
1.38	1.1	2.3	10	7.1
2.76	3.0	4.2	21.5	15
4.14	4.0	5.5	27.5	25.5
5.52	4.5	7.1	30	34
6.9	6.1	9.0	32.5	49
8.3	7.5	11.9	33	56
9.66	9.5	13	36.5	63
11.0	10.6	15.2	43	79
12.4	12.2			
13.8	13			
15.2	14.2			
16.6	15.4			

WALL G_3

Pressure $N/mm^2 \times 10^{-3}$	Deflection mm x 10^{-2}		
	A	B	C
0	0	0	0
1.38	0.9	1.1	0.8
2.76	1.9	2.6	1.8
4.14	2.7	3.8	2.8
5.52	3.6	5.0	3.7
6.9	4.4	5.9	4.5
8.3	5.0	6.8	5.3
9.66	5.8	7.7	6.1
11.0	6.6	9.1	6.8

WALL F_1

$N/mm^2 \times 10^{-3}$	Deflection mm x 10^{-2}		
	A	B	C
0	0	0	0
0.69	1.5	2.3	1.5
1.38	3.5	4.5	3.8
2.07	5.0	8	5.5
2.76	7.5	10	8.3
3.45	9.5	14	10.8
4.14	11.5	17.5	12.5
4.83	14	21	15.5
5.52	15.8	24.5	17.5
6.21	17.5	27.5	20

WALL F_1

Pressure $N/mm^2 \times 10^{-3}$	Strain $\times 10^{-6}$		
	S_1	S_2	S_3
0	0	0	0
0.69	6.7	9	7.3
1.38	10.7	19.8	12.2
2.07	26.3	40	29
2.76	39.2	55	41
3.45	48.6	67.2	52.5
4.14	59	81.6	64.2
4.83	69.6	96	76.8
5.52	82.8	111.6	89
6.21	93	127.8	101

WALL F_2

Pressure $N/mm^2 \times 10^{-3}$	Deflection mm x 10^{-2}				
	A	B	C	D	E
0	0	0	0	0	0
0.69	1.02	1.52	1.52	1.26	1.02
1.38	3.05	3.55	3.55	3.05	2.8
2.07	5.1	7.6	6.6	6.1	5.1
2.76	7.6	10.2	9.4	8.9	7.1
3.45	10.2	14.2	12	12	9.4
4.14	12.4	17	14.5	13.8	11.6
4.83	15	20	17.8	16.8	14.2
5.52	16.8	24	20.3	19	16
6.21	19.6	26.7	23	23	19
6.9	22.0	31.2	26	25.1	21

WALL F_3

Pressure N/mm x 10^{-3}	Deflection mm x 10^{-2}				
	A	B	C	D	E
0	0	0	0	0	0
0.69	1.3	1.7	1.5	1.5	1.5
1.38	7.0	4.2	7.5	3	3
2.07	5.5	6.5	5.7	5	5
2.76	7.2	8.5	7.5	7.0	6.8
3.45	9.5	11	9.5	8.7	8.5
4.14	11	13.5	11.2	11.5	11.3
483	12.8	16.5	13	12.5	12.4
5.52	15.2	18.5	15.5	14.5	14
6.21	17.5	21	17.5	16	15.7
6.9	19.3	23	19.3	18	17.8
7.6	20.8	25.3	20.8	19.5	19

F_3

Pressure $N/mm^2 \times 10^{-3}$	Strain				
	S_1	S_2	S_3	S_5	S_6
0	0	0	0	0	0
0.69	4.5	8.6	3.8	4.75	5.2
1.38	5.5	14.6	7.3	11	13.2
2.07	8.4	21.2	10.8	18.5	20
2.76	10.8	2.6	14.4	26.5	27.5
3.45	15.6	33.5	19.6	37.5	37
4.14	24.2	42	26.5	42	37.5
4.83	39	49	37.5	57.5	40

WALL H_1

Pressure $N/mm^2 \times 10^{-3}$	Deflection mm x 10^{-2}		
	A	B	C
0	0	0	0
0.35	1.0	1.4	1.4
0.69	3.0	3.5	3.3
1.04	4.7	6.0	5.0
1.38	6.8	8.6	6.9
1.73	8.0	11	7.9
2.07	9.2	12.8	9.5
2.42	11	15.3	11.2
2.76	13	17.6	13.4
3.1	15	20.6	15.4
3.45	17.3	24	17.5
3.8	19	25.5	20.2

WALL H_2

Pressure N/mm x 10^{-3}	Deflection mm x 10^{-2}	
	A	B
0	0	0
0.35	0.9	1.3
0.69	2.4	3.3
1.04	3.7	5.8
1.38	5.5	8.1
1.73	6.8	10.4
2.07	8.5	13.2
2.42	10	15.2
2.76	11.2	16.6
3.1	13	18.8

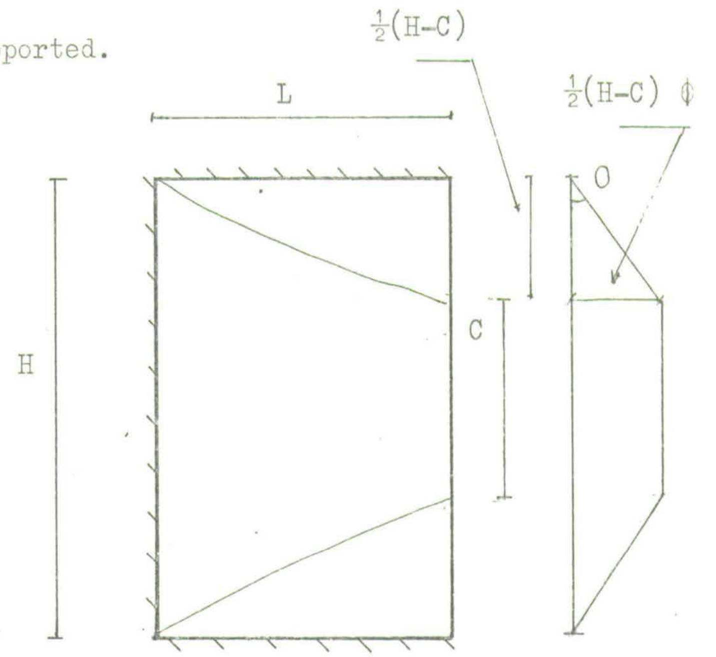
WALL H_2

Pressure N/mm x 10^{-3}	Strains x 10^{-6}				
	S_1	S_2	S_3	S_5	S_6
0	0	0	0	0	0
0.35	3.5	7.2	5	0.72	1.88
0.69	7.2	14.7	11.8	1.4	2.6
1.04	12.3	24	16.7	2.7	4.5
1.38	16.5	31.3	22.5	3.75	7.2
1.73	22.4	38.5	27	4.8	9.8
2.07	27.3	45.5	35	5.1	14.6
2.42	35	52	38.2	5.5	20
2.76	42	59	45.5	6.2	20.5
3.1	49	69	53	8.0	-

APPENDIX B.

(1) Panels 3 sides simply supported.

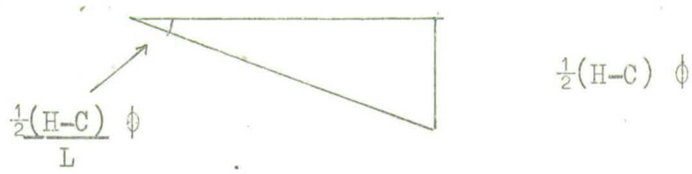
First Mode of Failure



W = Work done by external loads

U = Internal Energy

$\mu = \frac{Mp}{Mn} \geqslant 1$



$$U = 2M_n \phi L + \mu M_n \frac{H-c}{2L} (H-c)\phi$$

$$= M_n \phi\left[2L + \mu \frac{(H-C)}{2L} (H-C)\right]$$

$$W = q \phi \left(C\frac{(H-C)}{2} \frac{L}{2} + 2 \frac{(H-C)}{2} \frac{L}{3} \frac{(H-C)}{2}\right)$$

$$U = W$$

$$M_n \left(2L + \mu \frac{(H-C)^2}{2L}\right) = q \left[(H-C) \frac{CL}{4} + \frac{L}{6} (H-C)^2\right]$$

Putting $x = \frac{L}{H}$ $\gamma = \frac{H-C}{H}$ $\therefore \frac{C}{H} = 1-\gamma$

Multiplying by $\frac{1}{H}$

$$M_n \left(2\alpha + \frac{\mu\gamma^2}{2\alpha}\right) = q\frac{H^2\alpha^2}{6} \left[3\gamma (1-\gamma) + 2\gamma^2\right]$$

$$q = \frac{6M}{b^2} \frac{4\alpha^2 + \mu\gamma^2}{3\gamma - \gamma^2}$$

$$= \frac{6M}{b^2} \Phi(\gamma) \qquad \Phi(\gamma) = \frac{4\alpha^2 - \mu\gamma^2}{3\gamma - \gamma^2}$$

for $\Phi(\gamma)$ minimum $\frac{\partial \Phi}{\partial \gamma} = 0$

$$(3\gamma - \gamma^2)\, 2\mu\gamma - (4\alpha^2 + \mu\gamma^2)(3-2\gamma) = 0$$

$$3\mu\gamma^2 + 8\alpha^2\gamma - 12\alpha^2 = 0$$

$$\gamma = \frac{4\alpha^2}{3\mu}\left(\sqrt{1 + \frac{9\mu}{4\alpha}} - 1\right)$$

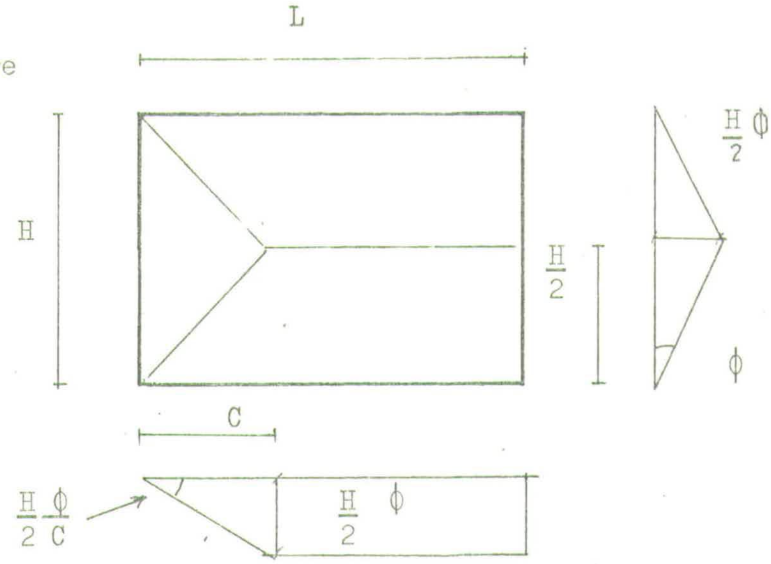
$$\Phi_{mm} = \frac{8\alpha^2}{3\gamma}$$

$$= q = \frac{6Mn}{L^2} \qquad \frac{8\alpha^2}{3\gamma}$$

$$M_n = \frac{qH^2\gamma}{16} \qquad \gamma = \frac{4\alpha^2}{3u}\left(\sqrt{1 + \frac{9\mu}{4\alpha}} - 1\right)$$

valid for $\gamma \leqslant 1$ ie $\alpha \leq \frac{3}{2} = 0.86$

(2) Second Mode of Failure



$\mu = \frac{Mp}{Mn} \geqslant 1$

$$W = q\,\phi\left[(L-C)\,\frac{H}{2}\,\frac{H}{2} + \frac{1}{3}\;H.C.\,\frac{H}{2}\right]$$

$$= \frac{q\,\phi\,H^2}{12}\left[\,3L - C\right]$$

$$U = 2M_n\,L\,\phi + \mu M_n\,\frac{H.H.\phi}{2C}$$

$$W = U$$

$$\frac{qH^2}{12}\left[3L-C\right] = \tfrac{1}{2}\left[4M_n\,L + \frac{H^2}{C}\,\mu\,M_n\right]$$

$$q\,(3L-C) = \frac{6}{H^2}\left[4M_nL + \frac{H^2}{C}\,\mu\,M_n\right]$$

$$= \frac{6}{H^2}\,M_n\left[4L + \frac{H^2}{C}\,\mu\right]$$

$$\therefore\; q = \frac{6M_n}{H^2}\left[\frac{4L + \frac{H^2}{C}\,\mu}{3L - C}\right]$$

Putting $\alpha = \frac{L}{H}$ $\gamma = \frac{C}{H}$

$q = \frac{6M_n}{H^2}\;\frac{4\,\alpha\,\gamma + U}{3\alpha\,\gamma - \gamma^2}$ denoting Φ as $\frac{4\,\alpha\,\gamma + \mu}{3\,\alpha\,\gamma - \gamma^2}$

for Φ minimum $\frac{\partial\Phi}{\partial\gamma} = 0$

$$(3\,\alpha\,\gamma - \gamma^2)\,4\alpha - (4\,\alpha\,\gamma + \mu)\,(3\alpha - 2\gamma) = 0$$

$$4\,\alpha\,\gamma^2 + 2\,\mu\,\gamma - 3\,\mu\,\alpha = 0$$

$$\gamma = \frac{\sqrt{\mu^2 + 12\,\alpha^2\mu} - u}{4\,\alpha}$$

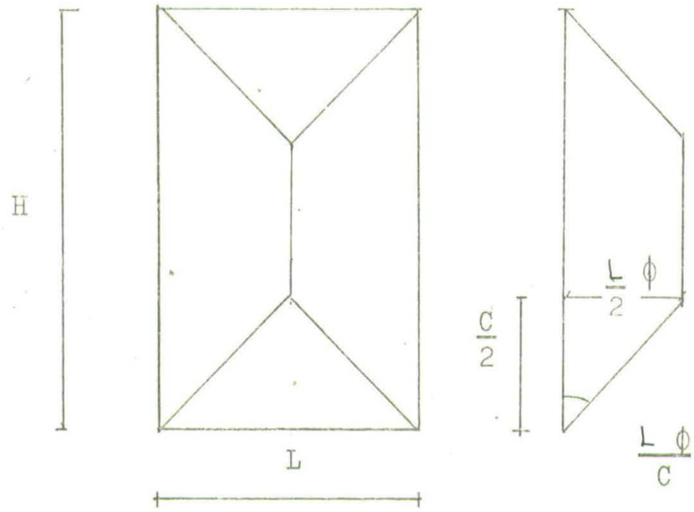
$$\Phi_{mm} = \frac{\mu}{\gamma^2}$$

$q_{min} = \frac{6M_n}{H^2}\;\frac{\mu}{\gamma^2}$ where $\gamma = \frac{\sqrt{\mu^2 + 12\,\alpha^2\,\mu} - \mu}{4\,\alpha}$

(3) Panels 4 Sides Simply Supported.

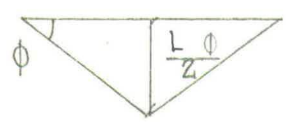
$Mp = \mu \ Mn$

M_n



$\mu = \frac{Mp}{Mn}$

$W = q \ \phi \ (\frac{1}{3} \ L \ C \ \frac{L}{2} + \frac{1}{2} \ L \ \frac{L}{2} \ (H{-}C)$



$U = 2M_n \ \phi \ (\ \mu H + \frac{L^2}{C)}$

$$q = \frac{24Mn}{L^2} \quad \frac{(\mu H + \frac{L^2}{C})}{3H - C)}$$

Putting $\alpha = {}^{H}/_{L} \qquad \gamma = \frac{C}{L}$

$$q = \frac{24 \ Mn}{L^2} \qquad \frac{\mu \ \alpha \ \gamma + 1}{3 \ \alpha \ \gamma - \gamma^2}$$

$$= 24 \ \frac{Mn}{L2} \qquad \Phi \ (\gamma)$$

for Φ (γ) minimum $\quad \frac{\partial \ \Phi}{\partial \gamma} = 0$

$\therefore \ \mu \ \alpha \ (3 \ \alpha \ \gamma - \gamma^2) - (\mu \ \alpha \ \gamma + 1) \ (3 \ \alpha - 2 \ \gamma) = 0$

$\mu \ \alpha \ \gamma^2 + 2 \ \gamma - 3\alpha = 0$

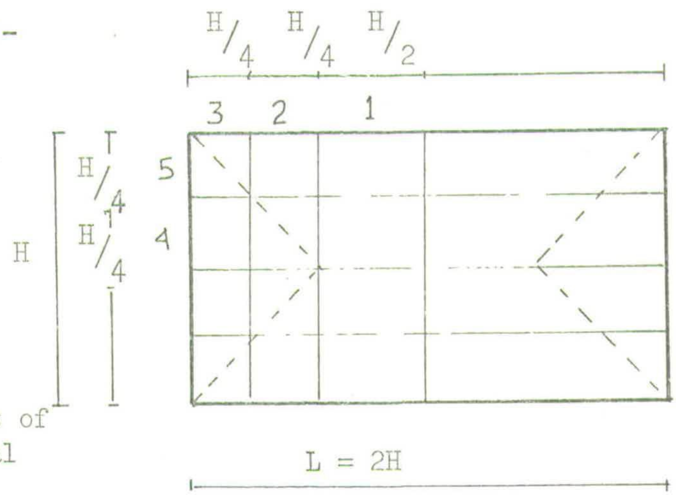
$0 \ \text{min} = \frac{1}{\gamma^2} \qquad \gamma = \frac{(\sqrt{1 + 3 \ \mu \ \alpha^2} - 1)}{\mu \ \alpha}$

$$q = \frac{24 \ Mn}{L^2} \quad \frac{1}{\gamma^2}$$

APPENDIX C.

Strip Method - numerical example:-

Slab 4 sides simply supported:-

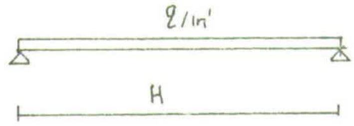


$L/H = 2$

K = moment Coeff.

μ = average ratio of moment of resistance in orthogonal directions.

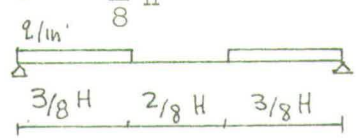
Strip 1 average loaded length = H



$$\text{moment} = 0.125\ q\ H^2$$

$$\text{moment volume} = 2 \times \frac{H}{2} \times H \times 0.125\ q\ H^2 \qquad = 0.1250\ qH^4$$

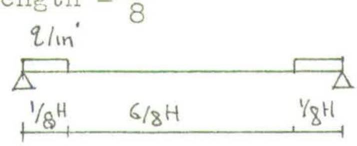
Strip 2 average loaded length = $\frac{3}{8}$ H



$$\text{moment} = \tfrac{1}{2}\left(\tfrac{3}{8}\right)^2 H^2 \times q$$

$$\text{volume} = 2 \times \frac{H}{4} \times H \times \tfrac{1}{2} \left(\tfrac{3}{8}\right)^2 q\ H^2 \qquad = 0.0350\ qH^4$$

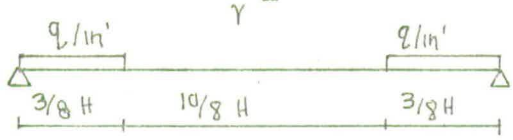
Strip 3 average loaded length = $\frac{1}{8}$ H



$$\text{moment} = \tfrac{1}{2} \left(\tfrac{1}{8}\right)^2 qH^2$$

$$\text{volume} = 2 \times \frac{H}{4} \times H \times \tfrac{1}{2} \times \left(\tfrac{1}{8}\right)^2 q\ H^2 \qquad = 0.0039\ qH^4$$

Strip 4 average loaded length = $\frac{3}{\gamma}$ H

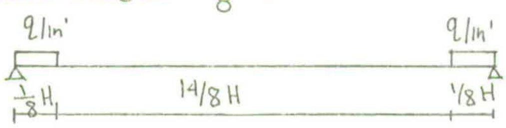


moment $= \frac{1}{2} \left(\frac{3}{8}\right)^2 q\ H^2$

$\mu = 3.08$

Volume $= \frac{1}{3.08} \times 2 \times \frac{H}{4} \times 2H \times \frac{1}{2} \left(\frac{3}{8}\right)^2 q\ H^2 \quad = \quad 0.0228\ qH^4$

Strip 5 average loaded length = $\frac{1}{8}$ H



moment $= \frac{1}{2} \left(\frac{1}{8}\right)^2 q\ H^2$

U = 3.08

Volume $= \frac{1}{3.08} \times 2 \times \frac{H}{4} \times H \times 2 \times \frac{1}{2} \left(\frac{1}{8}\right)^2 q\ H^2 \quad = \quad 0.0012\ qH^4$

$0.1879\ qH^4$

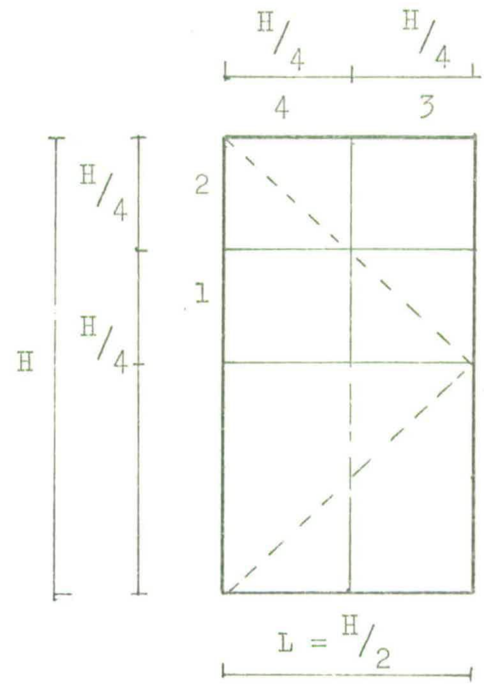
$K_H = \frac{0.1879 qH^4}{4H^2\ q\ H^2} = \frac{0.1879}{4} = 0.047$

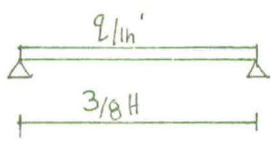
Panels 3 sides simply supported:-

$L/H = 0.5$

K = moment coeff.

μ = average ratio of moment of resistance in orthogonal directions.



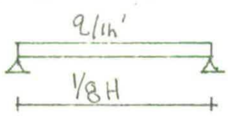


Strip 1

$$\text{moment} = 0.125 \left(\frac{3}{8}\right)^2 q\, H^2$$

$u = 3.45$

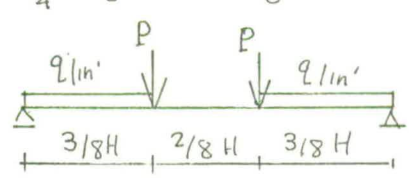
$$\text{volume} = \frac{1}{3.45} \times 2\,\frac{H}{4} \times \frac{H}{2} \times 0.125 \left(\frac{3}{8}\right)^2 q\, H^2 \quad = \quad 0.0009\, qH^4$$



Strip 2

$\mu = 3.45$

$$\text{volume} = \frac{1}{3.45} \times 2 \times \frac{H}{4} \times \frac{H}{2} \times 0.125 \left(\frac{1}{8}\right)^2 q\, H^2 \quad = \quad 0.0001\, qH^4$$

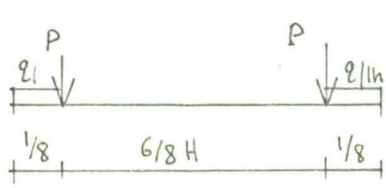


Strip 3

$$P = \tfrac{1}{4} \times \frac{3}{8}\, q\, H = \frac{3}{32}\, qH$$

$$M = \tfrac{1}{2} \left(\frac{3}{8}\right)^2 q\, H^2 + \frac{3}{8}\, PH$$

$$\text{Volume} = \frac{H}{4} \times H \times \left[\tfrac{1}{2} \left(\frac{3}{8}\right)^2 q\, H^2 + \frac{3}{8} \times \frac{3}{32}\, qH^2\right] \quad = \quad 0.0264\, qH^4$$



Strip 4

$$P = \frac{qH}{32}$$

$$M = \tfrac{1}{2} \left(\frac{1}{8}\right)^2 q\, H^2 + P \times \frac{1}{8}\, H$$

$$\text{Volume} = \frac{H}{4} \times H \left[\tfrac{1}{2}\left(\frac{1}{8}\right)^2 qH^2 + \frac{32}{8} \quad q\,H^2\right] \qquad = \qquad 0.0021\ qH^4$$

$$\underline{\qquad\qquad\qquad}$$

$$0.0295\ qH^4$$

$$\alpha_H = \frac{0.0295\ qH^4}{2H \times \frac{1}{2} H \times qH^2} = 0.0295$$

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