S U M M A R Y

• The object of the thesis was to investigate mortar properties influencing brickwork strength.

• The introductory chapter (one) is an examination of some examples of previous investigations into the strength of brick masonry under uni-axial compression. On the basis of the discussion, the main questions which emerged at the beginning of the project to be of importance are listed.

• The second chapter deals with a hypothesis for the criteria of splitting of brick masonry. Two theoretical models are developed and analysed, and the main parameters are discussed.

• In the third chapter an analytical investigation is attempted of the action and interaction between bricks and mortar. Expressions for the modulus of elasticity, critical cracking load and ultimate failure load are derived in terms of the properties of the individual components, and the main parameters are discussed.

• The fourth chapter evaluates both previous analyses (in Chapters 2 and 3) on the basis of their conclusions. Areas of action are discussed, and those of the first priority are determined as the stress-deformation properties of mortars.

• Chapter five begins with general remarks on existing methods for determining the deformation properties of some other materials. Then it introduces a new test which appears to be reliable and can facilitate the stress analysis of the specimens. The analysis is developed for two

basic cases of stress to yield new expressions for calculating the modulus of elasticity and Poisson's ratio, for which charts and tables are developed. Thirdly, the chapter shows how a new experimental technique can be developed with the benefits of some method of calculation, and simpler laboratory procedures, even with bricks which cannot be shaped in the laboratory.

- Chapter six represents mainly the results of an experimental investigation into the variation in compression failure characteristics of bricks due to a wide range of end joint conditions. The tests comprised, besides the main variables, differences in the final loading end conditions, in the directions of loading, and in the conditions of the brick surfaces. These tests were carried out with the object of giving answers to the questions posed at the end of Chapter one.

- The theoretical analysis in Chapter five holds true on the premise that the specimen is perfectly elastic. Also the limits between the basic states of stress referred to later as plane stress (disc or square plate) and plane deformation (cylinder or square block), were vague and open to question. The proposed new technique using square specimens was, however, the principal aim at this stage. All these questions are cleared up in Chapter seven, by carrying out a series of tests on steel specimens.

- Chapter eight is divided into four sections. The first gives a description of the programme of experimental work and the test results carried out on five conventional mortars, two types of model brick, and two types of brick-mortar assemblage. In the second and third sections analyses were carried out on mortars and bricks, respectively, as individual materials. The fourth section discusses the compressive strength of brick masonry as a function of the properties of its components. A comparison between the actual test results and calculated values is made, and the validity of the theoretical approaches to the determination of the compressive strength of brick masonry is discussed for the two principal cases.

Chapter nine, the concluding chapter, presents a summary of the new information given in the present work, the principal conclusions, and some suggestions for future research.

AN INVESTIGATION OF

MORTAR PROPERTIES INFLUENCING BRICKWORK STRENGTH

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1 9 6 8

A C K N O W L E D G E M E N T S

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INTRODUCTION

The various functions that brick masonry performs have been well known for a very long time. Recently significant changes and impressive developments in the manner of using brick masonry have taken place. Very distinct features of these changes are, that brick masonry is no longer thought of merely as enclosing space or providing shelter from weather, or of serving as bearing elements for small loads. On the contrary, a number of very high buildings in load-bearing brickwork or blockwork have been constructed. Examples have been described by Foster[44,45], Haller[53,56] and Hendry[60]. Such developments have made strength a requirement of first priority, especially when considering the economical use of brickwork in terms of the area of support. This produced the need for a more careful investigation of the strength properties of brickwork subjected to compression.

Looking back bricks have always been regarded as structural masonry units, with mortar as a material which is used to bind the bricks and in doing so acts as an adhesive and sealant. In correlating the strength of brickwork and the strength properties of the bricks and mortar a purely empirical relation based on experiments was deduced. The strength of the components was based entirely on tests in compression. Correlations for mortar may be conditioned or influenced by certain values of brick strength, and vice versa. On the other hand, the assemblage comprising the bricks and mortar has usually been treated, from the structural point of view, as an element of a single phase material. This treatment has been common in both design and research work. The result has been that, all the stresses and strains either calculated or measured are only the averages over certain areas or lengths of the assemblage containing bricks and mortar. This is usually justified with homogeneous materials, which is not the case with brick masonry.

These previous ways of approaching the problem of the strength of brick masonry under uniaxial compression did not appear to the present author either profitable or hopeful. Neglecting the heterogeneity was considered the most serious drawback responsible for the non-appearance of a failure criterion, and generally inexplicable observations.

Hence the general aim of the present work was to study the deformation properties and failure characteristics of a two-phase material having the structure of brick masonry. In other words, from now on, mortar will not be looked upon merely as a binding material which does not participate structurally in the assemblage.

It is likely, when looking on the assemblage as a two-phase material, that it would take a long time before reaching a perfect and complete understanding. But recognizing the heterogeneity of the assemblage and the fundamental properties of its constituents, even as individual homogeneous materials in the early stages, will certainly open the way for brickwork to be used more effectively as a load-bearing materi

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continued:

(44): Foster, D. Contemporary structural uses of burnt clays. The British Ceramic Research Association, Spec. Publ. 38: The uses of ceramic products in building, 1963. pp. 27-57.

(45): Foster, D. The use of structural brickwork for frameless high buildings. The Architectural Review, April, May 1962.

(53); Haller, P. Masonry in engineered construction. National Research Council of Canada. Tech. Translation 1270 [translated from Schweizerische Bauzeitung, 83(7). 103-107] Ottawa, 1967.

(56): Haller, P. The technological properties of brick masonry in high buildings. National Research Council of Canada. Tech. Translation 792 (translated from SCHWEIZ. BAUZ. 76-28, 411-419, 1958), Ottawa, 1959.

(60): Hendry, A.W. High rise load-bearing brickwork. The Architect Jl., 6 Sept. 1967. Vol.146: No. 10. pp. 611-619.

It is not intended by this to minimise the importance of non-heterogeneity in both bricks and mortar, or the validity of other functional requirements for brick masonry, it is rather a matter of determining a certain sphere of a project in rather a wide field of interacting problems.

However, in spite of the fact that this is the first time such a study has been carried out, it should be mentioned in advance, that the whole work encountered in the thesis cannot be claimed to be more than an adequate foundation for the extensive research required in this field. The author feels that further effort is essential for putting structural ceramics on the same footing as other structural materials.

C H A P T E R 1

EXAMPLES OF PREVIOUS WORK AND REMARKS ON THE STRENGTH OF BRICK MASONRY UNDER AXIAL COMPRESSION

1.1 ORIENTATION

Strength properties of a wall may be to resist one or more of the following:

1. Compressive forces
2. Transverse forces
3. Shearing forces

Following the lines along which the present investigation had been orientated, the review presented here was taken to be concerned with the failure characteristics of brick masonry under compression, with special reference to ultimate capacity.

If we consider buildings the real conditions are complex, and walls in compression can, clearly be restrained by adjacent members, and these restraints will tend to increase strength. The present work is not going to deal with such interactions. Reference can be made here to the work done under the supervision of Professor A.W. Hendry, at the Department of Building Science, University of Liverpool(61) and continuing now at the Structural Ceramics Research Unit, Department of Civil Engineering and Building Science, University of Edinburgh(62).

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(61): Hendry, A.W. Recent Research on load-bearing brickwork The Brit. Cer. Res. Association. Sp. Publ. 38, 1963, pp. 58-68.

(62): Hendry, A.W. Research in Structural Ceramics At Edinburgh University. Clay Craft and Structural Ceramics, Jan. 1965. pp. 142-144.

Therefore the review, in this chapter, will be confined to the behaviour under load of simple walls and piers or columns; the term column is used as defined in B.S.CP 111[20], i.e. isolated vertical load-bearing member. This limitation is necessarily a simplification of the most likely basic behaviour of load-bearing walls in the structure.

Again, if a wall or column is loaded uniformly without eccentricity, its load-bearing capacity is dependant, according to the traditional way of looking at a masonry assemblage, upon:

1. One or two of the mechanical properties of bricks.
2. One or two of the mechanical properties of mortar.
3. Size and shape of compression test specimen.
4. System of jointing or bonding.
5. Workmanship.
6. The slenderness of the member.

Counting the total possible factors inc included in the above groups that can affect the study, and a deeper insight into the extensive literature, make it necessary to consider mainly the influence of the mechanical properties of mortar and bricks in stout members.

However, the following is a brief review with assessments of the principal contributions in the sphere determined above. The contributors will be considered in chronological order and attention will be paid, mostly, to the considerations comprised in the philosophy of the present work. As will be shown, the majority of the references represent

(20): British Standard Institution. C.P. 111:1964 Structural recommendations for loadbearing walls.

rather sporadic experimental attempts in the United Kingdom and abroad to provide information on the problem.

1.2 SELECTED REVIEW

1884-1886: Howard[69]

The earliest work on the strength of brick masonry, so far as can be ascertained, appears to be that of Howard. It was later published by Bragg[13]. Howard carried out several series of pier tests, in which a study was made of various mortars, grades of brick, and methods of laying the bricks. A feature of his work was the laying bricks on edge, and in some cases breaking joints every third of sixth course, instead of every course. The main observations were:

1. The strength was found to vary with the height of the pier for both common and face bricks. Generally the strength decreases as the height increases (7.8-17.6% and 12.5-18.1% of the compressive strength of the bricks for common and face bricks respectively).
2. Laying the bricks on edge and breaking joints as mentioned above, increased the strength considerably.
3. As regards mortar, he used a narrow range of different mortars. The results, as analysed by Bragg, were not conclusive.

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(69): United States Report of tests of metals and other materials. 1884-1886. Engineering Record, March 22, 1913; Clay Working, March, 1913.

(13): Bragg, J.G. Compressive strength of large brick piers. Technologic Paper of the Bureau of Standards, Department of Commerce, September, 20, 1918, Washington.

1900: McCaustland(90)

This work of McCaustland was also reported by Bragg. His investigation was carried out on a series of piers of the same dimensions, the same bricks and mortar. The piers were reinforced laterally in the horizontal joints with steel plates, or wire mesh. It was shown that the efficiencies of the piers reinforced with iron straps and plates are less than those of the piers without reinforcement. The piers reinforced with mesh in every joint developed efficiencies of 46 per cent as compared with 30 per cent of those without reinforcement. However, there is a considerable drop in efficiency with the piers with mesh in every second joint whioh developed efficiencies of only 33 per cent.

1900-1916: Macgregor(91)

According to Bragg's review sets of piers of the same dimensions were tested. Common and hard-burned face bricks were used. The main variable was the mortar. In these tests piers laid in a mortar composed of 1 part (25 per cent lime and 75 per cent Portland cement) to 3 parts sand by volume developed the highest strength. Piers laid in mortar composed of 1 part (50 per cent lime and 50 per cent Portland cement) to three parts sand developed higher

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(90): McCaustland, E.J. Transactions of the Association of Civil Engineering of Cornell University for 1900. The data are recorded by Bragg as taken from Burr's Elasticity and Resistance of the materials of engineering, sixth edition, page 425.

(91): Tests made by Prof. Macgregor, J.S., Colombia University; Bulletin J, Hydrated Lime Bureau of the National Lime Manufacturing Association, U.S.A.

strengths than piers laid in 1 part cement to 3 parts sand mortar.

1916: Kreuger[86]

His tests were carried out on small piers and comprised a more comprehensive scope as regards the variables included. It was found that with bricks of various strengths laid in 1 part lime to 3 parts sand mortar, the piers developed strengths of from 18.5 to 26.5 per cent of the ultimate compressive strength of the bricks. A comparison by Bragg showed that Kreuger's results in Sweden were not comparable with the results of tests made in the United Sates, since the methods of testing the individual bricks are different. Kreuger's results were obtained from compression tests on halves of the same brick cemented together. It was explained that since the compression strength developed in this manner would be considerably lower than in the case of a single half brick tested flat, the efficiency of the pier would be correspondingly higher.

Tests of the mortars showed an increase in strength from 28 days to one year of 33 to 165 per cent, while the piers increased in strength in the same length of time only 6 to 17 per cent. The introduction of wire mesh in every joint increased the strength 88 to 100 per cent.

1918: Bragg[13]

With the object of investigating

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(86): Tests by Kreuger, H. at the Tech. High School in Stockholm. Tonind-Ztg. 40,1916; Clay Worker, July and August, 1916.

comprehensively the strength of brick masonry, he carried out his tests on large piers. Different bricks, mortars and grades of workmanship were used. Although the tests were carried out a long time ago, the present author believes that Bragg's results which covered and emphasised the previous ones are of interest. The conclusions were deduced from purely experimental tests, and since then, they have been inexplicable. In the light of the present work, his conclusion could be explained and justified from both theoretical and experimental aspects. The following is a summary of Bragg's main conclusions, and the peculiar ones will be discussed in the proper place.

1. The primary failure of brick piers is caused by a transverse failure of individual bricks.

2. The ultimate strength of the pier may be increased by any method of construction which will increase the depth of the component parts of the pier. This may be done by:

a. laying the bricks on edge instead of flat,

b. breaking joints every few courses instead of every course,

c. using bricks of more than ordinary thickness.

3. The strength of the pier may be increased by the introduction of wire mesh in all horizontal joints. The increase is slight, however, unless the mesh is used in every joint.

4. Varying the number of header courses used does not appreciably affect the ultimate strength of the pier.

5. The mortar joints should be made as thin as possible. They should be of uniform thickness. For this reason

regularity in shape of bricks is essential.

6. The ultimate strength of brick piers is proportional to the compressive and transverse strength of the bricks used in their construction. The transverse strength of the brick bears a close relation to the strength of the pier.

7. The kind of mortar used is important in its effect on the strength of brick masonry. A pure lime mortar is inefficient when a high compressive strength is desired. In a mortar of one part Portland cement to three parts sand, 25 per cent by volume of the cement may be replaced by hydrated lime without appreciably affecting the strength of brick piers. In other words the greatest advantage in the replacing of one part of the cement with hydrated lime was the easier working qualities of the cement-lime mortar.

8. Two empirical formulae,for use in computing the strength of brick masonry, were derived from the tests of this investigation. Both formulae represent the strength of the ultimate unit compressive strength of the pier as a constant, depending on the grade of the mortar, multiplied by the brick unit compressive strength when tested flat, on edge, or the brick modulus of rupture. A distinctive feature of the equations and constants given by Bragg (page 38 of his paper) is that the average increase in the constants for bricks tested on-edge compared with those tested flat is about 12 per cent. However, neither for the constants of these formulae, nor for that of modulus of rupture, is an explanation given.

1923: Whitemore and Hathcock[144]

In their tests as quoted by Swallow[133] on the strengths of hollow tile walls set in cement-lime-sand mortar, they found that walls in which the tiles were set with the axis of the holes vertical were much stronger than those with the tiles set with the holes horizontal.

1924: Ingberg[78]

His work was summarised in a memorandum prepared at the Building Research Station[14]. The properties of five kinds of building brick were discussed with reference to the resulting strength of masonry. Vertical cracks in the brick were taken to indicate the first sign of failure. In no case were well-defined planes of shear formed. The results brought out that there is a general increase in strength of masonry with inidividual strength of bricks. The degree to which the bond with the mortar is affected is an important factor. Bricks with rough surfaces developed higher relative strengths than smooth hard surface bricks. As the cement content of the mortar increases there is a consistent gain in strength. Hollow brick wall sections laid with brick on edge developed on the average approximately 80% of solid walls of comparable thickness.

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(144): Whitemore H.L. and Hathcock, B.D. U.S. Bur. Stand. Tech. Report No. 238, 1923.
(133): Swallow, H.T.S. Building mortars and bricks. The Brit. Cer. Res. Association. Heavy Clay Division Technical Note No. 57. pp.4-6.
(78): Ingberg, S.H. Factors affecting brick masonry strength Proc. A.S.T.M. Part (1924). No. 311, 1926.
(14): Brit. Cer. Res. Association. Memorandum on structural and load-bearing brickwork. (Confidential). April 1964.

1926: Stang and Others[129]

Their conclusions as summarised by Swallow were that the strength of solid walls was more closely related to the shear strengths of individual bricks than to any other individual property measured. Compressive strengths of half bricks tested flat were the next best measure. The compressive strength of wallettes was the best measure of the strength of brickwork, and the use of cement-sand mortar gave higher brickwork strengths than cement-lime-sand mortar, and much higher than lime-sand mortar. For solid walls strength varied approximately as the compressive strength of 2" diam. x 4" long cylinders of mortar cured on the walls. When discussing workmanship, it was mentioned that keeping the wall damp for seven days to cure the mortar had very little effect on the strength.

1929: J. Franklin Institution[79]

As stated in the B.C.R.Association memorandum, it was found that the average strengths of solid walls built with bricks having a compressive strength of 3,280 lb/in^2 were: with lime mortar 287 lb/in^2; cement-lime mortar 587 lb/in^2; and cement mortar 661 lb/in^2.

1929: Stang, Parsons, McBurney[130]

They found that the compressive strength tests on the wallettes gave a better indication of the wall

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(129): Stang, A.H. and Others. U.S. Bur. Stand. Tech. Paper No. 311, 1926.
(79): J. Franklin Institution, 208,556, 1929.
(130): Stang, A.H., Parsons, D.E., McBurney, J.W. Bureau of Standards, Journal Research, 3, 507, 1929.

strengths than similar tests on the bricks. The highest wall strengths were obtained when cement mortar was used. The strength of solid walls varied roughly as the cube root of the compressive strength of mortar cylinders, 2" diameter and 4" long, cured on the walls. Differences in workmanship produced great differences in strength. Varying curing conditions were studied. Solid walls were found stronger than hollow types.

<u>1938: Krefeld</u>[85]

He investigated the effect of the shape of the specimen on the apparent compressive strength of brick masonry, and found that a wall is proportionately stronger than a square pier. To obtain the maximum strength, the breadth should be at least six times the thickness. In his analysis, Swallow stated that a ratio between the strength of the piers with breadth: thickness ratios of 6:1 and 1:1 (i.e. square) was 2.5:1.

<u>1950: Davey and Thomas</u>[36]

They presented data which had been obtained over a period of 23 years at the Building Research Station on the strengths of brick piers and walls.

One of the aims of the experimental work was to determine the most suitable tests for inclusion in B.S. 1257. The strength of individual bricks was found to be dependant

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(85): Krefeld, W.J. Effect of shape of specimen on the apparent compressive strength of brick masonry. Proc. Amer. Soc. Testing Materials. Vol. 38, Part. I, 1938. pp. 363-369.

(36): Davey, N. and Thomas, F.G. The structural uses of brick-work. Structural and Building Paper No. 24. Structural and Building Division Meeting, Institution of Civil Engineers, 1950.

on the method of testing. Nevertheless, it was concluded that testing of the bricks bedded between mortars of different strengths (2,000, 4,000, 6,000, 11,000 lb/in^2) had shown no significant advantage over a simpler method in which bricks were tested between plywood. They concluded that the strength test requirements to be incorporated in the above mentioned specification, should apply only to bricks without frogs, which should be tested between plywood without any application of mortar. (Their results will be interpreted, later in Chapter 6).

For the results representing the relation between the brickwork crushing strength and the brick strength as assessed with the above mentioned method, an example is shown in Figure 1.1. No comment was given on the relation, and conclusions were limited to the slenderness ratio.

As regards the relationship between mortar strength and the strength of brickwork, a typical example is illustrated in Figure 1.2. From their group of graphs, they concluded that there is no advantage to be gained, when using bricks with a crushing strength of less than 3,000 lb/in^2, by using a mortar much stronger than 1,000 lb/in^2, or in the case of high strength bricks, 2,500 lb/in^2. The first part of the conclusion is justifiable as shown in the typical graph given in Figure 1.2 - the lower part. The upper part of the same figure shows that the second part of the conclusion is not so well founded. Curves for mortar mixes, considered by them suitable, were given. With these curves they advised that apart from saving the cement, it is useful to keep the cement content in the mortar within the limits suggested in their curves, because of the improved resistance of brickwork to cracking when a weak mortar is used.

1953: Butterworth[27]

In his review, according to Swallow, he pointed out the main conclusions to all the previous investigations it was that the strength of mortar can be far below that of the bricks without seriously reducing the strength of brickwork, so that except with very strong bricks, brickwork built with cement-lime-sand mortar differs little in strength from that built with cement-sand mortar. This was considered also as an important conclusion, since from every point of view other than strength the weaker mortar was considered preferable.

1955: Building Research Station Digest -75[23]

The digest suggests that the effects of variations in the strengths of bricks and mortar on the strength of brickwork can be predicted from results of tests on short square columns loaded axially.

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(27): Butterworth, B. The properties of clay building materials A symposium. Ed. A.T. Green and G.H. Stewart. Stoke-on-Trent. The Brit. Cer. Society, 1953. p. 383.

(23): Building Research Station Digest No. 75. (First Series) Strength and stability of walls. Her Majesty's Stationery Office, March, 1955.

1955: Parsons[104]

In this paper he emphasized some of the conclusions in his earlier paper, especially on the quality of workmanship. He showed that its effect on the compressive strength is greatest with strong mortar and low strength brick. He also gave two formulae for estimating the compressive strength of brick masonry. These formulae were, most probably, a modification of his earlier formula. The strength of solid walls was shown to vary as the cube root, and the fourth root of the compressive strength of mortar for superior and ordinary workmanship respectively. The cube strength replaced the cylinder strength, without any explanation.

1959: Angervo and Lehtonen[4]

As quoted by Swallow, they stated that the load-bearing capacity of brickwork increased with the strength of mortar up to a limiting value depending on the strength of bricks. They considered that the lime content of mortar should be at least 20% by weight of the binder content to ensure adequate workability.

1959: Haller[55]

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(104): Parsons, D.E. Building research in the United States. Proceedings of the Conference on the Building Research. National Research Council of Canada. Ottawa, October 1955. pp. 9-17.

(4): Angervo, K. and Lehtonen, J. Helsinki, 1959, Building Science Abstracts, 378, 1960.

(55): Haller, P. The properties of load-bearing brickwork in perforated fired bricks for multi-storey buildings. Dept. of Scien. and Indust. Research, Building Research Station Library Comm. No. 870 (translated from Schweizerische Bauzeitung, 1958. 76(28), pp. 411-419, Feb. 1959.

1960: Haller[54]

Schellbach[116]

Vogt[139]

According to the literature in hand, a considerable development in the way of thinking of the strength of brick masonry took place in the period 1959-1960. The development is reflected in the form of the ideas given in these four papers. It is the first time the present author found among the literature any mention of the difference between the rigidities of bricks and mortar, and its influence on the failure characteristics.

It can be said that, in spite of the fact that there are new insights in all three papers, Haller's can be considered the one which dealt, relatively to the others, to a far greater extent with the factors affecting the strength properties of brickwork.

Without showing anything about derivation Haller gave an expression for the transverse stress in the brick in terms of the properties of bricks and mortar (moduli of deformation, cross-sectional area, Poisson transverse strain factor of the brick and mortar, transverse stress in the mortar and the compressive stress on the Unit).

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(54): Haller, P. The physics of fired brick; Part One: Strength properties. Dept. of Scien. and Indust. Research. Building Research Station Library Comm. No.929 (translated from the Verband Schwezerischer Ziegel-und Steinfabrikanten Vszs, Zurich). Jan. 1960.

(139): Vogt, H. Considerations and investigations on the basic principles of model tests for brickwork and masonry structures Dept. of Scien. and Indust. Research. Building Research Station Library Comm. No. 932. (translated from German: Ziegelindustrie Jan. 1960).

Because of the difficulty of extrapolation for the state at failure from the elastic range, considered in the formula, the potentiality of the expression was considered by Haller himself to be limited. He concluded that it would merely indicate to the man on the job the extent to which the strength of brickwork can be affected by raising or lowering the quality of the bricks and mortar.

The papers also pointed out the factors which can increase the brick strength in compression. Similarly Haller's recommendations (1960) for increasing the strength of brickwork cover the others, and the following is a summary of those concerned with bricks and mortar only:

Improving quality of bricks by:

a. high compressive strength and tensile strength.

b. low scatter of the individual values from the mean value.

c. large cross section under tension, small cross section of perforations, free from cracks and without internal stresses.

d. level bed surfaces to avoid local increases of stresses.

e. regular shape and dimensions giving regular bond and uniform joints.

Improving quality of mortar by:

a. type of cement

b. quantity of cement

c. petrographic properties, particle shape and size grading of sand - avoid grading zones with an excessive amount of sand falling within fairly narrow grading limits, excessive content of clay, organic impurities.

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(116): Shellbach. The most important factors influencing the strength of brickwork. Brit. Cer.Res. Association. Trans. No. 557 (translated from Ziegelindustrie, 13, 841, 1960), Nov. 1961.

d. the lowest practical addition of water workability can be increased by a 10% addition of white lime with a slight reduction in strength.

1960: Hummel(73)

He made an attempt to clear some doubts which had arisen concerning the aptness of the German Specification Din 1056. The evaluation of the results included the compressive strength of the masonry with various mortars, which was measured on cube masonry assemblages.

Mortar of compressive strengths between 29 and 43 kg/cm^2 (412,5-611.7 lb/in^2) gave practically the same masonry values as mortars of compressive strengths 83 to 97 kg/cm^2 (1180.6-1379.8 lb/in^2) as shown in Figure 1.3.

His explanation for this phenomenon was a rare one of this kind. He said that this unexpected result is explained by the fact that the masonry compressive strength expresses a slab strength value of the mortar, not its cubic compressive strength. The slab strength increases compared with the cube compressive strength more in the case of mortars of low compressive strength than for those of higher compressive cub strength, thus tending to equalize the slab strengths, so that, especially where high grade clinkers are used, the range of masonry strength values is also narrowed.

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(73): Hummel, A. Tests on mortars for free standing chimneys. National Research Council of Canada. Tech. Trans. 886, Ottawa, 1960. (translated from German: Fortschr. U. Forschungen Im Bauwesen, Ser. D, (24): 15-31, 1956 , pp.13.

1962: Foster[45]

In his description of high load-bearing brick buildings, he reviewed shortly the factors that can affect the strength of the mortar, which in turn affect the strength of brick masonry. In another part of the paper he gave a statement the first part of which was considered by the present author to be important and should have been included in the main factors. That is, for high strength brick masonry it is essential to use a mortar of low transverse elongation and the compressive strength of this must be over 2,850 lb/in^2 after 28 days.

1964: Structural Clay Products Institute[131]

In 1963, a long-term research programme started in the U.S.A. It seems strange that the progress in thinking of 1960 was not taken into consideration during planning such a big scale work. The programme started, although with more care, on the conventional empirical basis. The object of the programme has been to provide better information on the strength properties of brick masonry.

The data given in the first progress report indicate that higher masonry compressive strengths are associated with higher brick compressive strengths all other factors being equal. The ratio of prism compressive strength to brick compressive strength tends to be higher for prisms built with lower strength bricks. This emphasises the importance of

(131): Structural Clay Products Research Foundation. A division of the Structural Clay Products Institute, Illinois. National Testing Programme: Progress Report No. 1; Compressive, flexural and diagonal testing of small scale four-inch brick masonry specimens. Oct. 1964.

the effect of mortar strength on masonry compressive strength, particularly in assemblages built with high strength units. In such masonry, the compressive strength of the mortar is the real limiting factor and, no doubt, accounts for the flattening out of the curve plotting brick strength against prism compressive strength when the former reaches around 12,000 psi.

From the data, Figures 1.4, 1.5, it would appear that the compressive strength of the mortar has a much greater, and more easily defined, effect upon the masonry prism strength than does the compressive strength of the brick units.

1965: Building Research Station Digest -61[25]

Under the heading of strength relationships between walls and mortars, it is stated that for a considerable range of bricks the optimum strength of brickwork is obtained with mortar mix proportions of (cement and lime): sand of 1:3 by volume, and that there is little advantage in using a very strong cement-sand mortar in most brickwork and blockwork.

The Digest illustrated a comparison between strengths of mortar and brickwork for a number of mortar mixes when a medium-strength brick is used. According to the Digest (Figure 1.6) although the mortar loses strength the effect on the brickwork is not nearly so marked.

In the case of high strength bricks the fall in strength of brickwork is much more pronounced as progressively weaker mortars are used. It is also recommended

(25): Building Research Station Digest (2nd Series) - 61. Strength of brickwork, and blockwork and concrete walls. Her Majesty's Stationery Office, 1965.

that in order to utilize the full capacity of high strength bricks (10,000 lb/in^2 or more) a 1:3 cement-sand mix is needed, while for lower strength bricks, mortars with increased proportions of lime can be used without any great loss in brickwork strength.

As regards the strength relationship between wall and brick, the ratio of the former to the latter is mentioned as varying from slightly less than half to slightly less than one-fifth, the lower limit being that sometimes found for high strength bricks used with a weak mortar.

As stated in the Digest, the permissible stresses incorporated in C.P.111:1964, for walls made of various bricks and mortar mixes are based on relationships of the kind illustrated in Figure 1.6,7.

As will be shown later, what was written in connection with blockwork is very important. Loading tests on walls of one block thickness built with low to medium strength blocks gave a ratio of wall strength to block strength that was usually much higher than that for low intermediate strength bricks. While for brickwork it is 0.1-0.5, it ranges for blockwork from 0.5 to 1.0 (Figure 1.7), and was usually greater than 3/4. It is mentioned in the Digest that the Code of Practice, based on limited evidence of this kind, allows varying increases in permissible compressive stress for walls built with blocks whose height:thickness ratio is between 3/4 and 3 (a ratio of 3/4 corresponds to the common brick).

This allowance for block shape is greatest for a height:thickness ratio of 2 to 3 and a crushing (wet) strength of up to 800 lb/in^2 (on gross area). For clay blocks of greater strength, tests indicate that the allowance should diminish as block strength increases, and should disappear when the latter exceeds about 3,000 lb/in^2.

1965: C. Monk, Jr.(30)

Parallel to the present work, and five years after the development in thinking of 1961, Monk made an analysis of previous work on the compressive strength of brick masonry. It seems also strange here, that his really new and useful contribution was given only in the introduction. In it he gave the results of a few tests, to show how the compressive strengths of couplets made from bricks of the same strength were affected by the use of different materials between the bricks. The range between the highest and lowest strengths was 0.97-0.42 of the compressive strength of the bricks. Here it should be mentioned as a point of important which will be referred later, that the brick strength was assessed in accordance with the A.S.T.M. C67.

He illustrated also the different modes of failure of a series of Hydrocal A.H. cubes (gypsum cement plus 20% P.C.) when tested with aluminium sheets and polythene as jointing materials. The modes were respectively shear and

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(30): C.Monk, B. Jr. A historical survey and analysis of the compressive strength of clay masonry. Structural Clay Products Institute, Illinois, May, 1965.

and splitting failures.

It can be fairly stated, that those experiments were not based on either a precise object, or any mathematical analysis. And this is presumably why the results were given very briefly in the introduction without any detailed analysis. On the other hand his main analysis for the factors affecting strength was adequate on the level of Haller's, so that there is no need to reiterate any of the conclusions here.

1965: Prasan, Hendry, Bradshaw[109]

Among the conclusions of crushing tests on storey-height walls $4\frac{1}{2}$ inches in thickness, the following were of importance as regards the compressive strength of brick masonry:

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

2. The effects of brick and mortar strengths on the strength of brickwork are generally in agreement with the work of the Building Research Station.

3. The brickwork piers having mortar joint thickness greater than $\frac{3}{4}$" were weaker than piers having normal joints.

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(109): Prasan, S., Hendry, A.W., Bradshaw, R.E., Crushing tests on storey-height walls $4\frac{1}{2}$ inches thick. Proceedings of the Brit. Ceram. Society No. 4: Load-bearing brickwork. July, 1965.

4. Increases in brickwork strength of over 60% were observed when every bed joint was reinforced horizontally.

1965: Simms[122]

He presented data which covers a wide range of building units. Wall strengths from the tests and relevant permissible loads from Code of Practice C.P. 111:1964 were used to indicate the range of load factors obtained from the various types of wall.

Among the results the ratios between the strengths of walls and the strengths of units (mostly the older forms of block of plain cuboid shape) were given. The ratio ranged between 0.45 and 1.1. The results of these walls are illustrated graphically in a new type of interpretation by the present author in Figure 6.23. It was claimed by Simms that the relatively low crushing strength of these blocks can be attributed to the premature failure in tension of the transverse webs which connected the vertical webs of the block. Since the weakness due to the shape of the block is less pronounced when it is built in the wall, it is thought that this phenomenon of individual blocks having an apparently low crushing strength might be one of the factors contributing to the high ratio of wall strength to block strength.

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(122): Simms, L.G. The strength of walls built in the laboratory with some types of clay bricks and blocks. B.R.S. Current Papers; Engineering Series 24 (reprinted from Transactions of the Brit. Cer. Society, July, 1965. pp. 81-92).

1966: Sinha[12]

This approach appeared by the end of Part One of the present work. It deals with an investigation into the splitting failure of brickwork by penetrating carefully inside , the structure of the assemblage, and by taking into account the individual properties of bricks and mortar. But it was conservative to some extent as regards some parameters which have been invariable by the force of tradition.

The derived formulae yielded many of the factors required for predicting the strength of brick masonry, but not the possible methods of increasing it. However in a similar manner to the present work prediction of strength was hampered by the problem of shortage of data.

1.3 DISCUSSION AND CONCLUSIONS

1.3.1. General

Historically, it can be said that there has been great deal with research on strength of brick masonry, for a long time. Although the forgoing review represents only few of the sporadic main contributions, it is felt that it is sufficient to form basis for discussion of some of the main points.

Starting with the main and common objects, two of them can be emphasised as common: the first has been the plotting of the brickwork strength against the mortar and brick

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(12): Sinha, B.P. Splitting failure of brickwork as a function of the deformation properties of bricks and mortar. Research Report: B.P.S./3/4, Structural Ceramics Research Unit, Department of Civil Engineering, University of Edinburgh, Feb. 1966.

properties, by considering each of the latter to be represented by one or, very rarely, two of the mechanical properties, as assessed by the loading tests incorporated in the specifications or codes. The second has been investigating the possible ways of an adequate use of brick masonry, which includes, achieving so far as possible the highest strength.

For both objects and **in the** majority of cases the common approach was through making experimental attempts to include one group, or more, of various mortars, grades of bricks, methods of bonding, different specimens, and workmanships. It should be pointed out, that much of the general influence of these groups on the strength of brick masonry, as based on judgment of field experience or research work, have been listed and described in detail by many of the given authorities. Among these authorities, special reference can be made to the original works of Davey and Thomas, Haller, Vogt, and Monk.

In a similar manner to the review, the discussion will be limited to the main points which emerged as of vital importance and peculiar to this particular work, with the inclusion of very few remarks, where necessary, on points not included in the review.

1.3.2. Strength of Brick Masonry and Loading Tests on Bricks, Mortar

From the repeatedly inexplicable

conclusions of past research a question of importance should have arisen; Why has the strength of brick masonry proved impossible to correlate with the strength of bricks and mortars as assessed by the standard loading tests?

The complete answer to this question is undoubtedly complex. It can be a profitable suggestion that the loading test should be first defined. Within the sphere of the present work, it can be defined as a means of assessing the structural performance of mortar or brick in the masonry assemblage under compression. The test can be considered desirable if the structural adequacy of the assemblage is only doubtful, which is not the case for brick masonry. Due to the fact that till a short time ago, the masonry assemblage had never been subjected to a rigorous analysis, the test has become necessary.

In conducting such a test, three other important questions usually arise. These are:

1. What criteria should be used to judge the success or failure of the material (mortar or brick) subjected to the test?

2. What should be the magnitude of the test load at which the criteria are applied?

3. What should be the details of the loading procedure?

As it appeared to the present author, these three questions have not aroused much attention. Most of the specifications or codes contain specified loading tests, and describe in varying degrees the test procedure. Nevertheless there has been the feeling of the lack of an adequate test. To justify this it is enough to refer to the non-complete achievement of the first object, given at the beginning of the discussion. According to the literature in hand it cannot be claimed that the great change in the manner of using brick masonry has caused any considerable concern even a little interest, in the field of assessing the structural quality of bricks and mortars. This is the case in both research and practical work. Moreover both specifications and codes seem to be in a state of non-differentiation between a strength test and a performance test. The result is that neither of the tests has been achieved satisfactorily.

A very distinct example is that the normal test on a brick is a strength test. At the same time the criteria of failure in a masonry assemblage under compression have never been used in any specification or code. If we call the test a performance test we find that the mode of failure of a single brick according to the specified test, bears no resemblance to that of a wall at failure load.

This brought the author to a conclusion, that so long as the failure characteristics of an assemblage are beyond changing, then adequate loading-test specifications might well be urgently needed. To formulate such tests the above mentioned three questions need to be answered properly and carefully. To do this the relative functions of bricks and mortar must be clarified.

1.3.3. Bricks

Two facts are well known about bricks. The first is that in the majority of investigations the chief structural property of brick as affecting the compressive strength of brick masonry, has been its compressive strength. Accordingly, attempts were made to correlate them. The second is that, in almost all cases, bricks are known to be stiffer than mortar.

These two facts together lead to a basic difference as regards their respective positions in a wall and in a standard loading test. During testing the loads are superficially of the same nature: that is compression. In reality the systems of loads withstood by the bricks are entirely different. While a brick, or half a brick when tested according to the standard load test is subject to lateral restraint from the platens of the testing machine, it is subjected to lateral squeezing out when a brick masonry assemblage is tested. Consequently, the modes of failure are completely different. In the first case it is a shear failure, and in the second it is splitting. Not only the modes of failure but also the value of the failure compressive strength are different. When we realise that one of the values can have a negative sign, and the other a positive sign, with respect to a certain value, called later the actual compressive strength, it can be understood that the difference between the two values may be too big.

for any correlation to be achieved. It may be claimed, here, that the use of plywood can compensate for this, but it will be shown later that this is absolutely incorrect.

However on the basis that the nature of testing should be compression, the possible range between strengths, the extent of similarity or divergence between all the failure characteristics, arguments about the necessity for more adequate tests, and suggested loading tests on bricks, are all fairly new questions to be dealt in detail theoretically in Chapters 2-4, and experimentally in Chapters 6,8.

1.3.4. Contradiction Between Mortar Definition and its Assessment

With mortar the situation is more vague. The ways of defining mortar and the methods of assessing its strength and suitability for brickwork have been contradictory, so that it does not seem strange if some investigators(10) call mortar the Cinderella of the building materials.

In a dictionary it is defined as: Mixture of lime, sand and water, for joining stones or bricks. In technical literature mortar may be defined as: Plastic material consisting essentially of a filler (sand) and bonding material (cement and/or lime) which hardens after application; it is used for bedding and jointing building units, such as bricks and blocks, and for surface finishes (plastering and rendering).

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(10): Bessey, G.E. Current developments affecting the design and use of mortar for building purposes. Welwyn Hall Research Association. Information Paper 17, November, 1965.

In this sense all plastering mixes containing fine aggregate should be regarded as mortar. But as indicated in the review, mortar within the sphere of the present work comprises mixes for jointing and laying bricks and blocks.

Thus in any case mortar has been looked upon merely as a bonding agent. And this is why in the past it has been mainly a building site product, dependent for its properties mainly upon the skill of the bricklayer, rather than a material which could be precisely specified and manufactured.

In specifications or codes, the requirements for mortar have been based largely on the average of what the craftsmen were known to use, and in practice the craftsman usually adjusted the specified proportions according to his available materials and skill.

Turning now to the method of assessing its suitability from the point of view of strength of brick masonry, the contradiction becomes clear.

The primary structural property which has been used to assess the mortar's contribution to a masonry assemblage subject to compression has been its compressive strength. Few investigators considered its tensile strength, which must surely be with the first consideration of a binding material.

In the author's opinion such contradictory considerations for the one material is responsible for the failure of mortar to be studied like other building materials, deeply and effectively. Before coming to a conclusion on this point something should be added. As the review indicated, most of the recommendations were for the use of mortars having moderate strength in most of the brick masonry work, high strength mortar being recommended for high quality bricks. This implies that there is a feeling or desire towards the achievement of higher strengths through using higher strength mortars.

All these together lead to the conclusion that the second consideration, with more care to cover all the mechanical properties should have been stressed a long time ago. At the same time the present author feels that the description of mortar as a purely binding material should be modified.

1.3.5. <u>Structural Function of Mortar in Brick Work</u>

Here, too, the problem of loading tests as against the state of the mortar inside a wall is very important.

In a standard loading test the compressive strength of mortar is assessed by using normally a cube, and occasionally a cylinder or prism. The tensile strength which is not very often used, is assessed by testing briquettes.

In the field of technology of materials it is well known that the cube compressive strength is far from being representative of the actual strength. If the common mode of failure of brick masonry is along the vertical plane lying between the vertical joints (as illustrated in Figure 1.8), it can be accepted that a compressive strength test should apply for assessing the mortar's structural participation, with only two main reservations. The first is that the failure strength of mortar in a wall which will represent actually the wall failure load, may be of a higher value than the conventional mortar cube strength. This is of course due to the relatively small height of the horizontal joint compared to the height of the cube. The second is that the number of horizontal joints, should exceed a certain minimum in order not to consider its effect. Thus it is fairly obvious that the cube compressive strength itself is far from being adequate. At this point it can be said that the compressive strength of prisms or cylinders, because of their lower value are much further away from being adequate.

It may be claimed that failure of mortar or concrete cylinders in compression is usually vertical splitting, and this would recommend the test because brick masonry fails also by splitting starting at one of the vertical joints. But it should be realised that splitting of a cylinder is absolutely different from the latter. In fact it lies in another field of study, so that there is no need to discuss it here.

As regards the tensile strength, the mode of failure of brick masonry suggests that the use of a tension test might be more reasonable. No theoretical justification can be found, and the only experimental results which were not considered by their original contributors, but have been interpreted by the present author are those of the Structural Clay Institute[131]. The interpretation is illustrated in Figure 1.9.

In the author's opinion, the results of McCaustland-1900, Kreuger-1916, Bragg-1918 and Prasan, Hendry and Bradshaw-1965 suggest provisionally and with some speculation, that the modulus of elasticity and Poisson's ratio of mortar may have great influence on the strength of brick masonry, even more than the influence of the presence of vertical joints. To clarify this briefly, it is conceivable that if two halves of a brick are put together with non-continuous hard backing top and bottom, and tested in compression, then the ultimate strength may be higher than the strength of a single brick tested under the same conditions. If under the same conditions with the two halves the hard backing is replaced by any soft material resembling mortar the strength will be much less. It cannot be claimed that the gap between the packing is ineffective in the first case, and that it is responsible for the reduction in the second case. As will be shown later in Chapter 6 it is the squeezing out due to softness which is mainly responsible for the reduction.

To sum up the structural participation of mortar was not considered to be simple. The structural properties of mortar, its structural role, the properties influencing the strength of brick masonry, the proper criteria for judging its success or failure, the formulation of loading tests, mortar simulation for practical purposes; these important points are dealt with theoretically in Chapters 2-5 and experimentally in Chapters 7 and 8.

1.3.6. Failure Criteria

From the preceding discussions on bricks it can be emphasised or added that the great differences in the relationships and behaviors of both bricks and mortar in a masonry assemblage and in compressive loading tests, are the main factors responsible for the non-appearance of failure criteria. This has invalidated to a large extent many of the results achieved to date.

The work of Haller, Vogt and Monk, confirmed later by the observations of Prasan, Hendry and Bradshaw yielded light on what happens between bricks and mortar inside an assemblage under compressive load. Clearly Sinha's work which appeared during the present work was a step forward towards recognizing some of the theoretical aspects.

However, at the beginning of the present work it was felt that knowledge of the mechanics of brickwork failure under axial compression was far from complete. Rigorous analysis for building up failure criteria, was considered inevitable not only for better understanding but also for planning profitable research work. This will be discussed analytically in Chapters 2-4.

1.3.7. Force of Tradition and Unchangeable Paramters

Under this heading bonding must be discussed. The literature on systems of bonding is extensive and available everywhere: an example is that of Caravaty[28] Usually the word "bond" when used with brick-masonry indicates the structural bond, or pattern of the mortar bond. It can undoubtedly be said that most bonds exist by force of tradition which differs from one country to another.

With all bonds the thickness of the horizontal mortar joint has been subject to many investigations, and it has been proved that its influence is very considerable. The height of a building unit or brick, on the other hand, was considered practically unchangeable.

In the author's opinion the contrary could have been considered, with great probability of being more profitable. For practical bricklaying the joint thickness cannot be less than a certain minimum value. On the other hand, the height of a brick might have been changed easily during the development of industry. The basis for this was indicated

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(28): Caravaty, R.D., Plummer, H.C. Principles of clay masonry construction. Students manual. Published by Structural Clay Products Institute, Washington, D.C. 1960. pp. 4-8.

at the earliest date by Howard's results of 1884-1886, whose tests were considered to have peculiar features, similarly Bragg - 1918, Ingberg - 1924, Davey and Thomas - 1950, and Simms - 1965. But it seems that what was first a custom and changed into prejudice later, has not given any chance for such parameters to be changeable or even studied in research work.

However in the light of these results which were considered as mere chance in their time, the height of a brick appeared at the beginning of the present project as a desirable factor to be investigated. This was encouraged later by the results of the theoretical studies. Then very near the end of the project, a similar objective was expressed by one authority, Haller[53], However, this question of height will be dealt with theoretically in Chapters 2-4, and experimentally to a far greater extent in Chapter 6.

1.3.8. Shape and Size of Brick Masonry Assemblage

Apart from the individual loading tests on mortar and bricks, it seems reasonable that some authorities have tried to develop some sort of field quality control test. It seems also probable that such tests can give a reliable assessment of the capability of the brickwork used.

Following what is usually done in such a case, it will be necessary to adopt a specimen in which all the influencial factors from the start of fabrication till the end of testing may be taken into account. Keeping this in mind and dealing with brick masonry, this time as a single material, the three questions which arose before for formulating a loading test emerge as vitally important in all the aspects discussed before.

The most important question will be the shape and size of specimen. Undoubtedly the big variety in shapes and specimens, employed before, shows how the results could be misleading. A particular example is that the Building Research Station Digest[23] suggests tests on short small columns, whereas Krefeld showed that the strength-ratio between a wall and a pier was 2.5: 1.

In this aspect the author came to the conclusion that the effect of this variable should be investigated. This was tackled to some extent in the experimental work described in Chapter 8.

1.3.9. Workmanship

The data given by Parsons - 1955, show the great influence of workmanship on the strength of brickwork. This is why it has been one of the important factors for the determination of the factor of safety.. On the side of workmanship it is generally known that the following factors reduce

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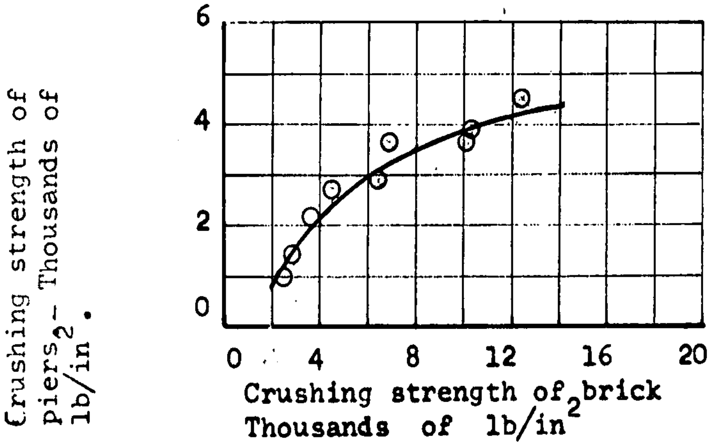
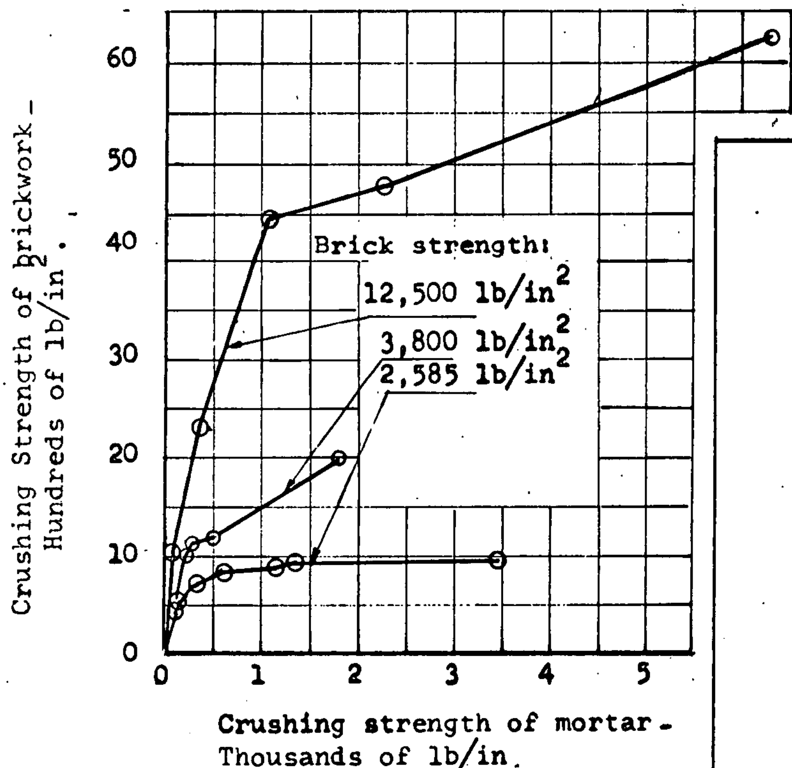
(23): Building Research Station Digest No. 75 (First series). Strength and stability of walls. March, 1955. Her Majesty's Stationery Office, Reprint, 1964.



Piers: 9" x 9" x 3'-0"

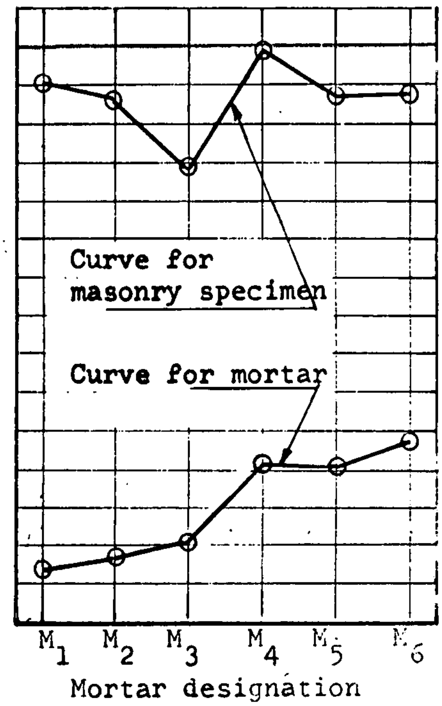
Mortar strength: 2.800 lb/in^2

FIGURE 1.1:
Relationship between brick strength and strength of brickwork piers. [Due to Davey and Thomas (36)]



Piers: 9" x 9" x 3'.0"

FIGURE 1.2:
Relationship between mortar strength and strength of brickwork [Due to Davey and Thomas (36)]

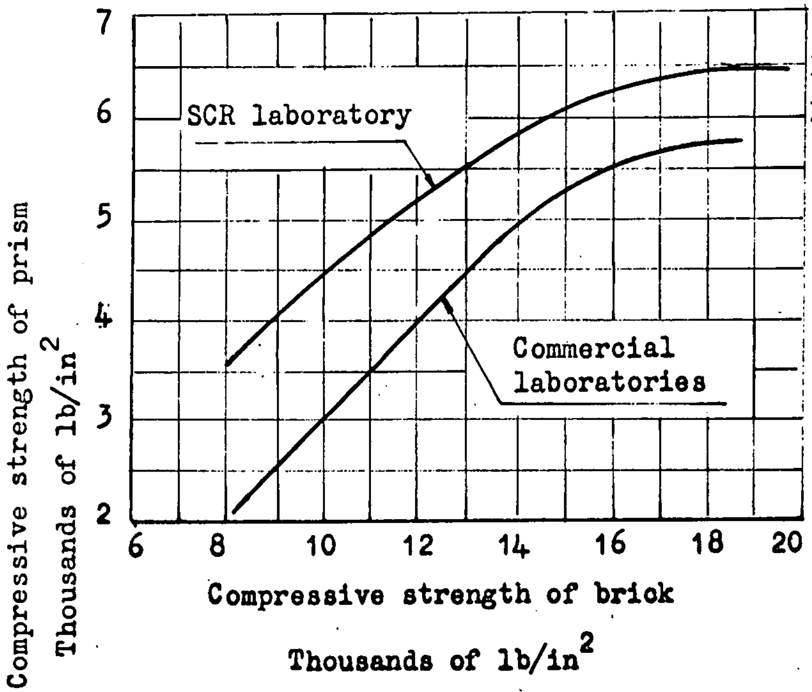


Compressive strength of M_1, M_2, M_3 = 29–43 kg/cm^2 M_4, M_5 M_6 = 83–97 kg/cm^2

Compressive strength of brick = 667 kg/cm^2

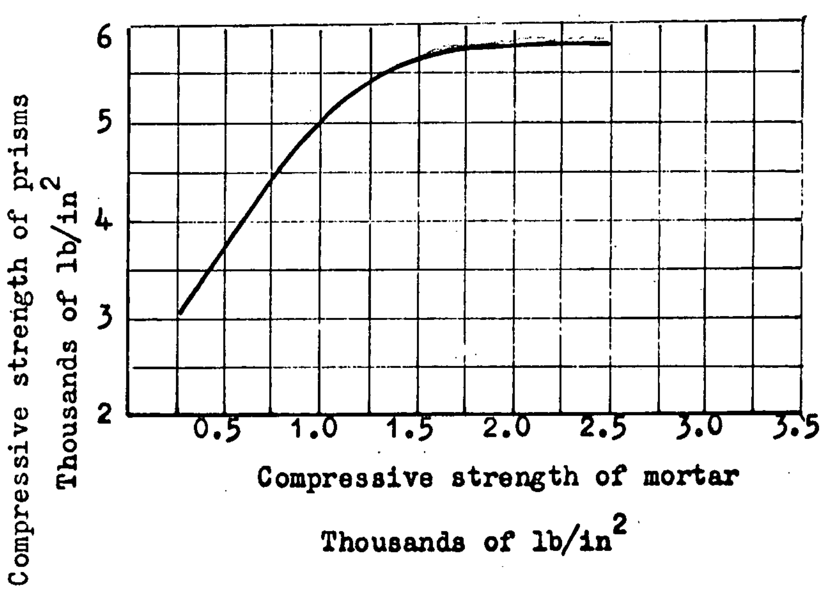
Side of cube : 38 cms.

FIGURE 1.3:
Compressive strengths of masonry cubes compared with mortar strength [Due to Hummel (73)]



Prism: 16" x 8" x 4"
Mortar: C.L.S. mortar $(1{:}\frac{1}{2}{:}4\frac{1}{2})$ of compressive strength 1292 lb/in^2

Figure 1.4:
Prism compressive strength as a function of compressive strength [Due to S.C.P. Institute (131)].



Prism:
Brick: Compressive strength = 11.771 lb/in^2

Figure 1.5
Prism compressive strength as a function of mortar compressive strength
[Due to S.C.P. Institute (131)]

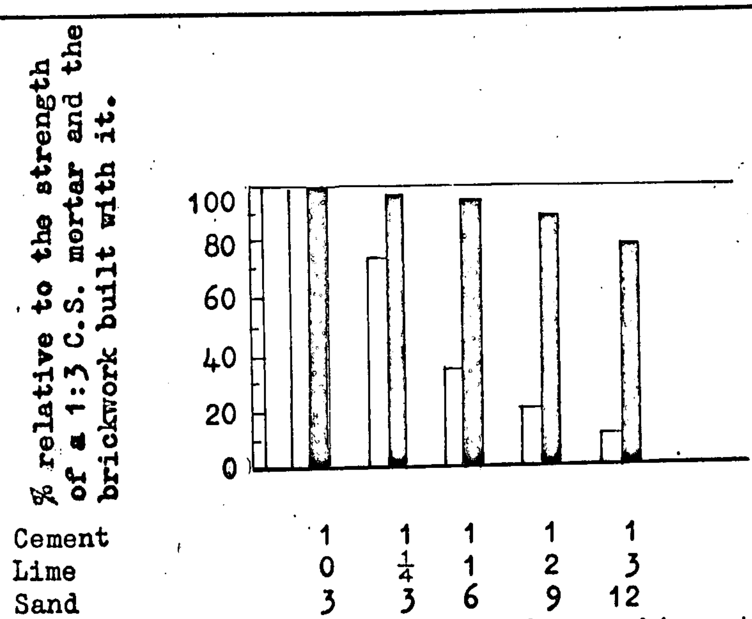


Figure 1.6 Effects of mortar mix proportions on the crushing strength of mortar and brickwork built with medium strength bricks [Due to B.R.S. Digest 61(25)]

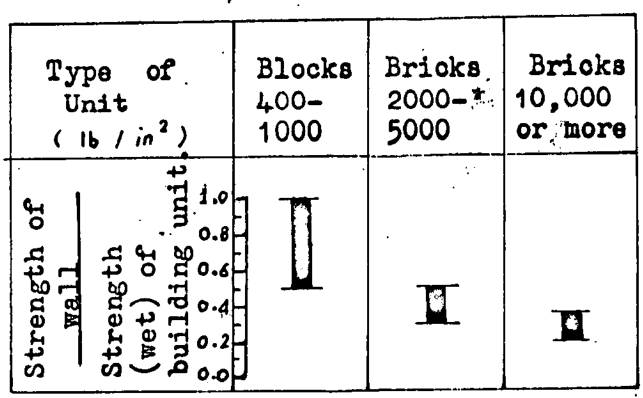


Figure 1.7: Ratio of strength of storey high wall to strength of building Unit. [Due to B.R.S. Digest 61(25)]

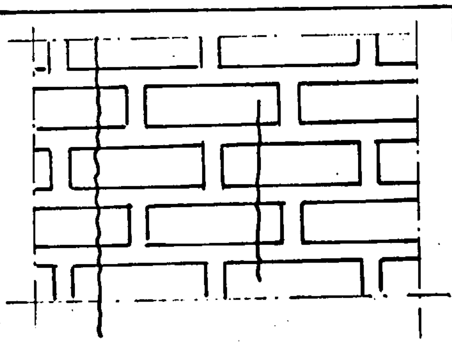


Figure 1.8: Line of failure between vertical joint (See 1.3.5 9.2.6.4)

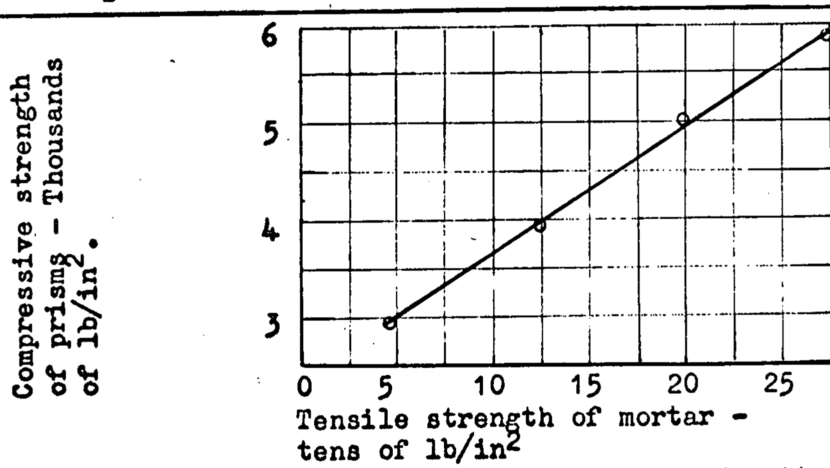


Figure 1.9: Prism compressive strength as a function of mortar tensile strength. (Figure is interpreted, by the author, from the data given in Ref.(131).

remarkably the compressive strength of brickwork:

1. Furrowing of bed joints.

2. Thick joints

3. Partial filling of vertical and collar joints.

Penetrating deeply into these factors emphasises strongly that mortar participates structurally besides being a bonding agent.

However all through the present work the author attempted to keep his workmanship consistent. To a great extent it can be considered more than good, which is the assumption in the theoretical analyses.

1.4 SUMMARY:

Summing up considerations of previous work relevant to the present project the following was concluded:

1. For at least half a century it has been known that in most cases brick masonry in compression failed by vertical splitting although the actual mechanism of failure was far from being understood.

2. The majority of research and design work dealt with deformation and failure characteristics of brick masonry in the same way as with a homogeneous or single material. In other words strains and stresses were taken as averages over finite lengths and areas.

3. It seemed probable that non-consideration of the internal stresses and strains, which are functions of the deformation and

structural properties of bricks and mortar, was a serious drawback responsible for what was called mysterious and inexplicable.

4. It appeared hopeful to consider brick masonry as an absolutely heterogeneous or composite material. At the same time it seemed impossible to tackle the strength of such a two-phase material from the stand-point of one phase only.

5. The loading tests incorporated in the specifications and codes in spite of their vital importance as regards the mysterious results were subject to doubt.

6. To start with, two solutions appeared possible. The first was to stick to the general meaning of the terms of reference of the present project and plan for extensive experimental work on mortars and brick masonry assemblages. This could have beeen done without any solid base or clear objectives. The second was to consider thoroughly a problem which had lasted for a very long time without achieving a proper solution. This might mean that it would take a longer time in the beginning, but in the long term it would be more profitable. Thus with the object of defining the short-comings of the previous research work on brick masonry under uniaxial compression and the remedies in an adequate future research the following questions emerged to be urgently investigated. And with these questions in mind the present work started.

<u>Question 1:</u>

Is it possible, even to some extent, to define failure criteria on the basis of theoretical speculations?

Question 2:

What are the possible approaches for tackling such a problem?

Question 3:

Is it possible to justify the theoretical aspects from previous experimental results?

Question 4:

What are the influential factors, not considered before that might help in solving the problem?

Question 5:

Is it possible, for time consideration, to list them according to priority?

Question 6:

Has the force of tradition influenced some paramters to be virtually unchangeable.

Question 7:

What could be done at the present stage to fulfil the requirements of brick masonry in its new function?

Question 8:

What is wrong with the present specifications and codes as regards the loading tests, and what is the range of error?

Question 9:

How to formulate new loading tests for mortar bricks, and masonry assemblages?

Question 10:

Is it necessary and possible for the loading tests to be the same in both research work and field work?

Question 11:

How to differentiate between the two groups of tests, and how to correlate between them?

Question 12:

What are the main differences between the properties of hardened conventional mortars?

C H A P T E R 2

A HYPOTHETICAL APPROACH WITH ITS ANALYTICAL TREATMENT FOR THE DEFORMATION CHARACTERISTICS AND MECHANISM OF FAILURE OF BRICK MASONRY

2.1. ORIENTATION

The present chapter discusses the the characteristics of deformation and mechanism of failure of a brick masonry wall on the basis of a completely hypothetical assumption in which the result of idealizing the wall structure is idealized into a series of mesh frames or lattices. On the basis of one of the analysed systems, the parameters influencing the strength of masonry were deduced and discussed.

2.1. ASSUMPTIONS AND NOTATIONS

2.1.1. Assumptions

The following assumptions are applicable to the walls analysed in this and the next chapters:

1. Bricks and mortar are individually homogeneous.
2. Each brick is identical to every other brick.
3. All joints, both horizontal and vertical, are completely and uniformly filled with mortar.
4. The wall is not slender.
5. No stress concentrations due to any irregularity.

2.1.2. Notation

a, b, c : dimensions of a brick in decreasing sequence.

t_1 : thickness of vertical joint.

t_2 : thickness of horizontal joint.

p : applied load on the wall expressed as pressure

P : applied load on a unit of the idealized internal system.

E : Young's modulus of elasticity.

ν : Poisson's ratio.

σ : normal stress.

ϵ : normal strain.

Suffix$_m$: indicates mortar, e.g. E_m = Young's modulus of elasticity of mortar.

Suffix$_b$: indicates brick.

First suffix$_t$: indicates tension, e.g. E_{tb} = Young's modulus of brick in tension.

No second suffix : indicates compression.

For the idealized internal structure:

A_1, A_2, A_3 : areas of the vertical, horizontal, and diagonal members respectively of a unit of the idealized internal system.

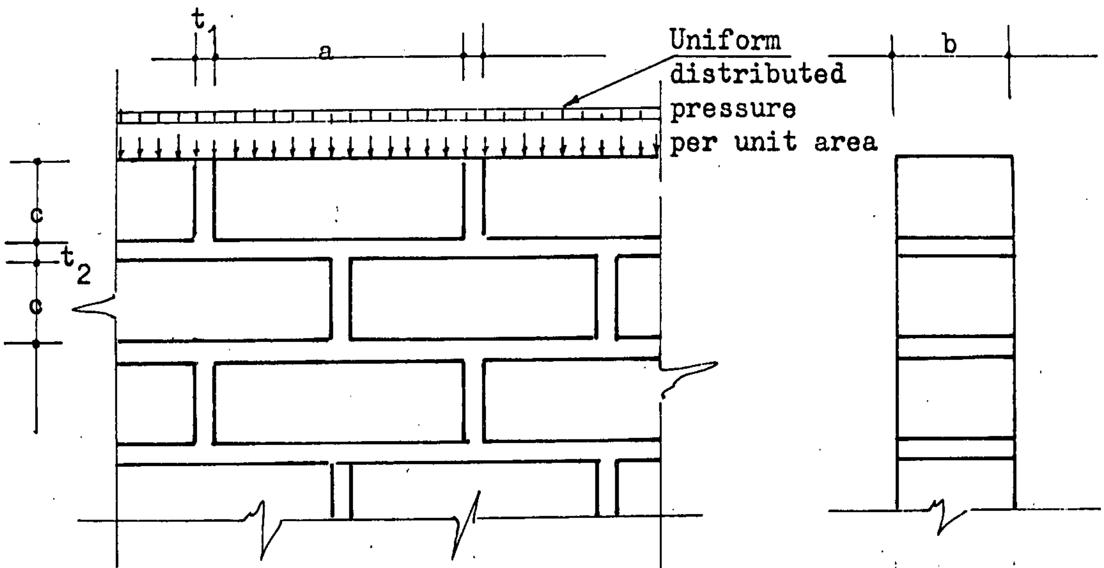
Suffixes 1,2,3 : indicate the members respectively, e.g. E_1 = Young's modulus of elasticity for the vertical member.

n : the ratio of the applied pressure to the induced lateral tensile stress (p/σ_2).

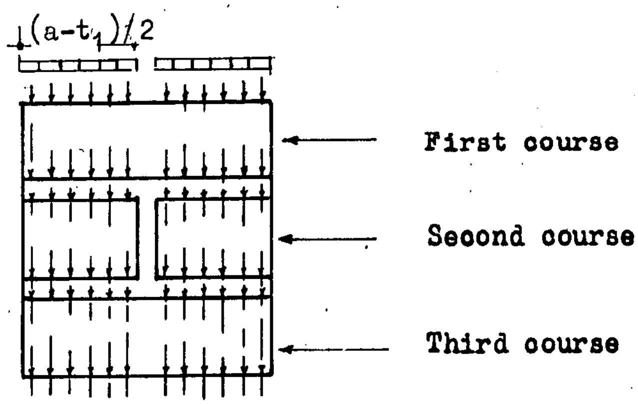
2.3. DISPLACEMENTS, DEFORMATIONS AND INTERNAL STRUCTURAL SYSTEM

2.3.1. Relative Displacements, Deformation Characteristics.

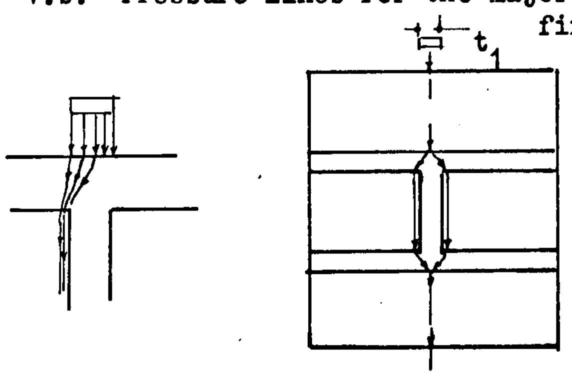
Consider a single leaf wall (Figure 2.1-a) made of bricks which are very stiff compared to the mortar and subjected to a uniformly distributed pressure. It can conceiv-



1.a Single leaf brick wall subjected to uniform pressure



1.b. Pressure lines for the major part of pressure on a brick on the first (top) course



1.c. Pressure lines for the smaller part of pressure on a brick on the top course.

FIGURE 2.1:

Pressure lines in a single leaf brick masonry wall.

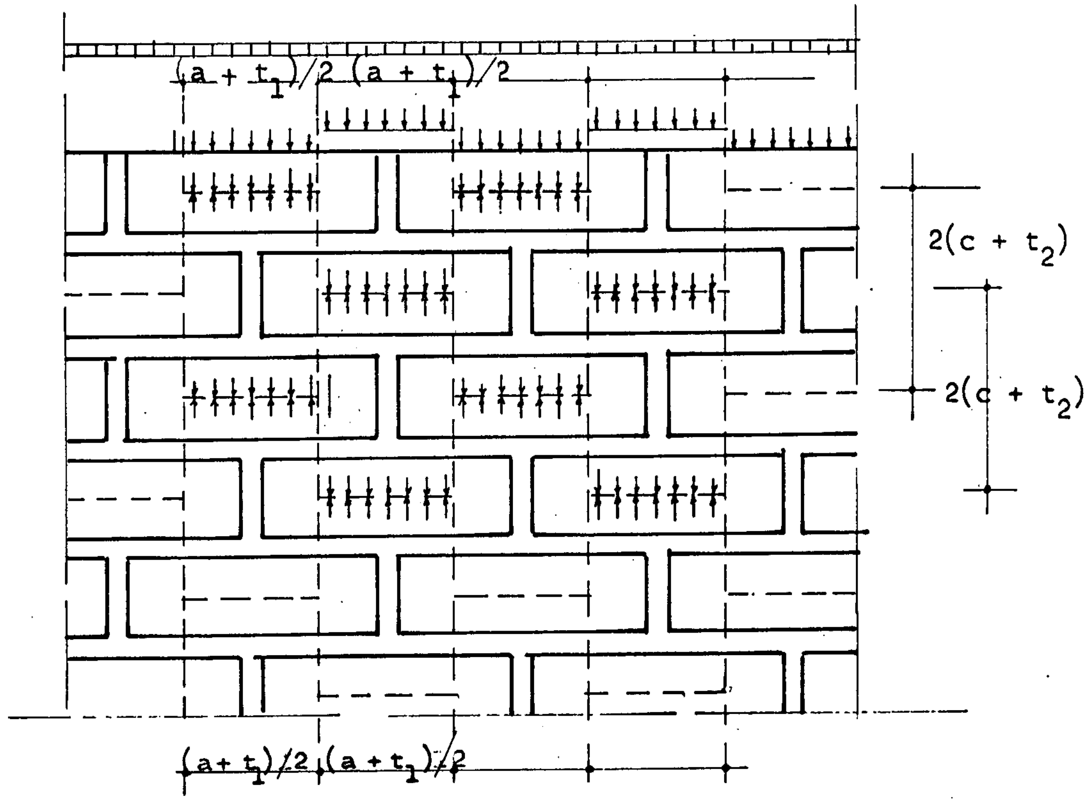
ably be, said that so long as the bricks are very stiff compared with the

mortar, the pressure on the brick in the first course can be divided into two parts as follows:

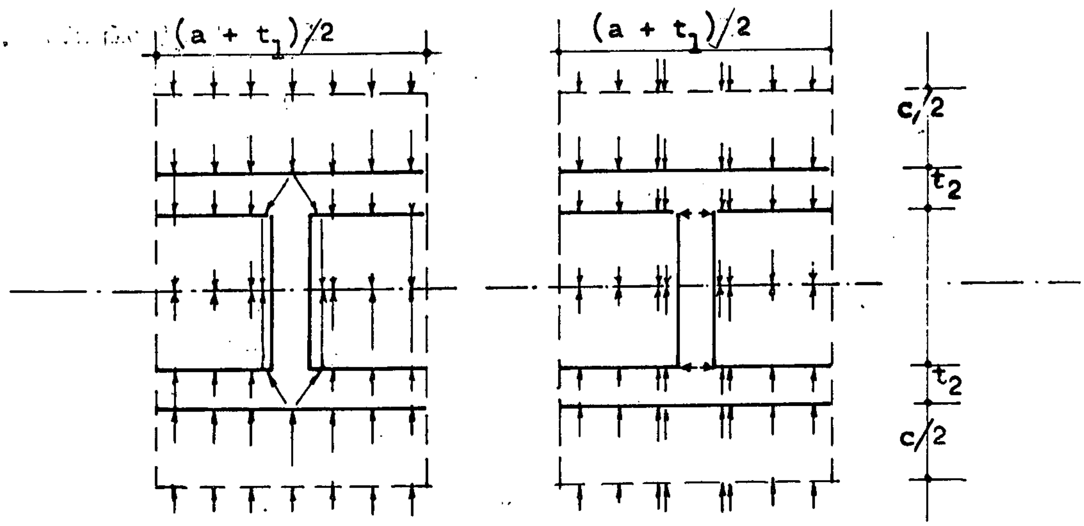
1. A major part over a width $(a - t_1)$. This part follows the bricks (Figure 2.1-b) in the first course, to the horizontal joint of mortar, to the two halves of bricks in the second course, to the horizontal mortar joint again, which in turn transmits it to the brick in the third course, similar in position to the first brick.

2. A smaller part of the pressure, namely the pressure over the width of the vertical joint (t_1) between the two halves of the bricks in the second course. Most of this part is not transmitted from the first to the third course through the vertical joint. Due to the fact that the elastic modulus of the bricks is much higher than that of mortar, this part follows inclined lines (Figure 2.1-C) of pressure to the bricks in the second course, then vertically through it, and again to the third course following a similar inclination, to be distributed on the brick in the third course, which is in a similar position to the first one.

If the wall is divided into similar and equal areas (Figure 2.2.-a) and these areas are considered as units then the forces acting internally on each unit can be treated exactly in the same manner as that mentioned above, as shown in Figure 2.2-b. The latter is again equivalent to Figure 2.2-C.



2.2.A. Single leaf brickwall divided into equal units.



2.2.b. Actual

2.2.c. Idealized for external pressure only (effect of squeezing out of horizontal joint not included)

FIGURE 2.2:

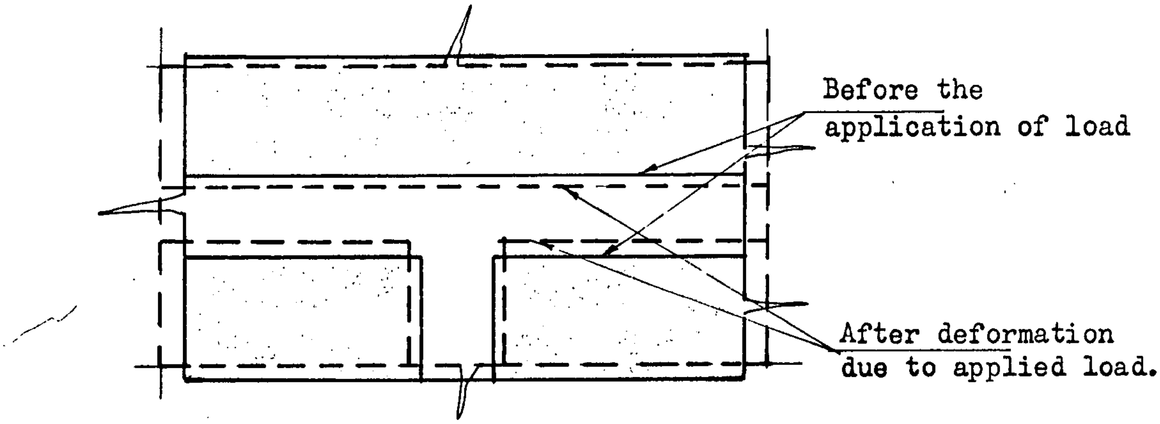
A single leaf brick masonry wall and the equivalent units.

Consider now the displacements and deformations due to the application of a vertical load. Both bricks and mortar expand laterally and contract vertically. Expansion is perpendicular to the direction of the applied load, while contraction is in its direction. Because of the assumed high stiffness of bricks relative to mortar, the former's intrinsic deformation under pressure would be comparatively small. But due to the very high co-efficient of friction, besides the bond acting between the bricks and the much less stiff mortar, lateral displacement in the brick takes place. The resulting displacement of bricks relative to their original positions can be represented as shown in Figure 2.3.-a, where the dotted lines take roughly the place of the full ones.

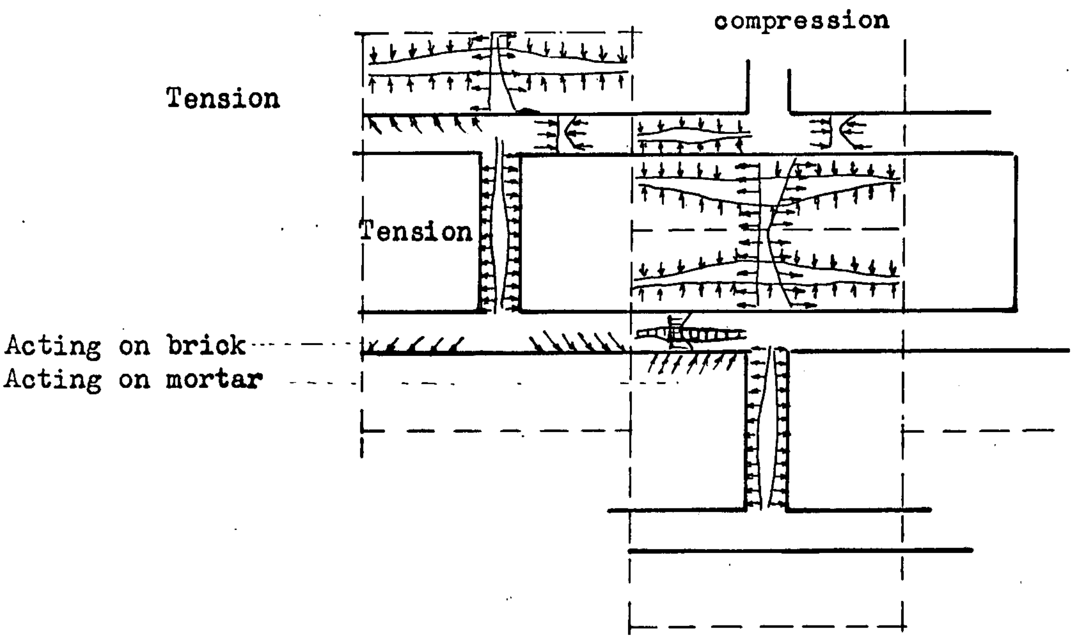
From the deformations due to these displacements, it is possible to deduce approximately the distribution of the forces acting either on the bricks or in the mortar joints. Figure 2.3-b shows the directions of the resultant forces. Actually each force at the contact surfaces between bricks and mortar at the horizontal joint is a resultant of external compression and tangential tension or compression. The former is for brick and the latter for mortar.

From these deformations and the approximate distribution of stresses the following can be easily noticed:

1. The tension in the brick is mainly due to the outward displacement of mortar in a direction perpendicular to the direction of the applied load.



2.3.a. Relative displacements of bricks due to uniformly distributed vertical pressure on the wall.



2.3.b. Approximate distribution of stresses inside bricks and mortar - deduced from the relative displacements.

FIGURE 2.3:

Internal deformations and distribution of stresses in a single leaf brick masonry wall.

2. Another lateral strain is caused in bricks, and this can be defined by Poisson's ratio of bricks. But due to the low stiffness of mortar compared to the high stiffness of bricks, this can be considered secondary. In this analysis it will be neglected.
3. Mortar is affected inversely as shown in Figure 2.3-b.
4. There is a probability that crushing takes place at the middle of the horizontal joint of the chosen unit. But again the mortar's effect on the brick is much pronounced than the reverse effect. This can be readily substantiated by the common mode of failure.

Considering now, one of the units together with the forces applied to it, it can be said that the deformations produced are approximately similar to the deformations of a closed portal frame as illustrated in Figure 2.4-a. The members of the frame have various widths and stiffnesses. The loads acting are approximately vertical pressure and four horizontal forces acting from inside in an outward direction. The frame cannot be in equilibrium without a tie at midheight or two ties near the supports. The force in the tie resembles, in the unit of the wall, an induced tension in the bricks and along the vertical mortar joint.

Therefore, it can be said that a uniformly distributed load on a single leaf wall produces primarily a vertical pressure, but the heterogeneity of the wall is responsible for inducing through this pressure horizontal forces. These forces try to pull each brick in opposite directions perpendicular

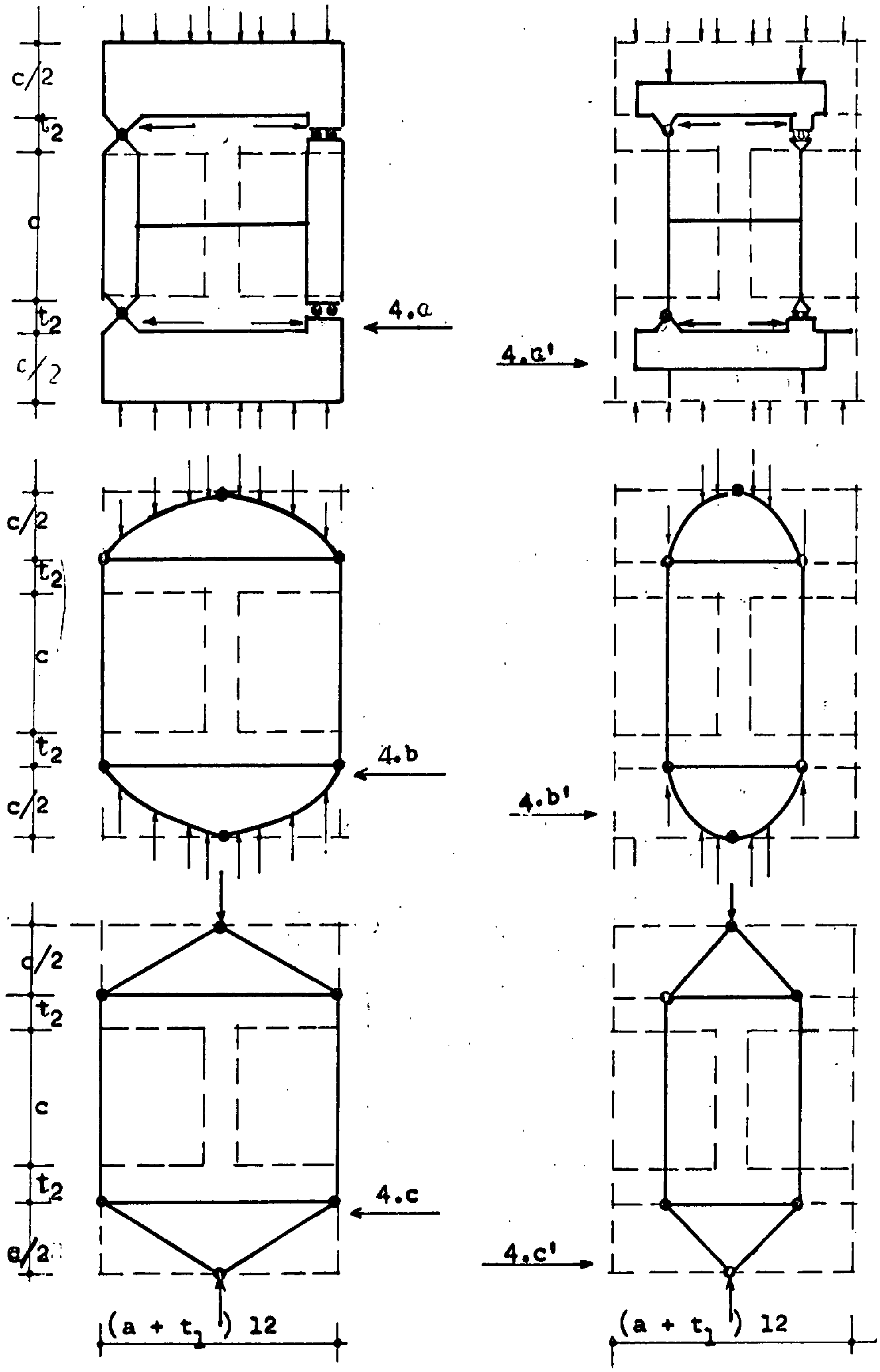


FIGURE 2.4:
Steps of idealization of the internal structural system of a single leaf brick masonry wall.

to the direction of the applied load, and at the same time try to increase the width of the vertical joints. The result is induced lateral stresses along the plane passing through the centre line of the vertical joints, and perpendicular to the plane of the wall. These lateral stresses decrease relatively at each vertical mortar joint and increase at each brick.

As the external vertical load increases, squeezing of the horizontal mortar joints increases, resulting in an increase in the outward pulling forces acting on the brick, and consequently an increase in the induced tensile stresses. At a certain limit the transverse pulling forces overcome the tensile strength of the brick material causing failure, starting along the vertical mortar joint, and then extending upwards and downwards dividing the bricks into two halves. This process occurs repeatedly in upward and downward directions along the same line producing ultimately the common vertical splitting of the wall under vertical compression. Naturally, the whole process can be repeated either at the same time of the vertical splitting or after it, with the result of two or more splits.

On the basis of the previous discussion it can be suggested that: the internal structure of a single leaf wall subjected to an external distributed pressure, approximates to a series of closed portal frames with ties as shown in Figure 2.4-a. Each frame inside the wall is subjected to a roughly vertical pressure and four horizontal forces acting in an outward direction. In the one frame, the stiffness of the members are not all the same, but the frames are themselves similar.

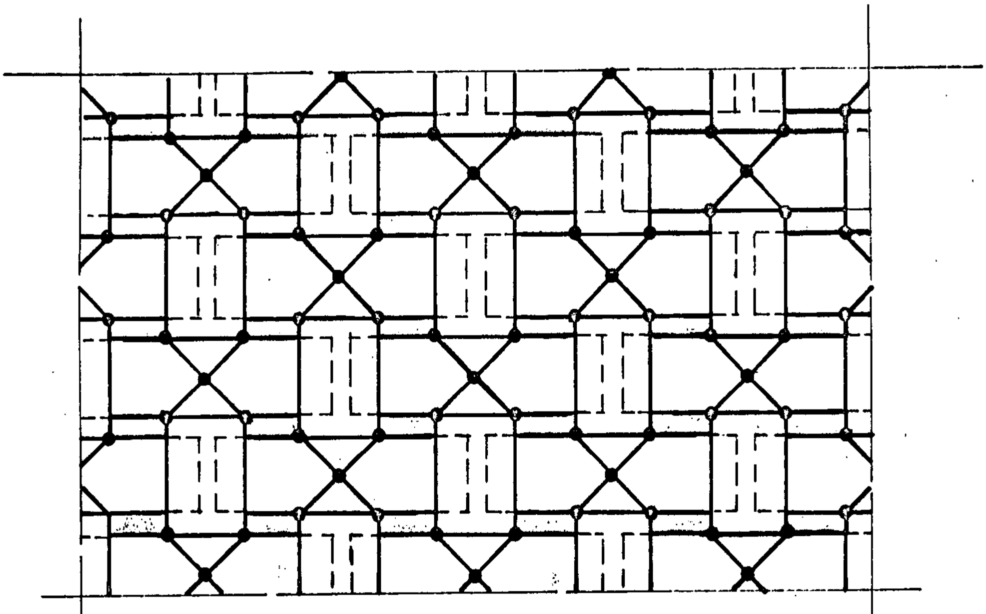
2.3.2. Idealized Internal Structural System

Looking toward a future investigation into the internal structure and failure characteristics of brickwork using a mechanical model, a major difficulty appears. This difficulty arises from the necessity of simulating the vertical pressure on one of the units of the wall by vertical forces, and four horizontal forces.

Searching for another representation that might be easier for this purpose, it seemed reasonable to replace the series of frames by a series of three-hinged arches connected with vertical members as shown in Figure 2.4-b. This again leads to the final idealized representation,that is a series of three-mesh-frames, Figure 2.4-c.

In order to make the oentre lines of the members and the centre lines of the elements of each unit coincide with each other, the three figures 2.4-a,b and c can be clearly developed respectively to be 2.b-a, b and c.

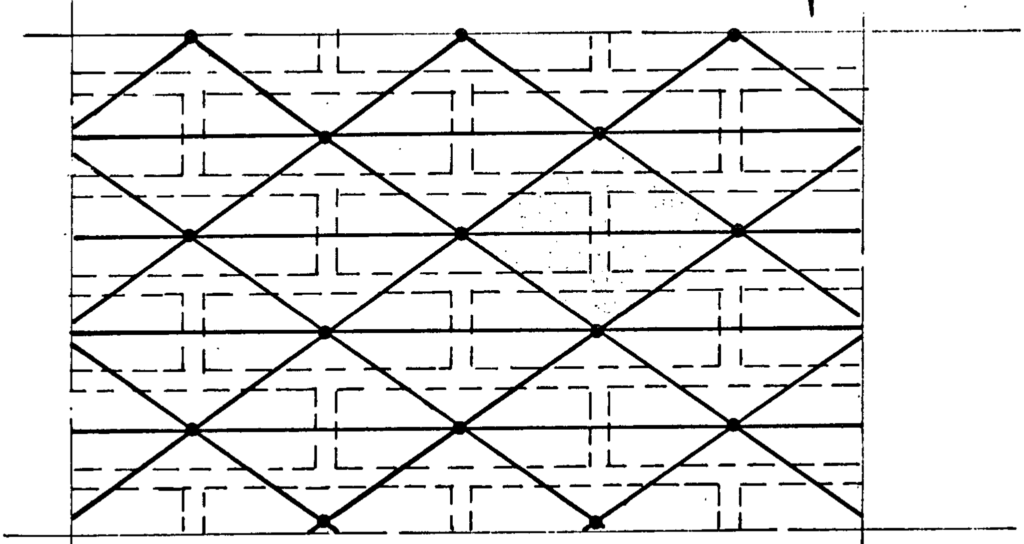
Finally, it is proposed that the internal structure of a single leaf wall be approximately represented by a series of open-mesh frames, each frame consisting of three openings. The dimensions of the frame differ according to the dimensions of the bricks and the thicknesses of the mortar joints. Figure 2.5.-a shows a single leaf wall and its idealized internal structure.



Idealised internal structure system in the form of three-space mesh frames.

Original masonry wall.

Idealised internal structure system



Idealised internal structure system in the form of one-space lattices with ties.

FIGURE 2.5:

Analysed cases of idealised-internal-structure system.

2.4 SIMULATION BETWEEN THE DIMENSIONS OF THE PROPOSED THEORETICAL STRUCTURAL SYSTEM AND THE INTERNAL STRUCTURAL SYSTEM OF THE WALL

The object of this section is to deduce the resultant forces which act on each of the units, and the resultant forces acting on each of the members of the unit. The former are related to the external pressure on the wall, while the latter are related to the internal forces inside the wall. However, each force is equal to area x stress.

Referring to Figure 2.b-c the single members and the forces acting on each unit are:

1. Two vertical members with vertical compression. The vertical compression is on an area of width $(a - t_1)/4 \times 2$ and length "b".

2. A horizontal member with horizontal tension. The tension is made up of two parts, as follows:

a. The major part due to squeezing out of mortar acting on an area of width "c/2" and length "b."

b. A minor part, due to pushing out of the halves of bricks, according to the lines of pressure. It acts on an area of width "c" and length "b".

3. If p is the unit pressure per unit area of the wall then the resultant force on each unit of a three-mesh frame P = $(a + t_1)/2 \times b \times p$

2.5 ANALYSIS OF THE PROPOSED STRUCTURAL FRAME SYSTEM

2.5.1. General

In order to determine the deformations and stress distribution resulting from the applications of external pressure on the wall by a frame model, the following are basic and helpful assumptions.

1. The behaviour of the masonry is approximately similar to the idealized mesh system.

2. The frame model structure used has members of appropriate cross sections and stiffnesses, designed so that each unit has the same relative vertical contraction and horizontal extension as the masonry assemblage.

These assumptions produce the following:

1. The model will reproduce on the unit scale the deformation of the assemblage.

2. Consequently from one, the distribution of the critical tensile stresses causing failure might be eventually studied by the aid of the model.

2.5.2. Analysis

The total load acting on an open-frame unit is deduced as:

$$P = \left(\frac{a + t_1}{2}\right) \times b \times p \tag{2.1}$$

For vertical equilibrium we have:

$$\left(\frac{a + t_1}{2}\right) \times b \times p = 2\left[\left(\frac{a - t_1}{4}\right) \times b \times \delta_1\right]$$

$$\text{i.e.} \qquad \delta_1 = \left(\frac{a + t_1}{p - t_1}\right) \times p \tag{2.2}$$

Assuming the tension over the height of the brick, and in the vertical joint, each individually to be uniform, and including the latter part in the former we get:

$$\left(\frac{a + t_1}{2}\right) \times b \times p \times \tfrac{1}{2} \times \tan\alpha = \frac{c}{2} \times b \times \delta_2 \tag{2.3}$$

But we have:

$$\tan\alpha = \left(\frac{a - t_1}{8} + \frac{t_1}{2}\right) / \left(\frac{c}{2}\right) = (a + 3t_1)/4c \tag{2.4}$$

Substituting from (2.4) in (2.3) we get:

$$\delta_2 = \frac{(a + t_1)\ (a + 3t_1)}{8\ c_2^2} \times p \tag{2.5}$$

Assuming that

$$\delta_1 = E_1 \times \epsilon_1 \tag{2.6}$$

$$\delta_2 = E_2 \times \epsilon_2 \tag{2.7}$$

Also if the vertical contraction of the diagonal member is neglected due to the fact that it is very stiff, as assumed at the beginning, then the vertical strain in the whole unit will be qual to the vertical strain in the vertical member. Consequently we get:

$$\gamma = \epsilon_2 / \epsilon_1 \tag{2.8}$$

Substituting from (2.6) and (2.7) in (2.8) we get:

$$\gamma = \frac{\delta_2}{\delta_1} \times \frac{E_1}{E_2} \tag{2.9}$$

Substituing from (2.2) and (2.5) in (2.9), we get:

$$\gamma = \frac{(a + 3t_1)(a - t_1)}{8c^2} \times \frac{E_1}{E_2} \tag{2.10}$$

Assuming $p/\delta_2 = n$, and substituting from (2.5) then:

$$n = \frac{8\,c^2}{(a + t_1)(a + 3\,t_1)} \tag{2.11}$$

Substituting from (2.11) in (2.10) we get:

$$\gamma = \frac{(a - t_1)}{(a + t_1)} \times \frac{1}{n} \times \frac{E_1}{E_2} \tag{2.12}$$

$$R = \frac{(a + t_1)}{(a - t_1)} \quad x \quad n \quad x \quad \frac{E_2}{E_1} \qquad (1.13)$$

2.6 DISCUSSION OF THE ANALYTICAL RESULTS

2.6.1. Relation Between Poisson's Ratio and E_2/E_1

It can be seen from equation 1.12 that for any given value of "n", Poisson's ratio increases as the ratio of E_1/E_2 increases. Generally it is conceivable to expect that the higher the ratio of E_2 in the cross section of the brick (beside the vertical joint) to E_1 in the horizontal joint the lower will be the value of ν.

This can be considered a general relationship between E_1/E_2 for all stages of loading before the failure of the horizontal tie of the frame which stimulates in a wall the vertical sections of halves of the bricks above and below the vertical joint. How far does this ratio change during loading? This is discussed in the following paragraphs.

Actually this ratio can be considered an uncertain term at this stage. While there is some data on Young's modulus for mortar under compression this modulus has never aroused attention for bricks either under compression or tension. Therefore any assumptions on this behaviour, now, can be done only on the basis of speculation.

As the pressure on the wall increases E_1 decreases. This is due to the fact that E_1 is a function of σ_1 which is again a function of the vertical pressure. It will decrease to a value which might be about one quarter of its initial value. This is based on the usually stress-strain curve for concretes and mortars.

On the contrary E_2 which is a function of δ_2 which is again a function of the squeezing out of the mortar, might be reduced to a much smaller extent. At this point it emerges clearly that experimental investigation of this hypothesis is far from being possible at present because of the extensive work required.

2.6.2 Relation Between Poisson's Ratio, R and n

There is no, or only very little, data on the ratio between the compressive strength of brickwork and the tensile strength of bricks or mortars. Therefore, the range for this ratio which can be applied in the present case is also vague. It is expected that as the pressure on the wall increases the induced tensile stresses increase with the result of a decrease in the ratio of the former to the latter. Referring to Equation 2.12 it can be seen that ν increases as "n" decreases.

2.6.3 Deformation of Brick Masonry and the Individual Properties

It has just been mentioned that Poisson's ratio may generally increase as "n" decreases. Considering the individual properties which can affect the value of "n" more than one factor can be found. In order to recognize these factors we have to consider the lateral stresses in brick masonry and correlate them with δ_2. This may be written in the following form:

$$\frac{c}{2} \times b \times \epsilon_b \times E_{tb} + \frac{t_1}{2} \times b \times p \times \frac{1}{2} \times \frac{t_1}{2} \times \frac{1}{t_2} = \frac{c}{2} \times b \times \delta_2 \qquad (2.14)$$

Because there is no slipping between bricks and mortar at the horizontal joints, and considering the lateral expansion in bricks resulting from the vertical pressure as a negligible term we get:

$$\epsilon_b = \epsilon_m = \frac{p}{E_m} \times \nu_m \qquad (2.15)$$

Substituting from (2.15) in (2.14) we get:

$$\delta_2 = \frac{E_{tb} \times \nu_m}{2\,E_m} + \frac{t_1^2}{4\,c \times t_2} \qquad (2.16)$$

From Equation (2.16) we find that, from the mortar's side, an increase in ν, and E_{tb}/E_m results in a higher value of δ_2, lower values of "n" and higher values of Poisson's ratio of masonry. In other words for minimum lateral strain, which will lead to delay in splitting, the mortar must be of a minimum ν_m value, and a maximum E_m value.

2.6.4 Poisson's Ratio and the Thickness of the Vertical Joint

The effect of the thickness of

the vertical mortar joint can be easily noticed from Equation (2.16). For a minimum value of δ_2, t_1 must be equal to zero. Consequently "n" can have a maximum value, with the result of minimum value for the Poisson's ratio of brick masonry. Although this is practically impossible it is theoretically conceivable.

2.7 FURTHER DEVELOPMENTS TOWARDS SIMPLER IDEALIZED INTERNAL SYSTEMS

As a further development towards a more simpler internal structural system, Professor Hendry suggested the unit shown in Figure 2.5-b as an alternative to the previous idealized unit. It is quite clear thatall the steps mentioned before (the relative displacements, deformation characteristic, ... etc.) are applicable here in the same manner. Finally, the proposal given in 2.3.2. may be replaced by the following one:

"The internal structure of a single leaf masonry wall may be represented by a series of lattices. Each lattice is of one opening with a horizontal tie at the middle. The dimensions of the lattice differ according to the dimensions of the bricks and thicknesses of the mortar joints. Figure 2.5-b shows a single leaf wall with its internal system idealized in the form of lattices.

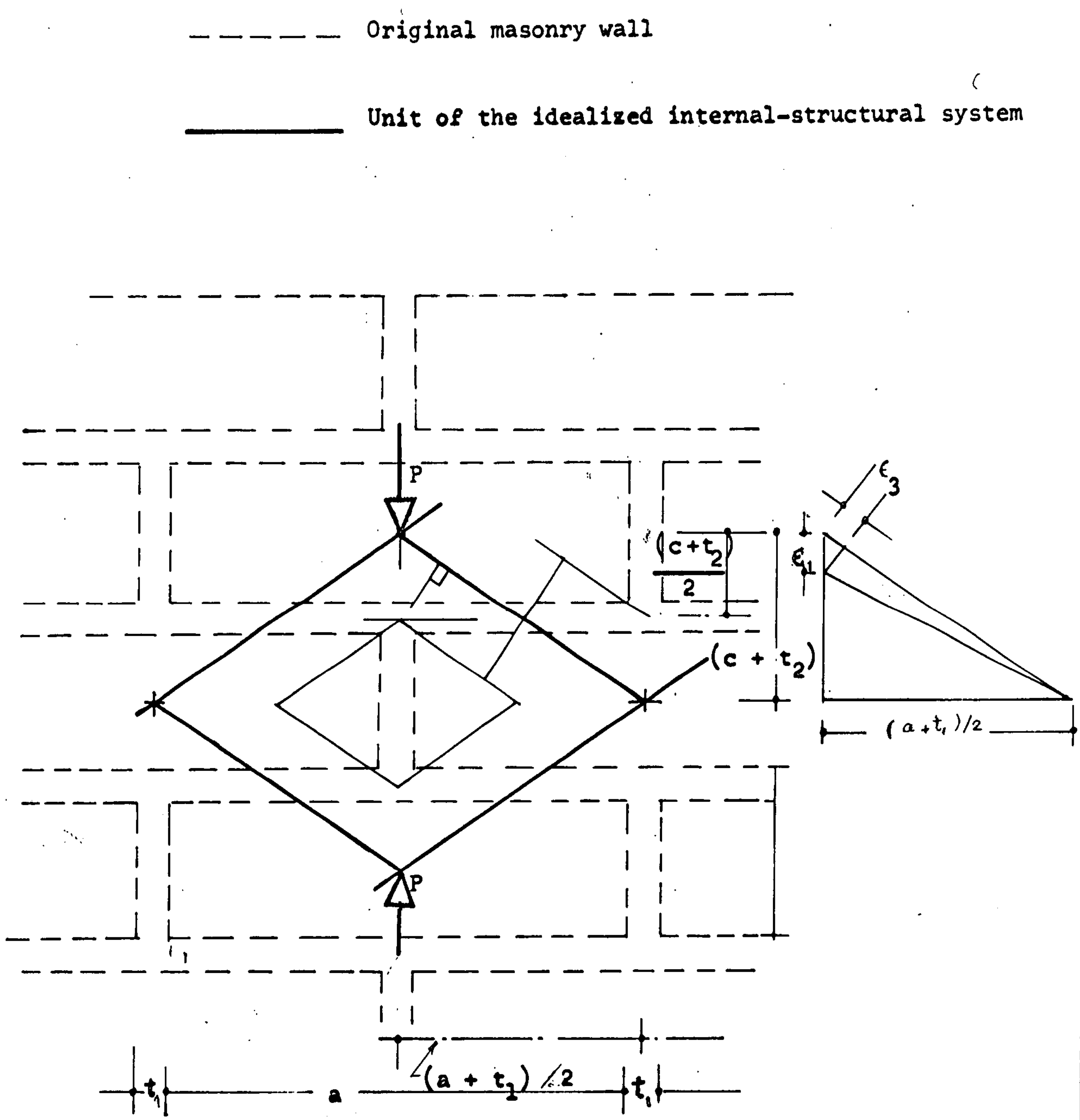


FIGURE 2.6:

A detailed one-space lattice with a tie

With the same assumptions as in 2.1.1, 2.5.2 and referring to Figure 2.6, the analysis can be carried out as follows:

$$P = \frac{a + t_1}{2} \times b \times p \qquad \text{resembles 2.1} \qquad (2.17)$$

For vertical equilibrium:

$$\left(\frac{a + t_1}{2}\right) \times b \times p = 2\left(\frac{c}{2} + \frac{t_2}{2}\right) \sin\alpha \times b \times \delta_3 \quad \text{or}$$

$$\delta_3 = \frac{a + t_1}{2(c + t_2)} \times \frac{1}{\sin\alpha} \times p \qquad \text{resembles 2.2} \qquad (2.18)$$

For horizontal equilibrium:

$$\left(\frac{a + t_1}{2}\right) \times b \times \tfrac{1}{2} \times p \times \tan\alpha = \frac{c}{2} \times b \times \delta_2 \quad \text{or}$$

$$\delta_2 = \left(\frac{a + t_1}{2c}\right) \times \tan\alpha \times p \qquad \text{resembles 2.3} \qquad (2.19)$$

But we have:

$$\tan\alpha = \frac{a + t_1}{2(c + t_2)} \qquad \text{resembles 2.4} \qquad (2.20)$$

$$\epsilon_3 = \delta_3/E_3 \qquad (2.21)$$

$$\epsilon_2 = \delta_2/E_1 \qquad (2.22)$$

$$\epsilon_1 = \epsilon_3/\cos\alpha \qquad (2.23)$$

Neglecting the contraction in the vertical direction resulting from the extension of the horizontal diagonal we get:

$$\nu = \epsilon_2/\epsilon_1 \qquad (2.24)$$

Substituting from Equations 2.19 - 2.23 in Equation 2.24 we get:

$$\nu = \frac{\sigma_2}{\sigma_3} \times \cos\alpha \times \frac{E_3}{E_2} \qquad \text{or}$$

$$\gamma = \frac{(c + t_2)}{c} \times \sin^2\alpha \times \frac{E_3}{E_2} \qquad (2.25)$$

Assuming $p/\sigma_2 = n$ and substituting in a similar manner as before we get:

$$\nu = \frac{1}{\left\{\left[\frac{(a+t_1)}{2(c+t_2)}\right]^2 + 1\right\}} \times \frac{1}{n} \times \frac{E_3}{E_2} \quad \text{resembles 2.12} \qquad (2.26)$$

$$R = \left\{\left[\frac{(a+t_1)}{2(c+t_2)}\right]^2 + 1\right\} \times n \times \frac{E_2}{E_3} \quad \text{resembles 2.13} \qquad (2.27)$$

Following this it is possible to make other representations for the brick masonry wall as illustrated in Figure 2.7.

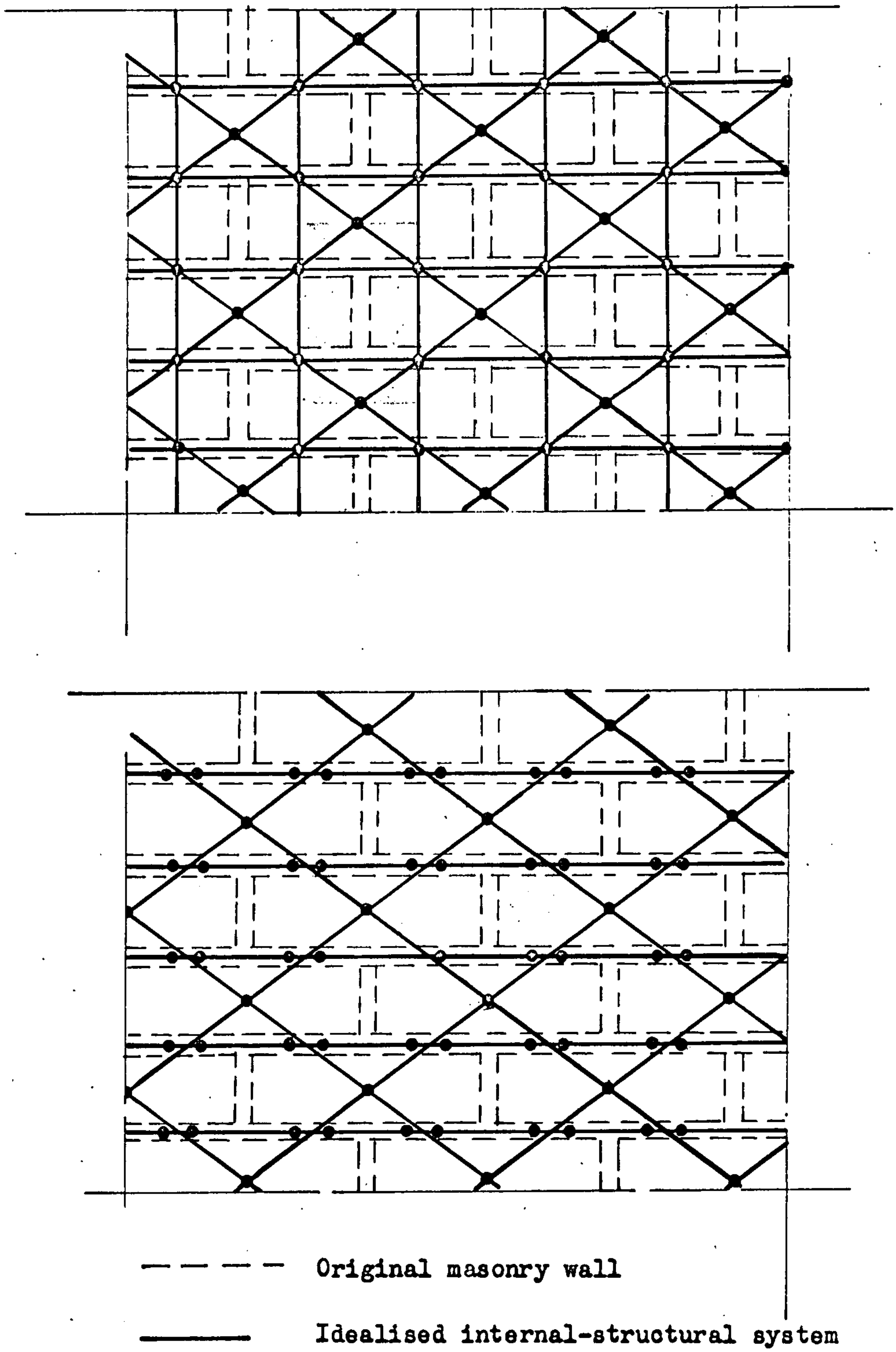


FIGURE 2.7:

Other possible idealisations for the internal structure system of a single leaf wall.

C H A P T E R 3

THEORETICAL ANALYSIS OF DEFORMATION AND FAILURE CHARACTERISTICS OF BRICK MASONRY BASED ON ACTION AND INTERACTION BETWEEN COMPONENTS

3.1 ORIENTATION

In this chapter an analytical investigation is attempted from an angle different to that in Chapter 2, namely by allowing the actual play of forces in individual components and the internal stress and deformation set up to be determined in terms of the individual properties of the bricks and mortar. Expressions for the modulus of elasticity, critical cracking load as a function of mortar properties only, and the ultimate failure load are derived and discussed.

3.2 NOTATION

a, b, c, p	are the same terms given in the notation of Chapter 2.
ν_b, E_b	: Poisson's ratio and modulus of elasticity for bricks.
E_{tb}	: Modulus of elasticity of bricks in tension.
ν_m, E_m	: Poisson's ratio and modulus of elasticity for mortar.
R_m	: $1/\nu_m$
ϵ_b, ϵ_m, ϵ_s	: Strain in brick, mortar and assemblage respectively.
f_{cm}, f_{tm}	: Compressive and tensile strengths of mortar respectively.
E_s	: Modulus of elasticity of assemblage.
ϵ_s	: Strain in assemblage
p_1	: Compressive vertical stress acting on the brick through the mortar, or stress developed between brick and horizontal mortar joint. (See Figure 3.2).

p_2 : Stress developed between horizontal part "H" and vertical part "V", of mortar strip. (See Figure 3.2).

p_3 : Tensile stress developed between brick and vertical joint along the interface of contact. (See Figure 3.2).

3.3 DEFORMATION BEFORE INITIAL CRACKING

Let the wall be divided into equal areas or units. The simplest one will be as shown in Figure 3.1, a brick surrounded by a strip of mortar having the same breadth as the brick. The thickness of the strip in the horizontal and vertical directions will be respectively half of the corresponding joints. Let the strip also be divided into four parts, two horizontal "H" and two vertical "V".

When the load is applied both mortar and bricks deform. Considering the lateral deformation while keeping in mind the assumption that bricks are rigid relative to mortar, the two horizontal parts "H" will tend to expand laterally more than the brick. By this expansion, the two vertical parts "V" become subject to horizontal forces pushing them away from the bricks. Consequently tensile stresses are induced between the brick and the two vertical parts of the mortar strip, and along the interface between them. For the vertical part "V" the horizontal forces and stresses are in equilibrium. Considering the individual deformations they can be given as follows:

1. Lateral strain in brick alone $= \frac{p_1}{E_b} \cdot \nu_b + \frac{p_3}{E_b}$

$$= \frac{1}{E_b} (p_1 \cdot \nu_b + p_3) \qquad (3.1)$$

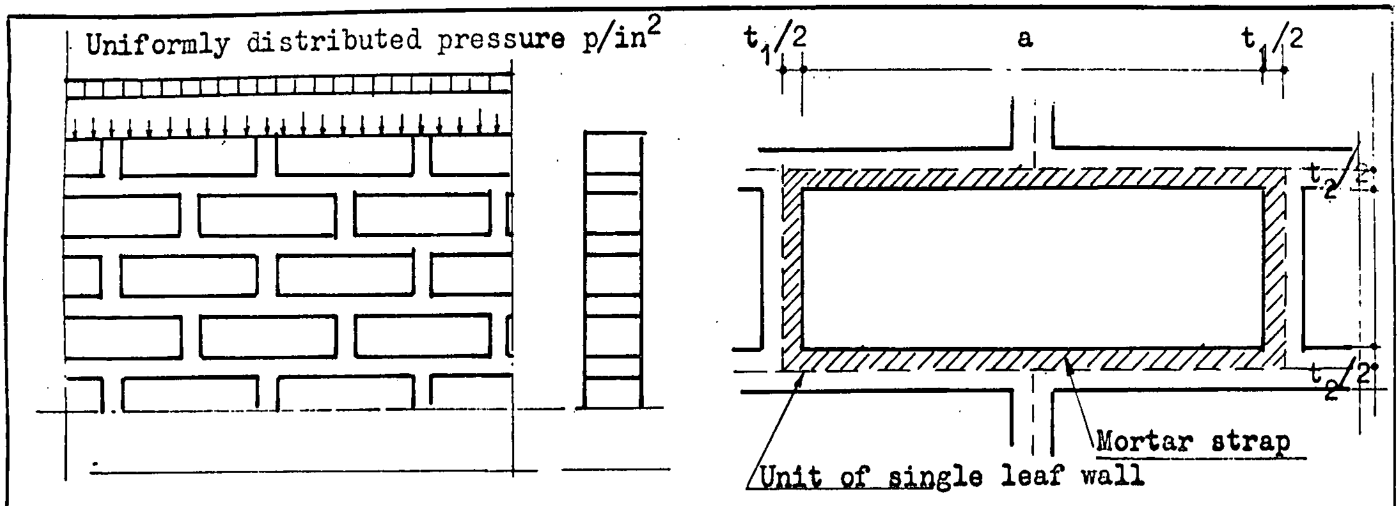
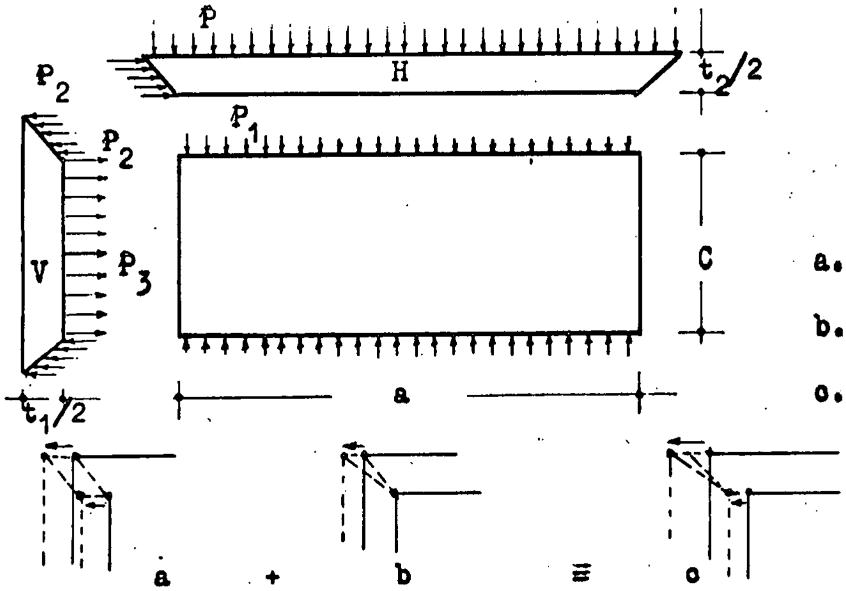


Figure 3.1 A single leaf wall brick masonry wall and the simplest unit



a. Position due to lateral strain in brick alone.
b. Position due to lateral strain in vertical mortar V.
c. Superposition of a and b (See equation 3.4).

Figure 3.2 Forces acting on individual parts of the unit

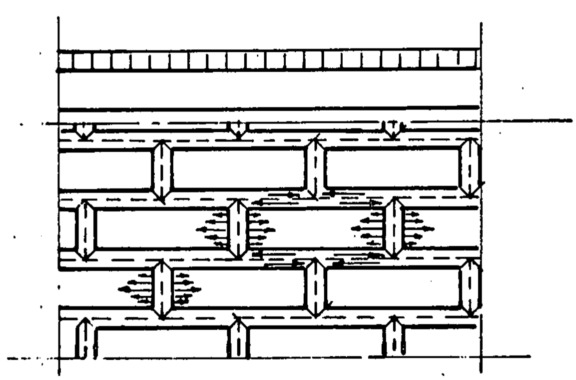


Figure 3.3 Lines of movement of mortar joints before initial cracking.

2. Lateral strain in mortar part "V" =

$$\frac{p_3}{E_m} + \frac{p \cdot t_1/2}{E_m} \cdot \nu_m \tag{3.2}$$

The second part is relatively small, so that it will be neglected in the following calculations. Also the effects of the horizontal parts "H" (of negative sign) at the top and bottom are considered negligible.

3. Lateral strain in mortar part "H" =

$$\frac{p}{E_m} \cdot \nu_m - \frac{p_2}{E_m}$$

$$\frac{1}{E_m}(p \cdot \nu_m - p_2) \tag{3.3}$$

At the stage of loading before the occurrence of any sign of cracking, the total lateral strain of the mortar horizontal part "H") is equal to the sum of the lateral deformations of the brick and the two mortar vertical parts "~~H~~". By super-position, as illustrated in Figure 3.2 and from equations 3.1-3.3 we get:

$$\frac{1}{E_m}(p \cdot \nu_m - p_2)(a + t_1) = \frac{1}{E_b}(p_1 \cdot \nu_b + p_3) \cdot a + \frac{p_3}{E_m} \cdot t_1 \tag{3.4}$$

For lateral equilibrium of one of the vertical parts "V" we have:

$$2\left(\frac{t_2}{2} \times b\right) \cdot p_2 = (b \times c) \cdot p_3 \qquad \text{or}$$

$$p_3 = \frac{p_2 \times t_2}{c} \tag{3.5}$$

From equations (3.4) and (3.5) we get:

$$\frac{1}{E_m}(p \cdot \nu_m - p_2)(a + t_1) = \frac{1}{E_b}\left(p_1 \cdot \nu_b + \frac{p_2 \times t_2}{c}\right) a + \frac{p_2 \times t_2}{c} \cdot \frac{1}{E_m} \cdot t_1$$

$$p = \frac{\frac{1}{E_b}\left(p_1 \cdot \nu_b - \frac{p_2 \times t_2}{c}\right) a + \frac{p_2 \times t_2}{c} \cdot \frac{t_1}{E_m} + \frac{p_2 (a + t_1)}{E_m}}{(a + t_1)\,\frac{\nu_m}{E_m}} \qquad (3.6)$$

Considering the vertical strains in the parts of the unit they are as follows:

1. Vertical strain in brick $= \frac{p_1}{E_b} + \frac{p_3}{E_b} \cdot \nu_b \cdot 2$

$$= \frac{1}{E_b}(p_1 + 2 p_3 \cdot \nu_b) \qquad (3.7)$$

2. Vertical strain in mortar horizontal parts "H" =

$$\frac{p}{E_m} - \frac{p_2}{E_m} \cdot \nu_m \cdot 2$$

$$= \frac{1}{E_m}(p - 2 p_2 \cdot \nu_m) \qquad (3.8)$$

From equations (3.7) and (3.8) we obtain the total vertical strain in the unit as:

$\epsilon_{s\ (vertical)}$ =

$$\frac{1}{E_b}(p_1 + 2 p_3 \cdot \nu_b) + \frac{2}{E_m}(p - 2 p_2 \cdot \nu_m) \qquad (3.9)$$

From equations (3.6) and (3.9) we

the modulus of elasticity of brick masonry as:

$$E_s = \frac{P}{\left[\dfrac{(P_1 + 2\,P_3 \cdot \nu_b)}{E_b} + \dfrac{2\,(P - 2\,P_2 \cdot \nu_m)}{E_m}\right]} \tag{3.10}$$

3.4 FIRST STAGE OF FAILURE AND CRITICAL CRACKING LOAD

As the external pressure on the wall increases, the horizontal thrust on both "V" parts increases, and, accordingly, the induced tensile forces alone the vertical joint increases. At a certain load a state of equilibrium no longer exists and failure may occur in one of the following ways:

1. If bond between the brick and mortar along the vertical joint is less than the tensile strength of mortar failure occur in bond of mid-height of the vertical joint.
2. If bond is stronger than the mortar tensile-strength failure occurs in tension at mid-point of the vertical axis of the vertical joint.

In either case this may be the start of the first stage of cracking. The distribution of stresses along the vertical mortar joint is undoubtedly non-uniform. Most probably it has the nature shown in Figure 3.3 with a maximum value at mid-height of the joint.

As the load increases the state of non-equilibrium continues, producing elongation in the crack. When the crack reaches the mid-height of the horizontal joints, it is assumed that the first stage of loading ends and a second stage of failure begins. Before shifting to the latter the first should be defined in terms of measured values.

Examining Equation 3.5 it can be seen that if the average values for p_2 and p_3 are considered, the crack is initiated when:

$$p_2 = \frac{c}{t_2} \cdot f_{tm} \tag{3.11}$$

But p_2 is produced as a result of the lateral strain in the horizontal mortar joint. Therefore:

$$p_2 = p \cdot \nu_m \tag{3.12}$$

From Equations (3.11) and (3.12) we get the following expressions for the first stage of failure:

$$p = \frac{c}{t_2} \cdot f_{tm} \cdot \frac{1}{\nu_m} \tag{3.13}$$

$$p = \frac{c}{t_2} \cdot f_{tm} \cdot R_m \tag{3.14}$$

It is proposed here to introduce a new term, the "critical cracking load". It may be defined as: the load at which a wall, reaching according to the assumptions incorporated in the present analysis, is cracked along the vertical mortar joint. Its value can be determined by either of the equations 3.13 and 3.14. General comments can be made as follows:

1. It can be seen that the critical cracking load is reached at a certain stage before complete failure: this will be discussed later. The zone between cracking and complete failure is small or large depending on the mortar and brick properties

and their relative dimensions, but the cracking load itself is a function of the mortar properties alone.

2. Although the formula expresses the cracking load, the author is inclined to consider it as a reliable indication of initial failure, when bricks of high rigidity are used. Here it can be claimed that it is not necessary to consider that a wall with such a crack is near failure, and that a wall with empty vertical joints can stand a considerable load not much lower than one having full vertical joints. In other words, the formula may indicate misleading values. The answer to this is that it should be remembered that the cracking load is reached because of the deformations of the horizontal joints. Therefore if a wall with empty vertical joints is loaded, deformation of the horizontal mortar joints will take place with no influence from the horizontal parts "H" to the vertical parts "V", due to the non-existence of the latter. At a certain stage the unit will be in the same condition as the unit of a wall with full vertical joints, from which the load was defined.

3. It can be seen that the value of the load is governed on the mortar's side, by its Poisson's ratio and tensile strength, provided other factors are invariable.

4. For the same mortar properties, the formulae show that the load is increased with increase of ratio c/t_2. In other words the load is increased as the brick height increases and the thickness of the horizontal mortar joint decreases.

5. The expressions give a theoretical explanation of the

results usually met within non-cementitious mortars. In reference (39), the tensile strength of the Sarabond mortar and the conventional mortar 1:1:6 were given respectively as 950 and 340 lb/in^2. Substituting from these values in the above expressions justifies the statement made in the same reference, that the strengths of load-bearing brick walls bear the ratio 4:1 (Sarabond: conventional).

3.5 SECOND STAGE OF FAILURE AND ULTIMATE FAILURE LOAD

This stage is considered to begin when the tensile strength of the mortar along the distance of the vertical joint between the centre lines of the horizontal joints, is reached. In this manner the forces acting on the unit start to be transmitted in a different way. As explained before in Chapter 2 the lines of force are assumed to be transmitted as shown in Figure 3.4. In other words the vertical mortar joint is structurally not working, and p_2, p_3 no longer exist.

As the load increases squeezing out of mortar increases, while it is affected not only by the increase in load but also by the high pressure resulting from the present lines of forces. As regards the brick it is assumed, for simplification with the present wall, that it is subjected to a state of biaxial stresses, and that the vertical pressure on the brick is uniform.

If we still assume that the intensity

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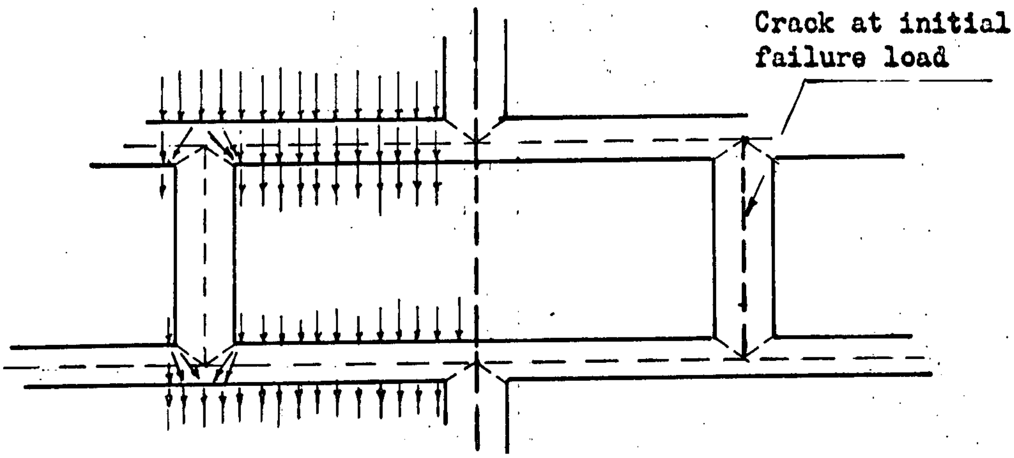
(39): The Dow Chemical Company. Sarabond. A.I.A. File No. 3.L.8 (NN). Michigan. U.S.A.



Lines of ultimate failure load (vertical splitting)

Figure 3.4
Lines of transmission of forces just after initial cracking

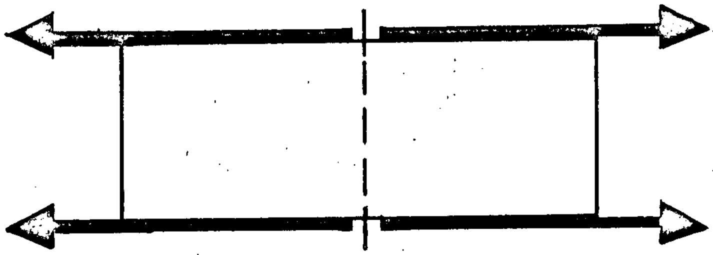


Figure 3.5
Nearest sketch for a brick and the lateral forces acting on it.

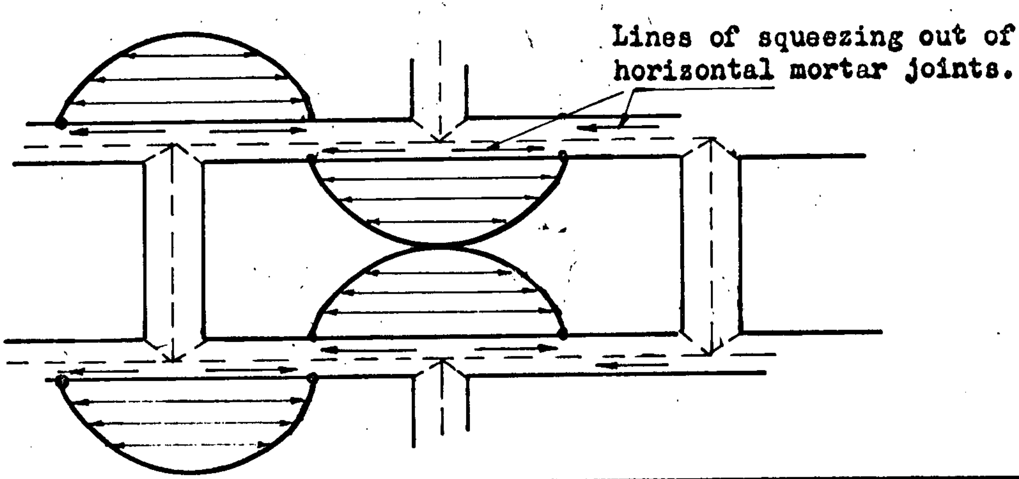


Figure 3.6
Possible distribution of lateral stresses acting on bricks.

of pressure is uniform we get:

Lateral strain in brick =

$$p \times \left(\frac{a + t_1}{a}\right) \times \frac{1}{E_b} \times \gamma_b \tag{3.15}$$

Lateral strain in mortar horizontal part "H" =

$$p \times \left(\frac{a + t_1}{a}\right) \times \frac{1}{E_m} \times \gamma_m \tag{3.16}$$

Before the occurrence of vertical cracks, or for equal lateral extensibility in brick and mortar Equations (3.15) and (3.16) give the following condition:

$$\frac{\gamma_b}{E_b} = \frac{\gamma_m}{E_m} \tag{3.17}$$

As the load increases the lateral extansion in both brick and mortar increases. Then at a certain limit the lateral movement of mortar becomes excessive, and due to the fact that friction between brick and mortar is of a high value in addition to the bond, the brick's maximum extensibility is overcome by the mortar's extensibility. At this limit the overall vertical splitting starts to propagate in both upwardsand downward directions along the line of crack, very rapidly. It is suggested that a brick in this stage of loading is very similar to a briquette under direct tension, as illustrated roughly in Figure 3.5. A more precise diagram for the internal structure and the tensile stresses in bricks is suggested in Figure 3.6.

Considering the conditions at failure we get:

$$p \times \left(\frac{a+t_1}{a}\right) \times \frac{1}{E_m} \times \nu_m - p \times \left(\frac{a+t_1}{a}\right) \times \frac{1}{E_b} \times \nu_b = \frac{f_{tb}}{E_{tb}} \quad \text{or}$$

$$\boxed{p_{ultimate} = \frac{\left(\frac{a}{a+t_1}\right)\left(\frac{f_{tb}}{E_{tb}}\right)}{\left(\frac{\nu_m}{E_m} - \frac{\nu_b}{E_b}\right)}} \qquad (3.18)$$

Equation (3.18) may be considered as the basic equation for the strength of a single leaf brick masonry wall, in terms of the properties of components and their relative dimensions. General remarks are:

1. At this stage the mortar's structural properties influencing the strength emerge as its deformation properties (E, ν).

2. For the bricks, it is the deformation properties, the tensile strength, and the modulus of elasticity in the lateral direction.

3. The influence of any of the factors incorporated in Equation (3.18) towards increasing or decreasing the value of ultimate strength can be easily noticed, in a similar manner to the cracking load.

4. Considering Figure 3.6 which indicates roughly the internal structural system at this stage and consequent

distribution of tensile stresses along the brick's height, the latter emerges indirectly as of importance. As the height increases, the average value for the tensile stress "f_{tb}" decreases, consequently the ultimate strength increases.

5. The formula gives also theoretical support to the previous experimental data; maximum failure load is obtained when the thickness of the vertical joint is zero.

C H A P T E R 4

EVALUATION OF THEORETICAL ANALYSIS AND CONSIDERATIONS FOR AREAS OF WORK

A short evaluation may be given as follows:

1. The idealized approach can be considered an advance on the action and interaction approach in the search for a complete deformation theory, in that it tries to account for deformation characteristics in both directions. At the same time it can represent deformation only by mechanical models, and tell us very little about the ultimate failure strength.

2. Although the case considered with the idealized approach was two-dimensional, there is clear evidence that it can be extended to the three-dimensional case. To clarify this a phenomenon which is usually met within the failure of brick masonry piers should be remembered. It is well known that the mode of failure of a pier under uniaxial compression is the vertical splitting into four more or less equal parts. This indicates what happens with a single leaf wall, can happen also with a pier, but in two directions perpendicular to each other. In other words it may be expected that the internal structural system of a pier can be represented by two series of mesh frames or open lattices. The two series are in two directions perpendicular to each other.

3. The action-and-interaction approach, on the other hand,

can be considered an advance on the idealized one in that it yielded formulae which may have much-needed useful application, especially, after the mechanism of failure has been more completely visualized.

4. In a similar manner to "3" the action and interaction approach may be extended to the three dimensional case of a pier. This might be the only way which will provide evidence not only of the mode of failure of a pier under uniaxial compression, but also an explanation of its low value compared with a wall (See Krefeld's observations, p. 1.10).

5. As regards the parameters which were established theoretically to have great influence on the strength, they can be listed as:

a. Deformation properties of mortar, in the hardened state, and bricks: E_m, ν_m, E_b, E_{tb} and ν_b.

b. Tensile strength of mortar and bricks: f_{tm}, f_{tb}.

c. Dimensions of mortar and bricks in both vertical and horizontal directions.

6. Regarding the way in which each of these parameters affects the strength, this has already been discussed and can easily be assessed from the nature of the expressions.

To decide on the most useful areas of experimental work, a comparison between the two approaches led to two main conclusions. The first: at that stage the success of the idealized approach was considered to be limited as to that of a visualizing tool. The second: in the hope of achieving much information and perhaps more conclusive results on the strength, the action and interaction approach was expected to be more profitable.

Summing up, two areas suffering from lack of data and in urgent need of experimental investigation, emerged as follows:

1. The structural properties of mortars, with special reference to the deformation properties.

2. Structural properties of bricks with special reference to its tensile strength and deformation properties.

As can be seen, these requirements involved an extensive experimental programme. Adding to this the fact that there were many questions which had arisen in Chapter 1, to be answered and the fact which emerged later that there was a need to develop new reliable methods of testing "1" was considered to form the main object of the investigation.

C H A P T E R 5

REMARKS ON PREVIOUS METHODS AND THEORETICAL ASPECTS OF A NEW PROPOSED METHOD FOR THE DETERMINATION OF THE STRESS DEFORMATION PROPERTIES AND TENSILE STRENGTH OF MORTARS

5.1 ORIENTATION

It has been concluded from the foregoing three chapters, that knowledge of mortar deformation properties (Poisson's ratio and modulus of elasticity) and tensile strength is an essential pre-requisite to the successful study or prediction of masonry deformation and strength. Referring to the previous methods of measuring the deformation properties, it was found that these properties have aroused very little attention for mortars. Even the recent draft of the British Standard Methods for testing mortars(19) does not include anything about these properties; consequently the need for a testing method arose urgently.

Before choosing a technique, especially for the deformation properties, and with the object of forming a good basis for the choice, it was necessary to study the methods and the forms of test piece which have been used with some other materials. Among the wide variety of construction materials, concrete emerged as the most suitable one in this respect.

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(19): British Standard Institution. Draft of Standard Methods of testing mortars. 66/3796. March 1966. (Issued for comment only).

5.2. GENERAL REMARKS ON PREVIOUS METHODS

5.2.1. General

Concrete, like mortar, has been extensively used in construction, but some of its properties are still imperfectly known variables. Poisson's ratio and the modulus of elasticity are two of these properties, and up to the present no single test procedure for measuring them has been standardised. An example of this is that the B.S. 1881: 1952[17] on concrete testing does not specify any one method to determine the modulus of elasticity, it only shows two ways in which it may be obtained. The resulting two values are not the same, and this makes such information open to question.

In a similar manner the situation with Poisson's ratio is confusing. Poisson's ratio of a material is usually defined as the ratio of the lateral strain to the longitudinal strain when the specimen is subjected to a direct compression or tension. Theoretically, this definition can apply only for an ideal material in which the ratio is constant. For a material like concrete or mortar this is not the case. Probably two reasons could be given. The first is that the stress-strain curves for compression and tension have been shown, by a few authorities, to be similar only up to low loads. The second is the creep effect primarily in one direction. Glanville[48], and Glanville and Thomas[49] showed, experimentally,

(17): British Standard Institution. B.S.1881:1952. Methods of testing concrete.

(48): Glanville, W.H. Studies in reinforced concrete: The creep or flow under load. Department of Scien. and Indust. Research. B.R.S. Tech. Pap. No. 12. London. His Majesty's Stationery Office, 1930.

(49): Glanville, W.H. and Thomas, F.G. Studies in reinforced concrete: Further investigation on the creep or flow of concrete under load. Dept. of Scien. and Indust. Research, B.R.S.Tech.Pap. No. 21. London. His Majesty's Stationery Office, 1939.

that while creep in the direction of loading is appreciable the lateral creep is so small that it can be neglected. Consequently, it can be said that Poisson's ratio for concrete and mortars is unlikely to be constant in the one test and at different stages of loading.

From this it becomes clear that for a material which might have different elastic properties the definition of Poisson's ratio is not fully valid. In fact if we foll the same rule for the modulus foeelasticity as adopted by the B.S.I. we could have specified or represented two values for Poisson's ratio. However, Poisson's ratio is usually assumed as one value or as values within a very narrow range.

Although the testing technique will be shown to have great influence on the results as indicated by the wide variation in measured values for the same mixes. It must be remembered that, in the main, the properties open to question must be affected by factors connected with the material under testing. Examples of these factors are the mix proportions, properties of constituents and conditions of curing. The effect of some of these factors, according to the method used in each case individually, has been investigated in the past few years. For the modulus of elasticity, reference can be made to Brown[22], Counto[35],

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(22): Brown, C.B. Models for concrete stiffness with full and zero continuity. Proceedings of an International Conference on the Structure of Concrete and its Behaviour under load. London, 1965. Cement and Concrete Association, Paper A1.

(35): Counto, U.J. The effect of the elastic modulus of the aggregate on the elastic modulus, creep and creep recovery of concrete. Mag. of Conc. Research, Vol.16:No.48, Sept. 1964. pp. 129-138.

Elvery and Evans[40], Hansen[57], Hughes[72], Illston[77], Neville[101] and Plowman[107]. As regards Poisson's ratio, although attention has been drawn to some of the influences on its value by Anson[5], Anson and Newman[6], Newman[100], Plowman[107] and Simmons[119], no correlations have yet been discovered to connect the value of the ratio with the mix proportions or any other property.

Surveying the methods commonly used in different laboratories as described in the available literature, it was possible to conclude that the modulus of elasticity and Poisson's ratio have usually been measured by static or dynamic loading methods. Both of them have been discussed by many authorities, and reference will be made to them in the proper places throughout the following remarks.

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(40): Elvery, R.H. & Evans, E.P. The effect of curing conditions on the physical properties of concrete. Mag. of Conc. Research. Vol.16. No.46, March 1964. pp. 11-20.

(57): Hansen, T.C. Theories of multi-phase materials applied to concrete, cement mortar and cement paste. Proceeding of an International Conference on the Structure of Concrete and its Behaviour under load, London. 1965. Cement and Concrete Association, Paper A2.

(72): Hughes, B.P. & Chapman, G.P. The deformation of concrete and micro-concrete in compression and tension with particular reference to aggregate size. Mag. of Conc. Research. Vol.18: No. 54. March 1966, pp. 19-24.

(77): Illston, J.M. The delayed elastic deformation of concrete as a composite material. Proceedings of an International Conference on the Structure of Concrete and its Behaviour under load. London 1965. London. Cement and Concrete Association. Paper A3.

(101): Neville, A.M. Creep of concrete as a function of its cement paste content. Mag. of Conc. Research. Vol.16: No. 46. March, 1964. pp. 21-30.

(107): Plowman, J.M. Young's modulus and Poisson's ratio of concrete cured at various humidities, Mag. of Conc. Research. Vol. 15: No. 44. July 1963. pp. 77-82.

(5): Anson, M. An investigation into a hypothetical deformation and failure mechanism of concrete. Mag. of Conc. Research. Vol.16: No. 47. June 1964. pp. 73-82.

(6): Anson, M. and Newman, K. The effect of mix proportions and method of testing on Poisson's ratio for mortars and concretes. Mag. of Conc. Research. Vol. 18: No. 56. Sept. 1966. pp.115-130.

Alternative methods, but not commonly used, are the determination of Poisson's ratio from triaxial tests, or from the elastic moduli in torsion and compression or tension. These will not be included in the reamrks for two reasons. The first is their rare use, and the second is the inherent inaccuracy of these methods, resulting from difficulties with apparatus in determining both moduli, and the ill-conditioned equation so far as Poisson's ratio is concerned. For more details about these methods reference can be made to reference (119), in which previous work is quoted and discussed.

5.2.2. Static Tests

There are wide variations in static tests, but usually they are carried out on cylinders or prisms in compression, and beams in bending. Sometimes tension tests are used by employing bobbins, cylinders, or prisms with embedded studs. A recently developed variant is the necked specimen tested in compression. In spite of the fact that the static tests are used more than the dynamic tests, they have each individually certain contentious aspects associated with them.

5.2.2.1. Compression tests

1. With either cylinders or prisms the loads are usually applied uniaxially. In spite of this the specimens can be

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Continued:
(100): Newman, K. Criteria of the behaviour of plain concrete under complex states of stress. Proceedings of an International Conference on the Structure of Concrete and its Behaviour under load. London, 1965. Cement and Concrete Association. Paper F1.
(119): Simmons, J.C. An investigation of Poisson's ratio for concrete. Thesis submitted to the University of London for the degree of Master of Science, 1954.

subject to eccentric loading or inaccuracies occurring during preparation. A defect which can affect the results considerably is a lack of parallelism, and any attempt to achieve accurate capping is usually accompanied by complications due to greater bulk. Reference can be made to Ahmed,[2] and Troxell[138].

2. Even if the effect of non-parallelism is overcome, the tests are accompanied by a phenomenon which affects the whole deformation and can be attributed mainly to the characteristics of the normal testing machine. When measuring deformations it is traditional[21] to regard the load applied by the testing machine as the independent variable to be increased or decreased at will, or in better circumstances at a constant rate. Consequently the strain measurement becomes the dependent variable. Such a situation is the natural outcome of the volution of the test machine as an apparatus for applyin load rather than producing deformation. It is a fact that the deformations both in the direction of loading and perpendicular to it are greatly dependent on the loading conditions. This will be discussed

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(2): Ahmed, S. Effect of capping on the compressive strength of concrete cubes. Mag. of Con. Research, Vol.6: No. 19, March, 1955. pp. 21-24.

(138): Troxell, G.E. The effect of capping methods and end conditions before capping on the compressive strength of concrete test cylinders. Proceedings of the Aerm. Soc. for Testing Materials, 1942, Vol. 41. pp. 1038-1044. Discussion: pp. 1048-1052.

(21): Brock, G. Concrete: Complete stress-strain curves. Engineering. Vol. 193, No. 5011, May 1962. pp. 606-608.

in detail in Chapter 6 . Moreover it has been found recently that the testing machines themselves have great inlfuence on these values. In this respect reference can be made to Atherton[8], Cole[32,33] Newman[99], and Sigvaldason[117].

In spite of all the efforts already made, however, the point still needs more clarification. It is enough here to mention that the effects of the seatings, platens, and specimen alignment start to be felt at the interfaces of loading, in other words at the very starting point of receiving the load.

3. The above-mentioned phenomenon is usually accompanied by another consequent phenomenon which can affect the measured strains considerably. This is the variation of the vertical stresses over the cross-sectional area. Therefore the assumption that

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(8): Atherton, M.J. Some experiences with a commercial compression testing machine. Mag. of Conc. Research. Vol. 17: No. 50 March 1965 pp. 45-46. Discussion: Contribution by Ackroyd, T.N.W. Vol. 17: No.53. Dec.1965. p.2

(32): Cole, D.G., The accuracy of compression testing machines and other factors which affect the accuracy of concrete strength tests. Cement and Concrete Association, Tech.Rep.TRA/389,1965.

(33): Cole, D.G. The relationship between the apparent variation in compressive strength of concrete cubes and inaccuracies found in the calibration of compression testing machines. Proceedings of a Symposium on Concrete Quality. Cement and Concrete Association, London, Nov. 1964. pp. 155-161.

(99): Newman, K. Concrete control tests as measures of the properties of concrete. Proceedings of a Symposium on Concrete Quality, London.1964. Cement and Concrete Association, 1966 pp. 120-138.

(117): Sigvaldason, O.T. The influence of the testing machine on the compressive strength of concrete. Proceedings of a symposium on Concrete Quality. Cement and Concrete Association, London, Nov. 1964. pp. 162-171.

readings at points in the outer planes of a speciment represent the average values for the whole cross-section is not absolutely correct. In spite of the fact that this was recognized[31,58] a long time ago very little attention has been paid to the effect of this on the modulus of elasticity or Poisson's ratio. Most researches were almost entirely limited to the influence on the ultimate compressive strength.

4. Because of the heterogeneous composition of a material like concrete it is difficult to be certain that the measured strain is the strain in the material and not only in one of the constituents. As an example Cooke and Seddon[34] found errors up to 80% when using inappropriate gauge lengths. It is necessary, therefore, to use quite a long gauge length.Keeping in mind the variation in the vertical stresses, discussed above, it becomes clear that the measured lateral strain might be subject to doubt. This is due to the fact that it provides a measure of the sum of several lateral strains over small lengths subjected to different vertcial pressures, averaged out as the strain in the whole basic length and for all the components of the material.

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(31): Coker and Filon. A treatise on Photo-elasticity. (second edition). The Cambridge University Press, Cambridge, 1957. pp. 583-588.

(58): Hast, N. Measuring stresses and deformations in solid materials. Central by Reciet Esselte ab Stockholm 1943. B.R. Station Reference Library No. Y9D1 340345.

(34): Cooke, R.M., Seddon, A.E. The laboratory use of bonded-wire electrical-resistance strain gauges on concrete at the Building Research Station. Mag. of Conc. Res. Vol. 8: No.: 22. March 1965. pp. 31-38.

5. The necked specimen adopted by Barnard[9], is more or less a cylinder with enlarged ends. He considered that the specimen gave a considerable improvement in the technique over prisms in that the cracking was always confined to the regions where strains were being measured, while with prisms failure or cracking often occurred in the upper and lower halves. Furthermore, a more uniform state of deformation and a more homogeneous gauge length were obtained with the necked specimens than with the prisms, where first cracks inevitably appeared at the corners where the trowelled surface showed less cracking than the other faces.

6. Considerable precautions were taken by Brock[21] and Hsu[75] when studying the stress-strain relation, mainly by measuring the loads through capsules or cells inserted between the specimen and the machine, using distribution blocks, and calibrating the loading frame to measure strains rather than measuring them by extensometers mounted on the specimen. Although this can give more reliable readings as regards the loads and vertical strains, both problems of lateral restraint at the interfaces of contact and unequal pressure still exist. To avoid this a bigger specimen would be necessary, with the consequent requirements of larger apparatus.

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(9): Barnard, P.R. Researches into the complete stress-strain curve for concrete. Mag. of Conc. Res. Vol. 16: No. 49. Dec. 1964. pp. 203-210.

(75): Hsu, T.T.C., Slate, F.O. Sturman, G.M., and Winter, G. Microscoping of plain concrete and the shape of the stress-strain curve. The Jl. of the Amer. Conc. Inst. Vol. 60: No. 2. Feb. 1963. pp. 209-223.

7. Comparing some of the previously published data on a great variation from one authority to another. For example Davis and Troxell[37] showed that the value of Poisson's ratio increased at low stresses. Morice[95] showed that at low stresses Poisson's ratio was very small, but as the stress increased the concrete behaved more like a metal and gave a value of between 0.15 to 0.20. After Morice, Simmons[119] showed in his main series that there was no evidence of a change in the value of static Poisson's ratio for applied stresses up to 750 lb/in^2. Then from his further tests he stated that for the mix studied the value of Poisson's ratio was sensibly constant over a range of stress from 200 to 3000 lb/in^2, but at higher stresses it was found to increase.

5.2.2.2. Tension tests

8. Although the modulus of elasticity in tension was shown by some authorities[26], [97], [43], [80], [137], [145] to be almost similar to that in compression up to about 50-60% of the failure load, one fact should be remembered about these tests. That is, the range of mixes for which this similarity was found in all cases was very small.

9. A major drawback of the tensile test is that one is handicapped by the difficulty of obtaining with certainty a true

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(37): Davis, E. and Troxell, G.E. Modulus of elasticity and Poisson's ratio for concrete and the influence of age and other factors upon these values. Amer. Soc. for testing Materials. Vol. 29. Part II. Technical Papers, 1929.

(95): Morice, P.B. Ph.D. Thesis submitted to the University of London, 1952.

(26): Burr, W.H. Tests made at Columbia University. Engineering Record, Vol. 54., No. 3. Dec. 1906.

uniaxial loading. This difficulty had been met before by the author(97), but the problem had been previously tackled comprehensively by Evans(43). Some of his tests were carried out on pure tension specimens. In the majority of cases he found a wide divergence indicating that eccentricity of loading was present. This indicated to him that the tension was not pure, but was combined with the effects of bending, to which the normally low tensile strength was attributed.

10. Todd(137) also was convinced from Evans' results and his own preliminary tests that eccentricity caused a large amount of error. He tried to devise a new form of tension test in which eccentricity of loading can be neutralized by an external bending moment, but his test was somewhat tedious.

11. Sometimes it is claimed that it is easier with a tension test to overcome the end effects resulting from the machine platens as is usually encountered with compression tests. In fact

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continued:

(97): Morsy, E.H. Plain and reinforced concrete from aggregates other than Egyptian ordinary gravel. Thesis submitted to Cairo University, Faculty of Engineering, for the degree of Master of Science, 1963.

(43): Evans, R.H. Extensibility and modulus of rupture of concrete. The Structural Engineer, Vol.24: No.12, Dec. 1946. pp. 636-659.

(80): Johnson, A.N. Direct measurement of Poisson's ratio for concrete. Proceedings of the Amer. Soc. for Testing Materials. Vol.42;No. Part 2, 1924. p

(137): Todd, J.D. The determination of tensile stress-strain curves for concrete. Proceedings of the Institution of Civil Engineers. Part 1, Vol.4: No.2. March, 1955. pp. 201-211.

(145): Williams, G.M. Some determinations of stress deformation relations for concrete under repeated and continuous loading. Proc. A.S.T.M. Vol.20, Part 2, 1920.

this is not absolutely true. Very recently tests were carried out in a manner different from the usual one, in which studs were used with the speoimens. In these new tests, Hughes and Chapman[71] and Spetla[126] used modern high-strength glues applied at the ends. In spite of the difficulty and care required in carrying out the tests the results are very interesting.

The results showed a phenomenon which is very similar to the results of compression tests on concrete specimens of the same shape. In order to avoid the influence of the gluing strip on the tensile test specimen (cylinder or prism) and the influence of the state of stress at the ends, the slenderness ratio must not be less than two for cylinders and three for prisms. This restricts the transverse strains in the end parts of the glued-on specimen. Above these two values of slenderness ratio it was possible to ascertain the true direct tensile strength. Considering the values at these ratios as basic ones then the percentage increase of the apparent direct tensile strength was obtained up to 140% higher than these basic values, and only above these two values was it found that the concrete attains practically constant values of the tensile strength.

12. Another example showing the dependence of the results upon

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(71): Hughes, B.P. and Chapman, G.P. Direct tensile test for concrete using modern adhesives. Rilem Bulletin, No.26. March. 1965. pp. 77-80.

(126): Spetla, Z. and Kadlecek, V. Effect of slenderness on the tensile strength of concrete cylinders and prisms. Rilem Bulletin. New Series No. 33. Dec. 1966. pp. 403-412.

the method of testing in tension was given by Saul[113], due to Schumann and Tucker[115], The average tensile strengths due to two different methods of loading (not given in his report), while all other conditions were kept constant, were 430 and 320 lb/in^2.

13. Following 7-12 it can be emphasized that measuring the strains in both directions or the tensile strength in a direct tensile test with a material like concrete or mortar is not reliable and might be expensive.

5.2.2.3. Bending tests

14. With static tests on beams, the flexural strength in itself is a tensile stress determined indiredtly. Due to the bending there are both compressive and tensile stresses, and this would appear to affect the measured values, making both the strains and the stresses different from those in the tension test. Comparing previous contributions on the flexure test and the tension test provides ample evidence that the situation is confusing.

15. Squire[128], as quoted by Todd[137], found that the actual failure load of a plain concrete beam loaded in bending is not the same as the failure load when calculated from the stress-strain curve obtained from a pure tension test, and may very well be

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(113): Saul, A.G.A., A comparison of the compressive flexure and tensile strengths of concrete, Cement and Concrete Association. London. June 1960. Tech. Rep. TRE/333.

(115): Schumann, L. and Tucker, J. U.S. Dept. of Commerce, National Bureau of Standards. Paper No. R.P. 1552. Vol.31. 1943, p. 107.

(128): Squire, R.H. Structural Engineer. Vol. XXI, p. 211, 1943.

double that figure.

Blakey[11] from his extensive tests stated that there is little evidence of any marked deviations from a linear tensile stress-strain relationship below cracking the cracking stress, although there is some indication that the stress distribution over the depth of a beam may be more nearly parabolic than linear on the tension side. From the whole analysis he came to the conclusion that the tensile stress calculated from the breaking load of an unreinforced beam is a quite fictitious and incorrect estimate.

Oladapo[103] found that the stress-strain relation on the tension face of a beam was linear only up to a much smaller strain, which he considered to be in disagreement with Blakey's results. Then by introducing an approximation, to the early part of the curves, he suggested that the modulus of elasticity is higher in compression than in tension by about 17%

Contrary to Blakey and Oladapo, Welch[142] found that the load-strain relation for concrete specimens tested in flexure at the standard rate of loading was almost always slightly curved.

16. Another drawback in flexural testing is that the results are not usually uniform, having high coefficients of variation. This was clearly shown by Blakey[11], Dewar[38], Grieb and Werner[52]

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(11): Blakey, F.A. and Beresford, F.D. Tensile strains in concrete. Part 1: Report No. C2.2-1 1953, Part 2: Report No. C.2.2-2, 1955. Melbourne, Australian Division of Building Research.

(103): Oladapo, I.O. Cracking and failure of plain concrete beams. Mag. of Conc. Res. Vol.16: No.47, June 1964. pp. 103-110.

(142): Welch, G.B. Tensile strains in unreinforced concrete beams. Mag. of Conc. Res. Vol.18: No.54, March 1966. pp. 9-18.

(38): Dewar, J.D. The indirect tensile strength of concrete of high compressive strength. Cement and Concrete Association, Tec. Rep. TRA/377, 1964.

(52): Grieb, W.E. and Werner, G. Comparison of the splitting tensile strength of concrete.with flexural and compressive strengths. Public Roads. Vol.32: No.5, Dec. 1962. pp. 97-100.

and Wright[147]. Considering the fact that the conclusion is common for separate series of tests using different approaches and methods, other differences between the tests would undoubtedly make the discrepancy greater. Such influencing factors include the beam depth, the way of applying the load, and the rate of loading. All these factors relate broadly to the testing technique.

A distinguished example of the first factor is that Wright[148] found a reduction of about 30-33% in the flexural strength when the depth of the beam increased from 3 to 8. inches for a span depth ratio of three. The value of the reduction was found by him to vary according to whether it was central or third-point loading. Blakey's results also led him to conclude that the relation between the flexural strength and the depth-span ratio is open to question.

As regards whether the beam is loaded centrally or at the third points of the span, the strengths obtained from the former were found to be higher than the latter by 20-25% and 11.4-12.2% according to Wright[48] and Morsy[97] respectively. This was explained by Garwood[47] as due to the fact that with third point loading the whole of the middle third is under the maximum stress so that there is greater

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(147): Wright, P.J.F. Crushing and flexural strengths of concrete made with limestone aggregate. Research Note RN/3320/P.I.F.W. Dept. of Scien. and Indust. Research, Road Research Laboratory, Oct. 1958.

(148): Wright, P.J.F. The effects of the method of test on the flexural strength of concrete. Mag. of Conc. Research, Vol. 4, No. 11, pp. 67-76.

(47): Garwood, F. Appendix topaper by Wright, P.J.F. (See reference 148).

opportunity for weak points to have an effect. Under central loading only the plane below the load is under maximum stress.

An interesting example of the influence of the rate of loading is that an increase in the apparent failure strength was found by Wright[148] up to 15% due to an increasing rate of loading.

Another factor, although of less importance, is the effect of direction of casting on the results. As stated by Saul[113], Shacklock and Keene[116], carried out some tests in accordance with the B.S.:1952, and other tests with beams on their sides. Although it was not absolutely significant there was a tendency for those tested on their sides to appear weaker than the others. This implies to the author that if specimens were tested upside down an even greater discrepancy might be expected.

17. From "16" it does not seem strange that a comparison between flexure and tension tests shows a wide range of contradictory variations. Blakey[11], Humphrey[74], Evans and Saul found that the tensile strengths are slightly below the flexural strengths, Liddicoat and Potts[89], and Todd[137] showed that the flexural strength can have a value of 1.56-2.41 times the tensile strength.

18. The above remarks suggests that it is unreliable to employ the bending test for the determination of Poisson's ratio. It is probable that this led Liddicoat and Potts to state that the

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(116): Shacklock, B.W. and Keene, P.W. The comparison of the compressive and flexural strengths of concrete with and without entrained air. Cement and Concrete Association, London, Dec. 1957. Tech. Rep. TRA/283, p. 10.

(74): Humphreys, R. Direct tensile strength of concrete. Civil Engineering and Public Works Review. Vol. 52. No.614, Aug. 1957. pp. 882-883.

(89): Liddicoat, R.T. and Potts, P.D. Laboratory Manual for Testing Materials. The Macmillan Company, New York, 1961.

test described in their book should be employed with materials of constant modulus of elasticity.

19. A technique which was proved by Smith[25] to be successful is the one originally developed by the Portland Cement Assocation. The technique involves the determination of the compressive stress-strain properties in flexure with no tension zone created in the test specimen. This is accomplished by applying two compressive forces to a prism which then resembles that part of the beam specimen above the neutral axis. In spite of the merits of the technique such as the relatively small size of the test specimen and the possibility of using knife edges as supports it use is more suitable for concrete compression zones in beams under bending.

20. Following 14-18, and in a similar manner to the tensile test, it can be said that what has been usually claimed for the flexural or bending test might not be absolutely true. The usual claim is that with this test the various factors affecting the results as regards the testing technique are much less than in the case of compression tests. In fact the popularity of the bending test can be attributed totally to its simplidity.

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(25): Smith, R.G. The determination of the compressive stress-strain properties of concrete in flexure. Mag. of Conc. Res. Vol.12: No. 36, Nov. 1960. pp. 165-170.

5.2.3. Dynamic Tests

It is the trend nowadays, in most laboratories to adopt one or two dynamic methods of testing concrete. The two main methods used with concrete are the resonance and the pulse velocity methods. They have been discussed by many authorities, among them Jones[81, 82, 83], King and Lee[84], Liddicoat and Potts[89], Philleo[106], Simmons[118,120] and Stutterheim[132]. These two methods were originally developed for homogeneous elastic bodies[81], then they were applied rightly or wrongly to concrete. In spite of their increasing popularity, the main object of these tests must be remembered.

The main object as applied to concrete and allied materials, is to provide a reliable estimate of the quality of concrete actually in a structure, without relying solely on results from test specimens which are not necessarily representative of the structural concrete. Due to the fact that there can be no direct measurement (apart from load testing) of the strength properties of structural concrete,

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(81): Jones, R. Non-destructive testing of concrete. Cambridge University Press. 1962.

(82): Jones, R. The non-destructive testing of concrete. Mag. of Conc. Research. No.2. ., June 1949. pp. 67-78.

(83): Jones, R. and Gatfield, E.N. Testing of concrete by an ultra sonic pulse technique. Road Research Tech. Pap. No. 34. Dept. of Scien. and Indus. Research, Road Research Laboratory, London. Her Majesty's Stationery Office, 1963.

(84): King, W.H. and Lee, I.D.G. An alignment chart for the evaluation of the dynamic modulus of elasticity and Poisson's ratio of concrete. Mag. of Conc. Research, Vol.7: No. 21, Nov. 1955. pp. Discussion Contribution by Bradfield, G. March 1956. pp. 39-45.

it is therefore necessary to measure some other physical property of the structural material which are related to strength and which can be obtained by non-destructive methods.

1. The pulse methods are applicable to structural material in situ, and are not restricted to laboratory specimens. This is considered, sometimes, as an advantage. Applying them to concrete, the dynamic modulus can be calculated when the density and Poisson's ratio are known. For this reason many research workers attempted to use the pulse velocity itself as a criterion of the quality of concrete. Even after these attempts, however, it was found[81] that there is no unique relation between the pulse velocity and the strength of concrete, so that severe limitations applied, when the method was used on concrete.

2. Consequently from "1" it can be said that for the particular objectives of the present work a major drawback of the pulse method is that it is doubtful whether any practical results can be obtained from it. This will be emphasized if we take into consideration the fact that Poisson's ratio is one of the main properties open to question, especially as no relation could be found

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continued:

(106): Philleo, R.E. Comparison of the results of three methods for determining Young's modulus of elasticity of concrete. Journal of the American Concrete Institute. Jan. 1955. pp. 461-469.

(120): Simmons, J.C. Poisson's ratio of concrete: a comparison of the dynamic and static measurements. Mag. of Conc. Research, Vol.7: No.20, July 1955, pp. 61-68. Discussion: Vol.8:No.22 March 1956.

(132): Stutterheim, N. Lochner, J.P.A. and Burger, J.F. A method for determining the dynamic Young's modulus of concrete specimens developed for corrosion studies. Mag. of Conc. Research, Vol. 16: No.16, June 1954. pp. 39-46.

by Simmons[119] between Poisson's ratio and any other property of concrete.

3. Another drawback is that if the paste and aggregate differ in elastic properties the formula used for calculating the modulus of elasticity is invalid and the results are likely to be misleading. Since the pulse velocity is a characteristic of the concrete independent of size and shape of the specimen, the velocity itself would appear to be a more significant property than something calculated incorrectly from it.

4. There is only one possibility, involving no little difficulty, for the determination of Poisson's ratio by applying the pulse method, but in this case a combination of the pulse method and the resonant frequency[116], [118] is required.

5. With the resonance method applied to concrete, the property usually determined is the dynamic modulus of elasticity, from which a certain relationship enables Poisson's ratio to be derived. The specimen is the standard specimen prescribed for flexural or compressive tests. The tests are normally carried out within the laboratory.

6. With the resonance methods, the deviations of the values are usually due to the effects of damping, heterogeneity and anisotropy. For concrete is has been found that the correlation for longitudinal damping is negligible. While the variations in the results likely to arise from non-uniform distribution of aggregate are quite small, it is difficult to achieve consistent curing conditions. As regards anisotropy, it sometimes leads to

dynamic Poisson's ratio being slightly lower than the dynamic Poisson's ratio as calculated from the resonant frequency in combination with the pulse velocity, although the actual value is usually reproducible[81].

7. Jones, in his discussion of the relation between the dynamic modulus and the strength of concrete, reviewed that investigations carried out by earlier investigators. He came to the conclusion that no general relation exists between the dynamic modulus of elasticity and the flexural or compressive strength. He added, however, that limited correlations were obtained where changes in the dynamic modulus and strength were produced by changes in the age of concrete, the degree of compaction, and the water-cement ratio, or by deterioration. Then these limited correlations were considered to form the basis of the main laboratory application of the resonance method as a means of assessing the durability of concrete.

8. Beside the preceding discussion another question of importance arises. That is how far the dynamic modulus and dynamic Poisson's ratio compare with the values measured statically. A comprehensive comparison has been made by Elvery and Furst[41], Jones[81], Philleo[106], Simmon[118], and very recently by Anson and Newman[6].

9. In the majority of cases it was found that the dynamic modulus is greater than the static modulus. Apparently, the E_d is the tangent modulus at zero stress. An empirical relation was given by Jones[81], who stated that the relation

might be different according to the constituents.

The same form of relation was obtained experimentally by Elvery and Evans. All the relations between E_s and E_d had the same form of equations, but for each condition certain constants were given. Moreover it was concluded that even that group of relations applies only to the one type of aggregate used.

Elvery and Furst[41] studied also the effect of compressive stress on the dynamic modulus of concrete. He found that a small reduction of the modulus occurs when concrete is loaded but that no subsequent reduction occurs while the load is maintained for periods up to six months. A similar reduction also occurs on releasing the load and with subsequent loading cycles. This effect which is independent of the age of the concrete after 14 days, is approximately proportional to the applied stress and becomes successively smaller for subsequent loading cycles. The reduction in dynamic modulus is always small being about 5%

An explanation of that can be given from the fact that the stress strain curve for a material like concrete or mortar is not a straight line. While the static modulus can be computed at selected different stresses, the dynamic modulus is calculated from almost infinitesimal stresses. The difference between the two moduli depends upon the extent to which the curve departs from a straight line and the particular

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(41): Elvery, R.H. and Furst, M. The effect of compressive stress on the dynamic modulus of concrete. Mag. of Conc. Research. Vol. 9: No. 27. Nov. 1957. pp. 145-150.

type of the static modulus. The difference can be more appreciated if it is borne in mind that there is no general basis for computing the static modulus, resulting in wide variations in different laboratories. As was pointed out before, some laboratories find the secant dompressive modulus at stresses ranging from zero to 2,000 lb/in^2, while others use compressive or flexural stresses of 15-25 or 50% of the ultimate. This explains how far the results can differ.

It is worth mentioning a few of the experimental results obtained before by other investigators.

The resonant and static moduli for mild steel as quoted by Philleo[106], were equal to 30.165×10^6 and 30.000×10^6 respectively.

Simmon[118] found that for steel, copper, brass and dural the dynamic modulus was respectively 30.50×10^6, 17.0×10^6, $14.50\text{-}15.00 \times 10^6$ and 10.40×10^6, while the static moduli were 30.30×10^6, $18.00\text{-}18.70 \times 10^6$, $14.00\text{-}14.80 \times 10^6$ and $9.5\text{-}10.70 \times 10^6$. Both result show a slight discrepancy between the resonant and static moduli.

For concrete Philleo quoted the results obtained by Powers[108]. In thirteen cases out of fourteen he found the straight line at the origin had a slope equal to the resonant modulus, coinciding with the stress-strain curve from

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(108): Powers, T.C. Measuring Young's modulus of elasticity by means of sonic vibrations. Proceedings of A.S.T.M. Vol. 38, Part II, 1938. p. 460.

a cylinder, in the region of the origin. In his case the curves started to diverge at stresses equal to 5% of the ultimate stress. In the same investigation 38 comparisons were made between the resonant modulus and the static modulus at about $\frac{1}{3}$ the ultimate strength computed from central deflections of prisms centrally loaded as beams. No discrepancies were noticed other than experimental errors.

Simmon also found that in almost every case the static value was appreciably less than the dynamic value, particularly for leaner mixes. This confirmed the earlier results quoted by him due to Takabayashi. Evidence was clear in the figures that E_d is greater than E_s for most of the moderate values but the points did not fall on a very well defined line. From the whole analysis he came to the conclusion that the dynamic tests do not necessarily measure identically the same properties, including the modulus, as the static tests.

10. As regards Poisson's ratio, the work done on its determination by the dynamic tests is not much. However it is assumed that there is a possibility of using combinations of the sonic and resonant frequency techniques applied to some isotropic materials with the same results as the static tests. The reason for this is that this assessment was made possible by the ability of the material to undergo alternative tests for confirming the values of both the elastic moduli.

Nevertheless the results given by Simmon from his measurements of Poisson's ratio by two different dynamic methods were contradictory, even with metals. The dynamic values of Poisson's ratio for steel, copper, brass, and dural, were respectively, for the two methods of test (0.288, 0.290), (0.358, 0.223), (0.355, 0.610) and (0.337, 0.423). The corresponding static values were respectively, 0.287,(0.337-0.348,) (0.340-0.400),and(0.280-0.360). Then he came to the conclusion that the values of Poisson's ratio determined from dynamic tests were not necessarily equal and that both might differ from the static value even with metals.

Again the situation with concrete is more complicated and the previous results indicate great discrepancies.

Jones[81] used the ultrasonic pulse velocity and longitudinal resonant frequency for the determination of Poisson's ratio. He found that the value varied from about 0.2 to 0.3 depending on the aggregate and the proportions of the mix. Then he suggested that errors might be involved in the use of the more usual value of 1/6. He did not, however, quote any results comparing dynamic tests with static tests.

Following him came Simmon's results[119]. From the measurements of Poisson's ratio using two different methods he found that one method gave more consistent results than the other, but the difference between corresponding values being in no case less than 16%. He also stated that no relation could be found between the values of dynamic Poisson's ratio and any other property, which was the same conclusion as given by him for the static Poisson's ratio.

11. In general it might be claimed that the dynamic methods have a major merit in their general simplicity. In addition they have reached the stage where they could be easily developed with a fair measure of standardization. But in the author's opinion this would not be of practical use unless correlations between dynamic tests and well-defined static tests were established, so that the readings could be converted easily to the actual values.

At the same time it is worth mentioning that it has been possible, very recently, to increase the accuracy of the dynamic tests. This can be achieved by the use of oomplex-non-destructive tests, as dealt with by Skramtaey[124]. It is enough, here, to say that real complications are included, as the name implies.

5.2.4. Conclusions

Looking back at previous methods, described briefly in the preceding remarks, it was concluded that the properties in question have not yet been assessed by tests which are well defined. In the author's opinion neither the static methods nor the dynamic ones seem to be adequate for highly precise measurements. While the static methods have the greater variability of the results due to errors or imperfections in the techniques of testing, the dynamic ones provide mainly a reliable estimate of quality.

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(24): Skramtaey, B.G. and Leshchinsky, M.Yu. Complex methods of non-destructive tests of concrete in construction and structural works. Rilem Bulletin, New Series No.30, March 1966.

A general conclusive remark can be added. There is ample evidence that the stress-deformation properties of concrete are not perfectly known because there has been no precise procedure. Moreover, the fact that there is no strictly specified method incorporated in the specifications can be attributed to this. From this point it can be emphasized that in comparing values of these properties determined from previous tests an important thing should be kept in mind: that is the means by which each property was determined, and the manner in which such properties may be related to the structure we are considering.

In the meantime it seems that a comprehensive programme of work is required to investigate the influence of the method of testing on the assessed values.

However, because our knowledge of the elastic constants of mortars, especially as regards Poisson's ratio, is very little or might be nothing, while theoretical speculations have shown their vital importance, and with the hope of achieving so far as possible reproducibility of the results, the author attempted first to adopt a more reliable method. At the same time a concentrated effort was decided in order to secure the knowledge of these properties in a more systematic way.

5.3. CHOICE OF SCOPE OF TESTING TECHNIQUE

In choosing a technique for testing, the principle of first priority was thought to lie in the exclusion of the influence of the loading equipment (by minimizing the area of interference between the platens and the specimen); in ensuring as far as possible satisfactory reproducibility; economical means of producing a large number of specimens in the one set; and at the same time, in maintaining the reliability of both laboratory performance and method of calculation.

The technique which appeared most convenient and reliable for achieving the above considerations was the indirect tension test, Figure 5.1-a. A fact which is not widely known and was pointed out by Evans[(42)], is that the method was proposed in the first place (1942), and adopted as a standard test (Japanese Industrial Standard A.1113-1951) in Japan. At about the same time the test was introduced in Brazil by Carneiro[(29)]. Since then it has been referred to by many authorities as the Brazilian test, and it has been established as a satisfactory method for the measurement of the tensile strength of concrete.

Existing knowledge of its use in the laboratory has been that the test is performed by loading a cylinder in compression diametrally and along two opposite generators. The stresses set up from this condition over the

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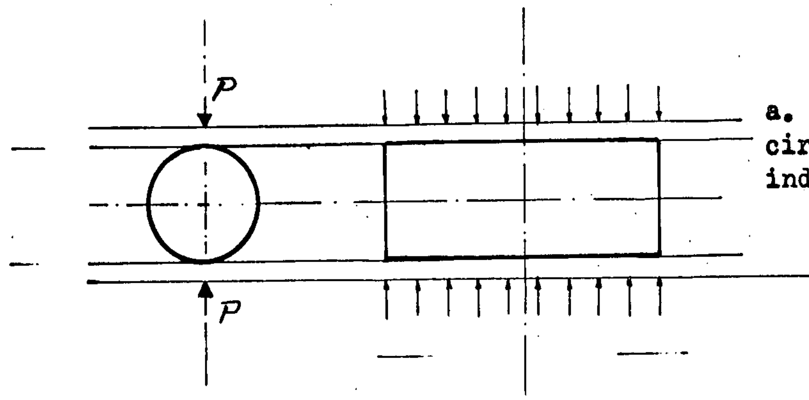
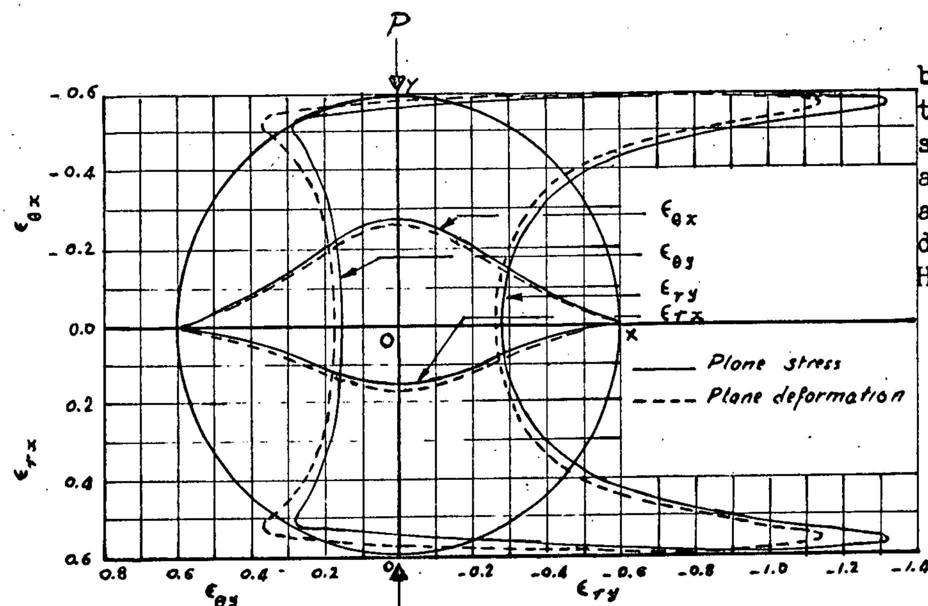
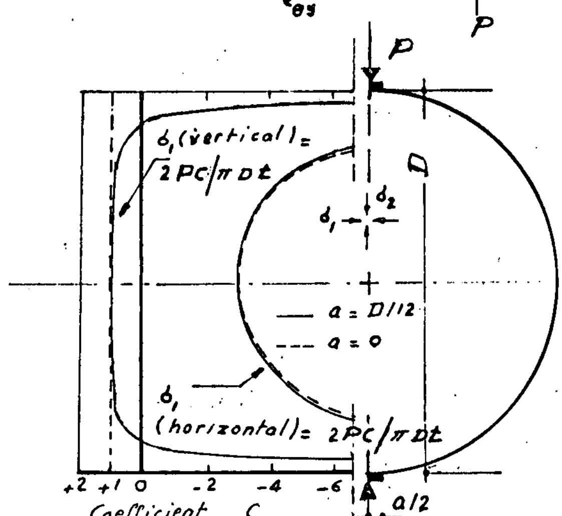
(42): Evans, R.H. Contribution to Comments on an indirect tensile test on concrete cylinders. Mag. of Conc.Research. Vol.8: No.22. March 1956. pp. 48-49.
(29): Carniero, F.L.L.B. and Barrellos, A Tensile strength of concrete (Resistance a la traction des betons) R.I.L.E.M. Bull. 13, March 1953. pp. 99-123.



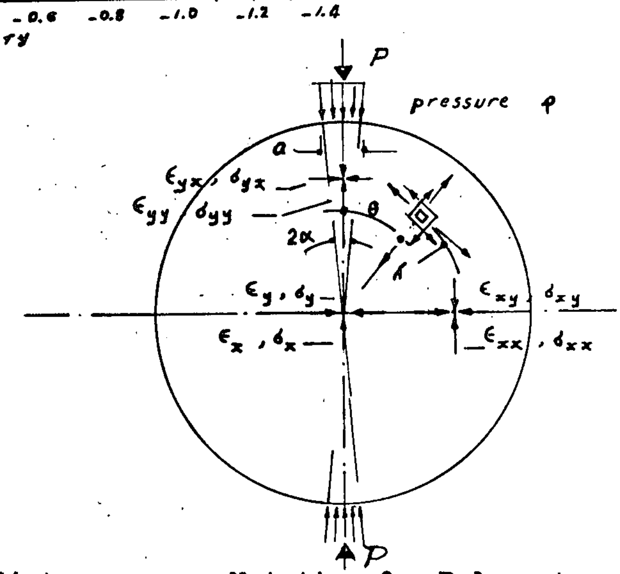
a. Method of testing a circular specimen for an indirect-tension test.



b. An example for the theoretical strain distribution along the vertical and horizontal diameters. Due to Hondros[67].



c. The influence of loading-strip on the distribution of vertical and horizontal stresses across the vertical diameter. Due to Newman[99]. (only a part of the figure is shown, to illustrate the conditions at the centre)



c. Notation for Polar stress components, and Cartisian stress components along the vertical and horizontal diameters. (The case analysed)

Figure 5.1:
Method distribution of strains, stresses , and notation for a circular specimen tested for an indirect-tension test.

majority of the area of the diametral plane of loading form a uniform tensile stress, which results in fracture along this plane. The tensile strength of the specimen can be assessed in terms of the applied load and the dimensions of the specimen.

Since the technique was originated, it has been recognised as reliable but both the original and recent recommendations have limited its use to measuring the tensile strength. Among the former were A.S.T.M., [3], Carniero [29], Mitchell [94], McNeely[92], Narrow and Ulberg [98], Simmon [121], Thaulow [135] and Wright [146]. Among recent recommendations which included new suggestions are Hiramatsu and Oka [66], Nilson [102], Rilem Bulletin[110], and Welch[141]

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(3): American Society for Testing and Materials. Tentative method of test splitting tensile strength of moulded concrete cylinders. N. C.496-62T. 1962. Suppliment to Book of A.S.T.M. Standards, Part 4.
(94): Mitchell, N.B.Jr. The indirect tension test for concrete Materials Research and Standards., 1961, (10, 780-788).
(92): Mc. Neely, D.J. and Last,S.D. Tensile strength of concrete Jl. of the Amer.Conc. Institute proceedings. Vol.60, No.6, June 1963. pp. 751-76.
(98): Narrow,I. and Ullberg, E. Correlation between tensile splitting strength and flexural strength of concrete. Proc.Amer.Conc.Institute. January, 1962, 60 1), pp. 27-28.
(121): Simmon, L. Quality control of pavement concrete using indirect tensile test). Constr. Review, 29 August, 1956.
(135): Thaulow, S. Tensile splitting test and high strength concrete test cylinders. Jl. of the Amer.Conc.Institute Vol. 28 No.7. Jan. 1957. pp. 699-706. Discussion: Proceedings Vol. 29, No.6. Part 2, December 1957, pp. 1315-1325.
(146): Wright, P.J.F. Comments on an indirect tensile test on concrete cylinders. Mag. of Concrete Research, Vol.7: No.20. July, 1955. pp. 87-96.

In fact, the stresses accompanying the conventional pattern when the load is applied diametrally to a cylinder could be of use beyond the determination of tensile strength. The pattern can become a biaxial, state providing a situation from which the stress-deformation properties could be assessed.

Before showing how the test can be developed for assessing the modulus of elasticity and Poisson's ratio, it was felt better to quote, shortly, the merits of the technique caused it to be chosen by the author. Although most of its merits as determined below were proved to be valid with the tensile tests, they can also be shown to apply with the new technique which will be proposed later.

1. The frictional restraint usually present at the interfaces of contact between test specimen and the machine platens is kept to a minimum.

2. It is considered to be the first test which gives a reliable measure of the tensile strength of a material like concrete.

3. It leads to smaller testing errors and less variation in the results than the direct tensile test and the flexural test.(38) (52)(38)(144) Although uncertainty might thereby be reduced, it is to be remembered that it is usually less uniform than the compression test.

4. It seems preferable to the standard test for flexure especially as the splitting tests are usually made on smaller compact specimens that are less susceptible to damage.

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(66): Hiramatsu,Y. and Oka,V. Determination of the tensile strength of rock by a compression test of an irregular test piece. International Jl. Rock Mechanics. Vol.3., Pergamon Press, 1966. pp.89-99.

5. Splitting failure occurs through the central portion of the specimen and it is less likely to be affected by surface imperfections, partial drying of specimens under test, or direction of casting.

6. The test usually gives results higher than those from direct tension tests, and lower than the modulus of rupture, so that the results of this test are not inclined towards an extreme.

7. Simplicity in performance.

The first to consider employing the technique for assessing Poisson's ratio and modulus of elasticity was Hondros.(67) If his test piece has not been inconvenient and uneconomical for practical purposes or long term research, and the calculating formulae not so complicated, the test could have been a useful device for measuring these deformation properties as well as the tensile strength. With the author's appreciation for Hondros's valuable work, this unsuitability has

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continued

(67): Hondros, G. The evaluation of Poissons ratio and modulus of elasticity of a material of low resistance by the Brazilian(indirect tensile) test with particular reference to concrete. Australian Journal of Applied Science, Vol. 10. No.3. September 1959. pp.243 - 268.
(102): Nilson, S. The tensile strength of concrete, determined by splitting tests on cubes. Rilem Bulletin. 1961(11), New Series. pp. 63-67.
(110): Rilem Bulletin, Materials and Structures, New Series. No.30. March, 1966. A new method of sampling, making, curing, and strength testing of concrete.
(141): Welch, G.B. Tensile splitting test on concrete cubes and beams. Civil Engineering and Public Works Review, Aug. 1965. pp.1161-1167.

been considered in the form of two main comments. In the author's opinion, they can be considered the main drawbacks which have discouraged the development and use of the test.

1. The test specimen suggested by Hondros was a disc of 24 inches diameter and 2 inches thickness. Such a specimen is not suitable for carrying out either a long research programme or careful routine tests with a considerable number of specimens in the one test.

2. Hondros used polar coordinates throughout his analysis, and this led him to derive the values of Poisson's ratio and the modulus of elasticity in the form of expressions which resulted in tedious calculation.

With these two comments in mind it was decided to make an attempt to develope for the indirect tension test, a better technique. The basis of the new technique lies in modifying the indirect-tension test in such a way that the shape and dimensions of the specimen, and the method of calculation, lead to a simpler laboratory technique, and an easier but still reliable method of calculation.

To avoid reference at this point to the next two sections, a fact about the chosen test specimen should be mentioned here: although it has not arisen before in the determination of the indirect tensile strength, it is of great influence when determining the deformation properties. When carrying out the test, there are two main factors as regards the specimen dimensions which affect the pattern of stresses, namely the diameter and thickness of the specimen. There is a certain limit for the thickness/diameter ratio below which the specimen is a disc, and above which the specimen becomes a cylinder. The stress analyses for the two cases are different, as illustrated graphically in Figure 5.1-b; while in the former the state of plane stress exists, in the latter it is replaced by the state of plane deformation. In spite of the complete identity in their laboratory performance and calculation of the tensile strength, they differ when calculating the modulus of elasticity and Poisson's ratio. With two different conditions in mind, therefore, the new technique will be introduced. Its introduction involves three stages of proposals for developing the indirect tensile test for the assessment of the properties in question, while justifying many of the above-mentioned considerations.

The first stage is the extension of the previous stress analysis in deriving new calculating formulae, and then simplifying so far as possible the steps of calculation. The second and third stages are concerned with developments in the method of testing in the laboratory at the same time complying with the theoretical aspect of the first stage.

5.4 NOTATION

1. Specimen and load.

D,R,t	diameter, radius and thickness of the disc respectively.
p	applied load expressed as a pressure.
a, 2α	projected width of the loaded section of the rim, and angle subtended at the origin.
P	(p.a.t) applied load.

2. Coordinates

Ox	horizontal axis of reference.
Oy	vertical axis of reference.
Oz	longitudinal axis perpendicular to the plane of the paper.
r	radial distance of a point from the origin.
θ	angular displacement of a point from the axis.

3. Coefficients, strains, and stresses

E	Young's modulus of elasticity.
ν	Poisson's ratio.
σ	normal stress.
τ	shear stress.
ϵ	strain
ϵ_θ, σ_θ	tangential strain and tangential s ress at a point r,θ.
$(\epsilon_{\theta x}, \sigma_{\theta x})$ or $(\epsilon_{xy}, \sigma_{xy})$	tangential strain and tagential normal stress at a point situated on the X axis (acting parallel to the Y axis).

$(\epsilon_{\theta y}, \sigma_{\theta y})$ or $(\epsilon_{yx}, \sigma_{yx})$	tangential strain and tangential normal stress at a point situated on the Y axis (acting parallel to the Y axis).
ϵ_r, σ_r	radial strain and radial normal stress at a point r, θ.
$(\epsilon_{rx}, \sigma_{rx})$ or $(\epsilon_{xx}, \sigma_{xx})$	radial strain and radial normal stress at a point situated on the X axis (acting along the Y axis)
$(\epsilon_{ry}, \sigma_{ry})$ or $(\epsilon_{yy}, \sigma_{yy})$	radial strain radial stress at a point situated at a point situated on the Y axis (acting along the Y axis).
ϵ_x, σ_x	longitudinal strain and stress components along strain and stress at the centre of the disc along the X axis.
ϵ_y, σ_y	strain and stress at the centre of the disc along the Y axis.
ϵ_z, σ_z	longitudinal strain and stress components along the Z axis.

4. Sign conventions

– (ve)	compressive effects.
+ (ve)	tensile effects.

5.5 MATHEMATICAL DERIVATION OF NEW EXPRESSIONS FOR CALCULATING MODULUS OF ELASTICITY AND POISSON'S RATIO

5.5.1 General

Various authorities, including Frocht(46), Hondros (67), Peltier(105), Timoshenko(136), Wright(146), have either carried out or discussed the stress analysis of a circular element subjected to concentrated forces at its boundary, so that it is not necessary to refer to the detailed method of the early stages of derivation. And, because the analysis will be extended further to new expressions it is felt to be enough to start from the end expressions as yielded from the stress analysis.

5.5.2 The Case of Plane Stress

For the disc shown in Figure 5.1-a subjected to a radially applied short strip of loading whose breadth is not more than 1/12 of the diameter (practically o- 1/12 give no difference as regards the stresses at the centre as shown in Figure 5.1.c) and where the state of plane stress exists, with its self weight neglected, the stress analysis yields the following four expressions for the stress distributions along the X and Y axes

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(46): Frocht, M.M. Photoelasticity. Vol. 2. John Wiley and Sons. New York. i9.

(105): Peltier, M. Theoretical investigation of the Brazilian test (Etude Theorique De L'Essai Brezilien). Rilem Bulletin. No.19, October 1954, pp. 33-74.
(136): Timoshenko, S. and Goodier, J.N. Theory of elasticity. McGraw-Hill Book Company, 1951.

$$\sigma_{\theta y} = \frac{2p}{\pi}\left[\frac{(1 - r^2/R^2)\sin 2\alpha}{(1 - 2r^2/R^2 \cos 2\alpha + r^4/R^4} - \tan^{-1}\frac{(1 + r^2/R^2)}{(1 - r^2/R^2)} . \tan\alpha\right] \quad (5.1)$$

$$\sigma_{ry} = -\frac{2p}{\pi}\left[\frac{(1 - r^2/R^2)\cdot\sin 2\alpha}{(1 - 2r^2/R^2 \cos 2\alpha + r^4/R^4} + \tan^{-1}\frac{(1 + r^2/R^2)}{(1 - r^2/R^2)} . \tan\alpha\right] \quad (5.2)$$

$$\tau_{r\theta} = 0 \quad (5.3)$$

$$\sigma_{\theta x} = -\frac{2p}{\pi}\left[\frac{(1 - r^2/R^2)\sin 2\alpha}{(1 + 2r^2/R^2 \cos 2\alpha - r^4/R^4} + \tan^{-1}\frac{(1 - r^2/R^2)}{(1 + r^2/R^2)} . \tan\alpha\right] \quad (5.4)$$

$$\sigma_{rx} = \frac{2p}{\pi}\left[\frac{(1 - r^2/R^2)\sin 2\alpha}{(1 + 2r^2/R^2 \cos 2\alpha + r^4/R^4} - \tan^{-1}\frac{(1 - r^2/R^2)}{(1 + r^2/R^2)} . \tan\alpha\right] \quad (5.5)$$

$$\tau_{r\theta} = 0 \quad (5.6)$$

Considering the case near the centre when the load is applied in compliance with the indirect-tension (Braxilian) test (strip width = 1/12 to 1/10 according to Wright and Hondros respectively) then:

On the Y axis:

$$\sigma_{\theta y} = \frac{2p\alpha}{\pi} \quad (5.7)$$

$$\sigma_{ry} = \frac{2p\alpha}{\pi}\left[1 - \frac{4}{(1 - r^2/R^2)}\right] \quad (5.8)$$

$$\sigma_{\theta x} = \frac{2p\alpha}{\pi}\left[1 - \frac{4}{(1 + r^2/R^2)}\right] \quad (5.9)$$

$$\sigma_{rx} = \frac{2p\alpha}{\pi}\left[1 - \frac{4r^2/R^2}{(1 + r^2/R^2)}\right] \quad (5.10)$$

Introducing Cartesian coordinates with coordinates equatrons (5.7), (5.8), (5.9), (5.10) can be written respectively in terms of the load and in Cartisian forms as follows: (TIR = 0), (= a/D)

$$\delta_{yx} = \frac{2P}{\pi t} \tag{5.11}$$

$$\delta_{yy} = -\frac{2P}{\pi tD}\left[\frac{4D^2}{(D^2 - 4r^2)} - 1\right] \tag{5.12}$$

$$\delta_{xy} = -\frac{2P}{\pi tD}\left[\frac{4D^2}{(D^2 + 4r^2} - 1\right] \tag{5.13}$$

$$\delta_{xx} = \frac{2P}{\pi tD}\left[1 - \frac{16r^2D^2}{(D^2 + 4r^2)^2}\right] \tag{5.14}$$

At the centre of the disc the stresses become as follows:

From equations (5.11) and (5.14) we get:

$$\delta_{yx} = \delta_{xx} = \delta_x = \frac{2P}{\pi t D} \tag{5.15}$$

From equations (5.13) and (5.14) we get:

$$\delta_{yy} = \delta_{xy} = \delta_y = -\frac{6P}{\pi t D} \tag{5.16}$$

From equations (5.15) and 5.16) we get:

$$\delta_y = -3\,\delta_x \tag{5.17}$$

Applying Hooke's law for a homogeneous and isotropic material: (equations 5.18) and (5.19) then:

$$\epsilon_x = \frac{1}{E}\left[\delta_x - \nu\delta_y\right] \tag{5.18}$$

$$\epsilon_y = \frac{1}{E}\left[\delta_y - \nu\delta_x\right] \tag{5.19}$$

Eliminating E from (5.18) and (5.19) we get:

$$\nu = \frac{\epsilon_x \delta_y - \epsilon_y \delta_x}{\epsilon_x \delta_x - \epsilon_y \delta_y} \tag{5.20}$$

Substituting from (5.17) and (5.19) in (5.20) we get Poisson's ratio in terms of the centre strains as:

$$\boxed{\nu = -\frac{3\epsilon_x + \epsilon_y}{\epsilon_x + 3\epsilon_y}} \tag{5.21}$$

Considering the absolute values of the strains at the centre and as counting for the sign conventions, equation (5.21) can be written in the form:

$$\boxed{\nu = \frac{3 - \dfrac{\bar{\epsilon}_y}{\bar{\epsilon}_x}}{3\dfrac{\bar{\epsilon}_y}{\bar{\epsilon}_x} - 1}} \tag{5.22}$$

Also eliminating from equations (5.18) and (5.19) we get:

$$E = \frac{\delta_x^2 - \delta_y^2}{\epsilon_x \delta_x - \epsilon_y \delta_y} \tag{5.23}$$

Substituting from (5.15) and (5.16) in (5.23) we get the modulus of elasticity in terms of the centre strains and the specimen constants as:

$$\boxed{E = 5.095 \times \frac{P}{Dt} \times \frac{1}{(\epsilon_x + 3\epsilon_y)}} \tag{5.24}$$

Using equations (5.22) and (5.24) Poisson's ratio and the modulus of elasticity could be respectively determined in terms of the strains at the centre and the applied load.

Equation (5.22) shows interesting characteristic. It depends absolutely on the value of the ratio between the vertical strain and the horiznntal strain regardless of the test specimen constans and the signs of the strains. Consequently it was possible to interpret equation (5.22) in the form of the tables and charts shown in Table (5.1) and Figure (5.2) respectively. Using these tables or charts makes it much easier to determine the value of Poisson's ratio, simply by determining the ratio of the vertical strain to the horizontal strain in terms of absolute values.

As regards the modulus of elasticity, an alternative to equations (5.24) and (5.23) could be used. In this case, the stresses at the centre of the disc and the applied load can be determined using equations (5.15) and (5.16), but this leads to more tedious calculations.

Employing equation (5.15), the tensile strength may be determined in the usual way.

5.5.3 The Case of Plane Deformation

In the case of plane deformation the sequence of deriving the calculating formulae is similar to the plane stress case, and equations 5.1 - 5.17 are applicable.

Then equations (5.18) and (5.19) may be written respectively as :

$$\epsilon_x = \frac{1+\nu}{E}\left[(1-\nu)\sigma_x - \nu\sigma_y\right] \qquad (5.25)$$

$$\epsilon_y = \frac{1+\nu}{E}\left[(1-\nu)\sigma_y - \nu\sigma_x\right] \qquad (5.26)$$

Eliminating E from (5.25) and (5.26) we get:

$$\nu = \frac{\epsilon_x \delta_y - \epsilon_y \delta_x}{(\delta_x + \delta_y)(\epsilon_x - \epsilon_y)} \tag{5.27}$$

Substituting from (5.1) in (5.27) we get:

$$\boxed{\nu = \frac{3\epsilon_x + \epsilon_y}{2(\epsilon_x - \epsilon_y)}} \tag{5.28}$$

In a similar manner to (5.22) equation (5.28) can be written in terms of the absolute values of strains as:

$$\boxed{\nu = \frac{3 - \dfrac{\bar{\epsilon}_y}{\bar{\epsilon}_x}}{2\left(\dfrac{\bar{\epsilon}_y}{\bar{\epsilon}_x} - 1\right)}} \tag{5.29}$$

Also eliminating δ_y from (5.25) and (5.26) we get:

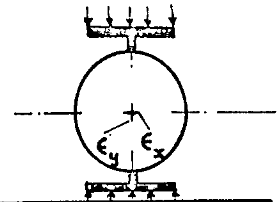
$$E = \frac{\delta_x (1 + \nu)(1 - 2\nu)}{\epsilon_x (1 - \nu) + \epsilon_y \nu} \tag{5.30}$$

Similarly eliminating δ_x from (5.25) and (5.26) we get

$$E = \frac{\delta_y (1 + \nu)(1 - 2\nu)}{\epsilon_y (1 - \nu) + \epsilon_x \nu} \tag{5.31}$$

$$\boxed{E = 0.6369 \times \left[\frac{P}{Dt}\right] \times \frac{(1 + \nu)(1 - 2\nu)}{\left[\epsilon_x (1 - \nu) + \epsilon_y \nu\right]}} \tag{5.32}$$

7.5.4. Tables for the Determination of Poisson's Ratio.

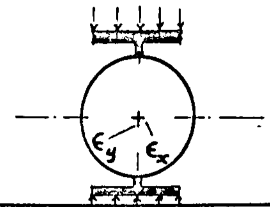


Poisson's Ratio Tables (Table 5.1).

Case: Plane stress.

$\bar{\epsilon}_y/\bar{\epsilon}_x$ +	P'R. *	$\bar{\epsilon}_y/\bar{\epsilon}_x$	P'R.	$\bar{\epsilon}_y/\bar{\epsilon}_x$	P'R.	$\bar{\epsilon}_y/\bar{\epsilon}_x$	P'R.	$\bar{\epsilon}_y/\bar{\epsilon}_x$	P'R.
1.000	1.0000	1.200	0.6932	1.400	0.5000	1.600	0.3684	1.800	0.2727
1.005	0.9901	1.205	0.6864	1.405	0.4961	1.605	0.3657	1.805	0.2707
1.010	0.9803	1.210	0.6806	1.410	0.4923	1.610	0.3629	1.810	0.2686
1.015	0.9701	1.215	0.6749	1.415	0.4884	1.615	0.3602	1.815	0.2665
1.020	0.9612	1.220	0.6692	1.420	0.4847	1.620	0.3575	1.820	0.2645
1.025	0.9518	1.225	0.6635	1.425	0.4809	1.625	0.3548	1.825	0.2626
1.030	0.9426	1.230	0.6580	1.430	0.4772	1.630	0.3522	1.830	0.2606
1.035	0.9335	1.235	0.6525	1.435	0.4735	1.635	0.3495	1.835	0.2586
1.040	0.9245	1.240	0.6471	1.440	0.4700	1.640	0.3469	1.840	0.2566
1.045	0.9157	1.245	0.6417	1.445	0.4663	1.645	0.3443	1.845	0.2547
1.050	0.9070	1.250	0.6364	1.450	0.4627	1.650	0.3412	1.850	0.2527
1.055	0.8984	1.255	0.6311	1.455	0.4591	1.655	0.3392	1.855	0.2508
1.060	0.8899	1.260	0.6259	1.460	0.4556	1.660	0.3367	1.860	0.2489
1.065	0.8815	1.265	0.6207	1.465	0.4521	1.665	0.3342	1.865	0.2470
1.070	0.8733	1.270	0.6157	1.470	0.4487	1.670	0.3317	1.870	0.2451
1.075	0.8652	1.275	0.6106	1.475	0.4452	1.675	0.3292	1.875	0.2432
1.080	0.8571	1.280	0.6056	1.480	0.4419	1.680	0.3267	1.880	0.2414
1.085	0.8492	1.285	0.6007	1.485	0.4385	1.685	0.3243	1.885	0.2395
1.090	0.8414	1.290	0.5958	1.490	0.4352	1.690	0.3219	1.890	0.2377
1.095	0.8337	1.295	0.5910	1.495	0.4318	1.695	0.3195	1.895	0.2359
1.100	0.8261	1.300	0.5862	1.500	0.4286	1.700	0.3171	1.900	0.2340
1.105	0.8186	1.305	0.5815	1.505	0.4253	1.705	0.3147	1.905	0.2322
1.110	0.8112	1.310	0.5768	1.510	0.4221	1.710	0.3123	1.910	0.2304
1.115	0.8038	1.315	0.5722	1.515	0.4189	1.715	0.3100	1.915	0.2287
1.120	0.7966	1.320	0.5676	1.520	0.4157	1.720	0.3077	1.920	0.2269
1.125	0.7895	1.325	0.5630	1.525	0.4125	1.725	0.3054	1.925	0.2251
1.130	0.7824	1.330	0.5585	1.530	0.4095	1.730	0.3031	1.930	0.2234
1.135	0.7755	1.335	0.5541	1.535	0.4063	1.735	0.3008	1.935	0.2216
1.140	0.7686	1.340	0.5491	1.540	0.4033	1.740	0.2986	1.940	0.2199
1.145	0.7618	1.345	0.5453	1.545	0.4003	1.745	0.2963	1.945	0.2182
1.150	0.7551	1.350	0.5410	1.550	0.3973	1.750	0.2941	1.950	0.2165
1.155	0.7485	1.355	0.5367	1.555	0.3943	1.755	0.2919	1.955	0.2148
1.160	0.7419	1.360	0.5325	1.560	0.3913	1.760	0.2897	1.960	0.2131
1.165	0.7355	1.365	0.5283	1.565	0.3883	1.765	0.2875	1.965	0.2114
1.170	0.7291	1.370	0.5241	1.570	0.3854	1.770	0.2854	1.970	0.2100
1.175	0.7228	1.375	0.5200	1.575	0.3825	1.775	0.2832	1.975	0.2081
1.180	0.7165	1.380	0.5160	1.580	0.3800	1.780	0.2811	1.980	0.2065
1.185	0.7104	1.385	0.5119	1.585	0.3768	1.785	0.2790	1.985	0.2048
1.190	0.7043	1.390	0.5079	1.590	0.3740	1.790	0.2769	1.990	0.2032
1.195	0.7983	1.395	0.5023	1.595	0.3712	1.795	0.2749	1.995	0.2016

+ Absolute ratio.
* Poisson's ratio.



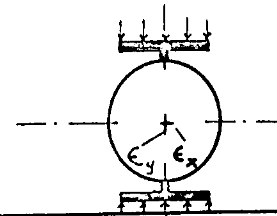
Poisson's Ratio Tables (Table 5.1. contd.)

Case: Plane stress (continued)

$\bar{\epsilon}_y/\bar{\epsilon}_x$ +	P'R. *	$\bar{\epsilon}_y/\bar{\epsilon}_x$	P'R.	$\bar{\epsilon}_y/\bar{\epsilon}_x$	P'R.	$\bar{\epsilon}_y/\bar{\epsilon}_x$	P'R.	$\bar{\epsilon}_y/\bar{\epsilon}_x$	P'R.
2.000	0.2000	2.200	0.1429	2.400	0.0968	2.600	0.0588	2.800	0.0270
2.005	0.1984	2.205	0.1416	2.405	0.0957	2.605	0.0580	2.805	0.0263
2.010	0.1968	2.210	0.1403	2.410	0.0947	2.610	0.0571	2.810	0.0256
2.015	0.1952	2.215	0.1391	2.415	0.0937	2.615	0.0562	2.815	0.0248
2.020	0.1937	2.220	0.1378	2.420	0.0926	2.620	0.0554	2.820	0.0241
2.025	0.1921	2.225	0.1366	2.425	0.0916	2.625	0.0545	2.825	0.0234
2.030	0.1906	2.230	0.1352	2.430	0.0906	2.630	0.0537	2.830	0.0227
2.035	0.1890	2.235	0.1341	2.435	0.0896	2.635	0.0529	2.835	0.0220
2.040	0.1875	2.240	0.1329	2.440	0.0886	2.640	0.0520	2.840	0.0213
2.045	0.1851	2.245	0.1316	2.445	0.0876	2.645	0.0512	2.845	0.0206
2.050	0.1845	2.250	0.1304	2.450	0.0866	2.650	0.0504	2.850	0.0199
2.055	0.1830	2.255	0.1292	2.455	0.0856	2.655	0.0495	2.855	0.0192
2.060	0.1815	2.260	0.1280	2.460	0.0846	2.660	0.0487	2.860	0.0185
2.065	0.1800	2.265	0.1268	2.465	0.0837	2.665	0.0479	2.865	0.0178
2.070	0.1785	2.270	0.1256	2.470	0.0827	2.670	0.0470	2.870	0.0171
2.075	0.1770	2.275	0.1245	2.475	0.0817	2.675	0.0463	2.875	0.0164
2.080	0.1756	2.280	0.1233	2.480	0.0807	2.680	0.0454	2.880	0.0157
2.085	0.1741	2.285	0.1221	2.485	0.0798	2.685	0.0446	2.885	0.0150
2.090	0.1727	2.290	0.1209	2.490	0.0788	2.690	0.0438	2.890	0.0143
2.095	0.1712	2.295	0.1198	2.495	0.0779	2.695	0.0430	2.895	0.0137
2.100	0.1698	2.300	0.1186	2.500	0.0769	2.700	0.0422	2.900	0.0130
2.105	0.1684	2.305	0.1175	2.505	0.0760	2.705	0.0415	2.905	0.0123
2.110	0.1670	2.310	0.1164	2.510	0.0750	2.710	0.0407	2.910	0.0116
2.115	0.1658	2.315	0.1152	2.515	0.0741	2.715	0.0399	2.915	0.0110
2.120	0.1642	2.320	0.1141	2.520	0.0732	2.720	0.0391	2.920	0.0103
2.125	0.1628	2.325	0.1130	2.525	0.0722	2.725	0.0383	2.925	0.0096
2.130	0.1614	2.330	0.1118	2.530	0.0713	2.730	0.0375	2.930	0.0090
2.135	0.1600	2.335	0.1107	2.535	0.0704	2.735	0.0368	2.935	0.0083
2.140	0.1587	2.340	0.1096	2.540	0.0695	2.740	0.0360	2.940	0.0077
2.145	0.1573	2.345	0.1085	2.545	0.0686	2.745	0.0352	2.945	0.0070
2.150	0.1560	2.350	0.1074	2.550	0.0677	2.750	0.0345	2.950	0.0064
2.155	0.1546	2.355	0.1063	2.555	0.0668	2.755	0.0337	2.955	0.0057
2.160	0.1533	2.360	0.1053	2.560	0.0659	2.760	0.0330	2.960	0.0051
2.165	0.1519	2.365	0.1042	2.565	0.0650	2.765	0.0322	2.965	0.0044
2.170	0.1506	2.370	0.1031	2.570	0.0641	2.770	0.0315	2.970	0.0038
2.175	0.1493	2.375	0.1020	2.575	0.0632	2.775	0.0307	2.975	0.0031
2.180	0.1480	2.380	0.1010	2.580	0.0623	2.780	0.0300	2.980	0.0025
2.185	0.1467	2.385	0.0999	2.585	0.0614	2.785	0.0292	2.985	0.0019
2.190	0.1454	2.390	0.0989	2.590	0.0606	2.790	0.0285	2.990	0.0012
2.195	0.1441	2.395	0.0978	2.595	0.0597	2.795	0.0278	2.995	0.0006
								3.000	0.0000

+ Absolute ratio.

* Poisson's ratio.

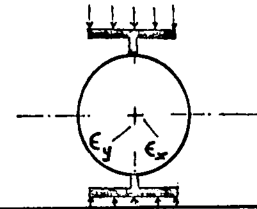


Poisson's Ratio Tables (Table 5.2)

Case: Plane deformation.

$\bar{\epsilon}_y/\bar{\epsilon}_x$ +	P'R. *	$\bar{\epsilon}_y/\bar{\epsilon}_x$	P'R.	$\bar{\epsilon}_y/\bar{\epsilon}_x$	P'R.	$\bar{\epsilon}_y/\bar{\epsilon}_x$	P'R.	$\bar{\epsilon}_y/\bar{\epsilon}_x$	P'R.
1.000	0.5000	1.200	0.4091	1.400	0.3333	1.600	0.2692	1.800	0.2143
1.005	0.4975	1.205	0.4070	1.405	0.3316	1.605	0.2677	1.805	0.2130
1.010	0.4950	1.210	0.4050	1.410	0.3298	1.610	0.2663	1.810	0.2117
1.015	0.4926	1.215	0.4029	1.415	0.3281	1.615	0.2648	1.815	0.2105
1.020	0.4901	1.220	0.4009	1.420	0.3264	1.620	0.2634	1.820	0.2092
1.025	0.4876	1.225	0.3989	1.425	0.3247	1.625	0.2619	1.825	0.2080
1.030	0.4852	1.230	0.3969	1.430	0.3230	1.630	0.2605	1.830	0.2067
1.035	0.4828	1.235	0.3948	1.435	0.3213	1.635	0.2590	1.835	0.2055
1.040	0.4804	1.240	0.3929	1.440	0.3197	1.640	0.2576	1.840	0.2042
1.045	0.4780	1.245	0.3909	1.445	0.3180	1.645	0.2561	1.845	0.2030
1.050	0.4756	1.250	0.3889	1.450	0.3163	1.650	0.2547	1.850	0.2017
1.055	0.4732	1.255	0.3869	1.455	0.3147	1.655	0.2533	1.855	0.2005
1.060	0.4709	1.260	0.3850	1.460	0.3130	1.660	0.2519	1.860	0.1993
1.065	0.4685	1.265	0.3830	1.465	0.3114	1.665	0.2505	1.865	0.1981
1.070	0.4662	1.270	0.3811	1.470	0.3097	1.670	0.2490	1.870	0.1979
1.075	0.4638	1.275	0.3791	1.475	0.3081	1.675	0.2477	1.875	0.1956
1.080	0.4615	1.280	0.3772	1.480	0.3064	1.680	0.2462	1.880	0.1944
1.085	0.4592	1.285	0.3753	1.485	0.3048	1.685	0.2449	1.885	0.1932
1.090	0.4569	1.290	0.3734	1.490	0.3032	1.690	0.2435	1.890	0.1920
1.095	0.4546	1.295	0.3715	1.495	0.3016	1.695	0.2421	1.895	0.1908
1.100	0.4524	1.300	0.3696	1.500	0.3000	1.700	0.2407	1.900	0.1896
1.105	0.4501	1.305	0.3677	1.505	0.2984	1.705	0.2394	1.905	0.1885
1.110	0.4479	1.310	0.3658	1.510	0.2968	1.710	0.2380	1.910	0.1873
1.115	0.4456	1.315	0.3639	1.515	0.2952	1.715	0.2366	1.915	0.1861
1.120	0.4434	1.320	0.3621	1.520	0.2936	1.720	0.2353	1.920	0.1849
1.125	0.4412	1.325	0.3602	1.525	0.2921	1.725	0.2340	1.925	0.1838
1.130	0.4390	1.330	0.3584	1.530	0.2905	1.730	0.2326	1.930	0.1826
1.135	0.4368	1.335	0.3565	1.535	0.2889	1.735	0.2313	1.935	0.1814
1.140	0.4346	1.340	0.3547	1.540	0.2874	1.740	0.2299	1.940	0.1803
1.145	0.4324	1.345	0.3529	1.545	0.2858	1.745	0.2286	1.945	0.1791
1.150	0.4302	1.350	0.3511	1.550	0.2843	1.750	0.2273	1.950	0.1780
1.155	0.4281	1.355	0.3492	1.555	0.2828	1.755	0.2259	1.955	0.1768
1.160	0.4259	1.360	0.3475	1.560	0.2812	1.760	0.2246	1.960	0.1757
1.165	0.4238	1.365	0.3457	1.565	0.2797	1.765	0.2233	1.965	0.1745
1.170	0.4217	1.370	0.3439	1.570	0.2782	1.770	0.2220	1.970	0.1734
1.175	0.4195	1.375	0.3421	1.575	0.2767	1.775	0.2207	1.975	0.1723
1.180	0.4174	1.380	0.3403	1.580	0.2752	1.780	0.2194	1.980	0.1711
1.185	0.4153	1.385	0.3386	1.585	0.2737	1.785	0.2181	1.985	0.1700
1.190	0.4132	1.390	0.3368	1.590	0.2722	1.790	0.2168	1.990	0.1689
1.195	0.4112	1.395	0.3350	1.595	0.2707	1.795	0.2157	1.995	0.1678

+ Absolute ratio.
* Poisson's ratio.



Poisson's Ratio Tables (Table 5.2 contd.)

Case: Plane deformation (continued)

$\bar{\epsilon}_y/\bar{\epsilon}_x$ +	P'R. *	$\bar{\epsilon}_y/\bar{\epsilon}_x$	P'R.	$\bar{\epsilon}_y/\bar{\epsilon}_x$	P'R.	$\bar{\epsilon}_y/\bar{\epsilon}_x$	P'R.	$\bar{\epsilon}_y/\bar{\epsilon}_x$	P'R.
2.000	0.1667	2.200	0.1250	2.400	0.0882	2.600	0.0555	2.800	0.0263
2.005	0.1656	2.205	0.1240	2.405	0.0874	2.605	0.0548	2.805	0.0256
2.010	0.1644	2.210	0.1230	2.410	0.0865	2.610	0.0540	2.810	0.0249
2.015	0.1633	2.215	0.1220	2.415	0.0856	2.615	0.0532	2.815	0.0242
2.020	0.1622	2.220	0.1211	2.420	0.0848	2.620	0.0525	2.820	0.0236
2.025	0.1612	2.225	0.1201	2.425	0.0839	2.625	0.0517	2.825	0.0229
2.030	0.1601	2.230	0.1192	2.430	0.0831	2.630	0.0510	2.830	0.0222
2.035	0.1590	2.235	0.1182	2.435	0.0822	2.635	0.0502	2.835	0.0215
2.040	0.1580	2.240	0.1173	2.440	0.0814	2.640	0.0494	2.840	0.0208
2.045	0.1568	2.245	0.1163	2.445	0.0805	2.645	0.0487	2.845	0.0202
2.050	0.1557	2.250	0.1154	2.450	0.0797	2.650	0.0479	2.850	0.0195
2.055	0.1547	2.255	0.1144	2.455	0.0789	2.655	0.0472	2.855	0.0188
2.060	0.1536	2.260	0.1135	2.460	0.0780	2.660	0.0464	2.860	0.0181
2.065	0.1525	2.265	0.1126	2.465	0.0772	2.665	0.0457	2.865	0.0175
2.070	0.1515	2.270	0.1116	2.470	0.0764	2.670	0.0450	2.870	0.0168
2.075	0.1504	2.275	0.1107	2.475	0.0755	2.675	0.0442	2.875	0.0161
2.080	0.1493	2.280	0.1098	2.480	0.0747	2.680	0.0435	2.880	0.0155
2.085	0.1483	2.285	0.1088	2.485	0.0739	2.685	0.0427	2.885	0.0148
2.090	0.1472	2.290	0.1079	2.490	0.0731	2.690	0.0420	2.890	0.0141
2.095	0.1462	2.295	0.1070	2.495	0.0722	2.695	0.0413	2.895	0.0135
2.100	0.1452	2.300	0.1061	2.500	0.0714	2.700	0.0405	2.900	0.0128
2.105	0.1441	2.305	0.1051	2.505	0.0706	2.705	0.0398	2.905	0.0122
2.110	0.1431	2.310	0.1042	2.510	0.0698	2.710	0.0391	2.910	0.0115
2.115	0.1420	2.315	0.1033	2.515	0.0690	2.715	0.0384	2.915	0.0109
2.120	0.1410	2.320	0.1024	2.520	0.0682	2.720	0.0376	2.920	0.0102
2.125	0.1400	2.325	0.1015	2.525	0.0674	2.725	0.0369	2.925	0.0095
2.130	0.1390	2.330	0.1006	2.530	0.0666	2.730	0.0362	2.930	0.0089
2.135	0.1380	2.335	0.0997	2.535	0.0658	2.735	0.0355	2.935	0.0083
2.140	0.1369	2.340	0.0988	2.540	0.0650	2.740	0.0348	2.940	0.0076
2.145	0.1359	2.345	0.0979	2.545	0.0642	2.745	0.0340	2.945	0.0070
2.150	0.1349	2.350	0.0970	2.550	0.0634	2.750	0.0333	2.950	0.0063
2.155	0.1339	2.355	0.0961	2.555	0.0626	2.755	0.0326	2.955	0.0057
2.160	0.1329	2.360	0.0952	2.560	0.0618	2.760	0.0319	2.960	0.0050
2.165	0.1319	2.365	0.0943	2.565	0.0610	2.765	0.0312	2.965	0.0044
2.170	0.1309	2.370	0.0935	2.570	0.0602	2.770	0.0305	2.970	0.0038
2.175	0.1299	2.375	0.0926	2.575	0.0594	2.775	0.0298	2.975	0.0031
2.180	0.1289	2.380	0.0917	2.580	0.0587	2.780	0.0291	2.980	0.0025
2.185	0.1279	2.385	0.0908	2.585	0.0579	2.785	0.0284	2.985	0.0018
2.190	0.1269	2.390	0.0900	2.590	0.0571	2.790	0.0277	2.990	0.0012
2.195	0.1260	2.395	0.0891	2.595	0.0563	2.795	0.0270	2.995	0.0006
								3.000	0.0000

\+ Absolute ratio.

* Poisson's ratio.

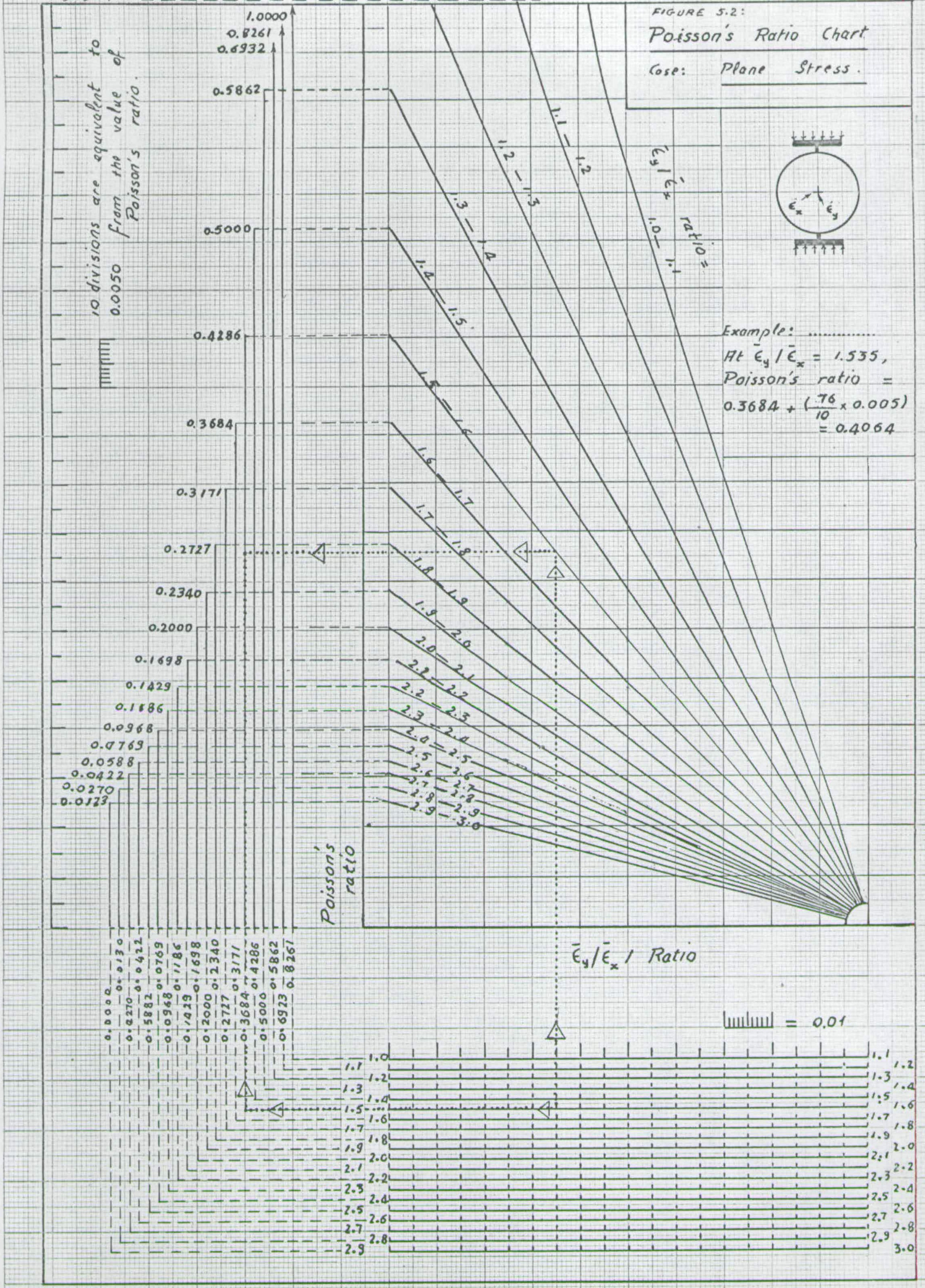


FIGURE 5.2: Poisson's Ratio Chart
Case: Plane Stress.

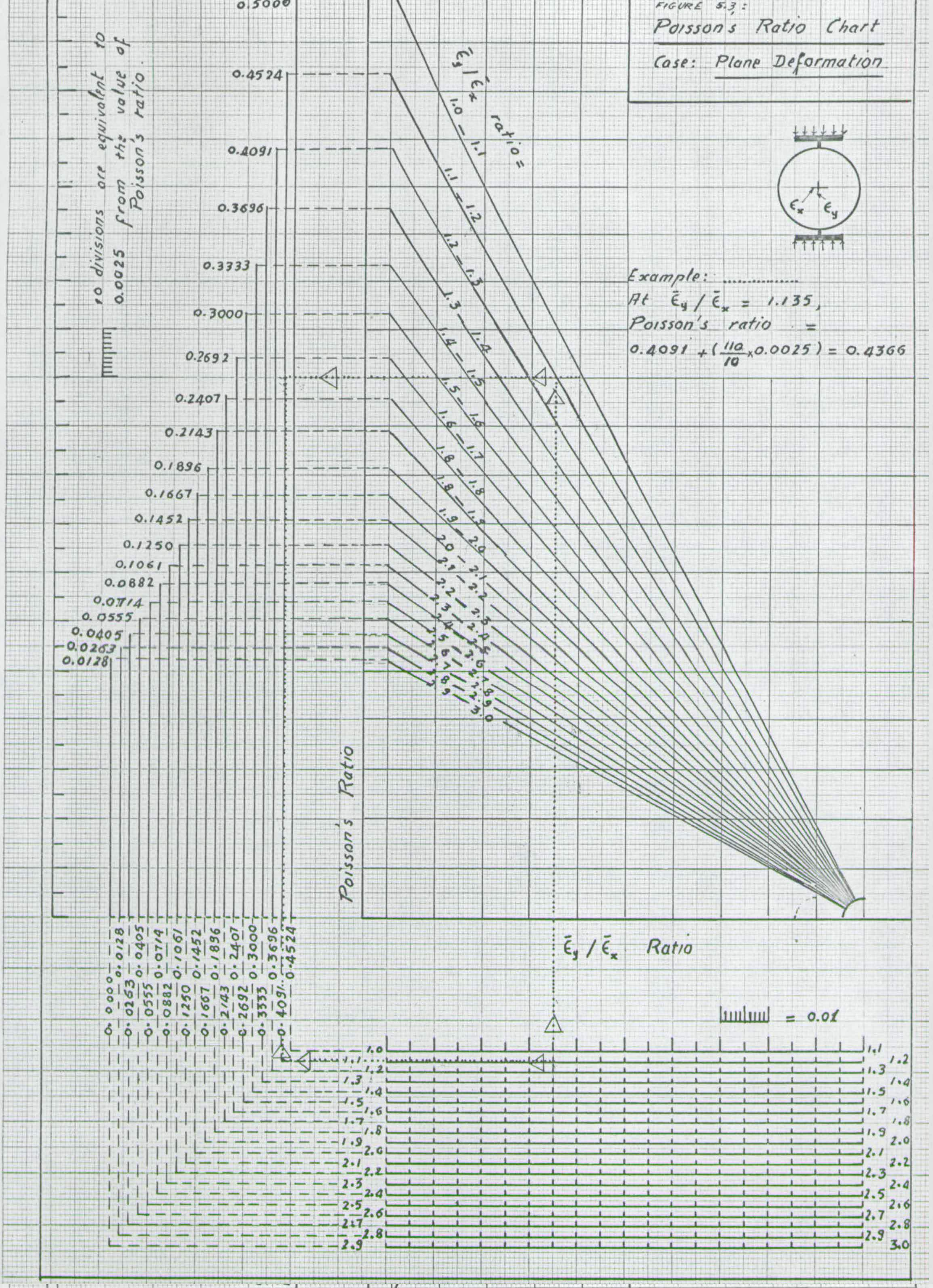


FIGURE 5.3: Poisson's Ratio Chart

Case: Plane Deformation

Substituting from (5.16) in (5.31) we get

$$\boxed{E \doteq 1{,}9107 \times \left[\frac{P}{Dt}\right] \times \frac{(1+\nu)(1-2\nu)}{\left[\epsilon_y(1-\nu) + \epsilon_x \nu\right.}} \qquad (5.33)$$

In a similar manner to that of plane stress, Poisson's ratio, for plane deformation may be calculated by applying any of equations (5.28), (5.29), Table 5.2 or the chart in Figure 5.3.

Also employing equation (5.32) or (5.33) the modulus of elasticity can be calculated.

5.6 NEW PROPOSED TECHNIQUE FOR MORTAR TESTING

In introducing the new technique it was not known which of the two possible conditions analysed would exist, but it was adopted in the hope that the case of plane stress would be valid. If not the alternative case to be considered was that of plane deformation.

With this condition in mind, the new technique was introduced. It consists of two stages for developing the indirect-tensile test and in order to be employed with the above derived formulae the following brief discussion is better given with this introduction:

Firstly, in his work, Hondros used discs of 24 inches diameter and two inches thickness, and the case of plane stress was assumed and shown experimentally to be valid.

However, these dimensions were chosen deliberately by him and it is not known if these are the limiting dimensions between a disc and a cylinder or not. As indicated before, because such dimensions are neither convenient nor economical it became essential to think about the possibility of using smaller dimensions.

In the author's view there were three possible diameters which emerged as the most suitable for this purpose, namely 6, 4 and 2.78 inches. The reason for this will be shown later in the second stage. As regards the thickness it was suggested by Ryder[112] that it should be one inch, which seemed to the present author quite reasonable.

At this point two questions of importance arose. The first, how far do the assumptions incorporated in the theoretical analyses apply? The second, what is the limiting case between a disc and a cylinder, in both in general and for the present suggested specimens? No theoretical answer could be found. Accepting the new dimensions was a matter depending on the experimental investigation, which will be dealt with in Chapter 7.

Secondly, it is the trend nowadays for some authorities[102][141] to investigate the possibility of carrying out the indirect tensile test using cubes rather than cylinders. Therefore, the second and further development was the use of square plates having the same outer dimensions as the proposed discs, as illustrated in Figure 5.4. This can have two further advantages. Firstly, it can be a step towards convenience in that it is easier to apply the load to a square plate across one of the sides than to apply it diametrally to a disc, and secondly the ordinary cubes for mortar and concrete casting and which are available in most laboratories could be used, by casting the mortar one inch in height. By this means making new circular moulds is avoided, and this is why the dimensions discussed above were chosen.

Again another two important questions arose. The first is how exactly a square plate or block can simulate to a disc or cylinder, under the present suggested conditions of loading, especially as regards the strains at the centre. For these strains both charts and tables were developed. The second question, of course, is the limiting case between a square plate and a cube. Two answers are required; theoretical and experimental.

For the theoretical answer we can turn to Goddier[51] and Frocht[46]. The former gave the theoretical stress distribution in a square plate under a pair of line loads. In his analysis he has shown that the magnitude of the tensile stress at the centre of the plate is nearly the same as in a disc under the same loadings. Frocht discussed, in detail, in part of his book the problem of the stresses produced by a concentrated load acting on the edge of a plate. Using Flamant's solution, and one of the consequent applications, it has been shown that the

continued:

(112): Ryder, J.F. Building Research Station. Personal communication.
(51): Goodier, J.N. Comparison of rectangular blocks, and the bending beams by non-linear distribution of bending forces. Trans. of American Society of Mechanical Engineers, Vol. 54, 1932, pp. 173-196.

that the average tensile stress along the vertical of a rectangular block under the considered case of loading is equal to that in a cylinder.

As regards the experimental answer, very little evidence could be found, Nilson[102] was the only one, according to the literature in hand, who investigated the similarity between a cylinder and a cube for assessing the tensile strength. He assumed, without giving the basis of his assumption, that exactly the same formulae can be used. For the cuse he used the following one: $\delta_x = 0.64\ P / a^2$, where δ_x, P, and "a" are the stress along the vertical, the load, and the length of side of the cube respectively. His experimental verification showed that the tensile strength of concrete can be determined with at least as much accuracy by the splitting test as by the cylinder splitting test. However, Nilson's results were few, and absolutely limited to the ultimate tensile strength, with no measurements of deformation.

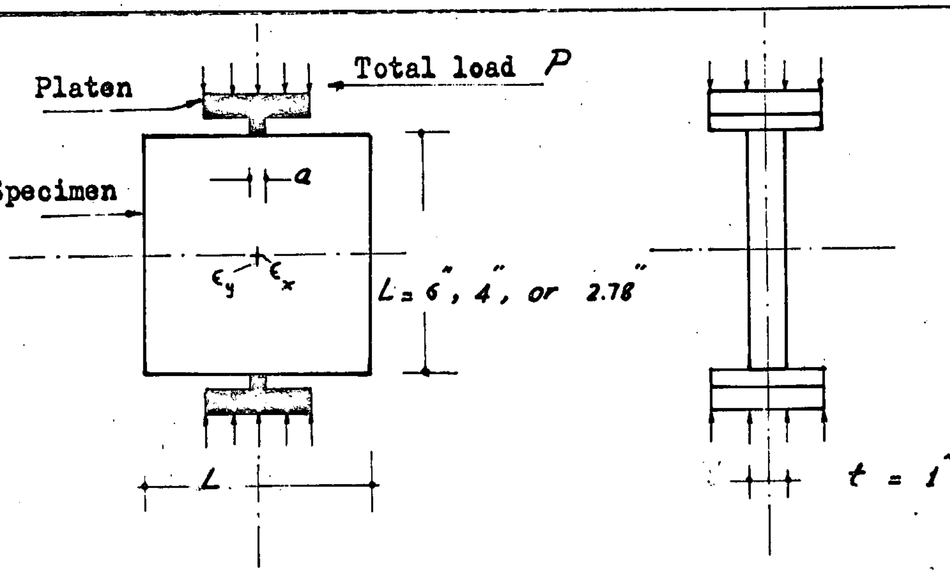
The experimental examination of the proposed testing technique will be dealt with in Chapter 7.

5.7 MISCELLANY: THE PROPOSED TECHNIQUE AND BRICK TESTING

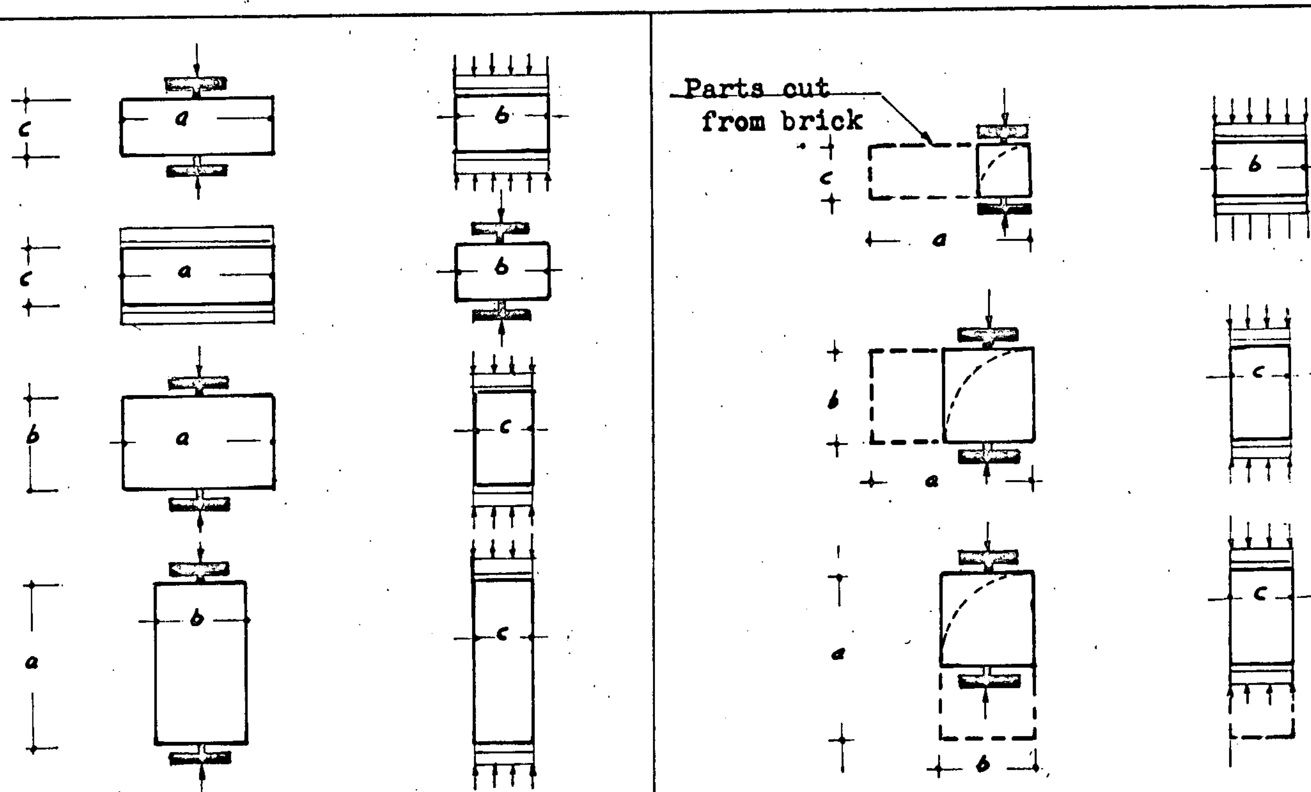
In the testing of bricks the need for information about the tensile strength, and stress-deformation relations became of importance. Therefore, before ending the theoretical studies incorporated in the present chapter, it was thought worth while to consider the request for bricks at least from the view-point of possible suggestions. The following proposals are put forward in the hope they will obviate the necessity for a complete theoretical study of brick testing, either in the present project or in future research.

With the merits of the new technique in mind it is thought that its modification to suit brick testing is both possible and useful. The requirement for modification arises from one main difficulty, the preparation of the specimen. This is due to the fact that a brick is not a castable material produced in the laboratory. On the other hand, preparation of the test piece from an actual brick, although accomplished with difficulty due to the brittleness of the brick, appeared somewhat easier.

In the following, some possible ways are suggested for brick testing. The main consideration in forming the test piece is to minimize so far as possible the planes of cutting. The calculating formulae derived before can be applied, each in the appropriate place. The suggestions are(a) For the tensile strength possible applications are shown in Figure 5.4.-a. (b) For the stress-deformation relations possible applications are shown in Figure 5.4.c.



a. Method and specimen for mortar testing [Stress-deformation relations and tensile strength]



a , b , and c are the dimensions of a brick in a decreasing sequence.

Tensile strength (using the whole brick)

Stress-deformation relations (using parts of bricks)

b. Extension of the proposed technique for brick testing.

Figure 5.4:

Method and test specimen of the proposed technique for mortar testing, and its extension for brick testing.

C H A P T E R 6

THE INFLUENCE OF END AND JOINT CONDITIONS REPLACING MORTARS OF DIFFERENT RIGIDITIES ON THE FAILURE CHARACTERISTICS OF BRICKS AND BRICK MASONRY

6.1 ORIENTATION

The object of the work described in this chapter was to investigate experimentally the following:

1. The influence of the relative rigidities of the two phases of an assemblage simulating brick masonry, on the failure characteristics of the assemblage.
2. The influence of brick height, which emerged from both theoretical analyses to be an important factor influencing the assemblage failure characteristics.
3. The disagreement between the failure characteristics of brick masonry and the failure characteristics of bricks when the latter are tested according to the standard loading tests.
4. Possible developments to increase the compressive strength of brick masonry.
5. The basic work upon which the present loading tests were incorporated in specifications.
6. The possibility of formulating a new loading test for bricks.

One of the chief intentions was to achieve these requirements by penetrating deeply into brick masonry assemblages on the lines of simulation. Shortly, the main part is a more detailed examination of bricks under simple compression. Compression was applied to the bricks through different materials simulating mortars, using two outer end conditions, three directions of loading, and three conditions of brick surface.

The scope can be considered relatively wide, but because of the limited materials and available measuring devices at the time of this series, the investigation was carried out by assessing the results in the first instance in terms of:

a. Apparent failure strength,

b. Mode of failure.

c. General observations during testing.

Then the calculated results were interpreted according to the different parameters concerned with the objects of this chapter, with discussions. Finally, conclusions are given with proposals which can be directly applied to brick masonry, achieving new and useful answers to some of the questions posed.

In spite of the conclusive results, the present chapters deals with work which is not claimed to be comprehensive. This is due to the fact that the tests represent only a relatively small effort undertaken with limited resources, in a new wide field of activities. It was felt by the end, that it could be a great advantage if the same work can be repeated, simultaneously on different sizes, relative dimensions of phases, original plastic mix, methods of extrusion, methods of burning and treatment.

6.2. EXPERIMENTAL WORK

6.2.1 General

The experimental work in this chapter was firstly intended to cover three shapes of specimens made from flat ground bricks with end bricks, as shown in Figure 6.1 (to the left). The idea of the end bricks arose from the feeling that the effect of the frictional forces between the platens and the specimen should be minimised. Due to the various materials, and their different characteristics however, it was not possible to do this for each case separately, because of the difficulty of the problem. Therefore, the only way that seemed possible was to standardize the end conditions by putting bricks of parts of bricks, according to the condition, of the same cross-section as the tested brick.

Then because of the unexpected results obtained after the first few tests, it was decided that a more complete investigation should be made. Consequently, the groups of specimens were extended as illustrated in Figure 6.1. In addition, both ground and super-ground bricks were used.

The majority of tests were done using the one-sixth-scale model bricks, and some with the one-third-scale bricks in the supplementary tests. Unfortunately, the latter tests were few because of the very limited number of bricks available, and the extreme difficulty in cutting and grinding them.

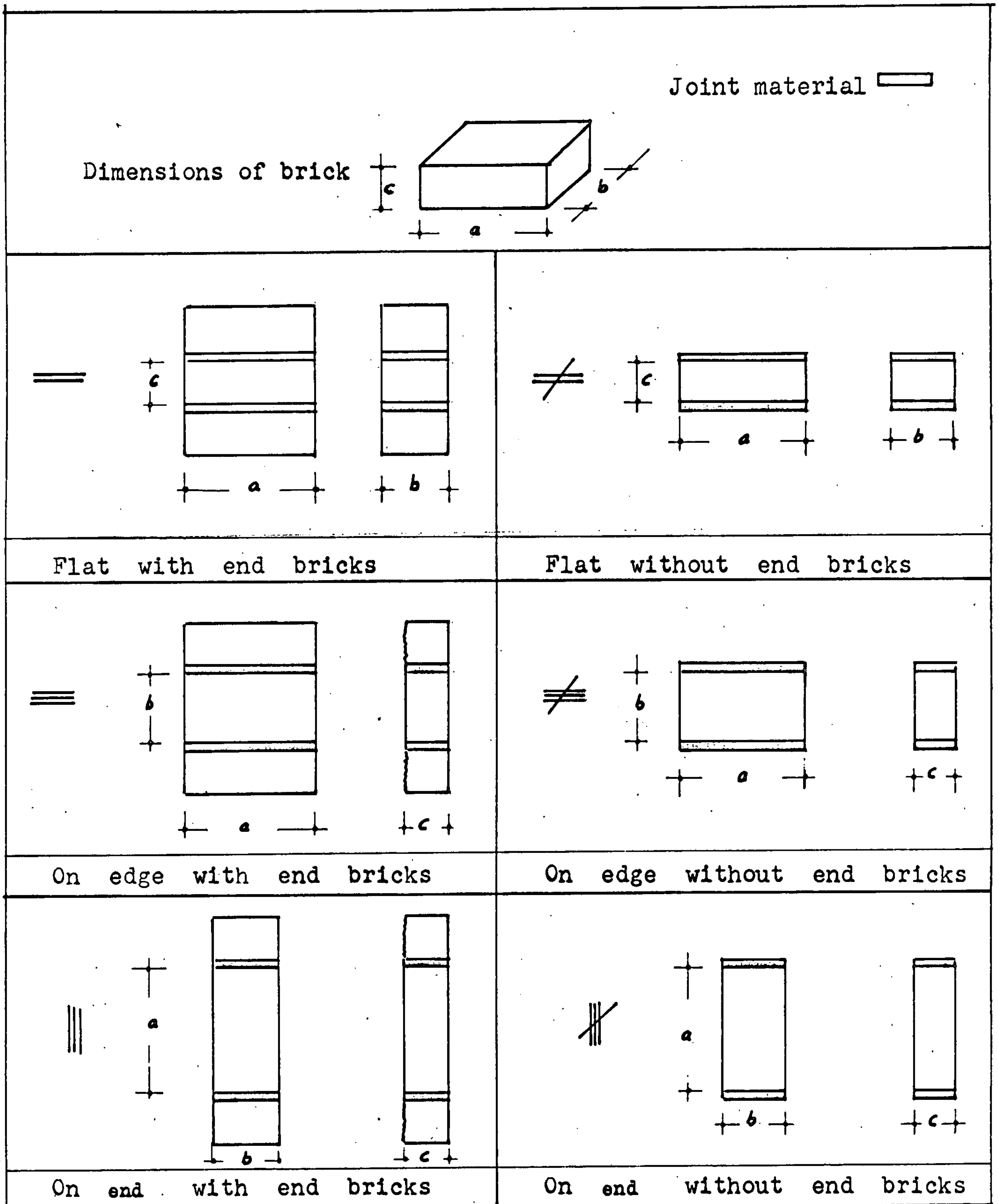


Figure 6.1 : Sets of specimens and notation.

6.2.2. Specimens

With each joint material (except brick joints and no joint material with super grinding) 72 specimens were made. They were divided equally between two categories, ground and rough. In each category the thirty six specimens formed three sets, each comprising two types of specimens as follows:

1. Six bricks tested flat with end bricks.
2. Six bricks tested flat with only the joint material as end condition.
3. Six bricks tested on edge with end bricks.
4. Six bricks tested on edge with only the joint material as end condition.
5. Six bricks tested on end with end bricks.
6. Six bricks tested on end with only the joint material as end condition.

Other tests were carried out with bricks super ground. The difference between grinding and super grinding will be shown later. Full details on combinations are given in table 6.1.

6.2.3. Materials

6.2.3.1. Bricks

The bricks were chosen from the same batch such that they had plane surfaces, with minimum variation and as far as possible surfaces at right angles to the axis of loading. The one-sixth-scale model bricks are shown in detail in Table 6.2.

Table 6.1

Scheme of tests for investigating the failure characteristics of one-sixth-scale model bricks.

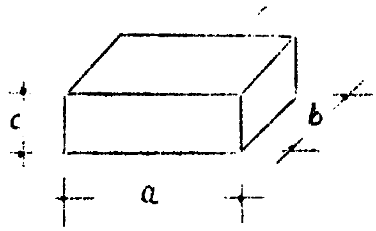
Joint material, thickness and condition of brick surface	Method of testing the brick	Flat		On edge		On end	
		with end bricks =	without end bricks	with end bricks ≡	without end bricks	with end bricks ∣∣∣	without end bricks
Steel	Ground bricks	1/8"	1/8"	1/8"	1/8"	1/8"	1/8"
	Rough "	1/8"	1/8"	1/8"	1/8"	1/8"	1/8"
Plywood	Ground "	1/8"	1/8"	1/8"	1/8"	1/8"	1/8"
	Rough	1/8"	1/8"	1/8"	1/8"	1/8"	1/8"
Hard-board+	Ground "	1/8"	1/8"	1/8"	1/8"	1/8"	1/8"
	Rough "	1/8"	1/8"	1/8"	1/8"	1/8"	1/8"
Hard-board+	Ground "	1/8"	1/8"	1/8"	1/8"	1/8"	1/8"
	Rough "	1/8"	1/8"	1/8"	1/8"	1/8"	1/8"
Plythene (6 layers)	Ground "	0.06"	0.06"	0.06"	0.06"	0.06"	0.06"
	Rough "	0.06"	0.06"	0.06"	0.06"	0.06"	0.06"
Rubber with fibres	Ground "	1/8"	1/8"	1/8"	1/8"	1/8"	1/8"
	Rough "	1/8"	1/8"	1/8"	1/8"	1/8"	1/8"
Pure rubber	Ground "	1/8"	1/8"	1/8"	1/8"	1/8"	1/8"
	Rough "	1/8"	1/8"	1/8"	1/8"	1/8"	1/8"
No joint material	Ground "	0.0	0.0	0.0	0.0	0.0	0.0
	Rough "	0.0	0.0	0.0	0.0	0.0	0.0
No joint material and super grinding	Super ground bricks	0.0 △	0.0 △	0.0	0.0	0.0 △	0.0 △
Brick joints and super grinding	Super ground bricks	1/4" △	1/4" △	1/4" △	1/4" △	1/4" △	1/4" △

Six specimens were tested from each, except those indicated by △ which were tested by four, and the total number of specimens was 638.

+ Indicates the direction of rough surface of hard-board with respect to the middle brick.

Table 6.2

Dimensions and cross sectional areas of $^{1}/6$ scale model bricks



Property		Dimensions			Cross sectional area		
		a inch	b inch	c inch	a x b inch^2	a x c inch^2	b x c inch^2
Mean value		1.472	0.691	0.472	1.0175	0.6952	0.3250
Range	Minimum value	1.462	0.684	0.453	1.0108	0.6124	0.3155
	Maximum value	1.495	0.697	0.485	1.0286	0.6734	0.3366
Standard deviation		0.038	0.012	0.0127	0.014	0.0284	0.0141
Coefficient of variation %		2.601	1.751	2.100	1.410	4.080	4.340
Number of specimens		20	20	20	20	20	20

6.2.3.2 Joint Materials

The joint materials were steel, plywood, hardboard with rough surfaces towards the inside, hardboard with rough surfaces towards the machine platens, 0.01" high gauge polythene (6 layers), rubber-with-fibres, pure rubber, brick material joints, no joint material, no joint material with super-grinding. The thickness of the joint materials was the same throughout the tests with only two exceptions, namely polythene and brick joints, because it was impracticable to insert more than six layers of polythene, and it was impossible to hold the small bricks on the rotating grinding machine. All details of joint materials are shown in Table 6.1.

6.2.4. Preparation of Specimens

6.2.4.1 Grinding

Bricks were ground flat by hand using sheets of carborundum paper on a glass plate as indicated schematically in Figure 6.2. The two grades of paper used subsequently were Emery cloth No.1 (rough)and Emery cloth No.0 (fine). Throughout the grinding process the ground face was checked on a glass surface so that it was plane with minimum variation and perpendicular to the other edges in the direction of loading. After grinding, a wire brush was first used to remove the fines. When the brush was found to produce some roughness, it was decided to remove the fines by thumb.

6.2.4.2. Super grinding

This was done by the author in the Geology Department, in three stages using three grades of carborundum abrasive. The first and the second were respectively grit - 120 and grit - 400 . Grinding in both stages was done on a mechanical rotating disc, using water. In the third stage grinding was done on a glass plate using grade grit - 600 in the form of very fine powder, and following the same technique as the ordinary grinding mentioned above. The third stage was also done in the wet state. Photograph 6.1 shows a brick during super grinding.

6.2.4.3. Preparation of jointing material

The jointing materials were cut to the required dimensions using different tools. Then any traces which might introduce irregularity, such as grease in steel, or roughness in plywood, were removed using different carborundum papers, acetone or washing. The rough surface of hardboard were kept in that condition in the hope that it would give some indication of the effect of the roughness.

6.2.4.4. Assembling

Throughout the tests assembling was

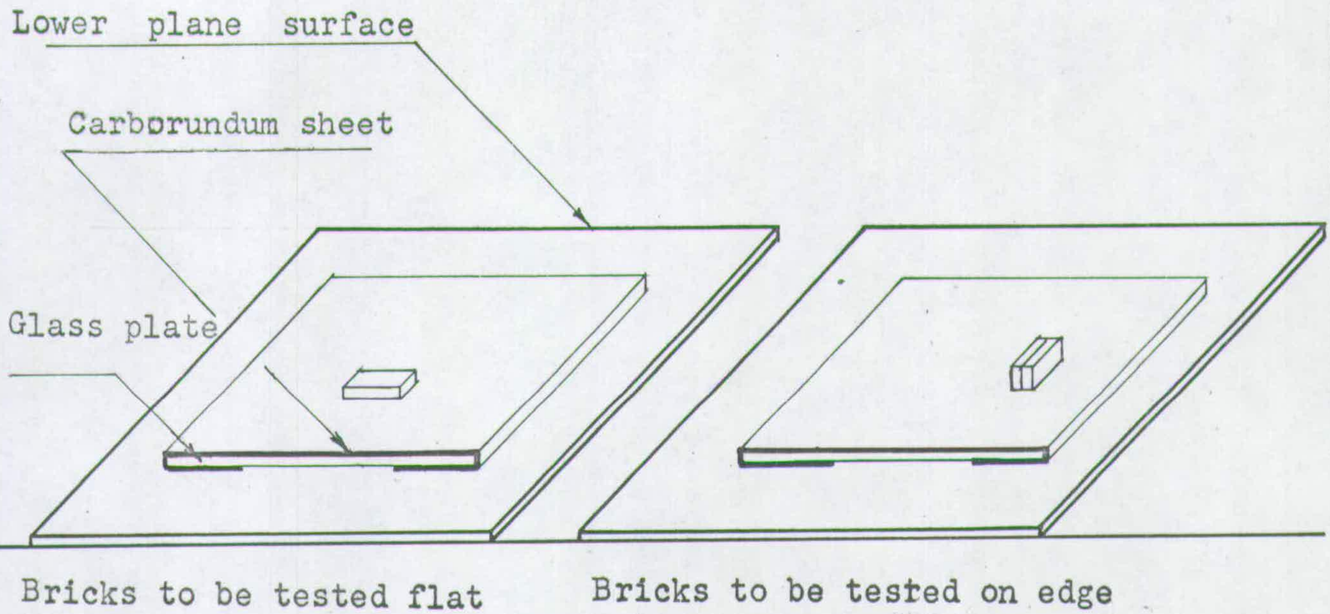


Figure 6.2 : Schematic sketch for grinding bricks by hand

Photograph 6.1 : Super grinding machine.

done by holding together the bricks and the joints, using sellotape on the sides in such a manner as did not affect the squeezing out of the joints. Sellotape at the top and bottom of specimens was completely avoided except in the case of polythene with specimens without end bricks.

6.2.5 Testing Machines and Testing Procedures

Testing was done by applying compression in the direction of orientation of bricks and joints until failure. An attempt was made at first to record the cracking load, but it became clear that this would be impossible with the machines used while recording other observations. Therefore, the observed results were limited to the ultimate failure loads, the mode of failure, and general notes.

As regards the testing equipment two machines were used. For specimens with expected maximum failure load less than two tons, the machine used was the Hounsfield Tensometer Type W (Photograph 6.2, 6.3). Its maximum capacity is two tons, and there are ranges of 1, $\frac{1}{2}$ ton, 500, 250, 125, $62\frac{1}{2}$ pounds. The scales are provided with a magnifier which helps in reading the load accurately. A main feature of the machine is that it applies the load horizontally. Never-the-less, throughout this chapter it will be consistently followed that "vertical" means the direction of applying the load with respect to the specimen. (Photograph 6.3).

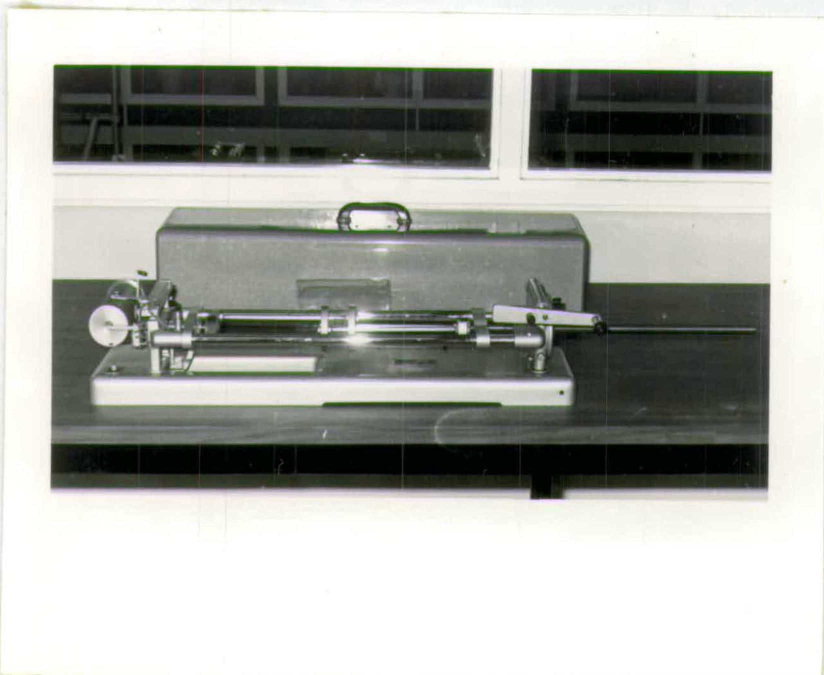
For specimens of expected failure loads higher than 2 tons the machine used was the Avery Universal Testing Machine 7104 CCCJ/DC7. Its maximum capacity is 100 tons and it can be adjusted for full scale loads of 5, 10, 20, 50 and 100 tons.

Hoping to obtain significant relations between the measured values an identical testing procedure was followed rigorously as follows:

1. The specimen was placed in the testing machine and either the compression attachments of the Hounsfield Tensometer were brought together or the lower platen of the Universal Machine was raised gently without any shock until the load was 0.2 ton.

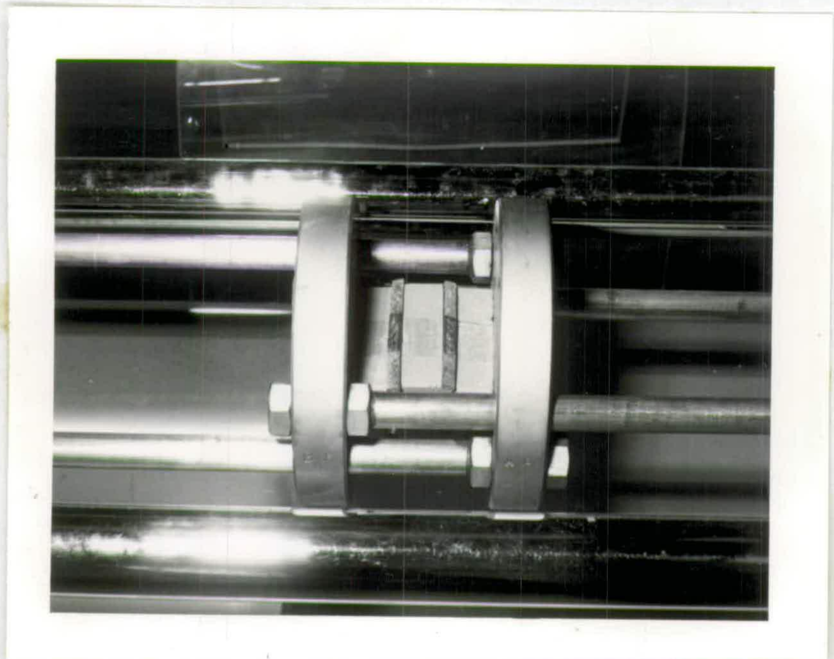
2. The load was increased until failure. The rate of increase was kept, so far as possible, constant. By using this careful and consistent sequence it was hoped to keep the effects of creep and rate of loading constant with ultimately reliable results.

Some bricks with some of the joint materials failed suddenly and very explosively, so that it was necessary to make a perspex shield for the



Photograph 6.2

General view of the Hounsfield Tensometer



Photograph 6.3

Detailed photograph of the compression attachments of the Hounsfield Tensometer and a specimen during testing.

Photographs 6.2,3:

Specimens during testing in the Hounsfield Tensometer.

Hounsfield Tensometer (Photograph 6.2). This allowed one to observe the behaviour of a specimen safely.

6.2.6. Laboratory and Temperature Conditions

No special or strict treatment was adopted other than wooden drawers ensuring that all the specimens after assembling were kept in the same place, so that both bricks and specimens were considered to be in a standard condition.

6.3 TEST RESULTS

In order to provide a simple way of studying the objectives of the work, the results are given in the form of families of tables as follows:

1. Effect of different joint materials on the apparent failure compressive strength. (Tables 6.3 - 6.8).
2. Effect of height (in the direction of loading) on the apparent failure compressive strength. (Table 9).
3. Effect of different joint materials on the mode of failure (Tables 6.10 and 6.11).

The actual observed results are too numerous to give in full, so with the object of convenience, the calculated results only are given, after being reduced into a statistically concise form. The statistical functions used in the present work are, the range, mean, standard deviation, coefficient of variation, and number of specimens. Although observations during testing form part of the results it was thought better to include them in the sequence of the general discussion.

The measured properties and indications of the above functions are well known, and it can be found in any reference on statistics(96)(127). The following are the formulae used in calculation.

If a set of readings or calculated values are indicated by X_I , X_2 , X_4 .... X_n, then we get:

Range:

The minimum and maximum value.

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(96) Moroney, M.J. Facts from figures. Penguin books. England 1964.

(127) Spiegel, M.R. Theory and problems of statistics. Schaum Publishing Comp. New York. 1961.

Mean $\bar{X}$:

$$(X_1 + X_2 + X_3 + \ldots\ldots X_n) / n$$

Standard deviation S :

$\sqrt{\sum (X - X)^2 / n}$ = the mean of the squares of the values - the square of the mean values.

Coefficient of variation V :

$$(s / \bar{x}) \times 100\%$$

Table 6.3

Effect of different joint materials on the apparent failure compressive strength of $^{1}/6$ scale model bricks tested flat with end bricks (═).

	Joint material	Range		Mean lb/in^2	Standard deviation	Coefficient of variation %	Number of specimens
		Min lb/in^2	Max lb/in^2				
Ground	Steel	6484	9451	8198	890.0	10.85	6
	Plywood	6000	7517	6773	687.0	10.14	6
	Hard board	5099	6902	6370	605.0	9.49	6
	Hard board	4989	5473	5114	163.5	3.13	6
	Polythene	2176	2780	2467	224.0	9.07	6
	Rubber with fibres	1494	1802	1699	119.0	7.00	6
	Pure rubber	901	1143	1014	84.0	8.28	6
	No joint material	4396	6418	5410	745.0	13.77	6
	No joint material and super grinding	4242	4791	4425	189.0	4.27	6
	Brick joints	4176	4440	4258	124.0	2.91	6
Rough	Steel	5759	7649	6766	578.0	8.50	6
	Plywood	6242	7407	6810	404.0	5.93	6
	Hard board	4660	6572	5744	678.0	11.80	6
	Hard board	4836	4836	5847	304.0	3.69	6
	Polythene	2088	2286	2207	89.0	4.00	6
	Rubber with fibres	1230	1495	1337	87.5	13.00	6
	Pure rubber	902	1077	1003	64.5	6.43	6
	No joint material	2616	3385	3037	334.0	10.99	6
	Brick joints	—	—	—	—	—	-

Table 6.4

Effect of different joint materials on the apparent failure compressive strength of $^1/6$ scale model bricks tested flat without end bricks (≠).

Brick	Joint material	Range Min lb/in^2	Range Max lb/in^2	Mean lb/in^2	Standard deviation	Coefficient of variation %	Number of specimens
Ground	Steel	6374	10045	7377	1230	16.67	6
	Plywood	5671	6044	5769	145.0	2.51	6
	Hard board	5319	5561	5385	194.0	3.62	6
	Hard board	5429	5759	5634	142.5	5.23	6
	Polythene	2725	3077	2884	136.0	4.71	6
	Rubber with fibres	1429	1604	1531	73.0	4.76	6
	Pure rubber	1097	1385	1209	106.5	8.80	6
	No joint material	7253	8352	7777	404.0	5.19	6
	No joint material and super grinding	5451	6352	6011	345.0	5.73	4
	Brick joints	3253	4220	3725	412.0	11.06	4
Rough	Steel	5640	7605	6795	492.0	7.25	6
	Plywood	5384	6990	6246	531.0	8.50	6
	Hard board	4340	5627	5230	423.0	8.08	6
	Hard board	4630	5715	5575	314.0	5.63	6
	Polythene	2108	3033	2814	215.0	16.70	6
	Rubber with fibres	1216	1495	1348	96.5	6.97	6
	Pure rubber	910	1055	956	68.0	7.11	6
	No joint material	7160	7671	7315	330.0	4.51	6
	Brick joints	---	---	---	---	---	-

Table 6.5

Effect of different joint materials on the apparent failure compressive strength of $^1/6$ scale model bricks tested on edge with end bricks (≡).

Brick	Joint material	Range		Mean lb/in^2	Standard deviation	Coefficient of variation %	Number of specimens
		Min lb/in^2	Max. lb/in^2				
Ground	Steel	4317	6702	5681	740.0	13.02	6
	Plywood	4897	6186	5831	462.0	7.92	6
	Hard board	4198	5545	4860	690.0	9.00	6
	Hard board	4575	5413	5015	298.0	5.90	6
	Polythene	2996	3319	3163	115.0	3.63	6
	Rubber with fibres	1869	2062	1970	82.6	4.19	6
	Pure rubber	1224	1643	1401	128.0	9.13	6
	No joint material T	4221	5993	4940	755.0	15.28	6
	No joint material and super grinding	3995	4189	4068	262.0	6.44	4
	Brick joints	2030	3029	2448	369.0	15.07	4
Rough	Steel	5155	5704	5343	349.0	6.53	6
	Plywood	5155	6831	5859	575.0	9.81	6
	Hard board	4446	5316	4902	290.0	5.90	6
	Hard board	3447	4640	4317	415.0	9.50	6
	Polythene	2706	3061	2860	120.0	4.19	6
	Rubber with fibres	1772	2320	1927	220.0	11.40	6
	Pure rubber	967	1224	1084	88.5	8.16	6
	No joint material	3609	4511	4081	281.0	6.88	6
	Brick joints	--	--	--	--	--	-

Table 6.6

Effect of different joint materials on the apparent failure compressive strength of $^{1}/6$ scale model bricks tested on edge without end bricks ($\not\equiv$).

Bricks	Joint material	Range		Mean lb/in^2	Standard deviation	Coefficient of variation %	Number of specimens
		Min lb/in^2	Max lb/in^2				
Ground	Steel	5542	7507	6266	653.0	10.42	6
	Plywood	4672	7088	5992	750.0	12.51	6
	Hard board	5220	6122	5697	288.0	5.05	6
	Hard board	4607	5445	4887	492.0	10.07	6
	Polythene	3866	4672	4290	292.0	6.80	6
	Rubber with fibres	2062	3125	2362	486.0	20.57	6
	Pure rubber	1547	2062	1782	198.0	11.11	6
	No joint material	5961	6637	6427	280.0	4.35	6
	No joint material and super grinding	4575	4994	4720	164.0	3.47	4
	Brick joints	3930	4253	4027	138.0	3.42	4
Rough	Steel	4961	6637	5261	630.0	11.97	6
	Plywood	5155	6218	5842	359.0	6.14	6
	Hard board	4801	6186	5563	490.0	8.80	6
	Hard board	4672	4962	4849	107.0	2.20	6
	Polythene	3061	4833	3828	620.0	16.10	6
	Rubber with fibres	3093	4124	3635	460.0	12.65	6
	Pure rubber	1128	1257	1213	68.5	5.64	6
	No joint material	4929	5252	5155	122.0	2.36	6
	Brick joints	--	--	--	--	--	-

Table 6.7

Effect of different joint materials on the apparent failure compressive strength of $^1/6$ scale model bricks tested on end with each bricks (|||).

	Joint material	Range Min lb/in^2	Range Max lb/in^2	Mean lb/in^2	Standard deviation	Coefficient of variation %	Number of specimens
Ground	Steel	3502	5493	4257	1330.0	31.20	6
	Plywood	4463	6180	5527	650.0	11.70	6
	Hard board	4257	6385	5206	764.0	14.60	6
	Hard board	3982	4532	4108	260.0	6.30	6
	Polythene	2678	3982	3226	440.0	13.60	6
	Rubber with fibres	1510	1922	1750	167.5	9.57	6
	Pure rubber	1373	1648	1521	105.0	6.90	6
	No joint material	3227	4394	3970	535.0	13.40	6
	No joint material and super grinding	2884	3570	3227	282.0	8.73	4
	Brick joints	3295	3502	3398	96.5	2.34	4
Rough	Steel	3570	4325	4062	250.0	6.10	6
	Plywood	3982	5767	4726	660.0	13.90	6
	Hard board	3639	5149	4360	503.0	11.50	6
	Hard board	3708	4257	3948	204.0	5.10	6
	Polythene	2746	3570	3009	316.0	10.50	6
	Rubber with fibres	1579	1716	1854	110.0	6.41	6
	Pure rubber	961	1304	1147	81.5	7.10	4
	No joint material	3201	3982	3249	332.0	10.2	4
	Brick joints	—	—	—	—	—	—

Table 6.8

Effect of different joint materials on the apparent failure compressive strength of $^{1}/6$ scale model bricks tested on end without end bricks (#).

Brick	Joint material	Range: Min lb/in^2	Range: Max lb/in^2	Mean lb/in^2	Standard deviation	Coefficient of variation %	Number of specimens
Ground	Steel	4120	5767	4657	590.0	12.60	6
Ground	Plywood	4669	6523	5584	645.0	11.50	6
Ground	Hard board	4600	6179	5275	642.0	12.10	6
Ground	Hard board	3708	5355	4371	625.0	14.20	6
Ground	Polythene	3845	5012	4508	900.0	19.90	6
Ground	Rubber with fibres	3708	5630	4565	665.0	15.40	6
Ground	Pure rubber	1922	3158	2449	365.0	14.90	6
Ground	No joint material	3433	4806	4108	475.0	11.50	6
Ground	No joint material and super grinding	3502	3708	3587	770.0 ~~77.0~~	21.48	4
Ground	Brick joints	3090	3639	3415	22.5	6.58	4
Rough	Steel	3878	5088	4517	625.0	13.80	6
Rough	Plywood	4600	5493	4943	346.0	6.90	6
Rough	Hard board	4394	5767	4897	578.0	11.80	6
Rough	Hard board	4180	4943	4405	260.0	5.90	6
Rough	Polythene	3982	4257	4142	115.0	2.70	6
Rough	Rubber with fibres	3982	5081	4337	440.0	10.14	6
Rough	Pure rubber	1922	3708	2711	545.0	8.90	6
Rough	No joint material	3639	4257	3856	230.0	5.90	6
Rough	Brick joints	—	—	—	—	—	—

Table 6.9

Effedt of height (in the direction of loading) on the apparent failure compressive strength of $^1/6$ scale model bricks - lb/in^2

Brick	Jointing material \ Method of testing	Flat		On edge		On end	
		$=$	$\neq$	$\equiv$	$\not\equiv$	$\vert\vert\vert$	$\not{\vert\vert\vert}$
Ground	Steel	8198	7377	5681	6266	4257	4657
	Plywood	6773	5769	5831	5992	5527	5584
	Hard board	6370	5385	4860	5697	5206	5275
	Hard board	5114	5634	5015	4887	4108	4371
	Polythene	2467	2884	3163	4290	3226	4508
	Rubber with fibres	1699	1531	1970	2362	1750	4565
	Pure rubber	1014	1209	1401	1782	1521	2449
	No joint material	5410	7777	4970	6427	3970	4108
	No joint material and super grinding	4425	6011	4068	4720	3227	3587
	Brick joints	4258	3725	2448	4027	3398	3415
Rough	Steel	6766	6795	5343	5261	4062	4517
	Plywood	6810	6246	5859	5842	4726	4943
	Hard board	5744	5230	4902	5563	4360	4897
	Hard board	5847	5575	4317	4849	3948	4405
	Polythene	2207	2814	2860	3828	3009	4142
	Rubber with fibres	1337	1348	1927	3635	1854	4337
	Pure rubber	1003	956	1684	1213	1147	2711
	No joint material	3037	7315	4081	5155	3249	3856
	Brick joints	—	—	—	—	—	—

Table 6.10

Effect of different joint materials on the mode of failure of ground $^{1}/6$ scale model bricks.

Jt. material \ Method of testing	Flat		On edge		On end	
	$=$	$\neq$	$\equiv$	$\not\equiv$	$\vert\vert\vert$	$\not\vert\vert\vert$
Steel	Mostly sh. failure	Sh.fail.	Sh.fail. in middle br.	Sh. fail.	Sh.fail. in middle br.	Sh. fail sometimes preceded by spl.
Plywood	"	"	"	"	"	"
Hard board	Spl. in outer prts. of middle br.	Spl. and sh. at outer parts.	Spl. at outer prts. of end brs.	Mostly sh.fail.	Sh. in middle brs.	Sh. and sometimes spl. at outer prts.
Hard board	Spl. at outer parts of end brs.	Mostly sh.fail.	Spl. in outer parts of end brs. slight sh. in middle br.	Spl. or sh. in outer prts.	Spl. in outer prts. and sh. in middle br.	Sh. fail.
Polythene	Spl.starting from middle br.	Spl.fail.	Spl.in end brs.	Spl.fail.	Spl.in end brs.	Spl.fail.
Rubber with fibres	"	Spl.mostly along axis	"	Spl.mostly along axis	"	Spl.mostly along axis
Rubber	"	"	"	"	"	"
No joint material	Mostly sh.	Sh.fail.	Mostly sh. in middle br.	Sh.fail.	Sh.in middle br.	Sh.fail.
No joint material & super grinding	"	"	Sh.fail.	"	"	"
Brick joints	Continuous sh. fail.	"	Mostly continuous sh. fail.	Continuous sh. fail.	Sh.in middle br. some times spl.in joints.	"

Br. = brick, Sh. = shear, Prt. = part, Fail. = failure, Spl. = splitting.

Table 6.11

Effect+ of different joint materials on the mode of failure of $^1/6$ scale model bricks.

Jt. material \ Method of testing	Flat		On edge		On end	
	=	≠	≡	≢	|||	$\not{\|\|\|}$
Steel	Mostly sh.fail.	Sh.fail.	Mostly sh.fail.	Sh.fail.	Sh.sometimes with spl.fail.	Sh.sometimes with spl. in middle br.
Plywood	Sh.fail. in middle br. sometimes with spl. in end br.	Sh.fail.	Sh.fail. in middle br. sometimes with spl.in end br.	Sh.fail.	Sh.fail.	Sh.fail.
Hard board	Spl.in outer parts of middle br.	Spl.and sh.fail. in outer parts.	Sh.and spl.fail. in outer parts of middle br.	Spl. accompanied by sh. at outer parts.	Sh. and spl.fail. in outer parts of middle br.	Sh.and spl. at outer parts.
Hard board	Spl.at outer parts of end br.	"	Sh. and spl.at outer parts of middle br. and spl. in end br.	Sh. at outer parts.	Sh. in middle brick.	Sh.fail.
Polythene	Spl. starting from middle br.	Spl.	Spl. in end brs.	Spl.	Spl.in end brs.	Spl.
Rubber with fibres	"	"	"	"	"	"
Pure rubber	"	"	"	"	"	"
No joint material	Mostly sh.preceded by spl.	Sh.fail.	Mostly sh. in middle preceded by spl. in end br.	Sh.fail.	Sh.fail. in middle br.	Sh.fail.

+ See notation in previous page.

6.4 INTERPRETATION OF AND DISCUSSION OF TEST RESULTS

6.4.1 General Discussion

As will be seen, the wide apread in relative stiffness of the selected joint materials has led to very different behaviour and failure characteristics in the specimens. Due to this and the other variables mentioned before it was thought better to discuss the results under separate headings representing groups of variables. These headings are:

1. Variation in apparent compressive strength of a brick due to different rigidities of joint materials.
2. Rigidity of joint material and mechanism of failure.
3. Variation in apparent compressive strength of a brick due to the three possible directions of loading (Effect of brick height).
4. Different modes of failure.
5. Effect of thickness of soft joint material on the apparent failure compressive strength of a brick.
6. Statistical discussion and scatter of results for the apparent compressive strength.
7. Comments on the standard compression test incorporated at B.S. 1257.(16)

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(16): British Standard Institution. B.S. 1257:1945. Methods of testing clay building bricks.

Before discussing each of these categories in detail, the following is a general discussion of some of the observations which were made during testing, but which were not sufficiently important to be included under the above-mentioned headings.

Rubber, and rubber-with-fibres

It was quite apparent to the naked eye that both rubber and rubber-with-fibres have the tendency to deform under very small loads compared with the ultimate failure loads.

Also it was easily seen, that if the bricks are as rigid as the machine platens, and if there is no friction acting between the surfaces of contact, the rubber layer could be easily extruded out at high loads.

As will be pointed out later, materials such as rubber produce with a hard brick a state of concentrated vertical stresses at the middle with an ultimate failure by vertical splitting. But for specimens with end bricks although failure was by vertical splitting it was not over the whole length of the specimen in all cases. The sequence of failure was as follows:

1. For specimens tested flat, failure happened at first, in the middle brick, then it was followed directly by failure in one or both of the end bricks, and almost along the same line.

2. For specimens tested on edge or on end the first splitting failure happened in one or both of the end bricks. Then it was followed, only in very few cases of specimens tested on edge, by failure in the middle brick. But with specimens tested on end it never happened that the middle brick failed. Photograph 6.6. shows typical failures of a set with rubber-with-fibres.

A point worth mentioning is that the internal friction which usually accompanies shear failure, which usually starts long before the actual failure, was not heard either with rubber or rubber-with-fibres. Indications of failure were heard as sharp reports very shortly before ultimate failure by splitting. These reports were very similar to the reports heard when testing cement mortar briquettes. The common number of reports with each specimen was one or two, and rarely, three.

Polythene

Specimens with polythene (6 layers) as joint material behaved in a more or less similar manner to the rubbers. The main thing, easily distinguished was that extrusion sometimes took place. It was marked by a form of sliding of one of the intermediate layers along its surfaces of contact with the two adjacent layers.

It could have happened, if friction on the top or bottom layer had not been created due to the increase in the load, that one or more layers flowed out complately, or diminished in thickness. It is also worth mentioning that the movement of this intermediate layer was accelerated at the beginning then retarded.

Hardboard

The compressibility of hardboard in both positions of its use was less visible to the naked eye. But generally it was more noticeable with the rough face than the smooth one. The audible reports were more than sharp. In fact failure happened explosively in all cases, so that it was necessary after the preliminary tests to make the protecting shield mentioned before. It was felt strongly when rotating the handle for applying the load that the resistance of the specimen to the load was much more than in the case of rubbers or polythene. It was also noticeable that failure happened in the outer parts of the hardboard layers rather than in the middle. This is simple because of the high friction produced in the middle.

Plywood

Specimens with plywood behaved in a manner between hardboard and steel as joint materials. Its compressibility was very slightly noticed by the naked eye. The internal friction in the bricks was clearly heard during the loading and long before failure. Generally, ultimate failure happened less suddenly than in the above cases of joint materials.

Brick, steel, and no-joint material

Internal failure was clearly heard in specimens tested flat and on edge with ground bricks. What was remarkable with some specimens tested on end especially with rough bricks, was an early splitting long before ultimate failure. It was easily seen and heard in the form of the sharp reports mentioned before in the majority of cases it was followed by the usual sound of shear failure, then ultimate failure. It will be shown later that the splitting failure here is not originated in the same manner as splitting with rubbers or polythene.

6.4.2. Variation In Apparent Failure Compressive Strength of A Brick Due to Different Rigidities of Joint Materials

1. For specimens tested flat without end bricks, the results show that it is possible to obtain virtually any apparent compressive strength by varying the joint material. As can be seen from Table 6.4 and Figures 6.3, 6.4 (to the right), the ranges in which the apparent strength can lie are 1209-7777 and 956 - 7315 lb/in^2 for ground and rough bricks respectively. An average value for the ratio of the maximum to minimum strength equals 7.04 : 1 which shows the wide spread of the results.

2. Trying to put the joint materials in order as regards the resulting apparent ultimate strength, it became impossible to keep one order for all the directions of loading and conditions of brick surface. Therefore, the order given in Table 6.3 was chosen arbitrarily and was kept the same throughout all the tables.

3. In order to represent the results graphically it was thought best to represent them as shown in Figures 6.3 - 6.8. The highest and lowest were located at first and joined by a straight line. Then each intermediate value was located on the Y axis, and a horizontal line was drawn to meet the inclined line in a point from which a vertical line was drawn to meet the X axis. Looking for a measurable property or a condition to put on the X axis so that the action produced as a result of the particulars joint material can be measured

by the abcissa, it was thought most suitable to call it the lateral restraint. For each joint material the lateral restraint can be quantitified as the percentage of increase or decrease in the apparent strength of the assemblage with respect to the actual compressive strength.

Consequently, it can be said that the relation between the apparent compressive strength of a brick and the lateral restraint produced by the end condition is linear. In fact for each lateral restraint there is one possible value for the apparent compressive strength, and for any obtained apparent strength there is a corresponding lateral restraint produced by the end conditions. This lateral restraint will be proved later to range between positive and negative values with respect to a certain one of these values.

4. Here arises a very important question. That is, which one of these values represents the actual compressive strength. Clearly the very wide range makes the answer very difficult. If we consider, for the moment, that the value obtained by the standard compression test is representative of the actual strength (taken as 100%), it can be seen that the end restraint produced by the joint material at the ends of a brick causes for ground and rough brick respectively either

an increase up to about 124.51 and 117.11% or a decrease down to 19.36 and 17.34 of the standard compressive strength. This is very interesting but what is even more interesting is that the value obtained from the standard test, as will be shown later, does not represent the actual strength which will be explained later. With respect to the correct actual strength the corresponding increase and decrease become respectively 191.17 - 179.82% and 29.18 - 23.50%.

5. It is also quite clear that the results quoted represent only the range covered by the materials used and not the whole possible range. Undoubtedly it is possible to obtain an apparent compressive strength lower than the value in the case of rubber by using a softer material. Similarly a higher value than that obtained with steel can be obtained by restraining the ends completely by a rigid clamping frame.

6. Comparing the different joint materials to see what the main difference in their features or behaviour it can be seen that it is the compressibility and the lateral deformation either outwards or inwards. In other words it is the deformation properties of the joint material at the end of a brick which influences its apparent strength. This is, of course, only when the coefficient of friction between the joint material and brick is considered constant. In other words when the friction along the surfaces of contact is acting and

and no sliding takes place.

7. Comparing the values obtained from similar specimens tested on edge it can be easily seen that all the above mentioned can apply in principle only, but not from the quantitative point of view. The apparent compressive strength ranges from 1782 to 6266, and from 1213 to 5842 lb/in^2 for ground and rough bricks respectively, as shown in Figures 6.5, 6.6... to the right. An average value for the ratio between the maximum and minimum strengths is 4.16. This corresponds to 7.04:1 for bricks tested flat. In other words the range becomes narrower. It is quite clear that the major variable parameter is the direction along which the load is applied to the brick. In other words the height of the brick. The most suitable place for explaining this phenomenon will be later when discussing the effect of height.

8. Similarly if we go further to specimens tested on end the same trend appears to be generally present as shown in Figures 6.7, 6.8 (to the right). But in fact if rubber is excluded it is found that the range becomes extremely narrow and even the order is distorted. The average ratio between the maximum and minimum strengths for ground and rough bricks is 1.92:1 instead of 7.04:1 or 4.16:1. Special reference should be made to the values obtained from steel and rubber-with-fibres.

They gave respectively apparent failure strengths of 4,657, 4,565 lb/in^2 with ground bricks and 4,517, 4,337 lb/in^2 with rough bricks. This shows considerable similarity between steel and rubber-with-fibres with these specimens, and will be also explained later.

9. In specimens with end bricks we can apply to some extent what has been mentioned above in "6." What is mentioned in "7" and "8" cannot be applied at all. This is due to the fact that the middle brick has much less influence, while the end bricks become of the controlling influence. This influence is more pronounced with softer joint materials. Again this will be pointed out later in the discussion of effect of height.

10. In the majority (33 out of 36 and the exceptional three are with tests on edge) of tests on edge and on end, the strengths of bricks tested with end bricks are less than the strengths of those tested without end bricks. (Comparison is easier between Tables 6.5, 6.7 and 6.6, 6.8 respectively). The difference is much more pronounced with soft materials than with hard materials in the joints. Referring back to the specimens without end bricks it was found that with rubber the apparent failure compressive strength of a brick tested flat is less than that of a brick tested on edge and

and much less than that tested on end. Therefore, in the case of specimens tested with end bricks, due to the fact that the height of the latter equals the smallest dimension of the brick which gave the minimum strength in the case of specimens with no end bricks, the end bricks become the controlling influence on the strength of the assemblage. Once their strength is reached the specimen cannot resist any more load. It is to be remembered here that the frictional restraint between the machine platens and the end bricks does not compensate even to a small extent for the squeezing out of the joint material between the middle and end bricks.

11. What was mentioned in 9 on specimens with end bricks and soft joint materials can be put in another way. Considering the middle brick loaded, then the lateral restraints acting on the outer end surfaces of the joint material are less with end bricks than in the case of machine platens. Also if we consider an end brick during the loading process, the middle brick produces frictional restraint on the joint material less than that produced if the other machine platen is acting. Finally, the apparent failure strength of the specimen with end bricks is in between the apparent strengths of the middle brick and

the end brick when each is loaded directly between the machine platens, usually much closer to the former value.

12. What has been said about specimens with soft material in the joints and end bricks can be said of similar specimens and hard joint materials in the joints, but in the opposite sense. In other words, the middle brick becomes the greater influence on the strength of the assemblage.

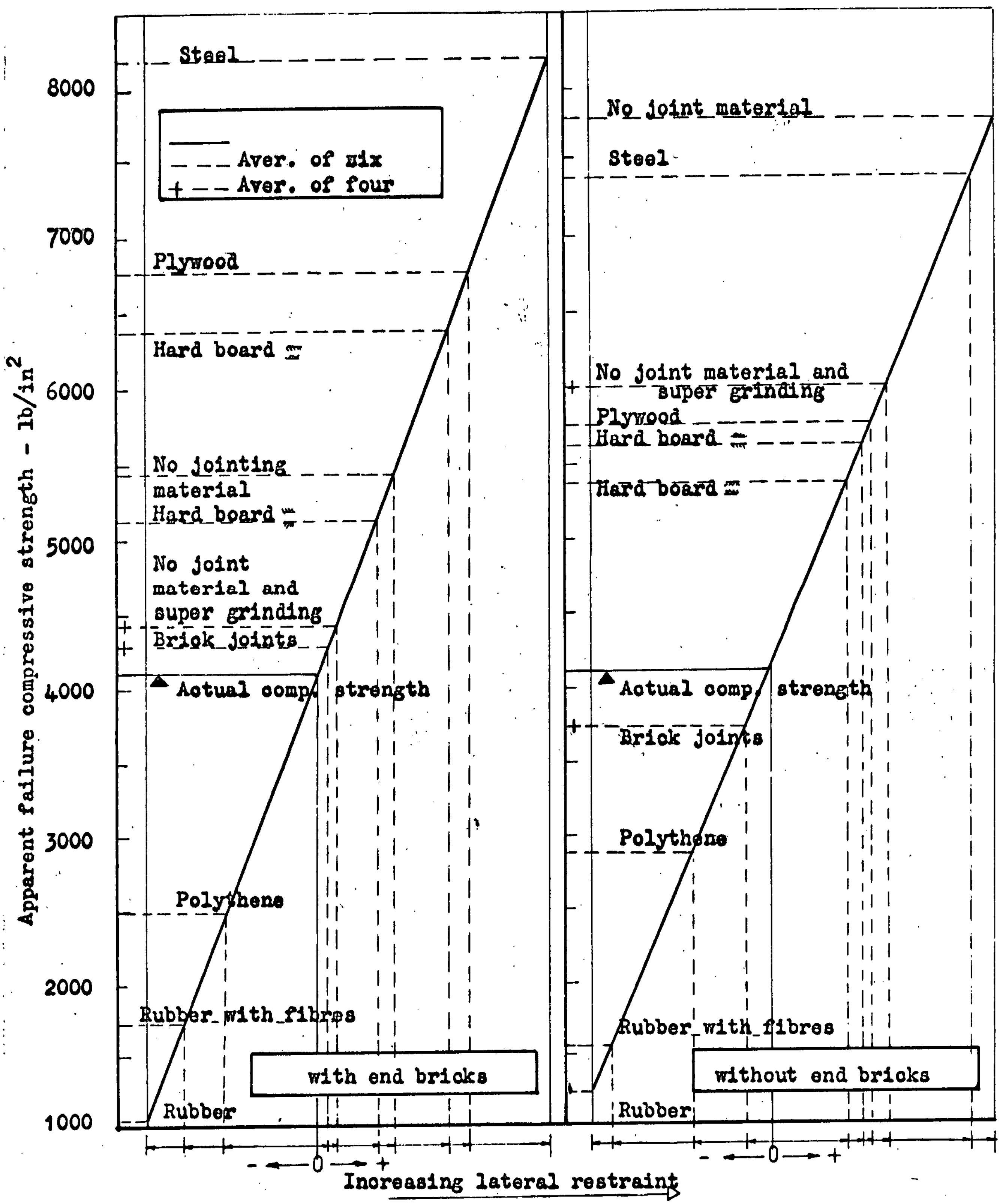


Figure 6.3:
Variation in apparent failure compressive strength of ground 1/6 scale model bricks tested flat.

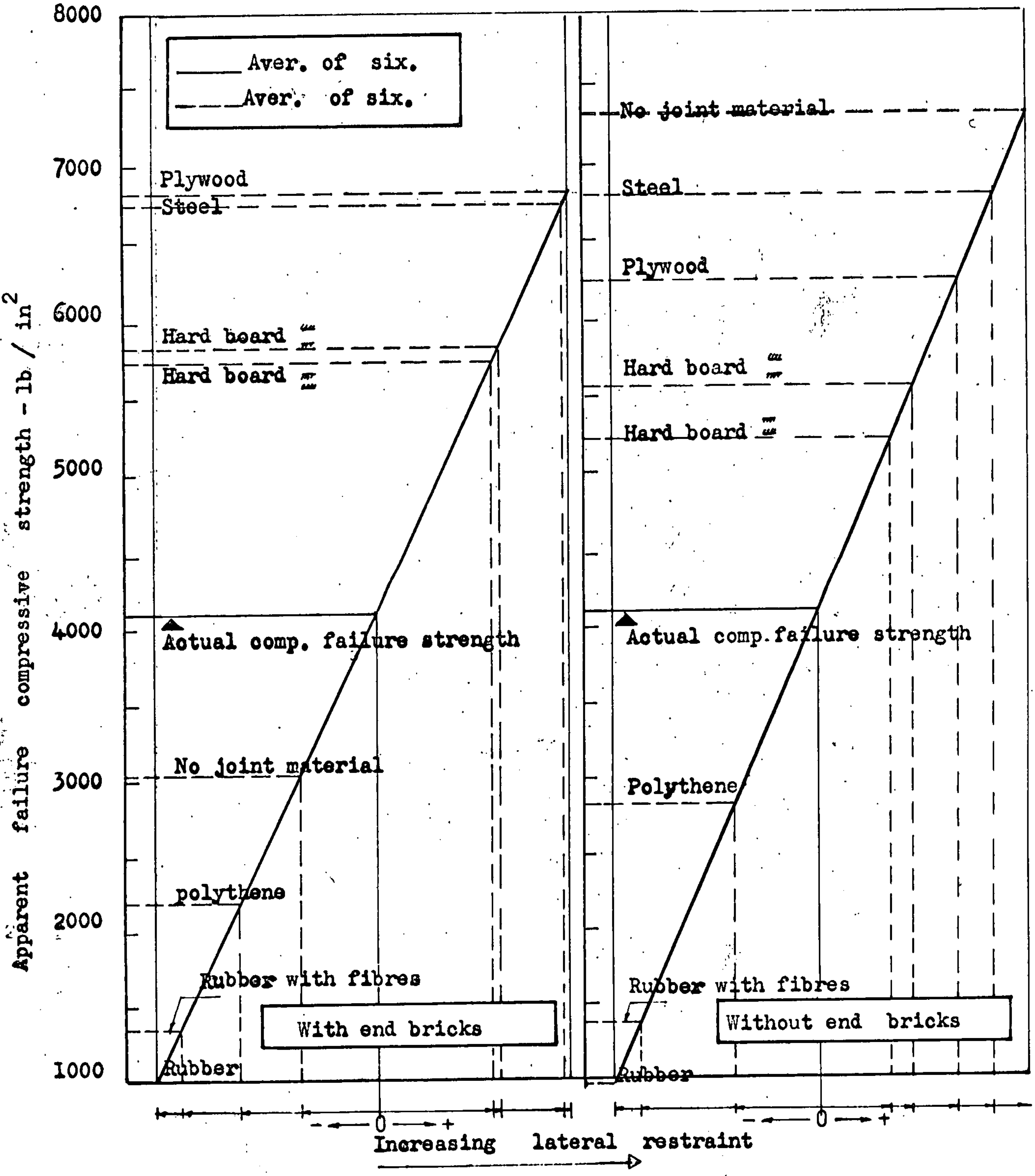


Figure 6.4: Variation in apparent failure compressive strength of rough 1/6 scale model bricks tested flat.

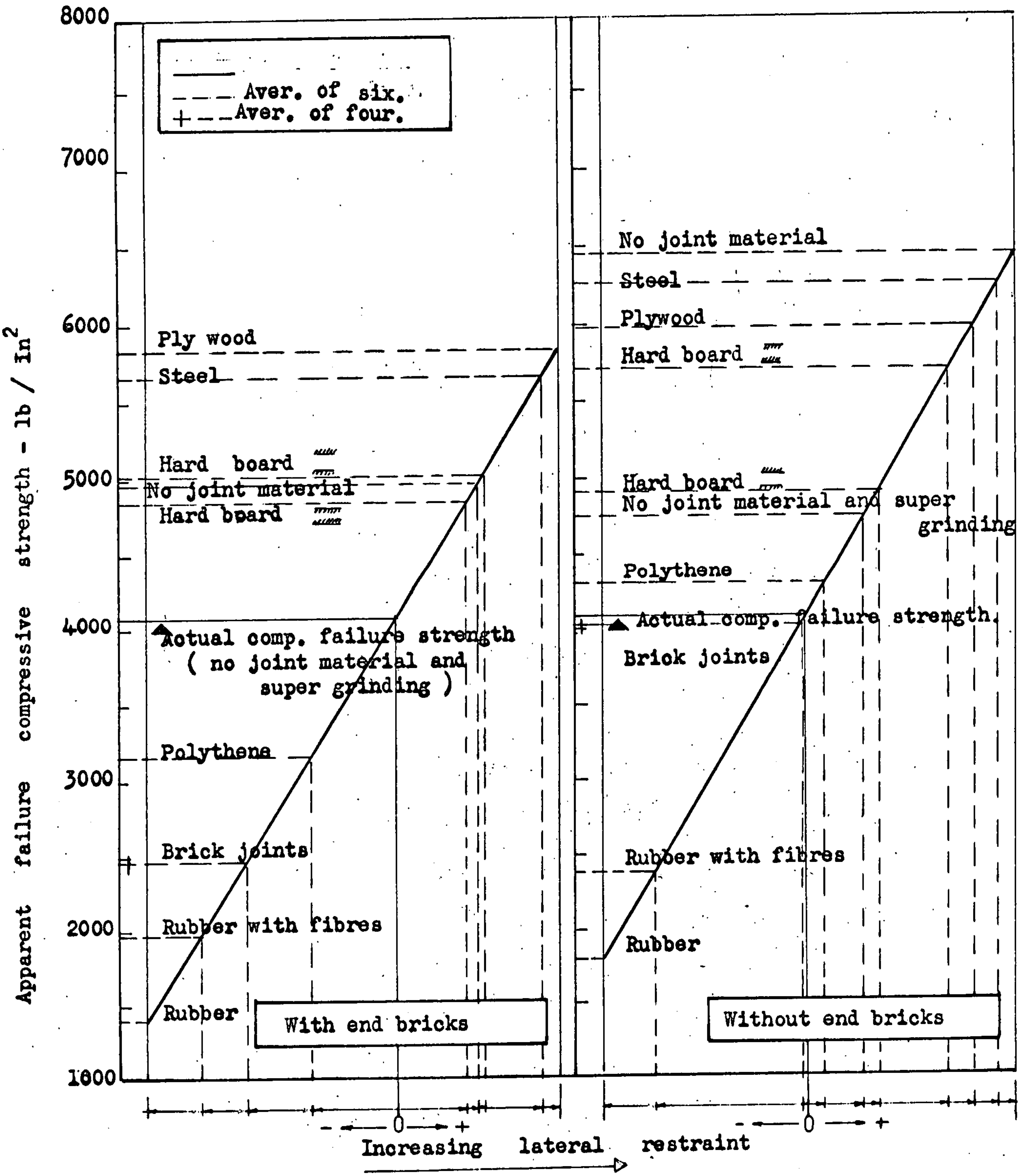


Figure 6.5:
Variation in apparent failure compressive strength of ground 1/6 scale model bricks tested on edge.

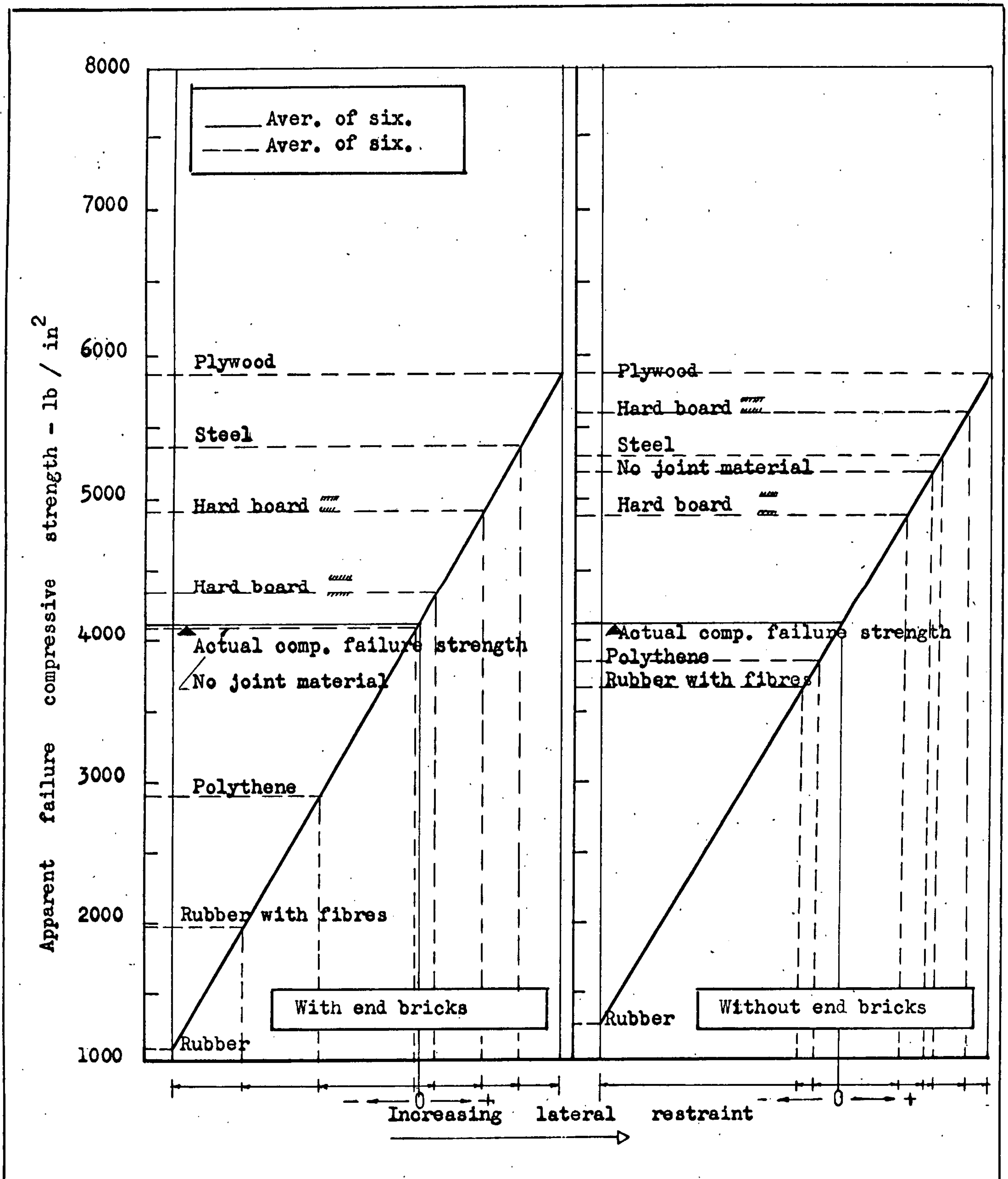


Figure 6.6:
Variation in apparent failure compressive strength of rough 1/6 Scale model bricks tested on edge.

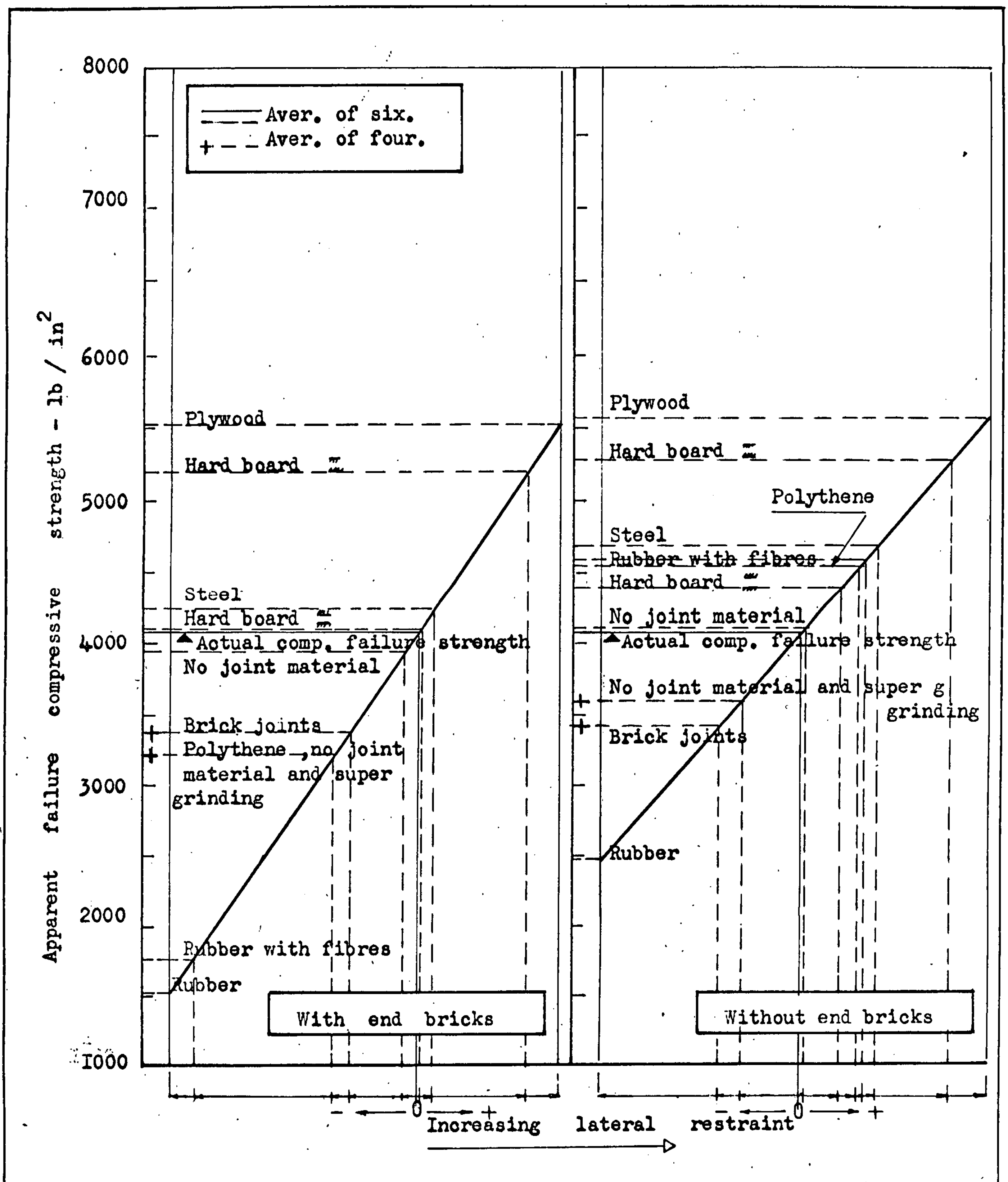


Figure 6.7:

Variation in apparent failure compressive strength of ground 1/6 scale model bricks tested on end.

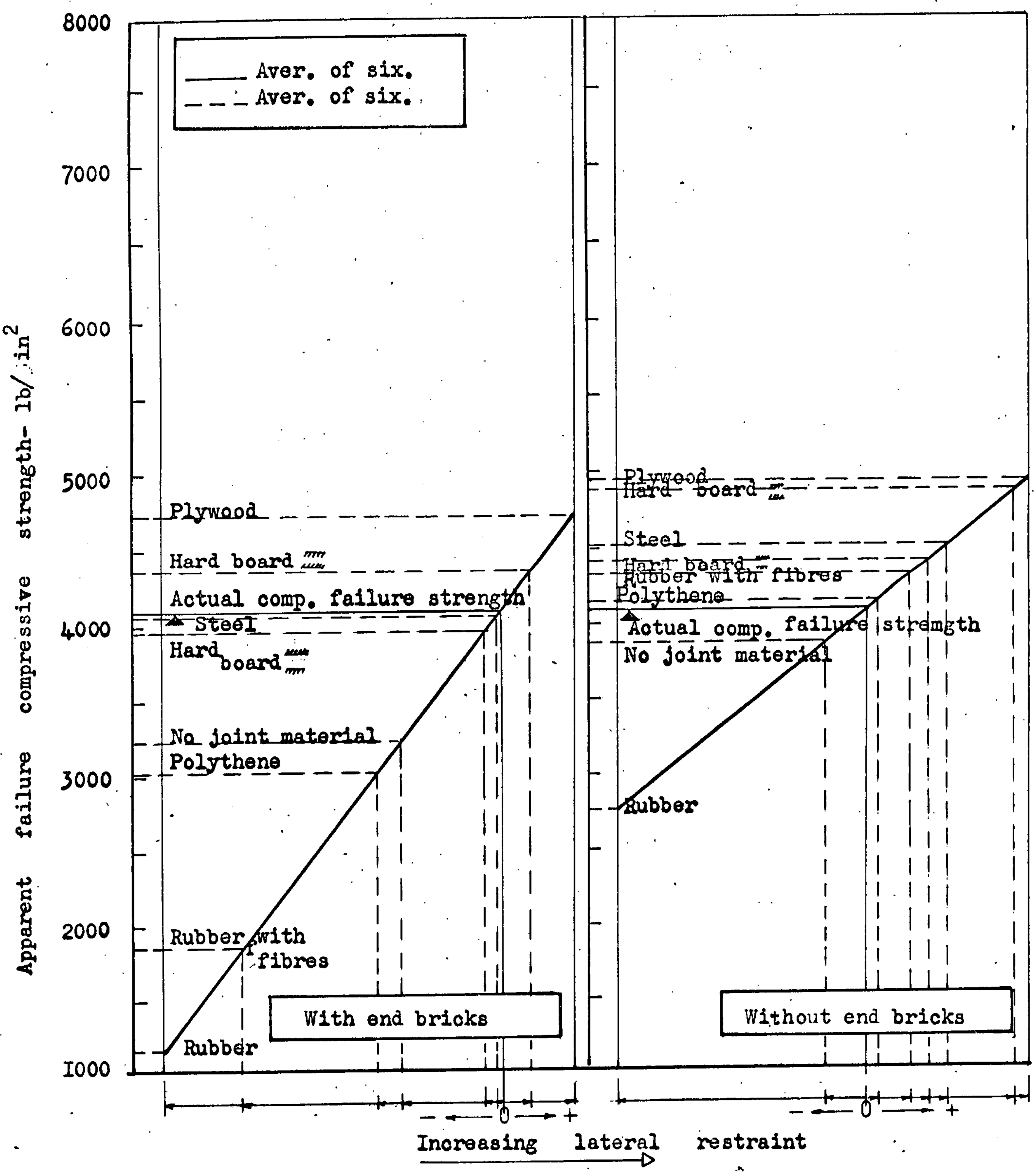


Figure 6.8:
Variation in apparent faioure compressive strength of rough 1/6 scale model bricks tested on end.

6.4.3. Rigidity of Joint Material and Mechanism of Failure

6.4.3.1. General

In 6.4.2. (4) an important question arose namely, which of the apparent failure compressive strengths obtained represents the actual strength? In fact, it is a difficult question and no answer can be given without penetrating deeply into the different conditions produced at the interfaces of contact between the joints and bricks. In such penetration, it first seemed reasonable to divide the joint materials into three categories, softer, of the same stiffness, and harder than bricks.

The insertion of different materials between specimens and the machine platens while a specimen is tested in compression has been of interest to some investigators for a long time. Because each investigator tended to be interested primarily in his own material, knowing the extensive experimental work that would have to be done to cover all possible and conditions, the problem resulting from the unknown wide range of coefficients of friction, and the difficulty in their determination, has become extremely complex. In fact, it has become vague. At the same time it seems impossible to treat the problem either theoretically or experimentally in a comprehensive manner to cover all ranges of rigidities and coefficients of friction.

However, in the light of previous work which was carried out with completely different materials, the mechanism of failure for the tests carried out can correspond to one of the following four failures:

1. Compression failure of bricks with soft end conditions.
2. Compression failure of bricks with rigid end conditions.
3. Compression failure of bricks with moderate end conditions.
4. Compression failure of bricks with ideal end conditions.

6.4.3.2. Soft end conditions and mechanism of failure

When a soft material like rubber either at the end of a brick in a specimen without end bricks or between the bricks in a specimen with end bricks is subject to compression it squeezes out. In the first stage of loading squeezing takes place close to the edges of the contact surfaces. If rubber is not prevented from lateral squeezing out it can as a rule carry only a very slight load compared to the present failure load. Also deformations become very marked in both directions. But because internally it is at least partially prevented from lateral movement, then as the load increases squeezing out becomes greater at the outer edges and a state of non-uniform distributed pressure is produced. The brick is then subject to two kinds of forces:

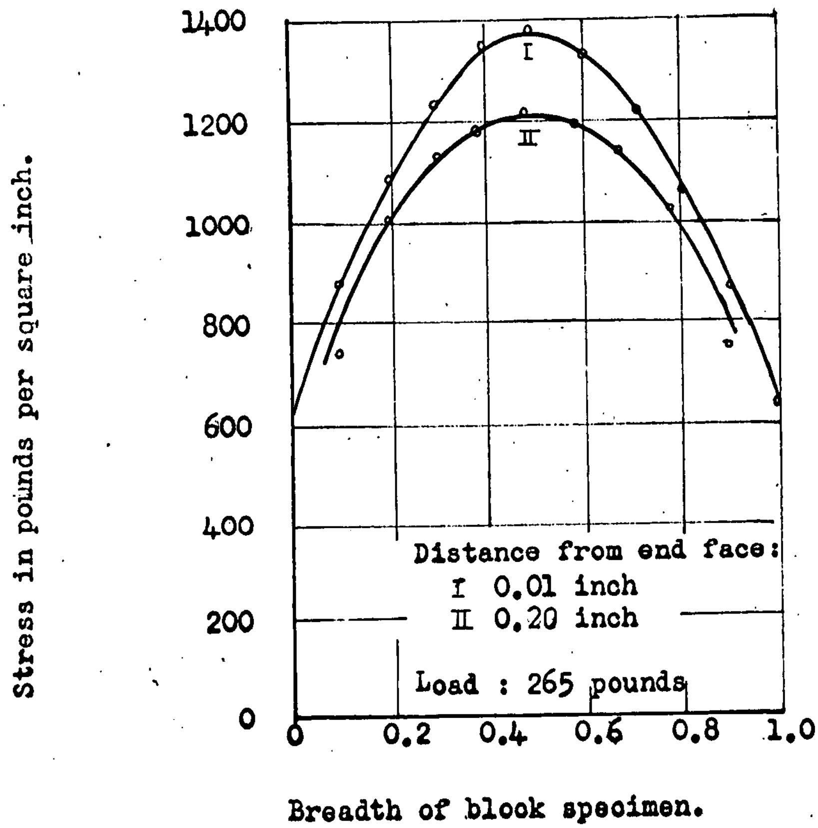
1. Non-uniform vertical compression with its maximum value at the middle.
2. Outward tangential forces.

Both of these forces set up transverse tensile stresses in the brick. They act together accelerating failure by splitting. Because the discussion of this phenomenon is too long to be quoted here, the following are only some of the results by previous investigators given in a very concise form.

Coker and Filon[31], using photoelastic technique, tested a square transparent specimen in compression, inserting rubber sheets between the specimen's ends and the loading platens. As illustrated in Figure 6.9. a vertical concentration of stress was found at the centre of the loaded faces. Photographs 6.4,5 from the present work show typical imprints produced in rubber-with-fibres joints.

Similar results were obtained by Hast[58]. Some of his tests were mainly to study the compression stress distribution under pressure plates with a soft layer interposed. Due to him, Figure 6.10 illustrates the results of his measurements for the distribution of normal pressure in intermediate layers of paraffin, soft rubber, and hard rubber. It can be seen that, on the whole, the curves of the materials run in a similar way. Comparing soft rubber and hard rubber it is clear that the concentration of stresses in the former is somewhat greater than in the latter. This is, of course, due to its greater squeezing out.

The outwards tangential stresses were also of interest to Hast. He found with the same specimens mentioned above that when testing rubber sheets between steel plates, with concrete cylinders as end surfaces (reference to be made to Figure 6.10-b,c) the friction between the concrete body and the material of the layer produced tensile stresses in the concrete body close to its end surface, due to extrusion of the rubber sheet.



Distribution of vertical compression at various distances from the end faces, when compression is applied between thin sheets of India rubber (soft material)

Specimen: a transparent block section 1"x1"x1/4".

Figure 6.9:

Concentration of the vertical compression at the middle due to intermediate layers of soft material. Due to Coker and Filon (31)

Photograph 6.4

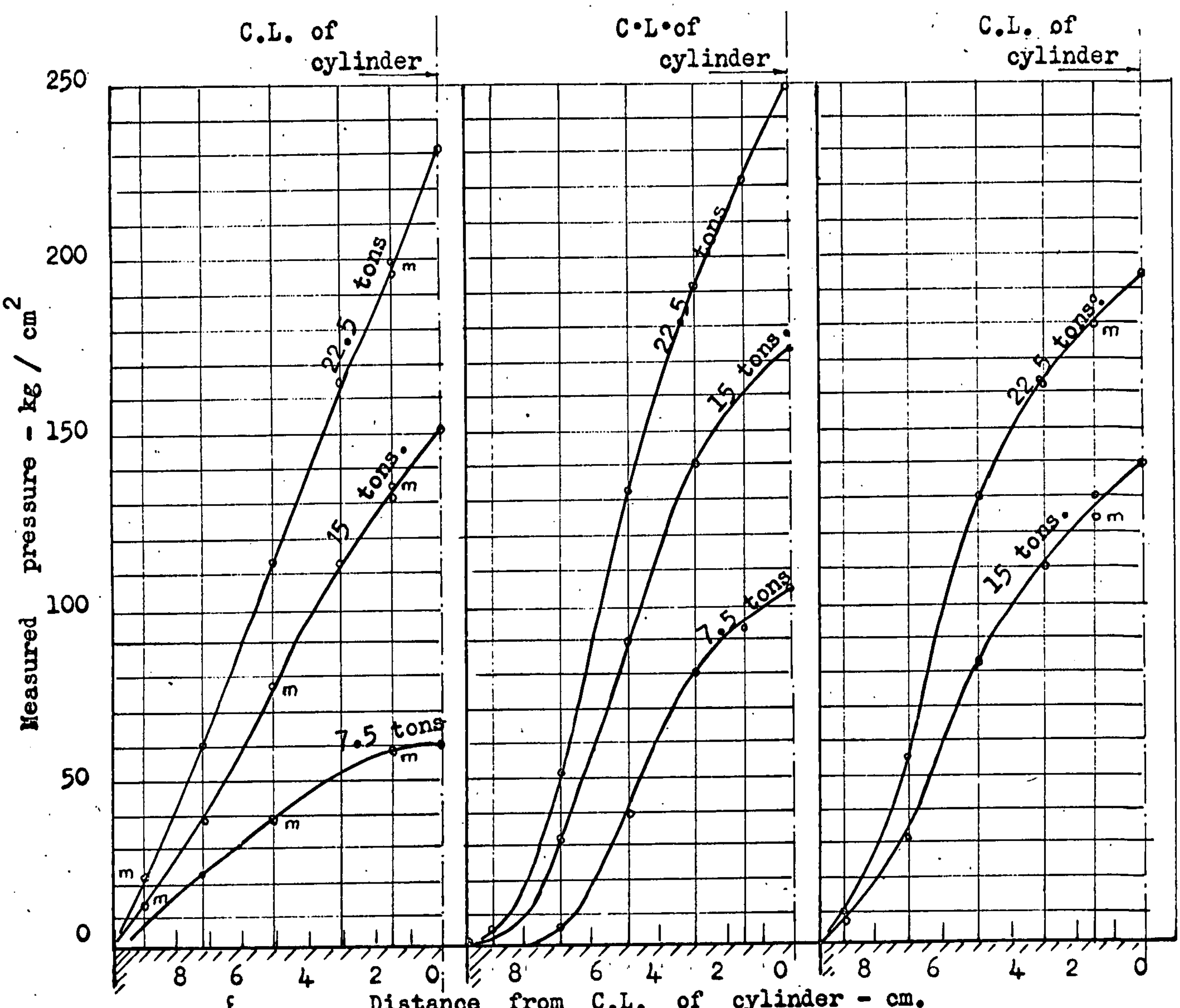
The imprints show how the outer parts of a soft joint material at the brick ends, squeeze laterally under compression, with a resulting greatest normal pressure in the centre. They also show how the cracks take place in brick starting from the ends at planes of contact with the horizontal (perpendicular to direction of loading) joints.

(Soft joint material ≡ rubber-with-fibres).

Photograph 6.5

Photographs 6.4,5:

Typical imprints in rubber-with-fibres (soft material) end joint material.



a. Parafin layer of original thickness 4 mm.

b. Soft rubber sheet of thickness 3 mm.

c. Hard rubber sheet of thickness 3 mm.

- The vertical represent the lines of location of measuring cells.
- The diameter of the test cylinder was 19.6 cms, its surface area 300 cm^2.
- The circles denoted by " m ", constitutes the mean value for all measuring cells located at equal distances from the centre.

Figure 6.10:

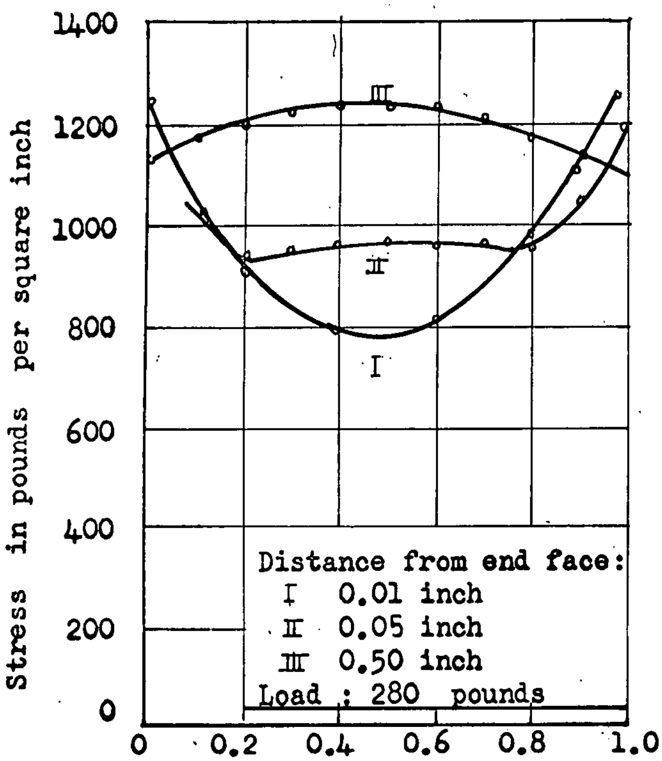
Distribution of normal pressure in intermediate layers of soft materials between hard test cylinders Due to Hast (58)

6.4.3.3. Hard end conditions and mechanism of failure

It is very well known that the ultimate failure compressive strength of a cube tested for compression in the customary manner is greater than that of a prism of the same material. The same phenomenon is produced here between bricks loaded along three directions, in other words when tested through different heights with hard joint materials. It can be easily noticed in Figures 6.13 - 6.16 that with steel and plywood, the apparent failure strength of a brick tested flat is higher than that tested on edge, and again the latter is higher than that tested on end.

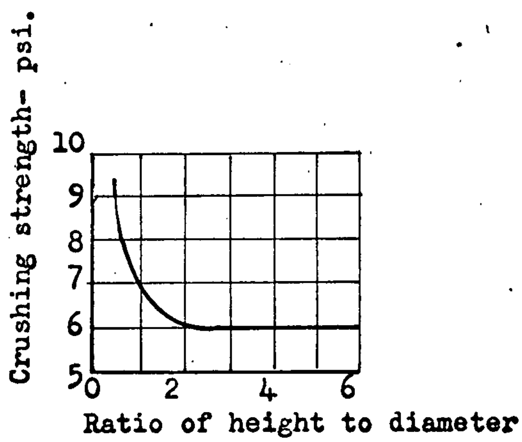
In fact, it has been found recently that the friction or adhesion between the machine platens which is in our case the hard end or joint material, and a brick specimen resembles a restraining band around the top and bottom of the specimen. This friction to some extent hinders the lateral elongation of the specimen under the load, with the production of a state of concentration of stresses at the edges. Consequently, the ultimate apparent failure compressive strength is increased. The shorter the height the greater the interference. Where the height of the tested brick is about 2.5 times its breadth this effect is no longer noticeable.

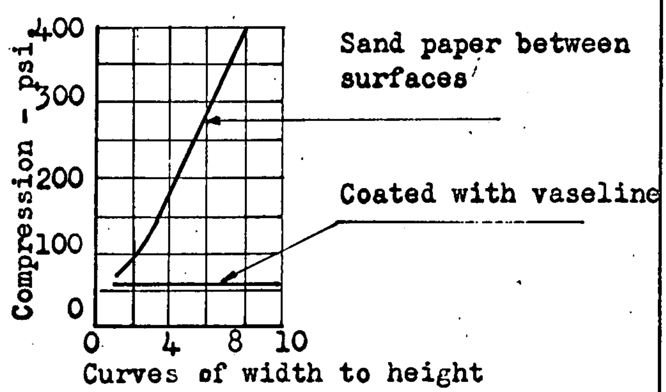
Figure 6.11.a illustrates the results obtained by Coker and Filon from their photo-elastic analysis of a transparent specimen. It was of the same dimensions as that described before specimen in the case of rubber, but tested with brass end. The concentration of stresses at the edges of the loading surface is very remarkable.



a. Distribution of vertical compression at various distances from the end face , when compression is applied between thin sheets ofbrass (hard material).

Specimen : a transparent block section 1" x 1"x1/4".





b. Variation of crushing strength as a function of the relative dimensions of specimen.
Specimen: Sand stone cylinder

c. Stress build-up due to head friction on rubber specimen. (an exagerated case of lateral restraint)

Figure 6.11:
Influence of hard end conditions on :
a. the distribution of vertical compression . Due to Coker and Filon (31)
b. the crushing strength for specimens of different heights . Due to Liddicoat @ Potts
c. the coeffect of lateral restraint. Due to Liddicoat and Potts (89)

As regards the influence of friction towards increasing the compressive failure strength, as a function of the ratio of height to lateral dimension, it was clearly illustrated by Liddicoat and Potts as shown in Figure 6.11 - c, b.

6.4.3.4. Moderate end conditions and mechanism of failure

The word moderate as introduced by the author means a material which can be considered as intermediate between soft and hard materials. It has been thought that materials such as plywood and hardboard behave moderately in that they have a tendency to produce a state of uni-axial compression.

Hast[58] stated that generally, porous wood fibre plates, "Treetex"*, etc. do not possess the tendency to squeeze out. He measured the pressure distribution under load plates with such intermediate layers, and found the pressure uniform. Then he stated, that with an intermediate layer of porous wood-fibres plates on the end, no great frictional forces can be transmitted, and therefore, the drawback which is usually in connection with steel loading platens is avoided. In other words, frictional forces preventing lateral elongation would almost disappear, and the spreading of the strength values becomes less.

As a conclusion he generalized that since a plate of these materials is very porous, from the start it adapts itself to local inequalities in the loaded surface of

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* Swedish compressible material.

the test bodies. But afterwards, he mentioned again that in this way the occur of course local variations of stress, but these as a rule, are of no importance for ultimate strength.

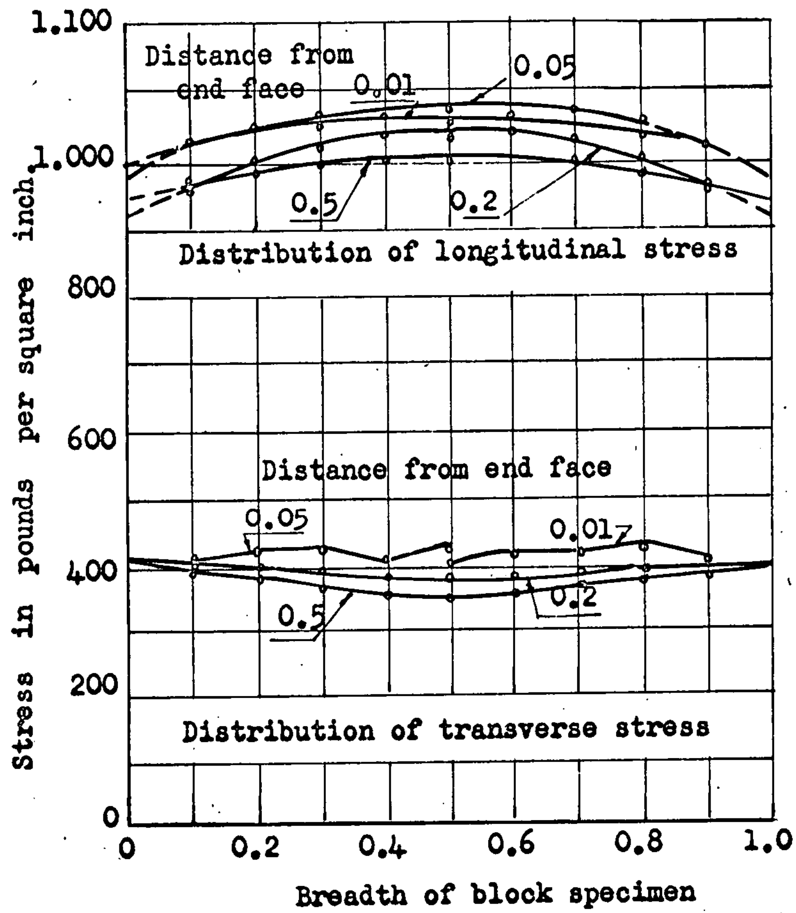
It can be easily seen that the results of the present work are in disagreement with Hast's results in abeolute terms, at least for bricks under compression. Figure 6.13-6.16 shows that such materials cannot have the property of producing a state of uniaxial compression for all directions of loading, whether flat, on edge, or on end. Plywood for example, behaved either in a similar manner to steel with a smaller rate of variation in strength between the three directions of loading, or with almost no variation. Comparing these values with the actual failure strength (explained later in 6.4.3.5.) it can be concluded that plywood does not eliminate frcitional restraint, but it sometimes produces a uniformly distributed restraint along the contact surface. Therefore, the author feels that Hast's conclusions apply only to his tests and cannot be considered as a general rule. According to the present tests plywood and similar materials are included in the hard category.

6.4.3.5. Ideal end conditions for failure under a state of uniaxial compression

It was one of Coker and Filon's main objects to reach the case of pure compression without the occurrence of any transverse forces due to end conditions. They concluded in the light of their tests that in order to produce this condition the load ought to be applied through an intermediate layer of the same material. Then from subsequent tests using different conditions, such as wider intermediate materials, they came to a more definite conclusion.

They added to the first condition that the intermediate end plates must be of sufficient thickness, to allow the disturbing effects to vanish within them. Also, the surfaces of contact must be plane surfaces of contact must be plane surfaces of a high order of accuracy. This was illustrated by them graphically as shown in Figure 6.12. As regards the slight effect appearing near the edges, they stated that the intermediate platens had shown considerable variation in stress at the end near the metal platens of the compression machine. But the specimen itself was very nearly under pure compression stress, as was shown by its almost perfectly uniform colour.

Accordingly, the only case from the present tests which complies with Coker and Filon's ideal is the brick tested on edge with end bricks, no joint material and super grinding. Therefore, the actual compressive strength for all the batch is considered to be 4068 lb/in^2. (See Table 6.5 and Photograph 6.11). The importance of the minimum depth of end platens as recommended above can be clearly seen when comparing this value of the ideal specimen, and the



Specimen : Square block 1" side and 0.254" thick , and and load is applied by blocks of the same material 0.4 deep , the same width and the same thickness.

Load : 260 pounds.

Figure 6.12:

Distribution of longitudinal and transverse stresses in the case of " the ideal case" as proposed by Coker and Filon. Due to Coker and Filon (31)

similar one tested flat. The latter equals 4425 lb/in^2. In other words it increased due to the insufficient depth of end platens relative to the smallest lateral dimensions, for diminishing the end restraint. Here the breadth of the specimen is "b", and not "a" as the ideal case.

It is to be remembered here that it may happen that one or some of the apparent compressive strength are equal or very near to the actual strength. But this happens only by mere chance and not according to any rule. An example of this is shown in Figure 6.7 to the right. It is clear that the strength of a ground brick tested on end without end bricks is very close to the actual failure strength. On the other hand the strength of a similar specimen with super grinding is less close. A possible explanation for this is that while the height of the latter is the main factor in reducing strength, the more frictional force in the former compensates for this reduction with ultimately a strength closer to the actual strength.

6.4.4 Variation in Apparent Failure Strength of a Brick Due Different Directions of Loading. (Effect of Brick Height)

It has been pointed out that

the apparent compressive strength of a specimen with soft joint material like rubber, is low for two main reasons. These are the transverse tangential stresses in combination with the concentration of the vertical stresses in the middle. Both are at the contact surface. Introducing the way of placing a brick in the machine and the direction of applying the load with respect to the brick surfaces, and terming this as the effect of brick height becomes very interesting. Figures 6.13 - 6.16 illustrate the apparent failure compressive strength for specimens tested without end bricks. For each joint material the apparent strength was plotted for the three possible ways of laying or testing, flatwise, on edge, and on end.

As the above mentioned figures show, the strength of a brick tested with rubber, rubber-with-fibres, and polythene at the ends increases gradually between the first two positions, flatwise and on edge, while between the positions on edge and on end the strength increases very rapidly. Ultimately, for tests on end, rubber-with-fibres and polythene produce apparent failure strengths very close to the apparent

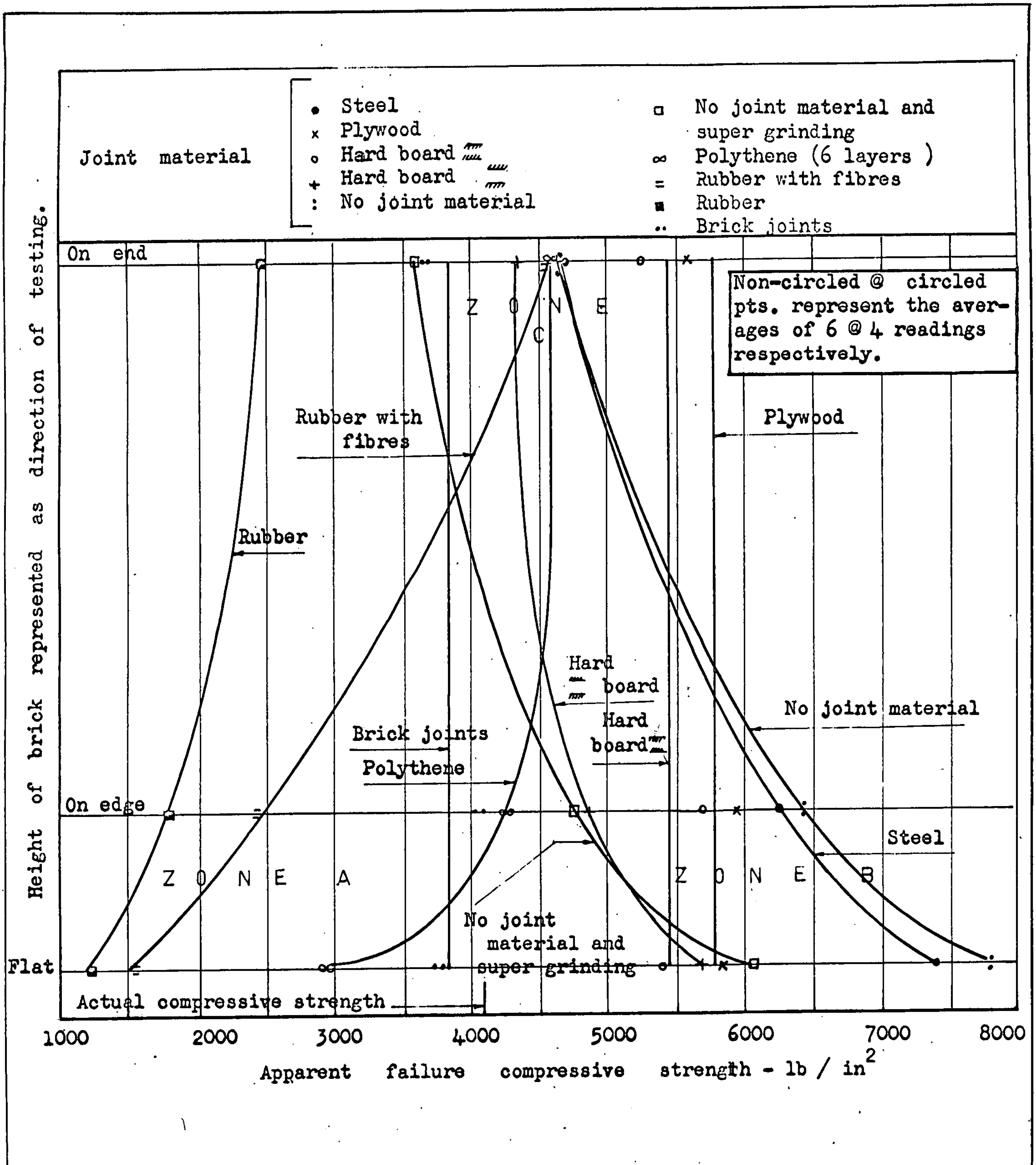


Figure 6.13:

Variation, due to effect of height, in apparent failure compressive strength of ground 1/6 scale model bricks tested without end bricks.

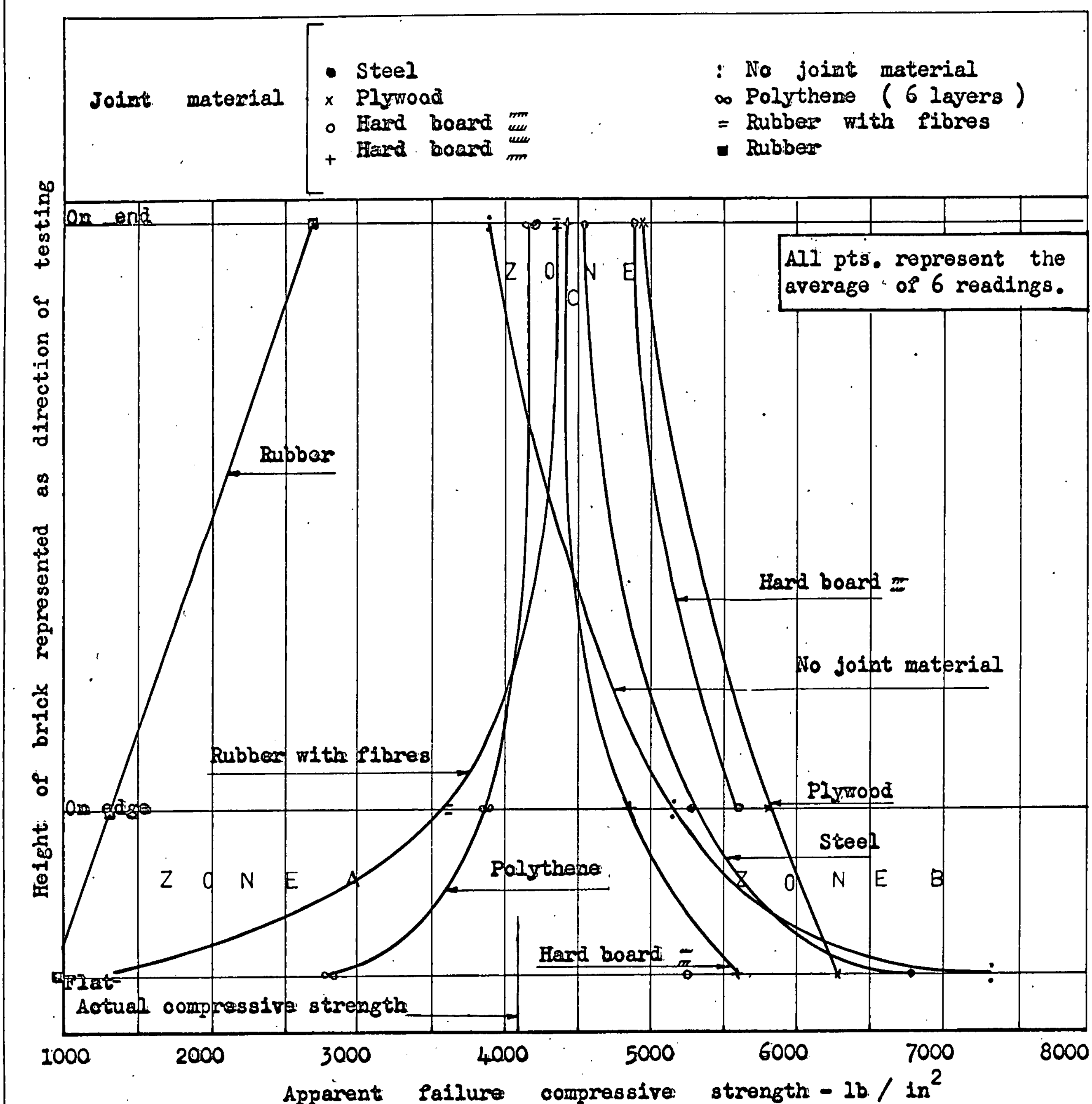


Figure 6.14:

Variation , due to effect of height , in apparent failure compressive strength of rough 1/6 scale model bricks tested without end bricks.

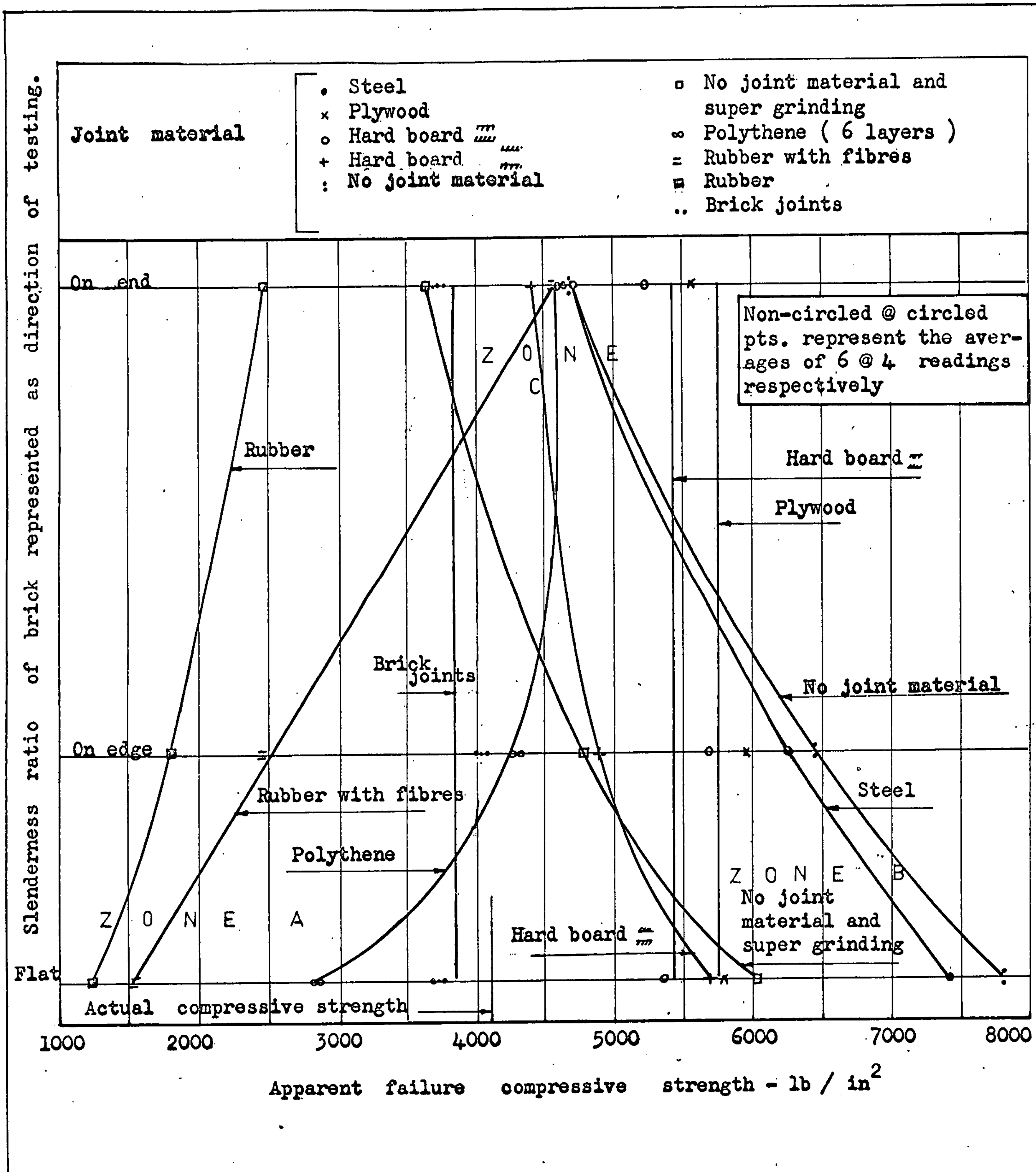


Figure 6.15:

Variation , due to effect of slenderness ratio , in apparent failure compressive strength of ground 1/6 scale model bricks tested without end bricks.

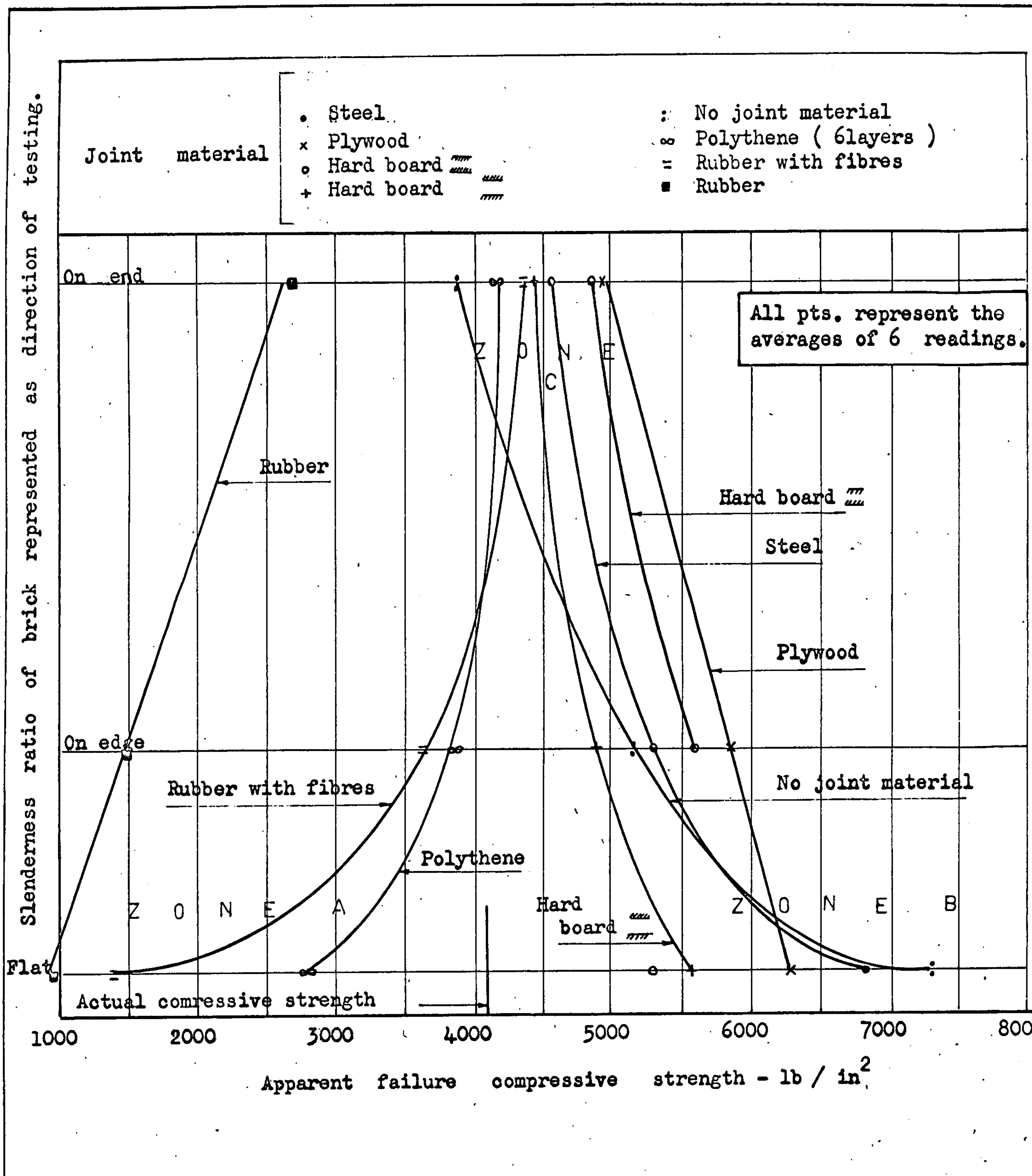


Figure 6.16:

Variation , due to effect of slenderness ratio, in apparent failure compressive strength of rough 1/6 scale model bricks tested without end bricks.

strength with steel. Another important remark is that in the on-end position the strength produced from rubber-with-fibres and polythene is very near to the strength of the brick tested on-edge with end bricks and super grinding. The latter is the actual compressive strength.

From this comparison it can be said that the ultimate degree of bulging-out of a brick due to the presence of a soft material of constant thickness at the end, which is interpreted as splitting failure, is influenced by two factors related to the way of testing the bricks. Considering the brick generally as a specimen subject to compression, then its size and shape must be well defined. If we consider now its shape only, then it is clear that there is no great variation in the shape, and to determine it quantitatively the term slenderness ratio is enough. That is the ratio between the height along the direction of loading, and the length of the smallest side in the other perpendicular direction. These can be put in other words as the height of the brick and its area of contact at the ends.

It seems possible to find combinations of these two factors such that each combination, individually, leads to diminishing or minimizing the quick splitting resulting from the presence of soft joint material. This is why it has been said that the height of the brick substantiates the squeezing out of rubber-with-fibres. In other words it substantiates for the combined action of tangential forces and a vertical concentration of stresses.

Accordingly, another important

question arises. That is how far are the transverse stresses distributed along the vertical and where is the horizontal plane of maximum value? Undoubtedly, this plane is neither the plane of contact with the soft joint material nor the plane at the centre. The first is simply because it is unlikely for this value to occur at the ends. The second is because it is inconceivable for the maximum bulging-out to occur in the brick at the farthest point from the source of bulging. The latter is the squeezing out of the soft joint material. What is quite believable is that the minimum lateral deformation can occur at the centre of the tested brick. And, as the height increases there will be a limit where bulging out dies before the centre, with a resulting zone symmetrically around the central plane where a state of uniaxial stress exists. The height of this zone is a function of the height and the relative rigidity of the brick and the joint material. Also it is not necessary for the state of uniaxial loading to exist for all stages of loading. But at least it exists at the first stages of loading. Finally, the apparent failure strength becomes more or less equal to the actual compressive strength.

It is worth giving here some explanations from a previous work which, to some extent, resembles the present case. Although complete similarity does not exist the author feels these results are quite helpful in visualizing the present situation. From their observations on the effect of splitting forces, Holister and

Florin[64,65] stated that in general, the splitting tensile force occurs at a distance of 2-4a below the edge of their loaded plate. This is illustrated graphically as shown in Figure 6.17-b,c. With increasing load concentration b/a and with increasing relative slab height h/b the depth of the resultant increases.

The statement, as well as the figures, shows strongly how the maximum plane of bulging can be at a short distance from the surface of contact, at which the splitting force acts. As regards how the transverse forces die after their maximum value and towards the centre, it is clear in the same figure that for the lowest value of b/a which represents the minimum load concentration, it dies completely before midheight of the plate in "b." and before the quarter point in "c". On the contrary for a load concentration of 29, in "c" the splitting forces die only at the fixed end.

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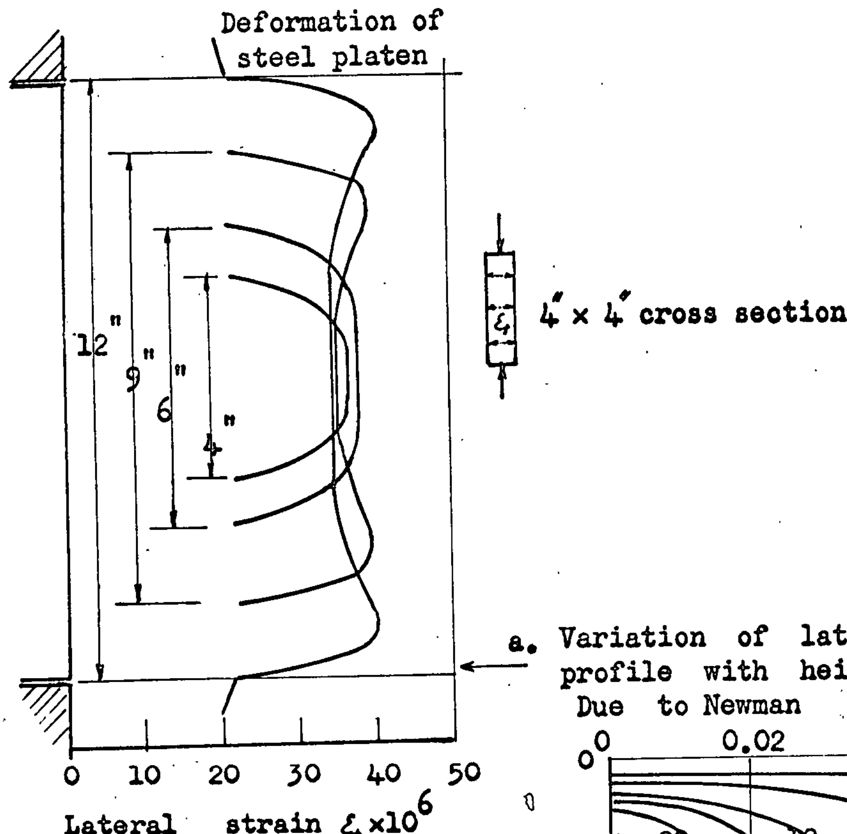
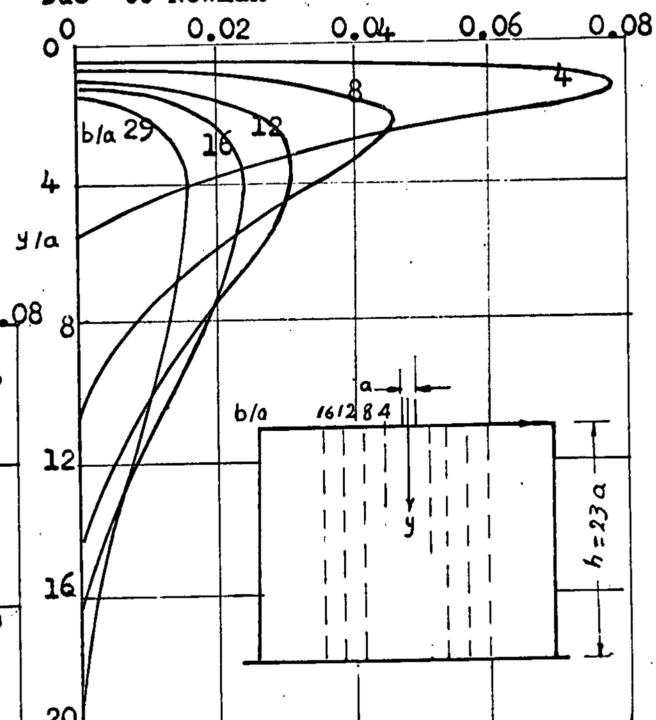
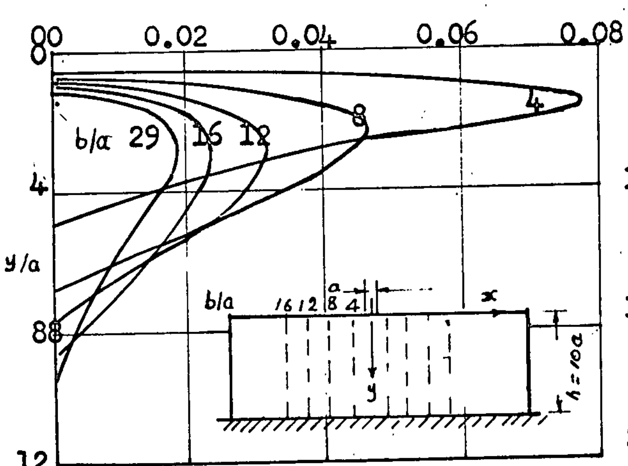
(64): Hiltscher, R. and Florin, G. Splitting and tear stresses in rectangular plates loaded at varying distances from the plate corner. National Research Council of Canada. Tech. Transl. No. 1137. Ottawa, 1964. (Translated from German
(65): Hiltscher, R. and Florin G. The splitting tensile force in rectangular plates fixed along one side and loaded at opposite side. National Research Council of Canada. Tech. Trans. No. 1224. Ottawa 1965.



a. Variation of lateral deformation profile with height of prism. Due to Newman





b. Splitting tensile stresses σ_x/q as a function of the distance y/a from the loaded edge for various load concentrations b/a for a constant tant model height of $h/a = 10$

c. Splitting tensile stresses σ_x/q as a function of the distance y/a from the loaded edge of various load concentrations b/a for a constant model height of $h/a = 23$

Figure 6.17:
a. Variation of lateral deformation profile. Due to Newman (99)
b, c. Splitting tesile stresses. Due to Hiltscher and Florin (64 65)

Similarly, it can be said in the present case, that the central zone which can be subjected to uniaxial compression is a function of the height of the brick along the direction in which the load is applied. Also, it can be stated that beyond a certain height or certain ratio of brick height to the least dimension and before buckling can take place, the splitting influence of the end joint material dies before the centre. Consequently, the apparent failure compressive strength becomes fairly independent of the end joint material. This can be seen clearly in Figures 6.13 - 6.16. Undoubtedly this is not absolute, and applies only for a range of smaller variation than the present range with respect to the joint materials.

As regards a hard joint material the contrary might be expected. For smaller heights of a brick maximum bulging occurs at the centre. For greater heights maximum bulging occurs at a plane near the end in a more or less similar manner to the case of soft joint materials. This has been confirmed by the very recently published data of Newman[99]. According to him the variation of the lateral deformation profile with height of prism specimen for an applied stress through a hard platen, is illustrated in Figure 6.17-a.

The author suggests that each of Figures 6.13 to 6.16, which show the effect of brick height on the apparent failure compressive strength and incorporate materials ranging from the soft to the hard, can be divided roughly into three zones as follows.

1. Zone A, in which the end joint material is soft compared with the brick. The controlling influence of the former is towards decreasing the apparent failure compressive strength of the latter.

2. Zone B, in which the end joint material is hard compared with the brick. The controlling influence of the former acts towards increasing the apparent failure strength is chosen as a basic strength.

3. Zone C, in which the joint material can be either hard or soft. So long as the buckling effect is not taking place it can have little or no effect on the apparent failure compressive strength. In the three zones, the actual compressive failure strength is chosen as a basic strength.

The above discussion explains what was pointed out before in 6.4.2 (10), where it was mentioned that with specimens tested on edge and on end, with soft joint material and with end bricks, the height of the latter becomes of the utmost controlling influence on the apparent failure strength. Undoubtedly, this has become clear now. While the middle brick is influenced by the soft joint material under the condition of zone C, the end bricks are influenced under the conditions of zone A. In other words, the destructive action of the joint material causes failure of one end bricks, before the middle one. Consequently, the latter's height is of small effect. This effect can be seen

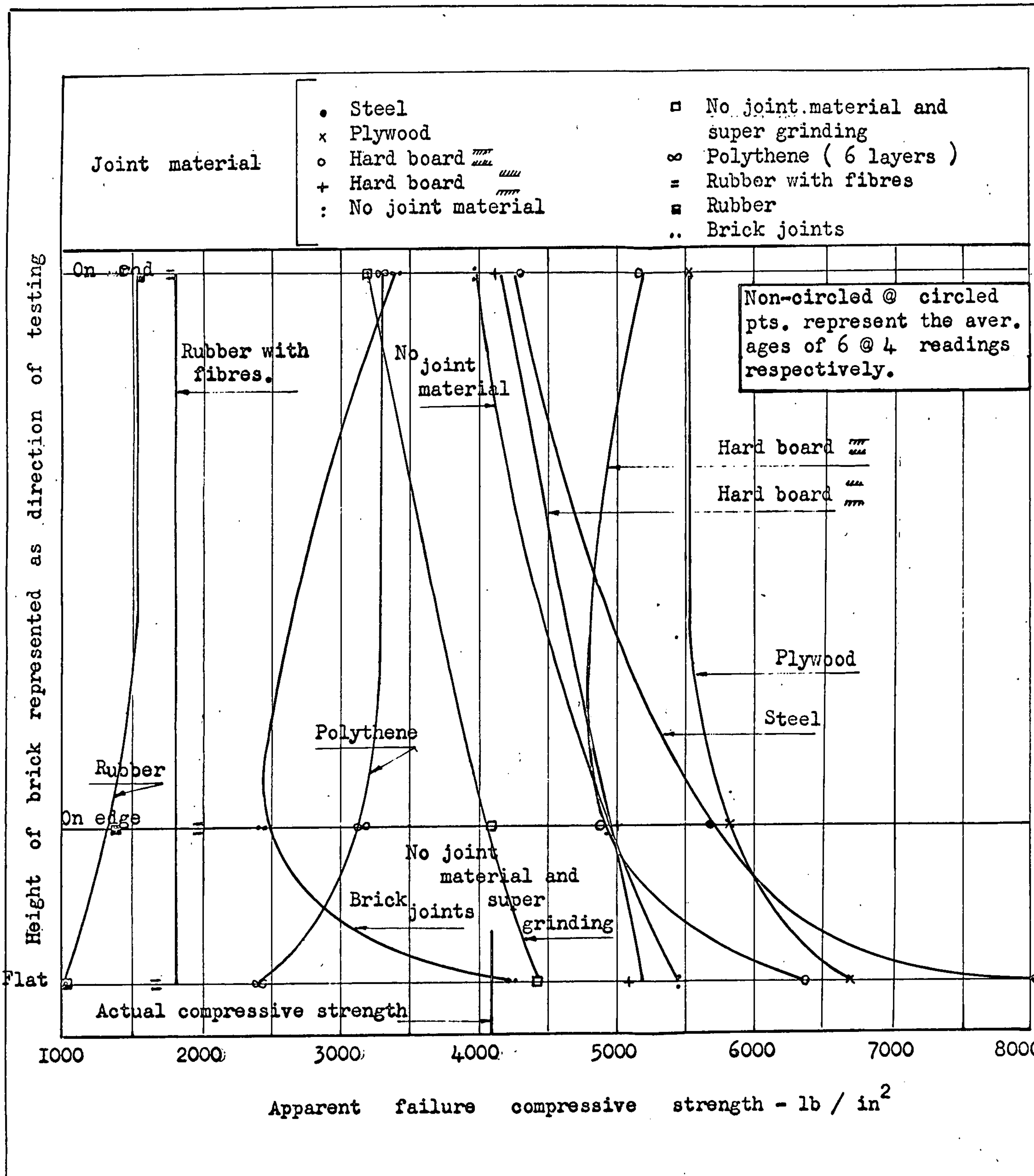


Figure 6.18:

Variation , due to effect of height , in apparent failure compressive strength of ground I/6 scale model bricks tested with end bricks.

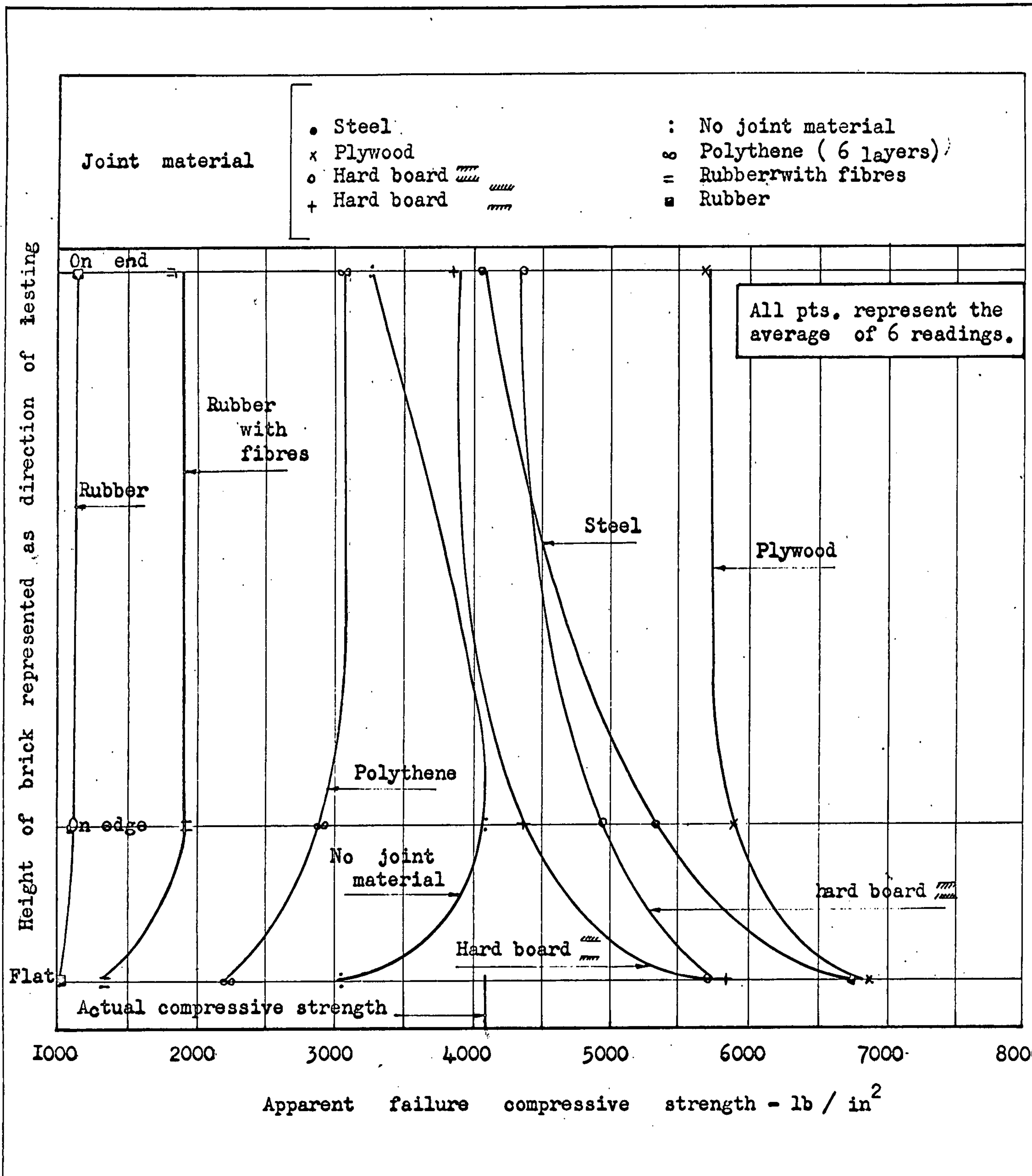


Figure 6.19:

Variation, due to effect of height, in apparent failure compressive strength of rough 1/6 scale model bricks tested with end bricks

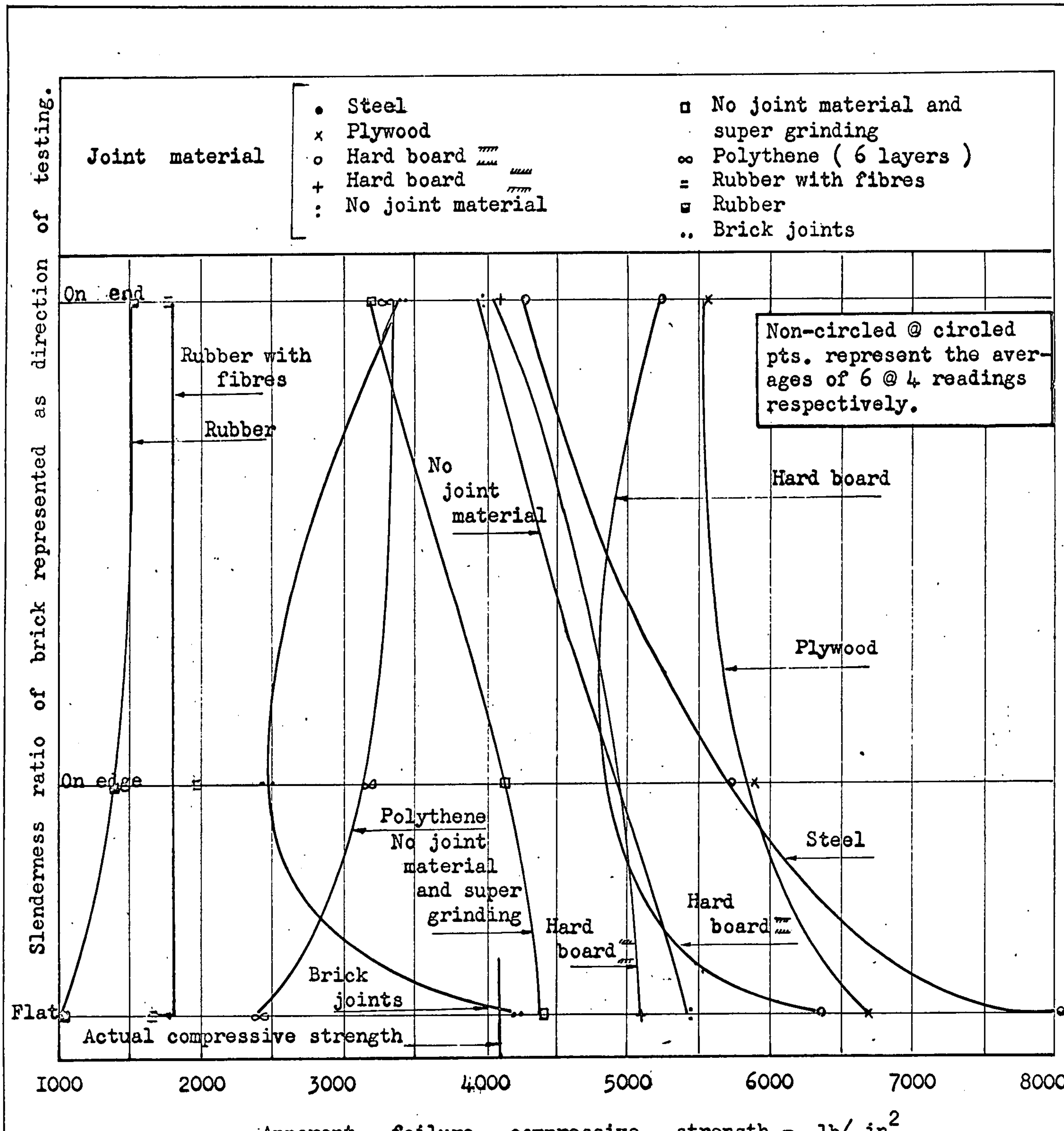


Figure 6.20:

Variation , due to effect of slenderness ratio , in apparent failure compressive strength of ground 1/6 scale model bricks tested with end bricks.

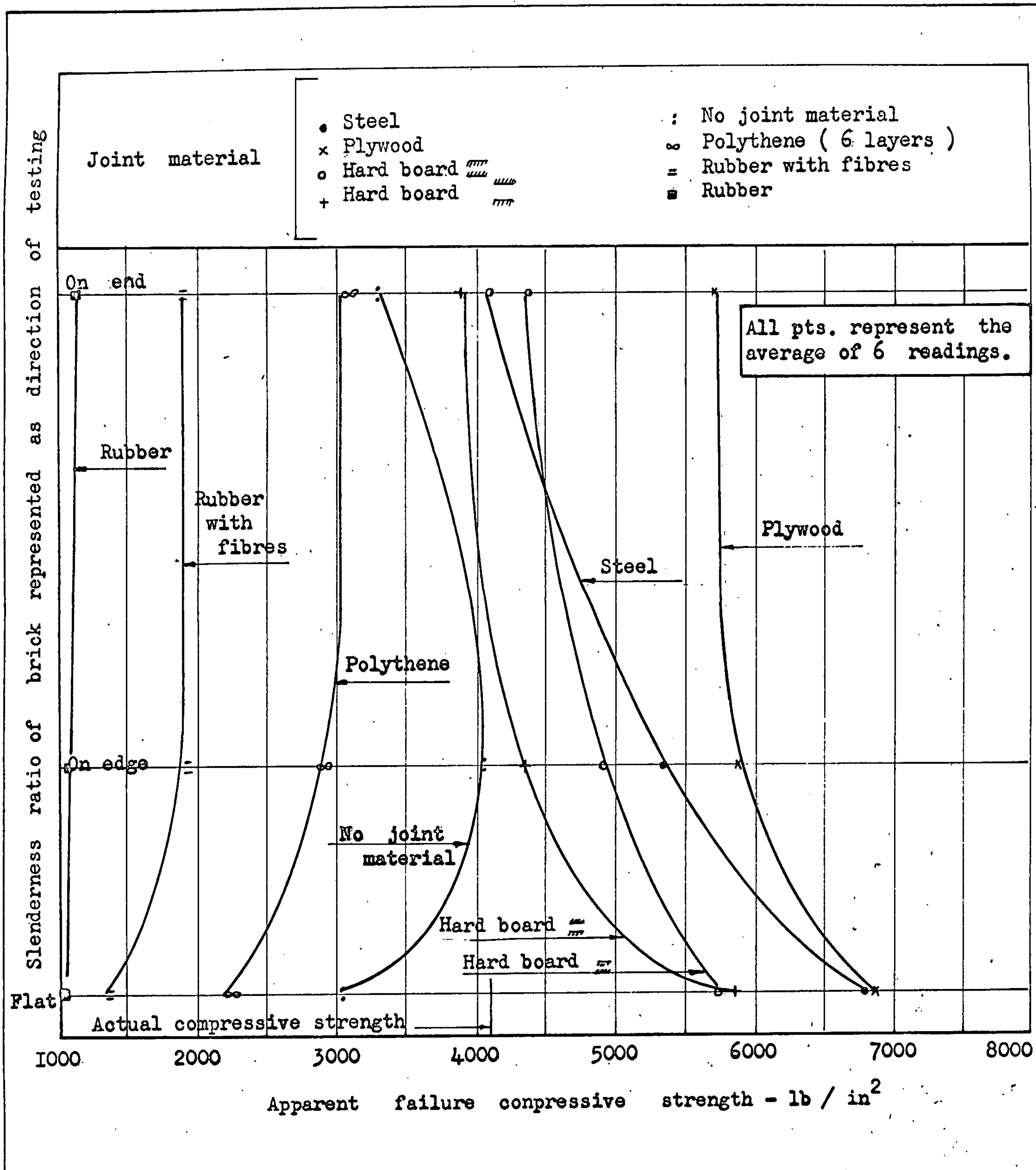


Figure 6.21:

Variation , due to effect of slenderness ratio , in apparent failure compressive strength of rough 1/6 scale model bricks tested with end bricks.

by comparing the specimens tested with end bricks and soft joints materials. Without exception specimens tested on edge and on end are of equal or slightly higher strength than similar bricks tested flat. The comparison is easier in Table 6.9.

Two very pronounced examples for the influence of brick height on the failure strength of bricks tested between softer joints are the results obtained at the Building Research Station[36][122]. The results were from real mortars and full scale bricks. In the first, two types of bricks were tested in compression. Six strength groups for each type, each were each used with five grades of mortar. The results are illustrated graphically in Figure 6.2.2.

In all tests, without exception, the strengths of bricks without frogs are higher than the corresponding ones (the same brick strength group, and the same grade of mortar) of bricks with frogs. No explanation was given for this in the above mentioned reference. But in the light of the present work, there can be only one explantion. That is the lateral squeezing-out of mortar, resisted in the case of no-frogs by a bigger

Brick strength group	Mortar	Bricks bedded in mortar of strength (lb / in^2) 2,000	4,000	6,000	11,000
With frogs	1,510 - 3,500	2,650	2,860	3,190	--
	3,510 - 6,000	3,820	4,300	4,810	--
	6,010 - 8,500	4,670	5,390	6,250	7,710
	8,5100-11,000	5,800	7,170	8,630	9,280
	over 15,000	8,360	9,450	10,180	12,210
Without frogs	1,510 - 3,500	3,110	3,470	3,660	--
	3,510 - 6,000	4,160	4,660	5,070	--
	6,010 - 8,500	6,100	7,280	8,280	--
	8,510 -11,000	7,190	9,040	10,220	11,000
	11,010 -15,000	10,580	10,570	12,330	12,800
	over 15,000	11,630	12,440	13,860	16,380

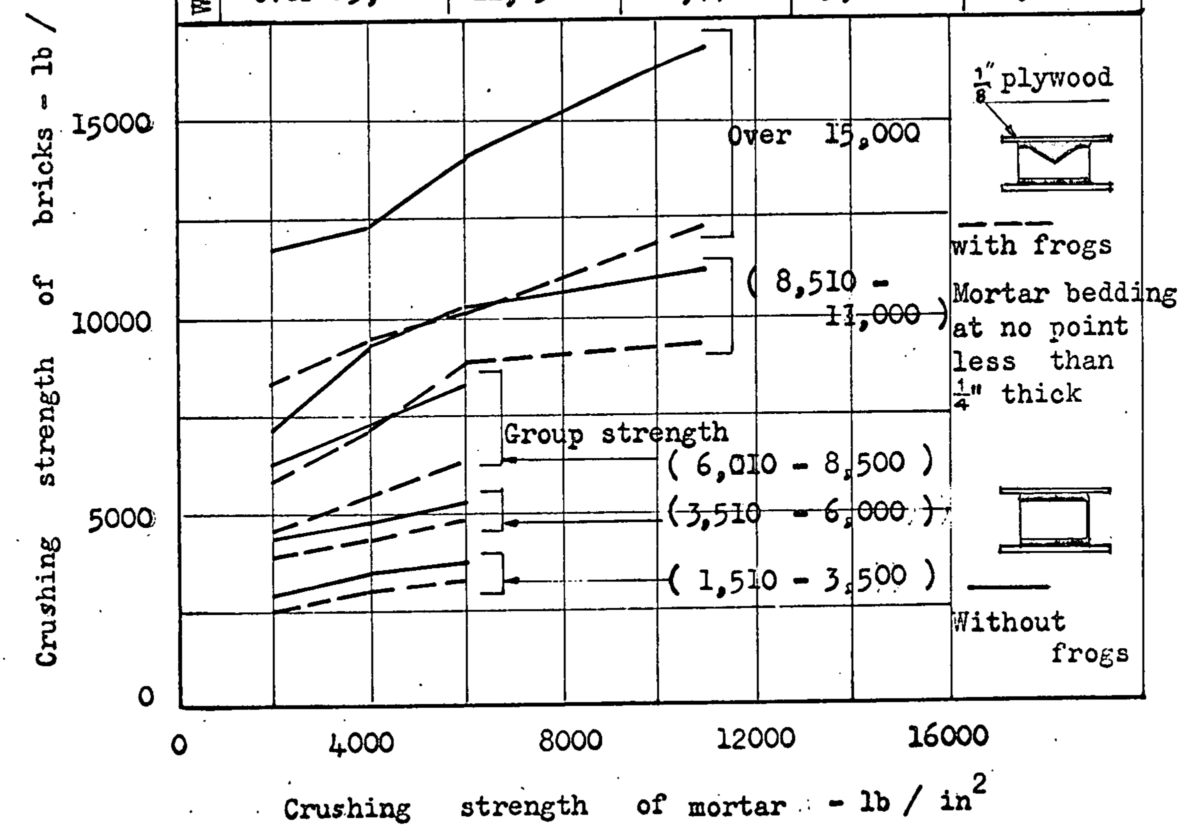


Figure 6.22:

Comparison between crushing strengths of bricks with and without frogs tested at Building Research Station. [After Davey and Thomas (36)]

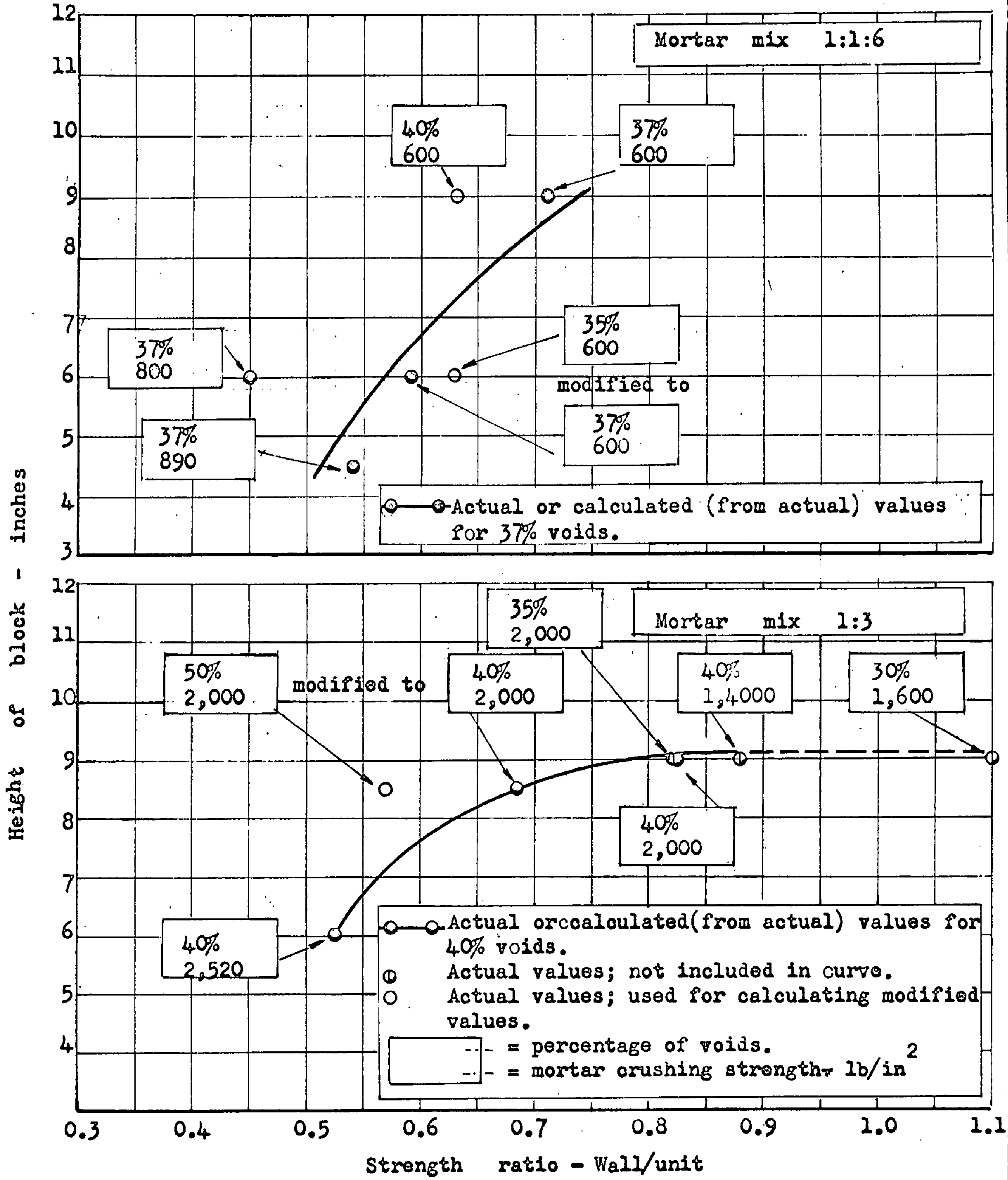


Figure 6.23:

Graphical interpretation of results for walls built with hollow clay blocks (horizontal voids). Due to Building-Research-Station Engineering Series No. 24. (122)

cross-sectional area than in the case of frogs. In other words it is the effect of height. The second example is the results quoted by Simms as indicated in Chapter 1, and illustrated graphically in Figure 6.23. It is clear from the figure that there is very little approximation resulting from the modification. The evidence is ample in the lower part of the curve that at a certain height the influence of mortar might be practically negligible. In the upper part the weakening influence is also found. It seems probable that the same trend could have happened if more tests were carried out.

Here the types of bricks produced commercially, become open to question. It is well known that fired bricks obtainable nowadays vary in shape, dimensions, and the nature and number of perforations or cavities. These perforations in solid bricks are made with the objects of reducing the weight, improving the bond between the bricks and the mortar, increasing the thermal insulation properties and increasing the compressive strength. Now the present tests and the foregoing discussion show clearly that the last one of these objects is not correct, so long as the others are kept unchanged. It can be maintained strongly that the solid brick in its oldest form has better dimensions than present-day bricks for resisting the destructive action of mortar which is traditionally weaker than bricks. In other words they are the best from the point of load-bearing capacity. If other objects are not equally fulfilled with the different types of bricks, then after this clarification, the question becomes: Could it be accepted as a principle to reduce the strength properties in favour of other properties? The question cannot be answered directly at the moment, due to the many factors involved. For an adequate answer we must go outside the field of the present investigation and consider the matter comprehensively from the point of view of pure economics. However, for this economic study, it is to be emphasized again that the net cross section of a brick, and not the gross section, is the main contributor to the strength of brick masonry with the conventional mortars.

6.4.5. Different Modes of Failure

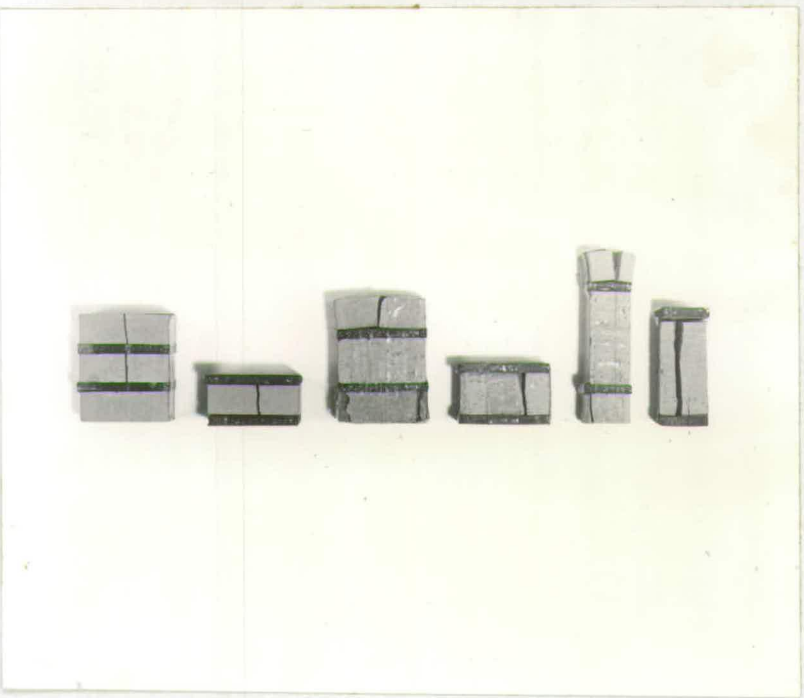
Similarly to the apparent failure compressive strength, the mode of failure is undoubtedly affected by the state of stress which, in turn, is a function of the end joint material. So long as failure is reached two modes of failure can be obtained, namely splitting and shear failures. Both can be obtained with any joint material, either soft or hard, but the mechanisms of failure are completely different. This can be classified roughly as follows:

1. With soft end-jointing material and flat bricks the mode of failure is undoubtedly splitting as illustrated in Photographs 6.6,7.

2. With soft end jointing material and bricks on edge or on end both splitting and shear failures can be obtained. There are many parameters which determine which of them is going to occur, such as the relative dimensions, the relative rigidities, and the coefficient of friction. All of these determine the degree of stress in both lateral and vertical directions. Vanishing of the former, or its dying out before the centre, and the plane where this occurs is of controlling influence. Generally, below a certain low ratio of the thickness of the soft joint material to the height of the brick, shear failure could be expected to take place. Otherwise, splitting failure takes place.

3. With hard end-jointing material and flat bricks the mode of failure is undoubtedly shear failure as shown in Photograph 6.8. This happens so long as the state of concentration of stresses pointed out in the next paragraph (4) does not exist. Even if splitting happens due to this cause it is usually followed by increasing resistance to the load, and again shear failure in the broken parts. In fact, what happens is that the brick is divided into two halves, and the situation becomes more or less testing two parts, each acting as a small brick.

4. With hard end-jointing material and bricks on edge or on end, both shear and splitting failures can occur. Splitting here occurs in a manner different to the one mentioned in (1), which was due to the presence of soft joint material at the



Photograph 6.6

The photographs show a typical compression failure by rupture or vertical splitting in the direction of the applied load.

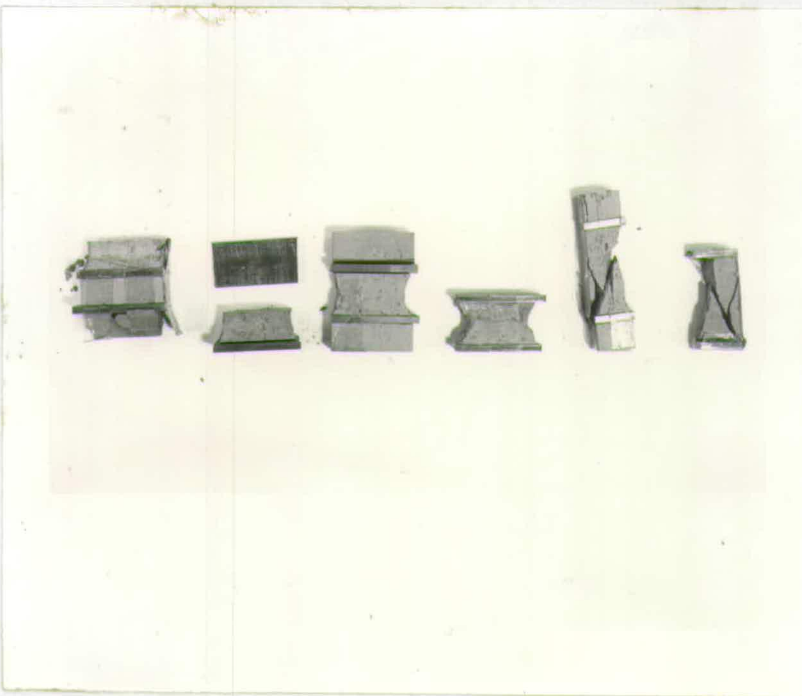
Note:

In all specimens on edge and on end with end bricks failure is in the end bricks. Reference is to be made to 6.4.5-1.

Photograph 6.7

Photographs 6.6,7:

Typical splitting failures of all sets of specimens due to soft joint materials.



Photograph 6.8

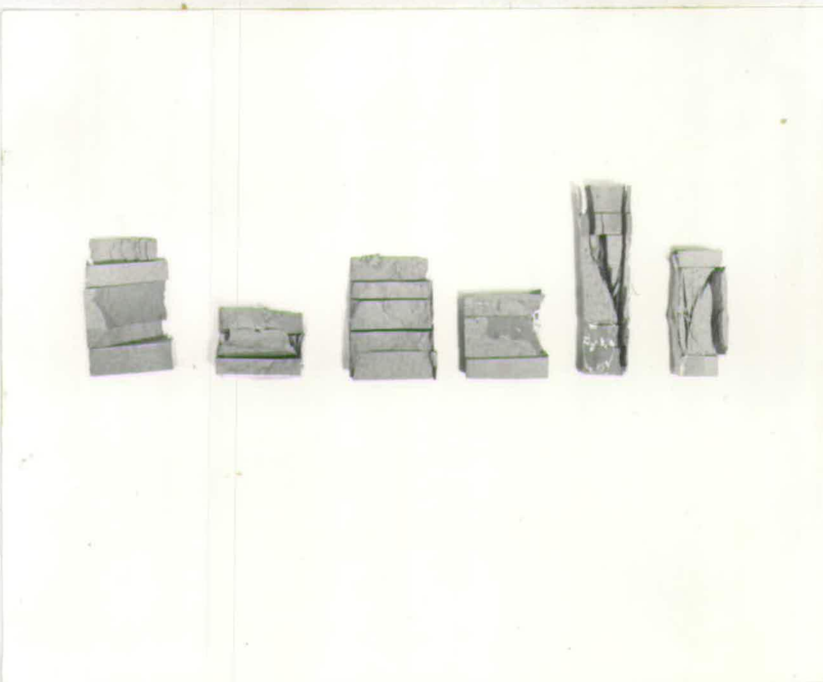
The photograph illustrates a typical compression failure with steel joint material. It is clear the the mode of failure is mostly shear.

Photograph 6.9

The photograph shows typical vertical splitting in two bricks due to concentration of loads at points. Failure is very similar to that resulting from an indirect tension test. To the right, the brick is tested between two steel joints. To the left it is tested directly between the machine platens.
Note: Enlargement is approximately to actual size.

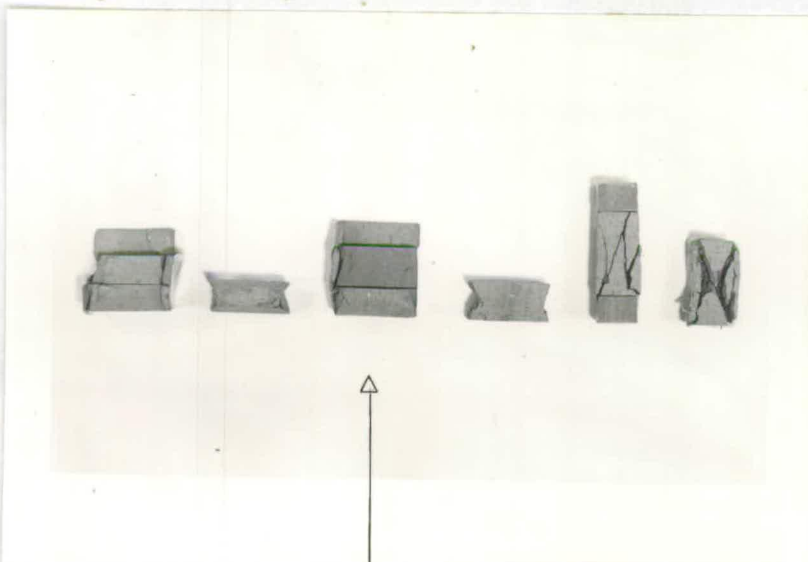
Photographs 6.8,9:

Typical shear failures (in most of the cases) and splitting failures (in few cases) due to hard steel joint material.



Photograph 6.10

Brick joints and super-grinding on all horizontal surfaces.



—ideal loading

Photograph 6.11

No joint material and super grinding on all horizontal surfaces.

Note:

From text, the ideal loading apply to the third from left. (Reference to be made to 6.4.3.5.)

Photographs 6.10, 11:

Typical failures due to super grinding at surfaces of contact, for the cases of brick joints, and no joint material.

brick ends. Here, when the brick is loaded, depending on the evenness of the bearing surfaces in the case of rough bricks and the degree of brittleness in the case of ground bricks, the total load might be transferred at one or more points. In other words sharp concentrations of loading happen at very early stages of loading. These concentrations cause splitting whose mechanism of failure is more or less the same as in the case of the indirect-tension test, which was discussed in detail in chapter 5. Photograph 6.9 illustrates typical failure of this kind.

6.4.6. Effect of Thickness of Soft Joint Material on the Apparent Failure Compressive Strength.

It has been the trend in all previous work to study the effect of the thickness of the bed mortar joints just by carrying out tests with various values for the thickness. This was, of course, because of the tradition that bricks are usually laid flatwise. In other words the height of a brick in the direction of bricklaying was considered invariable in all stages from the start of production until construction ends. The common conclusion from the previous studies[12][87][131] as regards the thickness was that the strength of brick masonry increases as the thickness decreases, and vice versa.

The same phenomenon was studied in the present tests with soft joint materials replacing mortars. But the study was done in a completely different manner. The variation in apparent failure compressive strength was interpreted as a function of the ratio between the thickness of the end joint material and the height of the brick in the direction of compression. Here the thickness is constant for each joint material individually but differs from one material to another, and the height of the bricks is variable. Figure 6.24 shows the variation in apparent strength with the height of the tested brick expressed as a function of the joint thickness to the latter for each of rubber, fibres and polythene. It can be said, as the graphs show, that the compressive strength varies approximately and invesely in a linear relationship form with the ratio of the mortar joint thickness to the brick height. This complies well with the theoretically derived formula (Equation 3.14 Chapter 3), the experimental results of Bradshaw and Hendry(12) and of Lenczner(87). All of these experimental results are shown at the top of Figure 6.24.

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(12): Bradshaw, R.E. and Hendry, A.W. The influence of mortar joint thickness on the strength of brickwork. Research Rep. REB/4. Structural Ceramics Research Unit. Dept. of Civil Engineering. University of Edinburgh, July, 1966.
(87): Lenczner, D. Strength and elastic properties of 9-inches brickwork cube. Transactions of the Brit. Ceramic Society. June, 1966. p.p. 363-382.

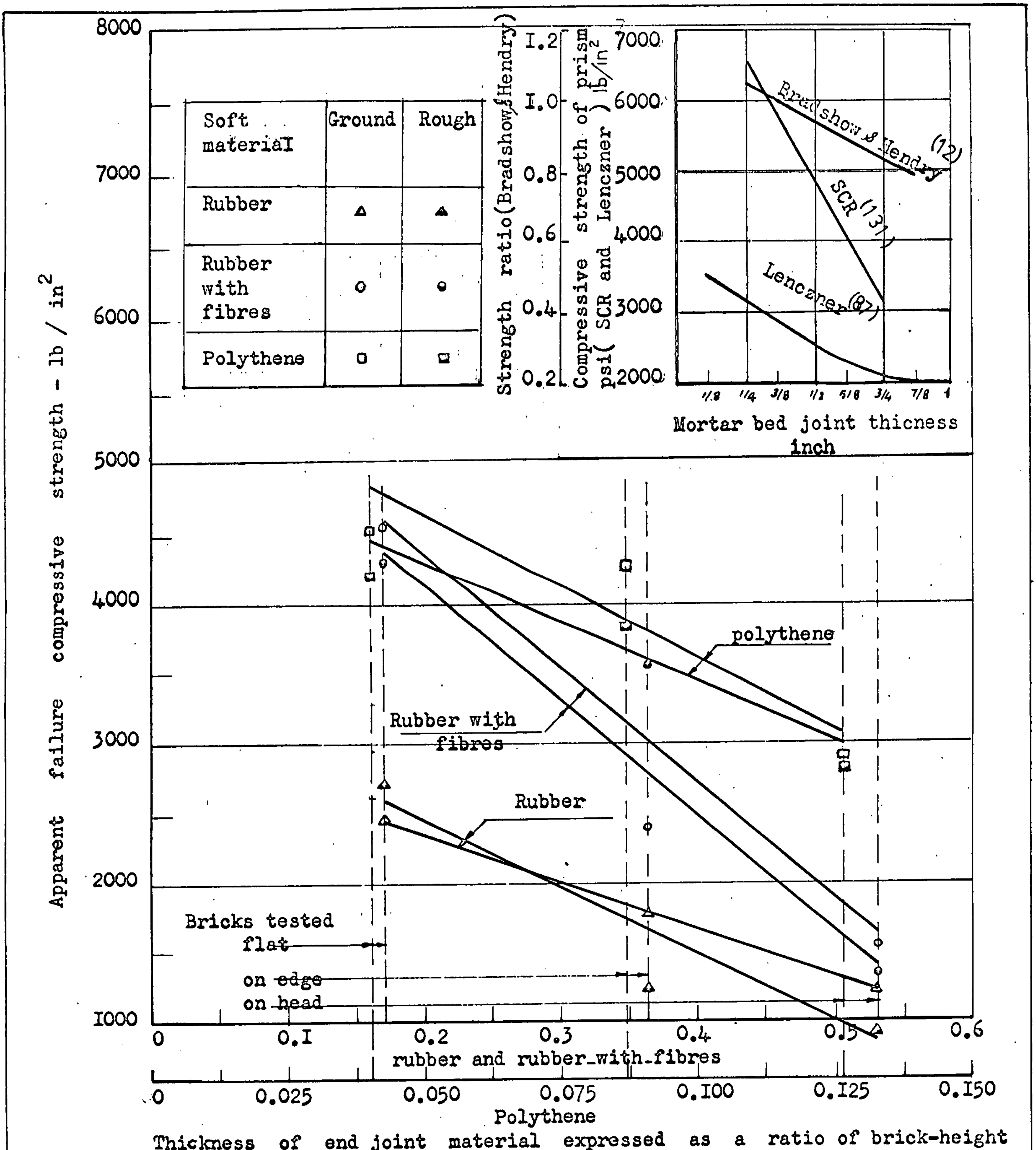


Figure 6.24:

Reduction in apparent failure compressive strength of bricks due to increase in thickness of soft joint material at ends.

6.4.7. Statistical Discussion and Scatter of Results For The Apparent Failure Compressive Strength

From the five representative and statistical functions mentioned before (page 6.15), only the coefficient of variation was chosen as the field of comparison. In fact, it can be considered the easiest of the statistical functions to use in comparing dispersions in a case similar to the present one, in which the range of difference between the materials is very wide. Very briefly, the coefficient of variation measures the spread of the data about the mean value, and is expressed as a percentage of the latter.

From Table 6.12 it can be seen that the coefficient of variation ranges from 2.20 to 31.20. But when dividing the results into groups, the 108 values can be divided as follows:

20 values of the coefficient ranges between	0	and	5,
47 " " " " "	"	5	and	10,
32 " " " " "	"	10	and	15,
6 " " " " "	"	15	and	20,
2 " " " " "	"	20	and	25,
none of the values "	"	25	and	30,
and				
1 value ....ranges	.....	30	and	35.

Summing up it is found that 99 values out of 108 values of the coefficient of variation are below 15, and 67 are below 10. Also out of those nine values above 15, 6 values are below 20.

Table 6.12

Statistical comparison between test results expressed as a function of the coefficient of variation of apparent failure compressive strength

Brick	Joint Material	Direction of testing						Average "V" for each joint material
		Flatwise		On edge		On head		
		=	≠	≡	≢	|||	⧻	
Ground	Steel	10.85	16.76	13.02	10.42	31.20	12.60	15.81
	Plywood	10.14	2.51	7.92	12.51	11.70	11.50	9.38
	Hard board	9.49	3.62	9.00	5.05	14.60	12.10	8.98
	Hard board	3.13	5.23	5.90	10.07	6.30	14.20	7.47
	Polythene	9.07	4.71	3.63	6.80	13.60	19.90	9.61
	Rubber with fibres	7.00	4.76	4.19	20.57	9.57	15.40	10.25
	Pure rubber	8.28	8.80	9.13	11.11	6.90	14.90	9.85
	No joint material	13.77	5.19	15.28	4.35	13.40	11.50	10.58
	No joint material and super grinding	4.27•	5.73•	6.44	3.47	8.73•	21.48•	8.35
	Brick joints	2.91•	11.06•	15.07•	3.42°	2.84•	6.58•	
	Average "V" for each sets of specimens	7.89	6.84	8.96	8.78	11.88	14.02	
Rough	Steel	8.50	7.25	6.53	11.97	6.10	13.80	9.02
	Plywood	5.93	8.50	9.81	6.14	13.90	6.90	8.53
	Hard board	11.80	8.08	5.90	8.80	11.50	11.80	9.65
	Hard board	3.69	5.63	9.50	2.20	5.10	5.90	5.34
	Polythene	4.00	16.70	4.19	16.10	10.50	2.70	9.03
	Rubber with fibres	13.00	6.97	11.40	12.65	6.41	10.14	10.09
	Pure rubber	6.43	7.11	8.16	5.64	7.10	8.90	7.22
	No joint material	10.99	4.51	6.88	2.36	10.20	5.90	6.81
	Average "V" for each sets of specimens	8.04	8.09	7.80	8.23	8.85	8.25	

+ V = coefficient of variation

• All values of "V" are for six specimens, except those indicated are for four specimens

In fact, if only one value for specimens with ground bricks and steel joints are tested on end with end bricks is excluded, the coefficient of variation for this set (quoted as 31.20 in Table 6.12) drops down to 6.33. Consequently, it can be said that the values of the coefficient of variation range between 2.20 and 21.48 and the number of values above 20 becomes two out of 108.

This indicates strongly that the bricks and joint materials used in the present experiments are more uniform than was expected. Clearly in this respect special reference can be made to the bricks.

From this it can be concluded that workmanship might be a key factor towards producing masonry of better uniformity. If the part of heterogeneity resulting from discontinuity (pointed out later in 6.5.6-3) during bricklaying is kept to a minimum, then brick masonry and its components can be considered, each individually, of less non-uniformity than is ordinarily assumed. But this is, of course, only valid for bricks similar to the type sampled for the present work.

6.4.8. Comment on the the Standard Compression Test Incorporated in B.S.: 1257

It is of importance before coming to any conclusion to know something about the basic work behind the choice of the present standard test and its scope

Davey and Thomas[36] stated that a comparative study of test procedures had been carried out at the Building Research Station, using different samples of bricks and mortars. For bricks without frogs, the methods comprised bricks rendered and embedded in mortars of various strengths, and between plywood. The results of the tests due to the different end conditions are summarized as quoted by them, in Figure 6.25.

They stated that it had been concluded from the study that bricks embedded in mortars having strengths of 2000, 4000, 6000 and 11,000 lb/in.^{2} had shown no significant advantage over the much simpler method in which plywood was used with no application of mortar. They concluded that the test incorporated in B.S.: 1257 for the case of bricks without frogs, require them to be tested between plywood without any application of mortar.

Referring to Figure 6.25 the author feels that the conclusion was based more on the simplicity of the test incorporated in the specifications, than on a comparison penetrating deeply into the actual failure characteristics. It could conceivably be accepted if the test was used when the mortar has a strength between 6,000 and 11,000 lb/in^2 with bricks of strength between 8,000 and 15,000 lb/in^2 and more, or if the mortar strength is near the brick strength and the latter is less than 8,000 lb/in^2. In fact it is misleading to apply the method when the mortar strength is between 2,000 and 11,000 lb/in^2, and the bricks are of high grade. A particular example is that the strength of a brick according to the standard test is 18,000, while its strength with mortars has the values 11630, 12,440,

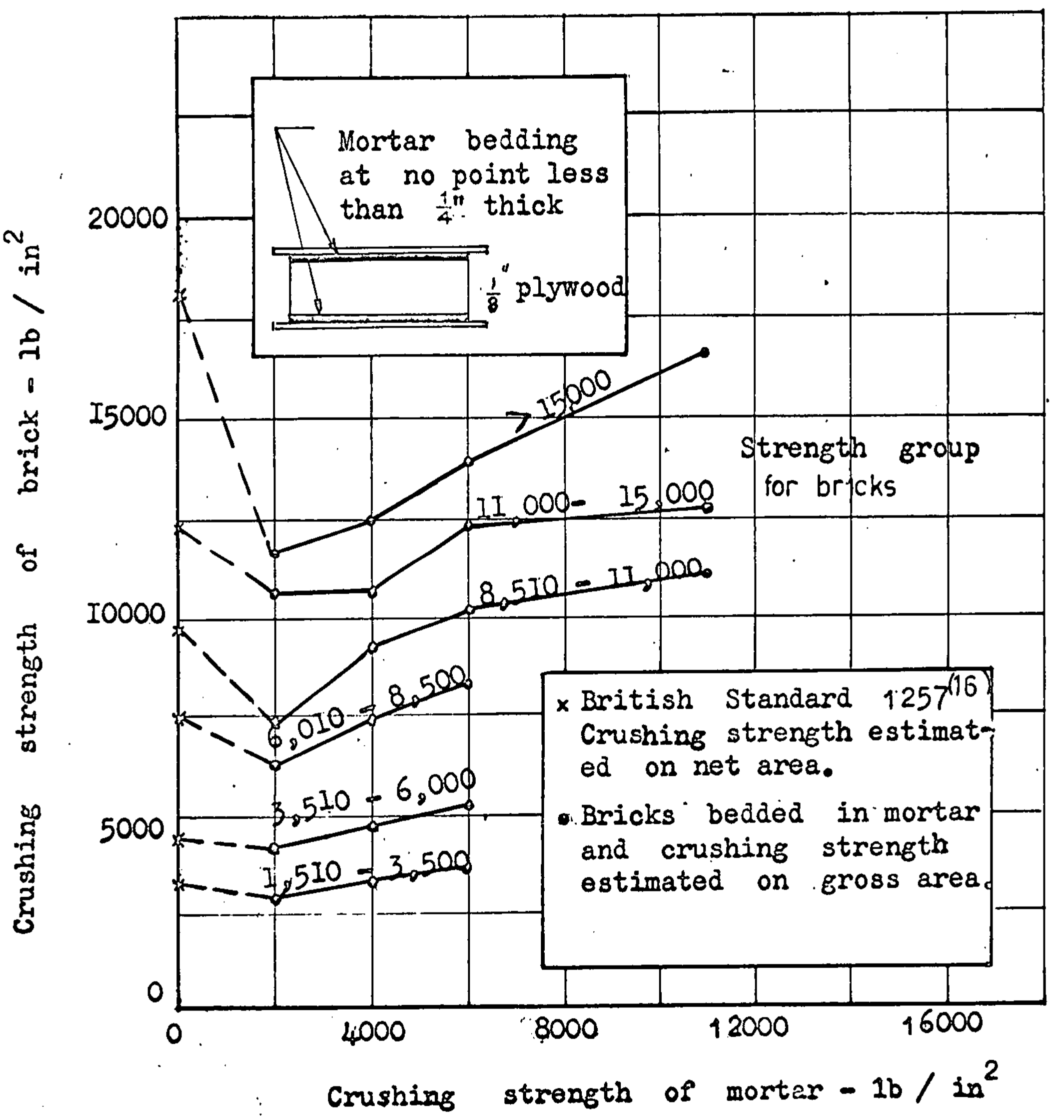


Figure 6.25:

Basic comparative studies of the standard compression test of bricks without frogs, B.S.S. 1257. [After Davey and Thomas (36)]

13860 lb/in^2. The discrepancy becomes more pronounced if we remember that the strength of a masonry assemblage decreases, within a certain limit, with the number of bed joints or courses.

Therefore, it can be emphasized that the ultimate compressive strength of a brick is at present usually assessed under standard testing conditions which are not well defined. This is as regards both its performance in masonry and its compressive strength as an absolute property.

6.5 GENERAL CONCLUSIONS AND PROPOSALS

6.5.1. General

On the basis of the previous discussions, which included the present results and some from previous investigators, the conclusions and proposals can be grouped under the following main headings:

1. Influence of rigidity of the end joint material on the failure characteristics of a brick under compression.

2. Influence of testing technique.

3. Proposed testing technique for assessing the strength of brick masonry under compression.

4. Effect of various sizes of bricks with end joint materials.

5. Influence of mode of bricklaying on the deformation properties and ultimate failure strength of brick masonry.

6.5.2. Influence of Rigidity of the End Joint Material on the Failure Characteristics of a Brick under Compression

1. In order to provide a uniform stress distribution on a brick in a compression test, and to eliminate either the restraint or the outward lateral stresses at the ends, the joint material used to transmit the load must be well chosen. In the choice, the principal considerations are whether the material is likely to be harder than the brick and prevent its lateral deformation, or softer and squeeze out increasing its deformation laterally.

2. In these circumstances the influence of the material between the brick ends and the machine platens is a function of the deformation properties, Poisson's ratio and modulus of

elasticity of the material in relation to the corresponding properties of the brick. The relative lateral deformations of the brick and the material determine ultimately the apparent failure strength and the mode of failure. This is true, if the effect of the platens of the loading machine is ignored, which with the end joint material produces another relative movement.

3. Considering the usual hard machine platen, it can be generally said that there are certain joint materials which can be designated soft. These expand laterally a greater amount than a brick and appear to increase the lateral expansion of brick. There are certain other joint materials which can be designated hard. These expand laterally a smaller amount than a brick, preventing to some extent the lateral expansion of the brick.

4. Whether the end joint material is soft or hard its influence can have a relation to its thickness, Posson's ratio, modulus of elasticity, and its coefficient of friction with the machine platen. The influence is much more pronounced in the case of soft material.

5. Theoretically, there must be combinations between these factors to produce a uniform state of uniaxial stress at the brick faces subjected to compression. But it is clear that it is extremely complex to develop such a theory. The simplest reason for this is that it is unlikely that the presupposed simultaneous occurrence of so many factors in the theory will happen in practice. This can be felt more if we

take into consideration the heterogeneity of brick itself as a material, and the great variation in the degree of surface roughness that may be expected.

6. Consequently from '5' above, it can be said that generally a brick is subject to restraining lateral forces from a hard joint material or outward lateral forces from a soft joint material at its load faces. The modes of failure produced are generally shear failure in the former and splitting in the latter case.

7. As regards the profiles, showing the distribution of stresses in both cases for both directions, they cannot be rigorously defined or stated, at least at this stage. But as the disdussion has pointed out, the squeezing out of soft layers at the brick faces generally produces vertical concentration of pressure and transverse lateral forces. The maximum value of the former is at the centre of the contact surfaces. The maximum value of the latter is unlikely to occur at the contact surface or the middle plane. It is most likely to happen at a small distance from the contact surface. In a similar manner, but in opposite directions, the case of a hard layer can be treated. Reference should be made to the work quoted before of Coker and Filon, Hast, Hiltscher and Florin, and Newman.

6.5.3. Influence of Testing Technique

1. As the results have shown, there is a very wide range for the value of the apparent failure compressive strength.

It is also possible to obtain virtually any of these values by varying the joint material inserted at the brick faces. With the assumption that has been accepted for a long time now, that the performance of a brick in masonry must be assessed by a compression test between two layers of another material, the very wide range means that the choice of this material is critical.

2. According to the B.S.:1257, the compressive strength of a brick is assessed by applying compression through two sheets of plywood on the brick faces. As has been strongly pointed out, plywood produces with a brick tested flat or on edge, a considerable degree of lateral restraint, with the result of a higher apparent failure strength. This restraint is quite significant, for it puts plywood in the same category as steel, at least within the limits of the present tests.

3. From this short comparison in '2', the ideal solution arises again. As was pointed out before, the ideal had been given by Coker and Filon, and was confirmed by the present tests. Practically, however, the ideal solution, with reference to be made to 6.4.3.5, is not possible especially if we remember that the compression assessment cannot be done by one or two specimens in the one series of tests. Besides, for each specimen three bricks are required super ground.

4. As the ideal solution is not practical, and taking into account the results obtained, the author suggests that the

actual compressive strength of a brick might be better assessed by using thin sheets of rubber-with-fibres, polythene, or hard board (with rough surfaces towards the machine platens) as transmitting load layers between the machine platens and the brick loaded faces and by testing the brick on edge or on end. Naturally the thickness of each material would need to be determined from many tests on full scale bricks. At the same time investigating the influence of heterogeneity of brick itself is required.

5. Again the suggestion given in '4' applies only if the compressive strength is required to be assessed. As has been proved theoretically in Chapters 2 and 3 and from the general analysis in Chapter 1 it is not the compressive strength of bricks which has the greatest influence from the brick side. It has also been shown, from the present results that when the joint material is softer than brick, the latter's tensile strength is of an utmost influence, especially with specimens tested flatwise. This is the best or nearest simulating case to brick masonry, from the side of their relative rigidities or stiffnesses.

7. There is no doubt that the accuracy of the testing machine is important. The minimum pronounced effects were pointed out in the discussion and previously in Chapter 5. Reference can be made to Cole[32,33], Sigvaldason[117], and Newman[99].

6.5.4. Proposed Testing Technique for Assessing the Strength of Brick Masonry under Compression

It is quite clear that it was

necessary to approach the problem from the angle of compressive strength, and the compression test. But it has been shown that when the compression is applied through soft joint layers at the brick loading faces, the scope of the test is completely changed, for either bricks or mortar being loaded through each other. In a masonry assemblage subject to an external pressure, the load is transmitted internally into the bricks mostly by lateral tensile forces before ultimate failure is reached. Thus it can be said that the standard compression test, even if it is carried out in the ideal way, does not provide a means of assessing structural performance when it is used with mortar, which is usually known to be softer than bricks. In fact the change in scope of the test internally in masonry assemblages is a main reason for the disagreement mentioned before in the orientation of this chapter.

Thus, it can be emphasized again that the right criterion by which to judge the success or failure of a brick in a test for use in masonry with a relatively softer mortar is the tensile strength.

At this point, and especially after these experimental results, the author has a strong feeling that it is time for the specification writers to think about changing the method of testing masonry bricks as regards the strength property. For agreement to replace the present disagreement between the strength properties of masonry assemblages and bricks, the latters structural performance would be much better assessed or judged by one of the following proposals for a standard load test:

1. A new compression test: This is by testing the brick flatwise with the insertion of softer sheets or layers which have the effect of producing conditions similar to those produced by rubber-with-fibres, polythene or similar material. The more resemblance between this material and the mortars the closer is the agreement obtained. Undoubtedly, this soft material can be standardized after sufficient investigations have been carried out. These investigations must cover a wide range of different soft materials, thicknesses, and grades of bricks. Reference should be made for a comparison between Photographs 6.6,12,13 and 6.14,15,18.

2. A standard tension test: Due to the fact that bricks are not manufactured in the laboratory, it may appear difficult to develop a testing technique, quite apart from the drawbacks discussed in Chapter 5. The author must refer here to his new proposed technique, and its extension for brick testing, as discussed in detail in Chapter 5. What can be added here, is that it may be useful to standardize that test so that a packing material is to be inserted between the specimen and the loading platens. This material will help to avoid the stress concentration discussed in 6.4.5(4).

It is of importance here to recall that it may not be possible to achieve one single material which would produce a completely perfect condition for assessing the compressive or tensile strength for all grades of bricks with all types of mortars. Undoubtedly there will be some disagreement between the failure characteristics of a brick under such a standard test and in a wall, but the disagreement will be only in the strength characteristics and within such narrow limits, that no comparison can be made with the present discrepancies. Reference must be made to the similarity between the increase in strength when testing brickwork on edge using cement-sand mortar and rubber-with-fibres. However this will be studied in more details in Chapter 8.

Taking into consideration usual conditions on site, there is no doubt that the first suggestion will be more suitable, but for research the author suggests that both methods must be used for correlations, until the point is clarified and an established agreement is achieved.

6.5.5. Effect of Various Sizes of Bricks with Loading Joint Material

As has already been stated, similar apparent compressive strengths may be obtained with certain combinations of joint material at the brick loading faces, and the way of testing the bricks. The latter, in fact, expresses the relative dimensions or areas with respect to the direction of applying compression and the perpendicular to it.

However, it must be emphasized that such similarity is purely fortuitous, would not be apparent, unless a wide spread in the rigidities of joint materials in the three directions of loading had been chosen.

Although the final apparent failure strengths may be identical, the stress system initiating failure will vary for different cases. Possible cases are, a uniform uniaxial compression, or a complex state of biaxial or triaxial stress. The lateral stresses in the latter two cases can be acting either inward or outward. As was discussed it is virtually impossible to visualise the sequence of the failure mechanism, and it is certainly difficult to define precisely and completely the states of stress from the beginning up to failure, but ultimately the failure characteristics problem is a matter of the combination of different thicknesses and ways of testing the bricks. As mentioned above the latter expresses the relative dimensions of the brick in the direction of loading.

From this point keeping in mind the results obtained by the Building Research Station (Figure 6.23) it can be said that the need is urgent for making similar tests on full size and scale model bricks of different grades. But for comparisons to be reliable, bricks of different sizes must be manufactured from the same plastic batch, and treated and dried in the same conditions. Undoubtedly, it would be of great value if bricks of different relative dimensions were used. It is now certain that the relative dimensions in the direction of loading, between the brick and the horizontal joint material, is of significant effect. It is to be remembered that the thickness of the layers used in the present tests was constant in most cases. Reference is to be made here as to the effect of thickness of the bed joints.

Also in connection with the effect of the brick height on the apparent failure strength, the author feels that the deformation profiles due to different joint materials for the three possible methods of laying, and for all stages of loading, need to be in estigated. This can be a step forward towards a more detailed analysis. Special reference to be made here to Hoister and Florin [64, 65], Newman [99], Hast [58]. Although the author appreciates that the above required work is extremely laborious and expensive, he feels that eventually this must be carried out.

6.5.6 Influence of Mode of Bricklaying on the Deformation Properties and Ultimate Failure Strength of Brick Masonry

1. At this stage, it can be stated forcefully that in studying the behaviour of brick masonry, complete heterogeneity must be assumed. The difference in nature between bricks and mortar and the lack of continuity of one relative to the other in a masonry assemblage are the main factors which affect deformations and accelerate failure by splitting.

2. One of the main results of the present tests is that the author has considered a possible modification of the internal structural system of brick masonry. With the conventional and available materials, modifications in brick-laying with the object of producing conditions counter-

balancing the heterogeneity effects in the assemblage as a whole can contribute significantly towards an increase in strength.

3. Before describing this modification, there is an interesting observation worth mentioning. This observation was made during bricklaying with the 1/6th scale model bricks, concerning slight bleeding and instability of the actual mortars. Heavy tapping during brick-laying and the possibility of sedimentation of cement particles during the first few hours after brick-laying can cause bleeding of the water in the mortar mix. This bleeding seems to take place directly under the bricks, leading to the formation of water pockets in the form of very thin layers on top of the bed joints and parallel to the bottom surface of the bricks. These water layers become air layers on drying out. This can result in deformation characteristics in the direction parallel to brick-laying, which are likely to be different along consecutive lengths of the height. This explains the lack of continuity mentioned above. In the same manner sedimentation in the vertical joint causes discontinuity in the horizontal direction, especially along the horizontal planes passing the upper parts of the vertical joints.

This conclusion was emphasized later during some subsidiary tests. Some of the masonry assemblages with cement-sand mortar collapsed 24 hours after building was finished. The author noticed that the wallettes were scattered into units. Each unit consisted of a brick and the bed mortar joint stuck on its upper face. Not one brick was stuck to the mortar joint on its lower face.

4. Ultimately, the author suggests in the light of all mentioned above, that modification of the internal structure may be achieved by laying bricks on edge. This leads to a noticeable reduction in the number of bed joints. As shown in "3" above, the smaller the number of horizontal joints the smaller the number of points of weakness contributing to heterogeity in the vertical direction. The benefit of greatest importance is, of course, to allow the lateral movement of the relatively softer mortar to be resisted by a vertical cross-section of bigger area. The same thing would also be achieved if the units used were of greater height than conventional bricks.

5. With prefabricated panels laid flat and grouted, it might be feasible to modify the internal structure by laying the bricks on end. Such a solution is undoubtedly impracticable with on site bricklaying, because mortar might not adhere to the bricks in the long vertical joints, and uniformity would

be difficult to achieve.

6. The author feels that the methods of bricklaying and bonding in the conventional dimensions used today are followed generally from force of habit, with little consideration of the case of laying bricks flatwise. By now the habit has hardened into a strong prejudice. The author strongly feels that the conventional brick height and the common way of laying bricks flatwise may be the suitable one only if mortars are generally stronger than the bricks. Unfortunately, masonry bricks are in general stronger than mortar.

7. As a long term future research topic the author suggests here that these conventional methods of bricklaying which originated empirically and have been perpetuated by habit must be reassessed. An investigation on a wide scale should be carried out to study all possible methods of bonding, combined with the two possible practical methods of bricklaying already described. Conventional masonry mortars would be used in the proposed investigation.

6.6. SUPPLEMENTARY TESTS ON SMALL PIERS AND WALLS

6.6.1 General

Before coming to any definite conclusions (Chapter 9) it was thought better to carry out some supplementary tests The object of these tests was to examine some of the main conclusions as applied to brick masonry, such as new proposed directions of bricklaying and the new proposed standard tests. The tests were few because of one main reason, that is the smaller number of bricks available. However, these tests can be divided into four groups as follows:

1. Studying the influence of rubber with fibres and plywood on the failure characteristics of one third scale model bricks.
2. Testing piers and couplets with the bed joint in a soft material like rubber-with-fibres.
3. Testing small walls with cement-sand mortar using 1/6 and 1/3 scale model bricks.

6.6.2. Variation in Failure Compressive Strength of One-third-scale Model Bricks

6.6.2.1. Materials, assembling, and testing

The nature of specimens and testing in this group is exactly the same as before, except that the bricks were super-ground. The joint material was rubber-with fibres and from the same sheets as used before. The dimensions and cross-sectional areas are given in Appendix 6.1.

6.6.2.2. Test results and discussion

The test results of this group are

given in Appendix 6.2 As can be seen from the table, some of the results comply with the previous results, some do not comply, and special reference is to be made to rubber with fibres and bricks tested on end. It was clear to the author, from the cross sections revealed on cutting the bricks and from the modes of failure, that the 1/3 bricks are different from the 1/6 bricks. They had a laminated internal structure which is most probably due to the method of extrusion. This resulted in a high degree of crushing.

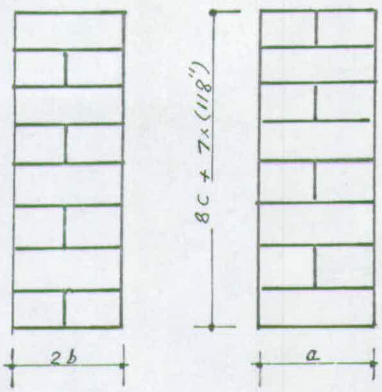
6.6.3 Piers and Walls With Rubber-with-Fibres in Bed Joints

6.6.3.1 Materials, assembling, and testing

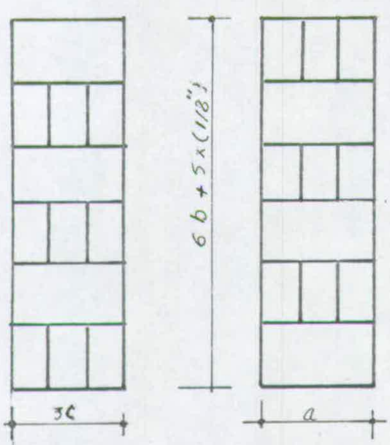
Nine small piers and six wallettes were assembled from the rough 1/6 scale model bricks. Both of these numbers were equally divided among the three systems of brick laying, flatwise, on edge, and on end. The horizontal joints were from the same rubber-with-fibres as used before. The vertical joints were empty. The specimens were assembled in the same manner as before, by using sellotape to keep them in place. Schematic sketches are shown in Figures 6.26, 27, and specimens after testing in Photographs 6.12,13. Testing was done as before by axial compression.

6.6.3.2. Test results and discussion

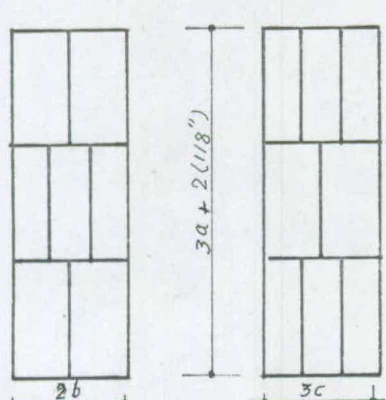
It may be interesting to begin with the mode of failure. With the piers with bricks laid flat and on-edge and the walls with bricks laid flat, the mode of failure is absolutely the same as the common failure obtained from previous studies on masonry. Vertical splitting was common to all these specimens without any exception.



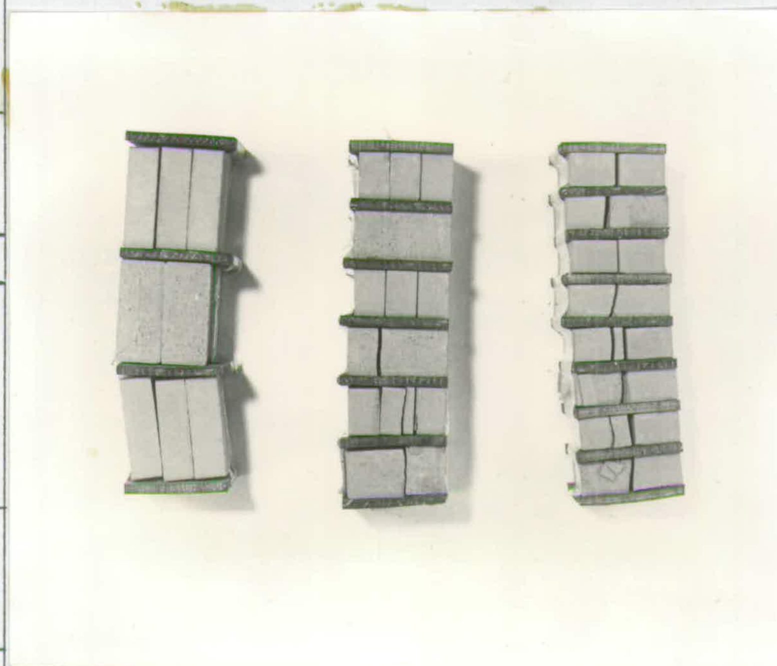
		Failure strength lb / in^2
		1010.0
		1040.0
		1193.0
Flatwise	Average	1081.2



		Failure strength lb / in^2
		1452.0
		1370.0
		1300.0
On edge	Average	1374.0



		Failure strength lb / in^2
		1080.0
		1210.2
		1080.0
On head	Average	1124.0

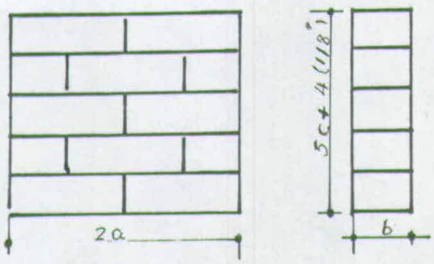


Photograph 6.12:

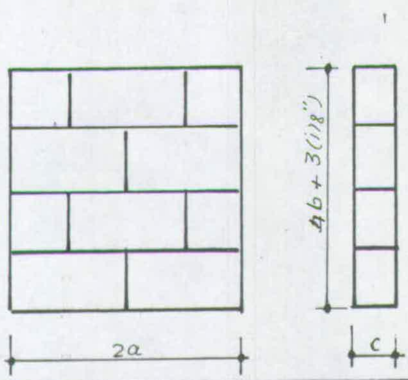
Typical failures of piers with rubber-with-fibres in bed joints.
The photograph illustrates complete similarity between modes of failure obtained from rubber-with-fibres and the common mode of failure of brick masonry in two cases, flatwise and on-edge.
In the third case distorsion occurred failure of any of the bricks.

Figure 6.26:

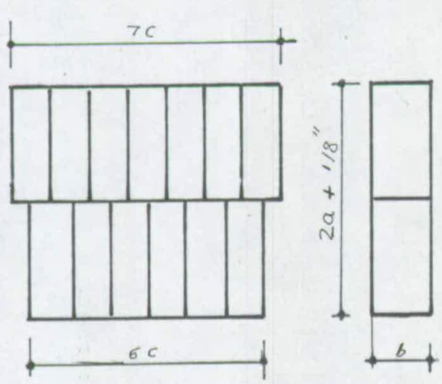
Schematic sketches of piers tested with rubber-with-fibres in bed joints.



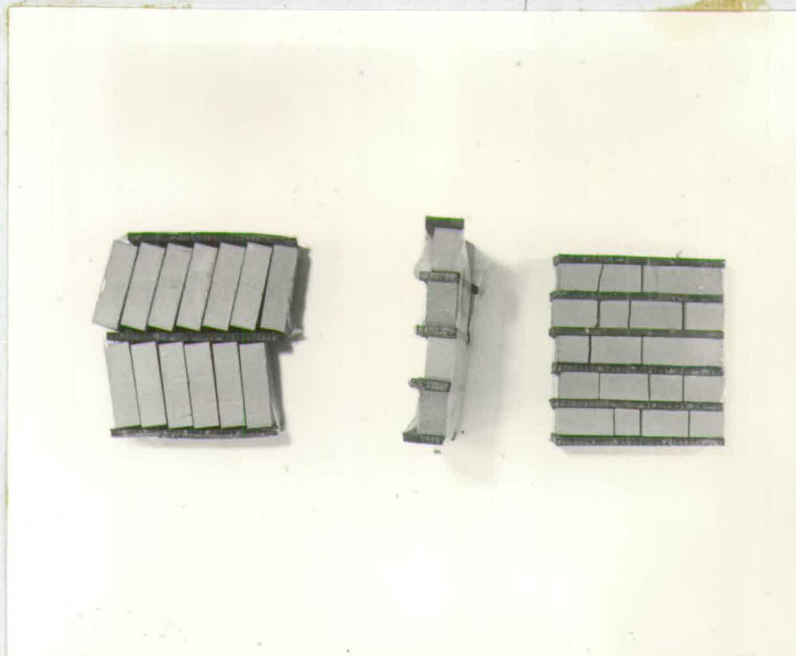
Bricks laid flat-wise



Bricks laid on edge



Bricks laid on end



Photograph 6.13:

Wallettes
Typical failures of wallettes with rubber-with rubber-with-fibres in bed joints.

The photograph illustrates complete similarity between modes of failure obtained from rubber-with-fibre and the common mode of failure of brick masonry, only in the case of bricks laid flat.

In the other two cases distorsion occured before failure of any of the bricks

Figure 6.27:

Schematic sketches of wallettes tested with rubber-with-fibres in bed joints.

The first indication of failure was heard very shortly before ultimate failure in the form of the sharp reports mentioned before. The reports were repeated and in no case were less than five or six. These repeated reports together with the mode of failure clearly imply that cracking or splitting started in one of the courses and then extended to other sources and finally become confluent and extended to the entire length of the specimen. The photographs clearly show that no crushing took place in any of the specimens.

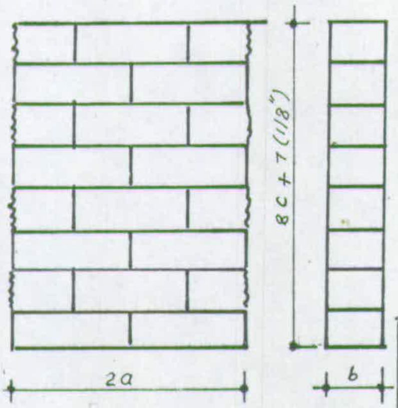
As regards the piers with bricks on end and wallettes with bricks on edge and on end, they actually did not fail. Only distortion took place. Although distortion can be considered a superficial failure, it occurred almost always at a higher load than the failure load per unit area of other specimens.

6.6.4 Small Wallettes With Cement-sand-mortar.

6.6.4.1. Materials, manufacturing and testing

Eight wallettes were made using one-sixth-scale, and one-third scale model bricks. The former were divided among the three methods of laying, but two systems with bricks on-end were used, divided between the systems. Figures 6.28 - 30 show the details of both respectively.

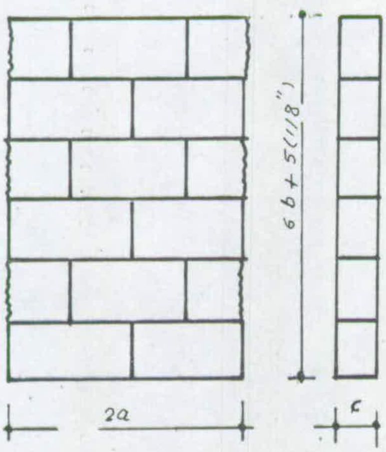
As regards manufacturing, the wallettes were built in the ordinary way after the bricks had been soaked for 24 hours in water. The wallettes with the one-third-scale bricks were tested after eight days and those with the one-sixth-scale bricks were tested after eleven days. Photographs 6.14 - 6.19 show specimens after testing.



Failure strength lb / in^2
1412.8
1700.0

Flatwise	Average	1556.4

Photograph 6.14: Flat

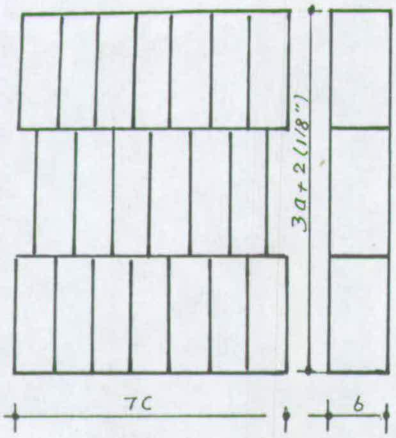


Failure strength lb / in^2
1870.9
1870.9

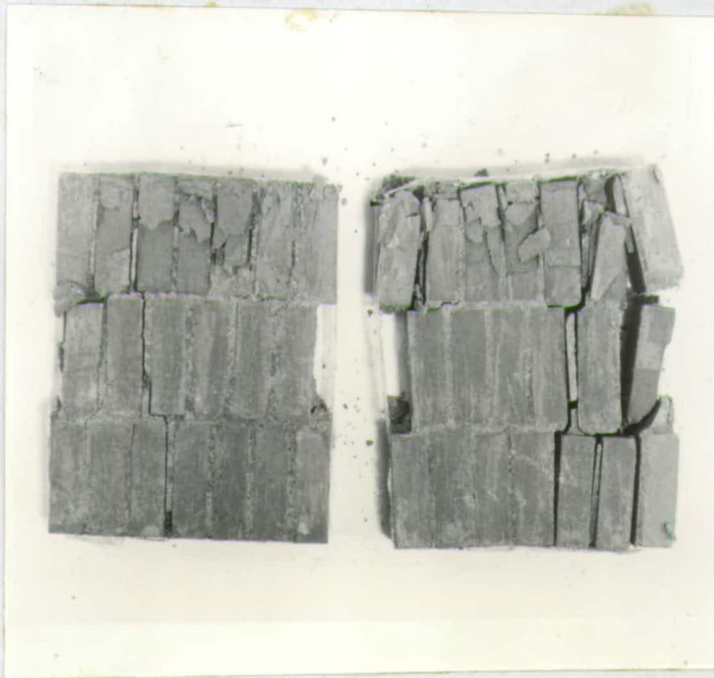
On edge	Average	1870.9

Photograph 6.15: On edge

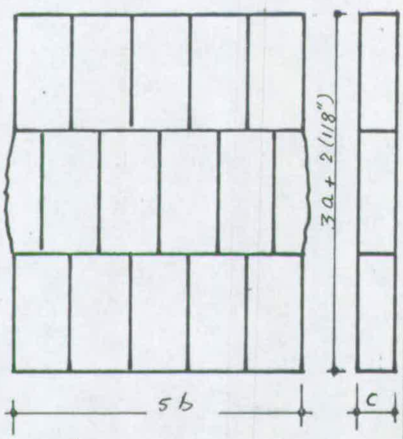
Figure 6.28:
Schematic sketches and failure strengths of small wallettes with cement-sand mortar (1:3) and 1/6 scale model bricks.



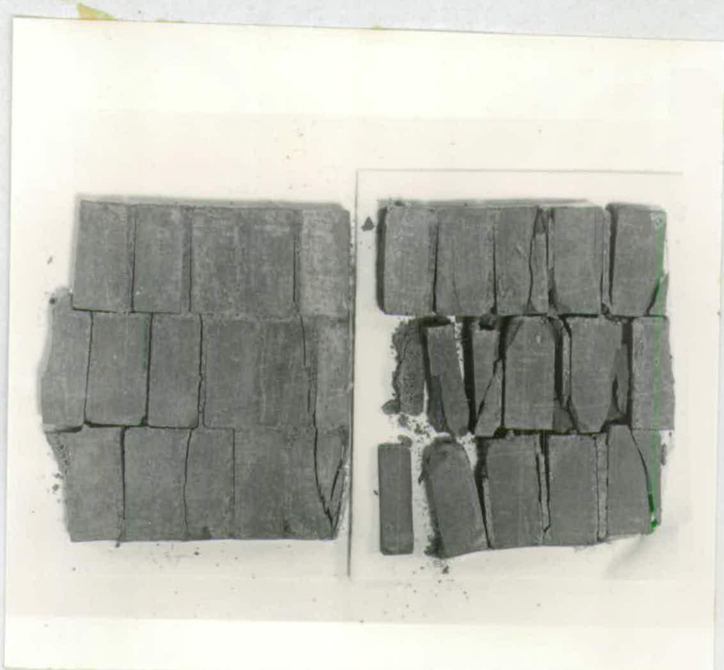
Flatwise	Failure strength lb /in^2
	1854.0
	1572.2
Average	1713.0



Photograph 6.16: On end



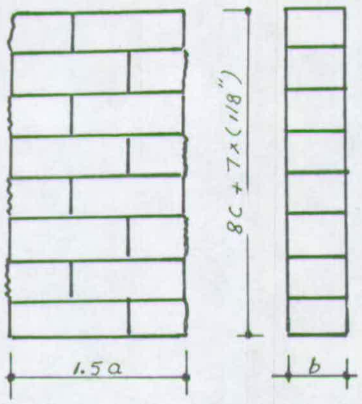
On end	Failure strength lb / in^2
	2107.0
	2080.0
Average	2093.5



Photograph 6.17: On end

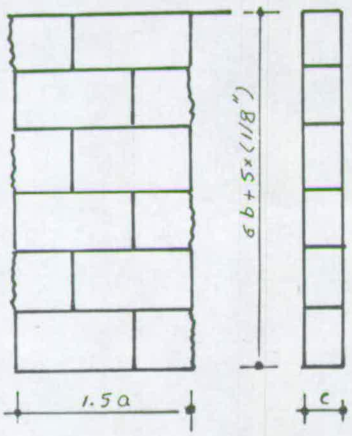
Figure 6.29:

Schematic sketches and failure strengths of small wallettes with cement-sand mortar (1:3) and 1/6 scale model bricks.



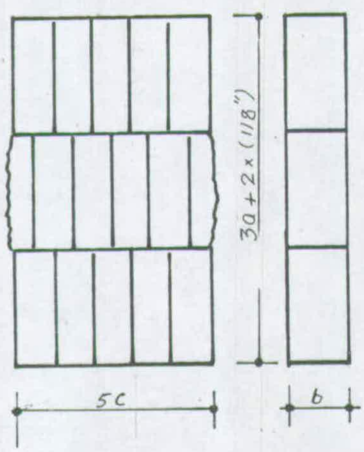
Failure strength lb / in^2
695.0
849.6
Average 772.3

Flatwise



Failure strength lb / in^2
1000.0
950.0
Average 975.1

On edge



Failure strength lb / in^2
1038.0
1089.0
Average 1063.0

On end

Photograph 6.18: On edge

Photograph 6.19: On end

Figure 6.30:
Schematic sketches and failure strengthes of small wallettes with cement-sand mortar (1:3) and 1/3 scale model bricks.

6.6.4.2. Discussion of test results

Comparing the different directions of brick-laying it was found that with on-edge bricklaying the failure strength increased to about 20.21% and 26.20% higher than those with flatwise bricklaying. The two values are for 1/6 and 1/3 scale bricks respectively. With on-end brick-laying these two figures both increase respectively to about 37.71%.

As regards the mode of failure it was the common splitting failure with flatwise and on-edge brick-laying but with on-end bricklaying, there was no consistent mode of failure. Shear, bond, splitting, and distortion failures or combinations of more than one sometimes took place in the one wall.

6.6.5. Conclusions from Supplementary Tests

While keeping in mind that the tests carried out as supplementary ones are very few, the following conclusions could be stated:

1. The results give some support, to the ideas revealed from the main tests reported in this chapter. The support is more pronounced in the increase in strength, which was up to about 40% higher than the strengths obtained from the common way of bricklaying.

2. Another important confirmation was the complete similarity between the modes of failures of wallettes and piers when using rubber-with-fibres, and the common failure splitting of masonry. This again supports the idea of testing individual bricks in compression between a soft material.

3. Rubber-with-fibres seems to be a suitable soft material. With on-edge bricklaying the increase in strength over that of flatwise laying is 21.30% and 20.21 - 26.20% for rubber-with-fibres and cement-sand mortar respectively.

4. The tests encourage the carrying out of further research on the lines proposed.

CHAPTER 7

EXPERIMENTAL EXAMINATION OF THE PROPOSED NEW METHOD FOR THE DETERMINATION OF STRESS-DEFORMATION RELATIONS IN MORTARS

7.1 ORIENTATION

In Chapter 5 the theoretical aspects of a proposed method for measuring the deformation relations (modulus of elasticity and Poisson's ratio) in mortars were discussed. It was concluded that acceptance of the method was subject to an experimental verification.

The present chapter deals mainly with this verification. The object of the tests was not so much to provide information as to examine experimentally, the possibility of using the technique, determining the limits between the plane-stress and plane-deformation cases, and to assess the deviations in the experimental tests when using a nearly isotropic material. With this principle established, a reliable measure of mortar properties can be obtained by using an identical technique of measurement.

Some of the experiments were designed to test the proposed dimensions of specimens as regards their compliance with both cases of stress. The others were designed to examine the similarity between a circular specimen and a square one as regards the strains at the centre.

7.2 EXPERIMENTAL WORK

7.2.1. Materials and Specimens

7.2.1.1. Materials

In the choice of materials for the

work described in this chapter, it was considered desirable to use one of as near ideal elasticity as possible. Mortar, like concrete, was not expected to fall into this category, but to a greater or a lesser extent, to exhibit properties of heterogeneity and anisotropy. As previously mentioned, it might manifest a non-linear stress-strain relationship when subjected to changes of load. Therefore, steel was chosen as a suitable material for the present purpose.

At first, attempts were made to make the specimens from steel of known properties. It was not possible, however, for any of the firms or suppliers with whom contact was made to provide accurate data on their products. Therefore, it was thought best to measure these basic properties in the laboratory. As these were actually the main measurements in the present series of experiments by which the theoretical analysis would be examined, one solution seemed to be most suitable. That was to measure the properties using specimens cut from the same original block, all having the same relative dimensions as used by Hondros. With these dimensions (diameter/thickness ratio = 12) it was proved that the theory is valid, with all the assumptions incorporated in it, for the plane-stress case. The values obtained from these steel specimens (D_1, D_3, D_5) were termed firstly, as the measured actual values, and the values E and ν derived from them by calculation were termed the theoretical values. Those obtained from specimens of the proposed new dimensions were termed the experimental values. (Table 7.1).

The steel specimens were supplied by Messrs. Colvilles Ltd., Motherwell, and they were re-machined to be in the best possible condition. Then for greater accuracy, the central parts either of the faces or edges, were polished to a mirror finish before starting the work.

7.2.1.2. Specimens

Nine specimens, six circular, and three square blocks, were tested. The final dimensions after re-machining and polishing are shown in Table 7.1.

7.2.2. Strain Gauges, Circuit Apparatus, and Testing Machines

7.2.2.1. General

After a survey of the methods of strain measurement, having regard to the base measuring length for each specimen and the limited space which there would be around the specimen during testing, it was thought best to use electrical resistance strain gauges. Then, because the tensile strains of mortars, discussed later, was expected to be very low (starting from 1 x 10^{-5}), it became desirable to measure strains with an accuracy of at least 1 x 10^{-6}. When electrical strain gauges are used, this degree of accuracy calls for refined techniques. This is because the defects, which are negligible when dealing with the larger strains ordinarily encountered in metals, become significant. Keeping this in mind, the foil type of strain gauge was chosen to satisfy these requirements, and it was decided to use it throughout all the tests whether with steel or mortar. The advantages of this type of gauge over wire gauges are well known(59)(143).

Because all the tests involved the particular case of measuring along two perpendicular directions, the ideal choice would have been the interleaved 90^{o} rosette, but because the minimum linear gauge comprised in a rosette, available at the time of this series, was $\frac{1}{2}$" the use of rosettes was not possible. The need for a smaller gauge length arises from the fact. that the strains encountered in this test changes appreciably over very short distances, so that the basic gauge length is limited to a maximum value of 0.067 of the diameter. However, during mortar testing rosettes were available, as will be seen later.

7.2.2.2. Strain gauges

Two types of gauges were used, as illustrated in Table 7.1. Both gauges comprise a copper nickel foil of parallel ribbons connected in series forming a grid pattern and terminating in two connecting tags. The tag ends are enlarged to facilitate the connection of the external circuits and are connected to the ribbon elements by tapering sections to reduce fatigue failures at these points. The grid pattern and the tags are mounted on an epoxy backing suitable for use in a temperature up to 100^{o}C. For full details to the instruotion manuals(143).

One important point is worth mentioning here - that is, Saunder's-Roe manual claims that the gauge instability, either the zero drift under no load, or drift under constant load, are carefully considered in the manufacture of the gauges. This was not found to be absoluately perfect, as will be shown later in the preliminary tests.

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(59): Hendry, A.W. Elements of experimental stress analysis. Pergamon Press, 1964.

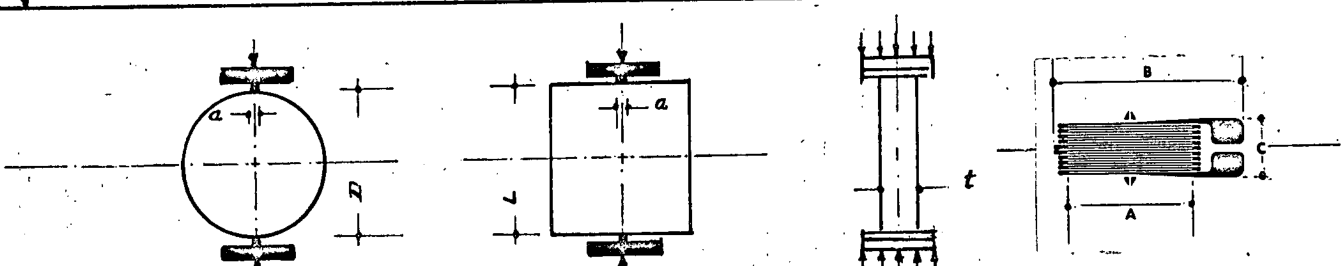
Table 7.1

Specimens, platens , and gauges.

Group	Object	Identification	Dimensions-inches	
			D or L	t
1 (Circular specimens)	Determination of the actual values of "E" and "ν" of steel.	D_1 (D/t = 11.7323)	5.750	0.4901
		D_3 (D/t = 11.6160)	4.004	0.3447
		D_5 (D/t = 11.1952)	2.782	0.2485
2 (Circular specimens)	Examinations of the proposed new dimensions.	D_2	5.750	0.9925
		D_4	4.000	0.9997
		D_6	2.780	0.9975
3 (Square specimens)	Verification of the similarity between circular and square specimens.	P_1	5.755	0.9880
		P_2	3.975	0.9937
		P_3	2.737	0.9988

↑ Details of specimens

↓ Details of gauges , and platens.



Schematic diagram of the experimental loading

Gauge

Specimen identification		D_1 , D_2 , P_1	D_3 , D_4 , P_2	D_5 , D_6 , P_3
Gauge dimension-inch	A	0.250	0.250	0.125
	B	0.400	0.400	0.208
	C	0.178	0.178	0.094
Width of loading rib - inch		0.500	0.330	0.230

7.2.2.3. Strain gauge measuring apparatus

The Strain Gauge Apparatus Type 1516, produced by Bruel and Kjaer-Denmark was used. It contains a 3 kc/s ascillator which feeds a Wheatstone bridge, consisting of two or four strain gauges. The strain measuring circuit was the full bridge. In the full bridge circuit two strain gauges were used, one acting as the active strain measuring element, and the second as the temperature compensating gauge. The other two bridge arms are formed by fixing resistors incorporated in the Balancing Unit Type 1531 (auxiliary equipment) used. A schematic diagram for the wiring circuit is shown in Figure 7.1.

With the apparatus the signed output voltage from the bridge is amplified in a four stage amplifier, the output of which, together with a portion of the oscillator voltage is fed to a phase sensitive rectifier circuit. The internal supply voltage is nominally a.c. 0.3, 1.0 or 3.0 volts, 3 kc/s. With each voltage there are four measuring ranges for either positive or negative strains, with an accuracy of 1.5% of full deflection. A centre-zero indicating meter connected to the output of the phase sensitive rectifier indicates the magnitude and sense of the bridge balance and hence the strain. The measurements were taken by employing the Balancing Unit Type 1531 produced by the same manufacturer, with the object of taking measurements from a number of strain positions. The unit has four independent sections, each with two built-in bridge arms and R and C balancing component.

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(143): Westland Aircraft Ltd. Saunders-Roi foil strain gauges: An introduction to foil strain gauge practice. Publication No. SP. 1191, January, 1966.

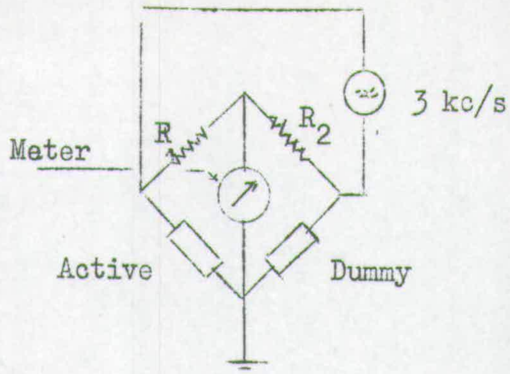
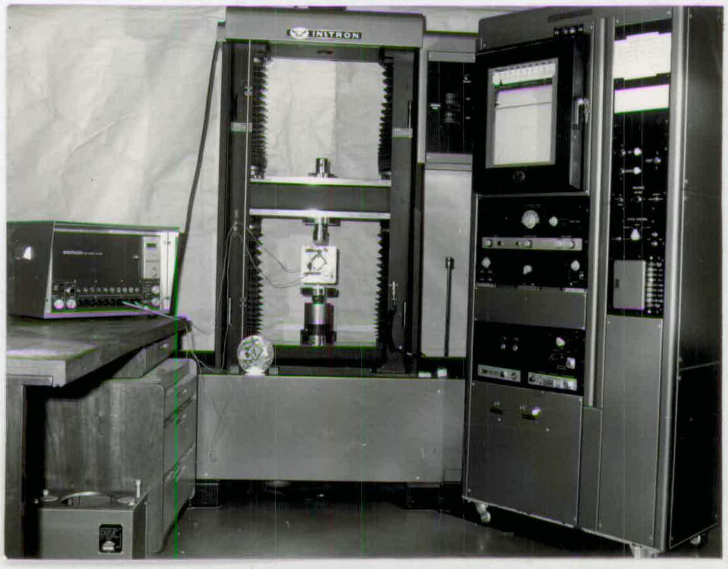
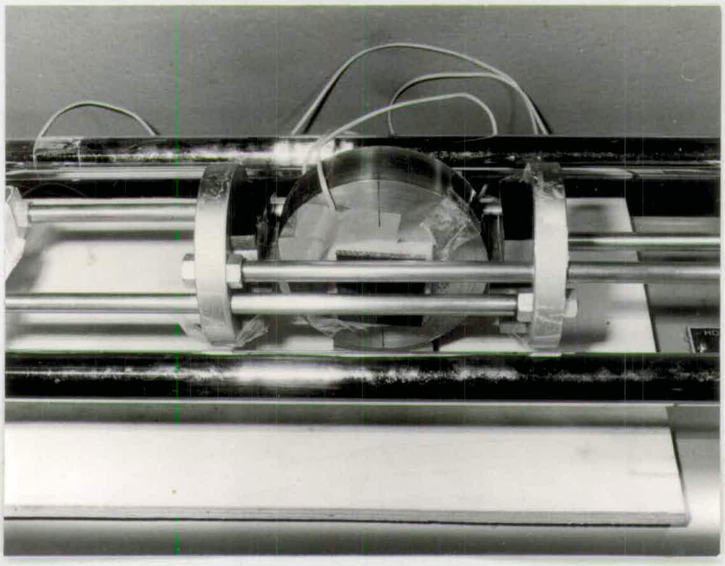


Figure 7.1:

Schematic diagram for the wiring circuit.





Photographs 7.1,2:
Steel specimens during testing.

7.2.3. Preparation of Specimen for Testing

7.2.3.1. Application and protection of gauges

It is well known that a most important procedure in electrical strain gauge work is the mounting of the gauge element on the specimen. Therefore, with the object of achieving the best results all possible precautions were taken. The exact position of the gauge was fixed by scribing two mutually perpendicular diameters or centre lines on the faces of the circular or square specimens, respectively. The central area of the specimen (machined and polished as described before), was abraded using a medium grade emery paper and rubbing in two perpendicular directions, so that a 90° lattice of scratches was left on the surface. After degreasing, cleaning thoroughly with trichlorethylene, the surface was treated with metal conditioner. Treatment was completed with 10% phosphoric acid solution, followed by a wash in weak ammonia solution (Neutralizer). Then the surface was dried thoroughly using hot air before fixing the gauge. The general techniques for application of electric strain gauges have been described in detail by many authorities, but because the procedure varies in detail according to the design of the gauge and the type of cement, the standard technique given by manufacturer[143] was rigorously followed together with general precautions, dictated by previous experience[11][34][59][68], and the additional precautions discussed later.

After attaching and wiring, each gauge

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(68) Hondros, G. The protection and manipulation of electrical resistance strain gauges of the bonded-wire type for use in concrete, particularly for internal strain measurement. Mag. of Conc. Research. Vol.9: No.27, December 1957. pp.173-180.

was checked for its resistance. Then with the object of avoiding the effect of earth leakage[59], the resistance of each gauge to earth was checked through the tags and the cable terminals.

With the aim of gauge protection, and avoidance of air current effects, the gauge and wires were coated completely with the Fleming G.C.101 Kit. It consists of a resin GCR 101 and a hardener GCH, which when mixed together form an epoxy two pack thixotropic coating. As will be shown later, it was found necessary to cover the whole area up to the beginning of the screened cable.

As a final precaution the cable was soldered to the specimen, a bag of foamed rubber was placed over the gauge and fixed by masking tape. Then before any connection to the measuring apparatus, the gauge resistance, as well as its resistance to earth were checked again through the wire terminals.

7.2.3.2. Troubles during testing

After preliminary tests, it was found that the results were neither consistent nor reproducible. Searching for possible sources of error, certain precautions emerged as necessary to avoid disturbances, especially at lower values of strain. Many of these precautions were additional to the usual precautions given in text books. Although some of them are of peculiar character, they seemed to be justified. In fact, after these sources of error were minimised, the results became reproducible and to a great extent consistent. Because of the considerable time spent with these precautions, the ones having the greatest influence will be summarised in the next paragraphs.

1. Air gap under the wire after its junction with the tage: At first it was thought enough to cover the strain gauge, the gauge junctions and the first part of the wire with a masking tape and a rubber band. Inspection showed that this was not enough to guard against local temperature fluctuations. Also the slightest touch or pull on the leads was found to alter the reading. Then it was thought better to solder a considerable length of the wire to the specimen, but unfortunately, this also did not help, and it was decided to waterproof the gauge, the junctions and the terminals of wire.

Firstly, a clear lacquer spray for woodwork was used, but with it another difficulty was met. It was not possible to have the insulation free from air bubbles, and the greatest bubble was usually formed between the junction and the rubber. The slightest pressure on this air bubble was enough to disturb the reading. Applying the spray many times, and removing the air bubbles each time, so that the space between the wire and the specimen became full, was found successful. Afterwards, Fleming G.C.101 was successfully used in one application.

2. Dummy gauges. A major trouble was found when making splices in the wires with the object of balancing. With splicing, a gap in the screen usually exists. This gap was enough to upset the balance of the strain bridge on sensitive ranges. Consequently, all wires of the dummy gauge were made slightly longer than the wire connected to the active gauge. Then balancing was done by cutting parts of the former rather than adding parts.

3. Zero drift: A feature of the Apparatus type 1516, which could have led to misleading results is zero drift. This took the form of a change in the reading of the pointer unassociated with deformation of test specimen. After a considerable effort in investigating the phenomenon, Blakey's conclusions were found quite applicable. He stated that drift occurs in most strain gauge equipment, usually to a negligible degree when the strains are not low, as in the case of tensile strains in concrete. In the present case the tensile strains along the X-axis are fairly low, especially at the early stages of loading. Therefore, the readings were affected to a considerable extent at the beginning. Briefly, there are two main reasons for zero drift, instrument and gauge drift. Both were discussed in detail by Blakey[11]. To minimise the instrument drift, which is usually associated with the measuring circuit outside the gauges, it was sufficient to allow the apparatus to warm up for one hour. As regards the gauge zero drift, it is almost wholly attributed to the drying shrinkage of the gauge bond and matrix. Although this results in a compressive strain in the wires of the gauge, the drift can be either tension or compression on the indicating metre, depending upon the relative rates of shrinkage in the active and dummey gauges. However, this could be minimised, by forced drying of the attached gauges using an infra-red lamp; consequently all the gauges were removed, and replaced by new ones. A further precaution was taken by attaching and drying both active and dummy gauges at the same time.

4. Zero shift: A defect in the readings which does not fall under any of the types mentioned previously, is a rather abrupt zero shift immediately after the commencement of the loading. With d.c.,

this has been attributed before[11], to a fault in the apparatus rather than a deformation of the specimen. More precisely it was attributed to a fault not yet positively identified in the galvanometer equilibrium. When the apparatus used has an internal a.c. bridge voltage, that cannot be the case, but it can be attributed to some change at the contact surfaces between the specimen and the loading platens, due to initial application of load. Extrapolation of the curves in such cases appears to be permissible in assessing the physical significance of the results.

5. Quality of soldering. After taking all the above precautions it was found that a badly soldered joint at one junction, can cause erratic fluctuations in the readings on the most sensitive strain measuring range. Resoldering reduced these fluctuations, and afterwards, care was taken during the soldering process.

7.2.4. Scope of Testing

As described before, the scope of testing was to compress a circular specimen along the vertical diameter or a square specimen along the centre line. A schematic diagram of the experimental loading was illustrated in Table 7.1, and specimens during testing in the Instron and the Hounsfield Tensometer are shown in Photographs 7.1 and 7.2.respectively.

7.2.5. Preliminary Tests on the Performance of Specimens, Loading and Gauges

7.2.5.1. Effect of different cushion materials on the measured strains

It is recommended in A.S.T.M. Standards when testing concrete by the indirect tension test, that soft packing strips should be inserted at the rim of loading. The object of this might be to avoid high compressive stresses in the vicinity of the loading area, or to assure uniform distribution. In the meantime recent tests[1][41] showed that the type of cushion matérial influences to a great extent the ultimate splitting tensile strength, and on the contrary to A.S.T.M. one of the main conclusions by Addinal and Hacket was that steel platens alone had given a stress distribution in his disc (araldite) most closely resembling the theoretical distribution. Adding to this contradiction, the fact that the present specimens are neither concrete nor araldite, it was thought of importance to study the effect of different loading conditions on the measured strains values. A disc (D_1) and a plate (P_1) were selected and tested using different cushions. The testing procedure was the same one followed in the main tests, which will be described in 2.6.1. The detailed results need not be given here, but they yielded most of the indications given by Hacket[1] disagreeing in only one important conclusion. However, the main observations from the present tests can be summarised briefly as follows: -

1. Both vertical and horizontal strains (ϵ_y , ϵ_x) at the centre were affected by the cushion material.

2. For the same load a softer cushion material produced lower values for the horizontal strain, and slightly lower values for the vertical strains.

3. The omission of any cushion material gave the highest value for the lateral strain, and higher values for the vertical strain..

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(1): Addinal, E. and Hackett, P. The effect of platen conditions on the tensile strengths of rock-like materials. Civil Engineering and Public Works Review, October, 1964. pp. 1250-1253.

4. The insertion of plywood or cardboard gave strains, at the fourth cycle almost the same as the strains due to no-cushion-material inserted, but the smaller range of values was obtained when using plywood, and this was contrary to Hacket's conclusions in this respect. The latter was that the closest distribution to the theoretical one, would be better obtained by applying the loads through the prepared steel loading platens without inserting any cushion materials, But his specimen was made from araldite..

An explanation of this contradiction might be to the relative rigidity between the specimen and the cushion material. It seems that in order to have the smallest range in values, there should be some difference, but not too much in rigidities. This would help to achieve a better re-distribution of the pressure at the loading rim. This was emphasised later, when the three discs complying with the assumptions incorporated in the theory of the plane-stress case, gave results very close to the actual properties of specimens.

7.2.5.2. Reproducibility of readings

After the decision to employ plywood as an insertion between specimen and platen, an additional disc and plate which were prepared from the same steel block were tested for the reproducibility of readings. Tests were carried out using gauges of 120 ohm resistance, and employing the Data Logger* which was available for a short time as a demonstration

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* The data logger will be described later in Chapter 8 .

model. Readings were recorded at ten increments. In no case was the specimen loaded to a value greater than 50% of the load which creates a maximum stress near to the steel's proportional limit. The load was decreased down to a load slightly above zero to keep the specimen in position and the strains were recorded at the same intervals. This process of recording strains at loading and unloading was repeated seven times for four positions of loading for each specimen. A typical example is given in Appendix 7.1. The results indicated that the strains in both vertical and horizontal directions could be measured several times, after the fourth cycle, without any variation in the results.

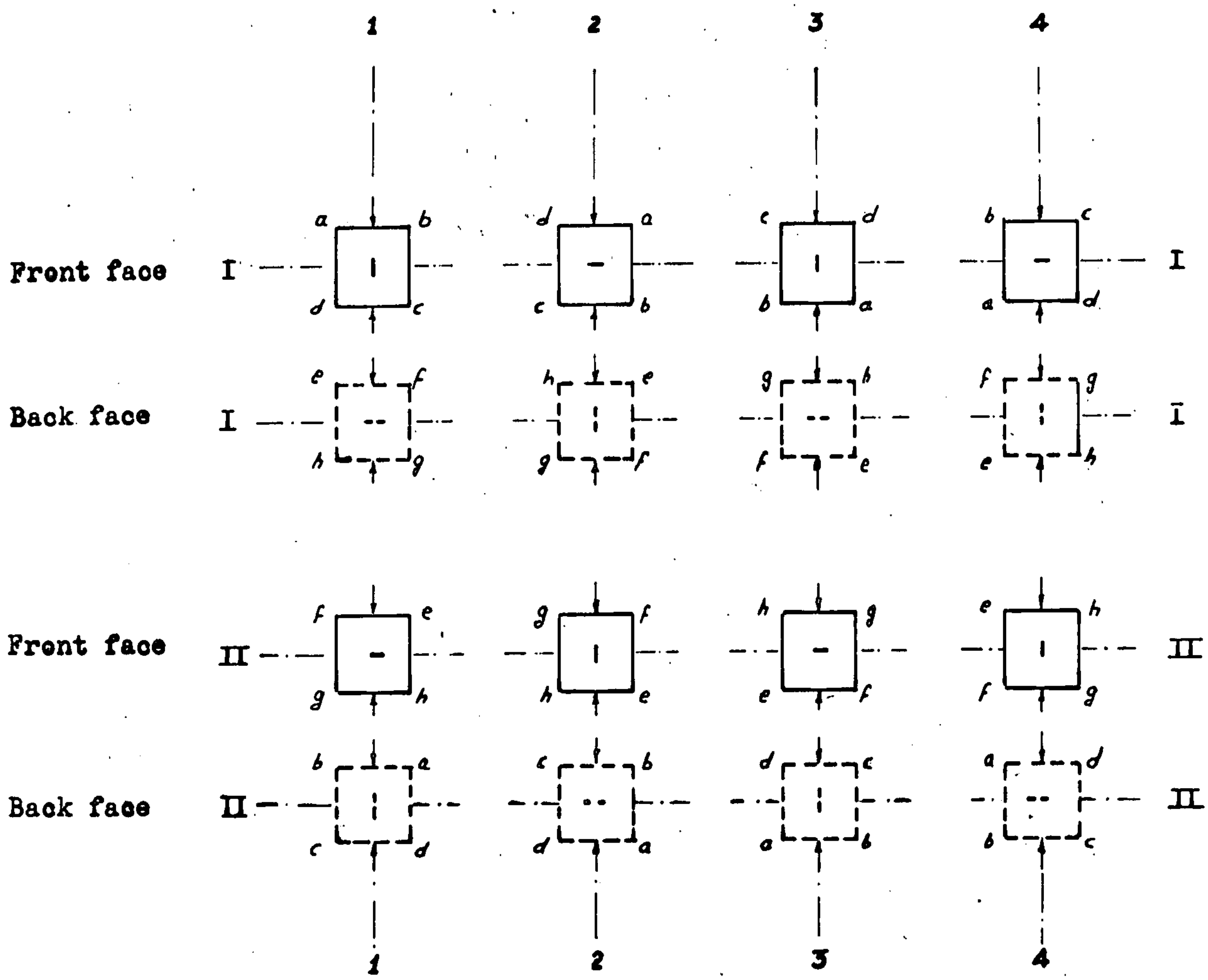
7.2.6. Main Tests: Procedure, Stages of Testing and Test Results

7.2.6.1. Procedure

During the testing procedure, the following were the main precautions all through the tests:

1. Readings were taken for two or four positions on each face. Thus at each loading, the strain is represented by four or eight readings according to whether the specimen was tested in the Instron or the Hounsfield respectively. Figure 7.2. shows a schematic sketch for the different positions.

2. With each position, the balancing process was carried out at the highest degree of sensitivity, by starting with the maximum range, and then altering the range, step by step, until balance is achieved at the lowest strain range, which corresponds to the highest sensitivity.



For tests with the Instron:

ϵ_x = the average of positions: I2 , I1 , II1 & II2

ϵ_y = the average of positions: I1 , I2 , II2 & II1
(readings are plotted on the figures)

For tests with the Hounsfield :

ϵ_x = the average of positions: I2 , I4 , I1 , I3 , II1 , II3 , II2 & II4

ϵ_y = the average of positions: I1 , I3 , I2 , I4 , II2 , II4 , II1 & II3
(an example showing the deduction of the average line is shown in Appendix 7.1)

Figure 7.2:

Schematic sketch showing calculation of the average positions

3. An identical testing procedure, was followed for all positions of one specimen, and for all the specimens. For each position the load was increased incrementally, and the strains ϵ_y, ϵ_x were measured at the end of seven to ten increments.

7.2.6.2. Stages of testing and test results

1. Readings for the determination of the actual values of E and ν: For these readings, the discs D_1, D_3 and D_5 were used. The measured values for the present, as well as the subsequent readings are too long to be listed here. The averages of these readings are illustrated graphically in Figures 7.3-7.5 and Figures 7.6-7.7 for tests carried out with the Instron and the Hounsfield respectively.

2. Readings for the examination of the proposed outer-dimensions; The circular specimens D_2, D_4 and D_6 were used. The average measured strains are illustrated graphically in Figures 7.8-7.10 and Figures 7.11-7.12 for tests carried out with the Instron and the Hounsfield respectively.

3. Readings for the examination of the similarity between circular and square specimens; The square blocks P_1, P_2 and P_3 were used. The average measured strains are illustrated graphically in Figures 7.13-7.15 and Figures 7.16-7.17 for tests carried out with the Instron and the Hounsfield respectively.

It should be mentioned here, that D_1,D_2 and P_1, due to their large diameter could not be tested in the Hounsfield. Consequently with the object of statistically proper comparisons, the following discussion will be limited to the tests with the Instron. Other results with the Hounsfield will help only as confirmatory ones.

7.2.7. Discussion of Test Results

7.2.7.1. Elastic constants of the steel used

At any stage of loading, the modulus of elasticity and Poisson's ratio could have been calculated by using respectively Equation 5.24 ($E = 5.095 \times \frac{P}{Dt} \times \frac{1}{(\epsilon_x + 3\epsilon_y)}$) and equation 5.21 ($\nu = -\frac{3\epsilon_x + \epsilon_y}{\epsilon_x + 3\epsilon_y}$), Equation 5.22 ($\nu = \frac{3 - \bar{\epsilon}_y/\bar{\epsilon}_x}{3\bar{\epsilon}_y/\bar{\epsilon}_x - 1}$), or developed tables or charts.

After the preliminary calculations it was found that account should be taken of the first readings of low strains. Therefore it was considered justifiable to plot an average curve for both the vertical and horizontal strains (ϵ_y , ϵ_x), Due to the fact that the material was steel, and the maximum loads were much lower than the loads which could produce a stress equal to the proportional limit, the ϵ_y and ϵ_x curves were both straight lines. At the same time any error in the load value, although avoided* in interpretation was considered eliminated. Consequently the calculate values of E and ν were constant at all stages of loading. These values are summarised in Table 7.3-a. Analysing the table together with the figures 7.3-7.5 lead to the conclusion that the steel block from which the specimens were prepared has a modulus of elasticity, and Poisson's ratio equal to 27.600 lb/in^2 ,

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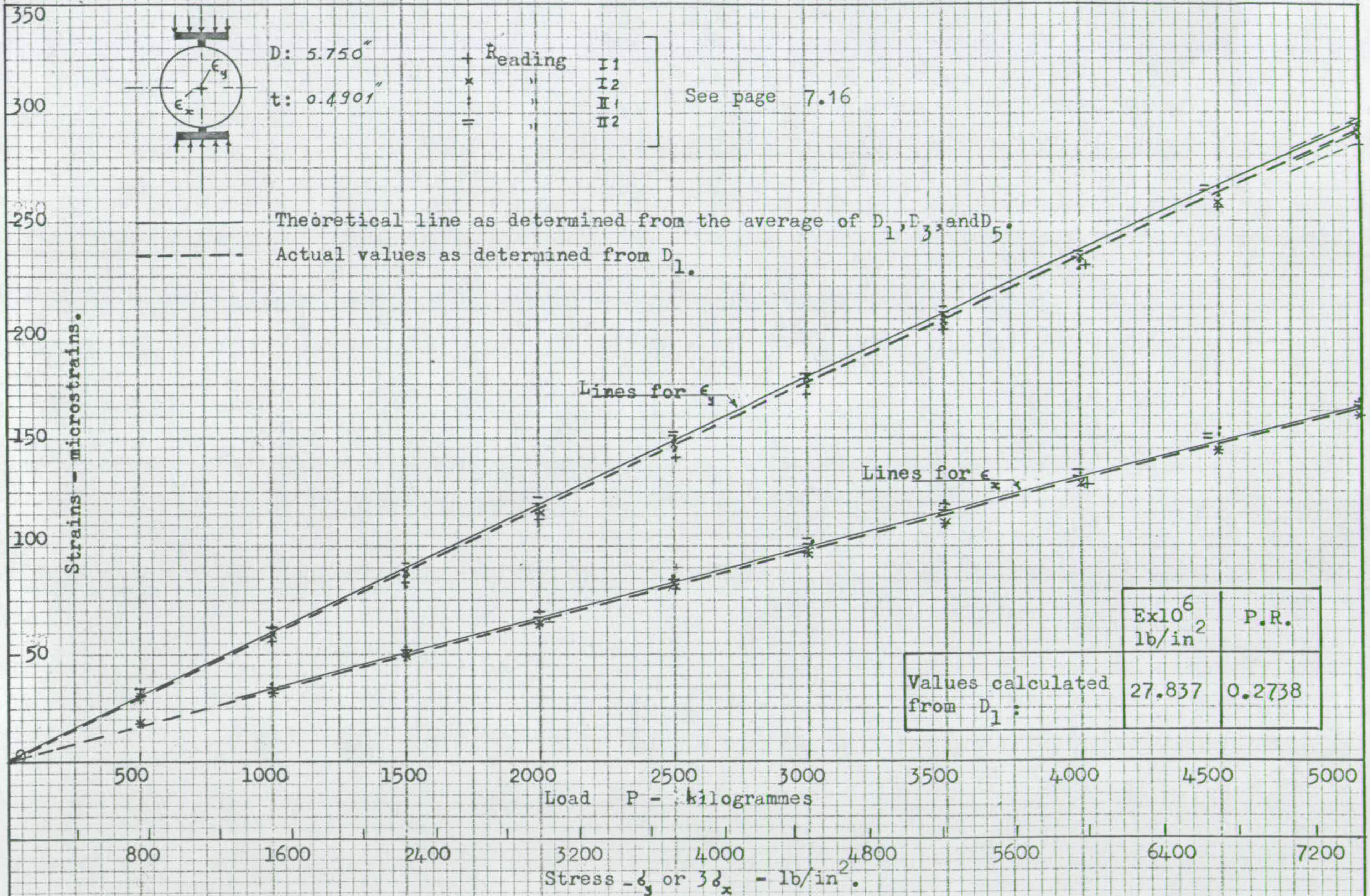
* Strains at some points are not plotted exactly on the vertical indicating the load. Their locations indicate the exact load reading, as shown by the machine indicator.



	$E \times 10^6$ lb/in^2	P.R.
Values calculated from D_1 :	27.837	0.2738

Figure 7.3 :

Actual deformation properties of steel as obtained from D_1. (Loading machine : Instron)

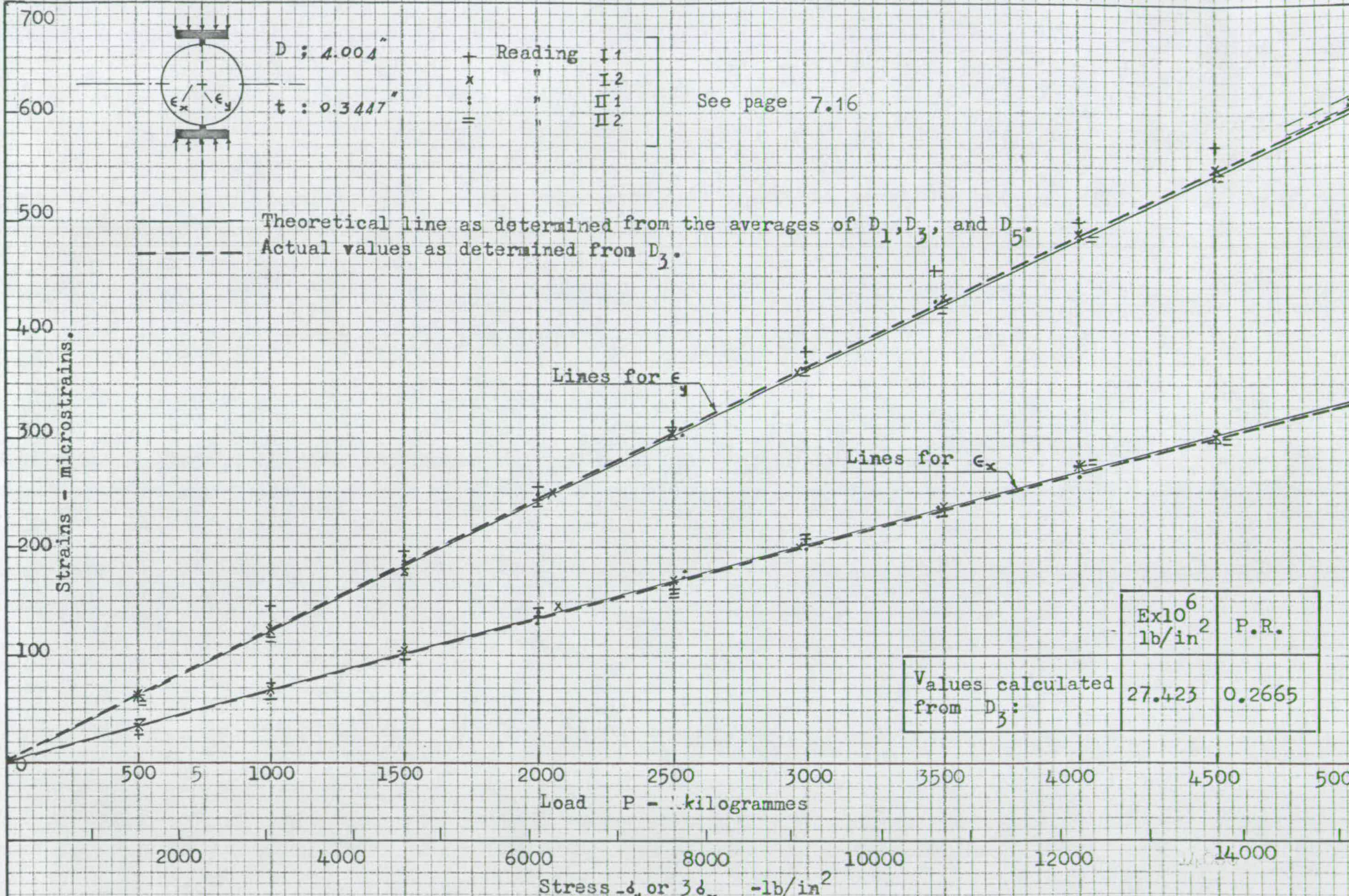


Figure 7.4 :

Actual deformation properties of steel as obtained from D_3. (Loading machine : Instron)

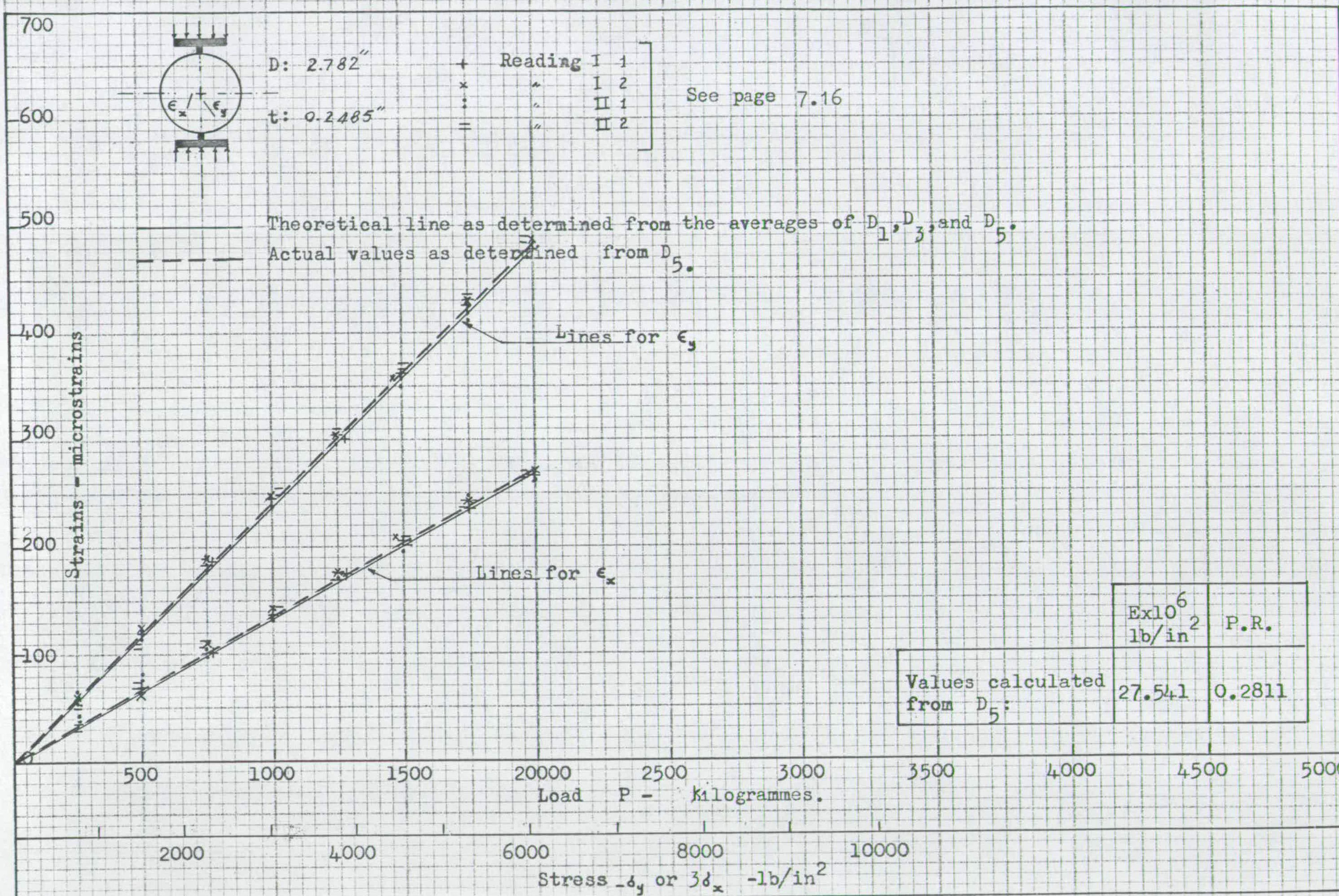


Figure 7.5 :
Actual deformation properties of steel as obtained from D_5. (Loading machine : Instron)

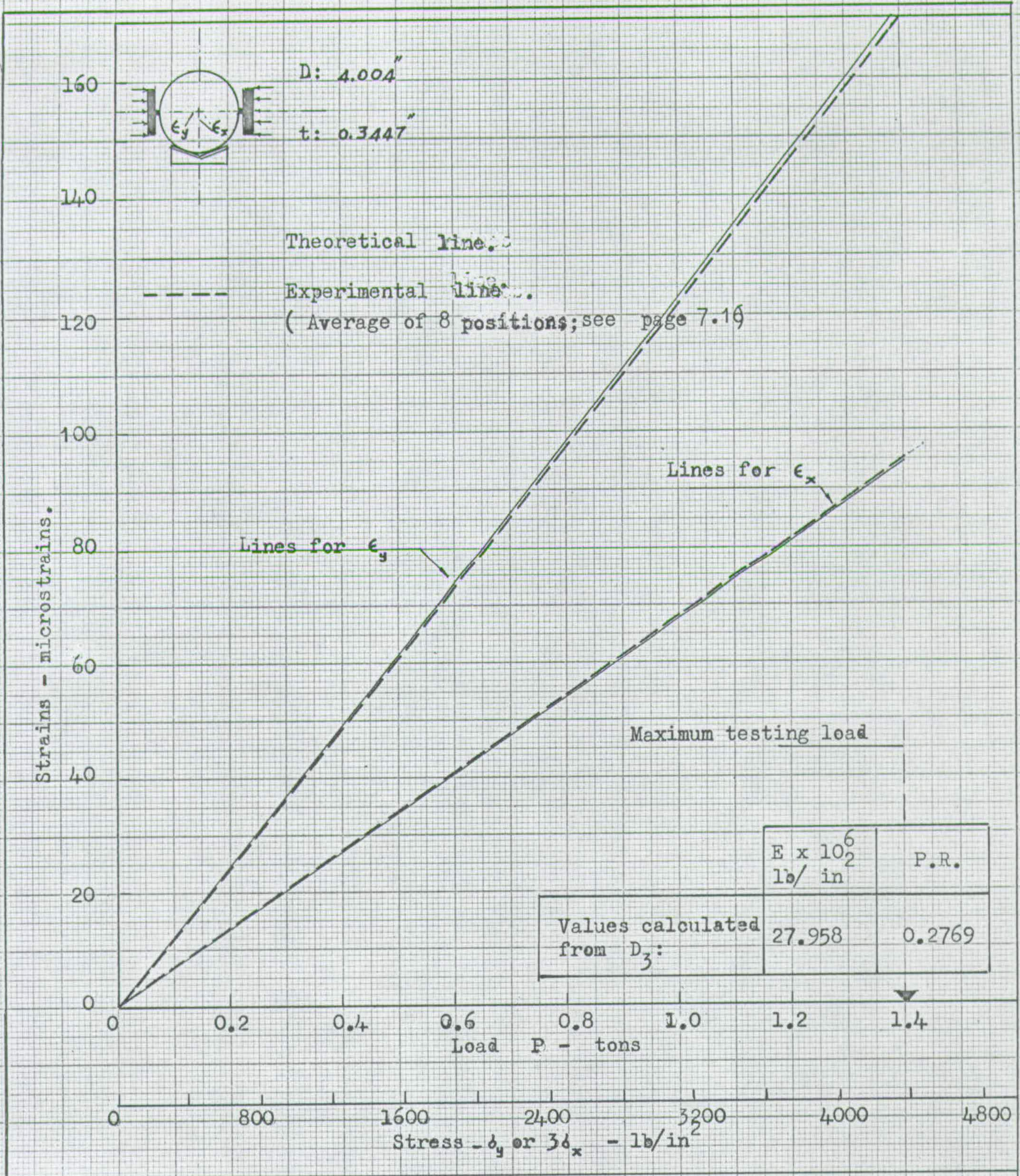
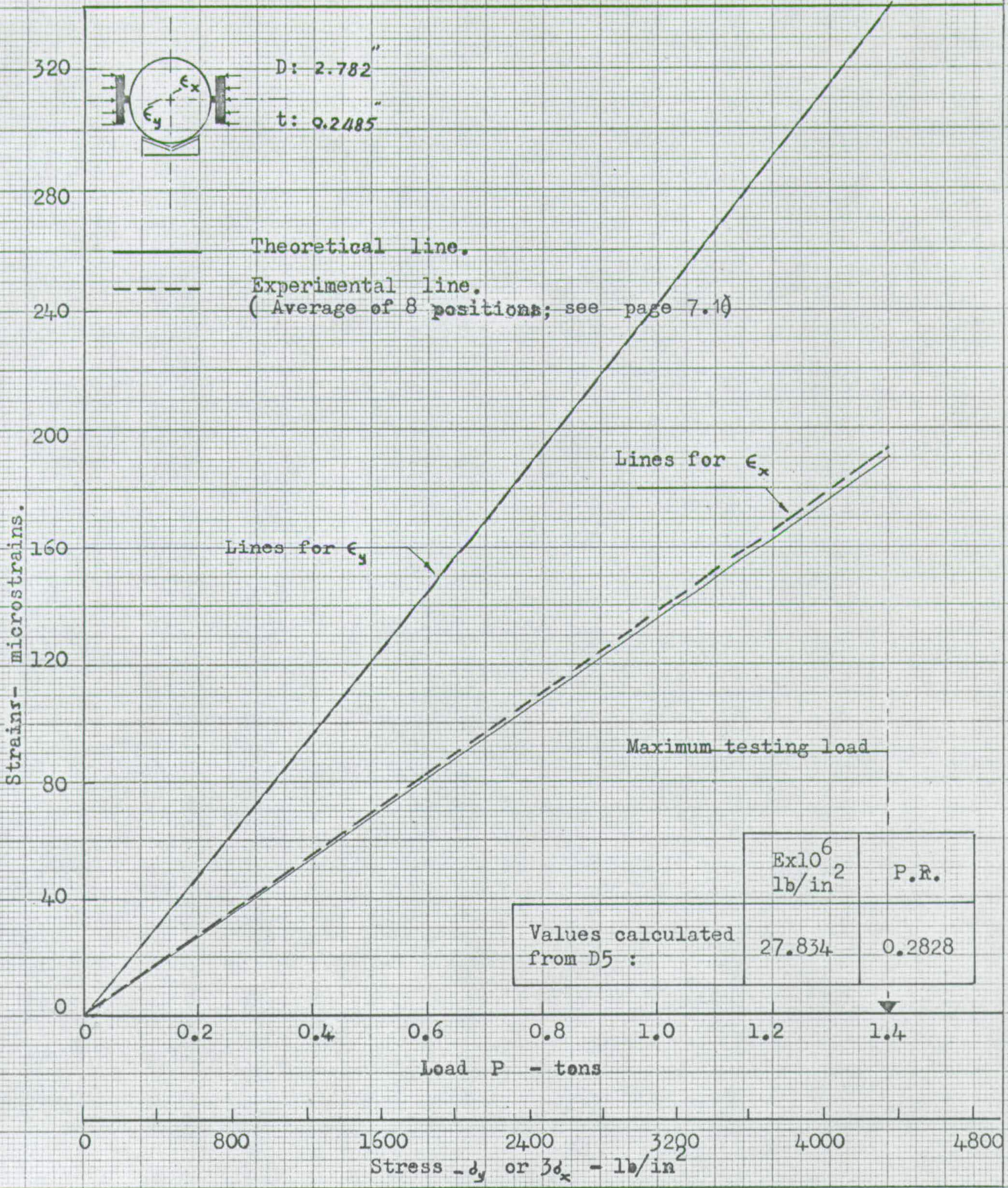


Figure 7.6 :

Actual deformation properties of steel as determined from D_3. (Loading machine: Hounsfield)



	Ex10^6 lb/in^2	P.R.
Values calculated from D5 :	27.834	0.2828

Figure 7.7 :

Actual deformation properties of steel as determined from D_5: (Loading machine : Hounsfield) and 0.2738 respectively. Each of these values represents the mean of the three discs. From now on all the strains deduced from these values will be termed the theoretical strains, and the lines connecting different points, the theoretical lines.

After this conclusion, the three discs were considered as specimens of known properties. The values of the theoretical strains were calculated, as for any other specimen, and at any load P as follows:

1. Substituting for P in equations for the case of plane stress we get, $\delta_x = \frac{2P}{\pi t d}$ (Equation 5.15), and $\delta_y = \frac{-6P}{\pi t D}$ (Equation 5.16) . .

2. Substituting for E and ν we get the theoretical strains as

$$\epsilon = \frac{1}{E}\left[\delta_x - \nu \delta_y \right]$$ (Equation 5.18)

$$\epsilon_y = \frac{1}{E}\left[\delta_y - \nu \delta_x \right]$$ (Equation 5.19)

As the figures show, these strains calculated from the mean are very close to the measured ones for each of the tests. Judging this in terms of the final calculated values, it was found that the range of deviation from the mean for the modulus of elasticity and Poisson's ratio as assessed from the individual discs is + 0.85% to - 0.64% and + 2.7% to - 2.7% respectively.

3. Confirmatory tests (Figures 7.6, 7.7) showed values of the modulus of elasticity and Poisson's ratio with maximum deviations from the actual values, equal to 1.29% and 3.32% respectively.

7.2.7.2. The proposed circular specimens dimensions and state of stress

With these dimensions (D_2, D_4 and D_6) it was not known which case of stress was valid. Therefore, theoretical values were calculated for both the plane-stress and plane deformation cases. The method of calculation for the former was the same one as given above in 7.2.7.1. As regards the latter the same sequence was used, applying the proper equations. In other words, instead of equations 5.18, and 5.19 the following two equations were used respectively.

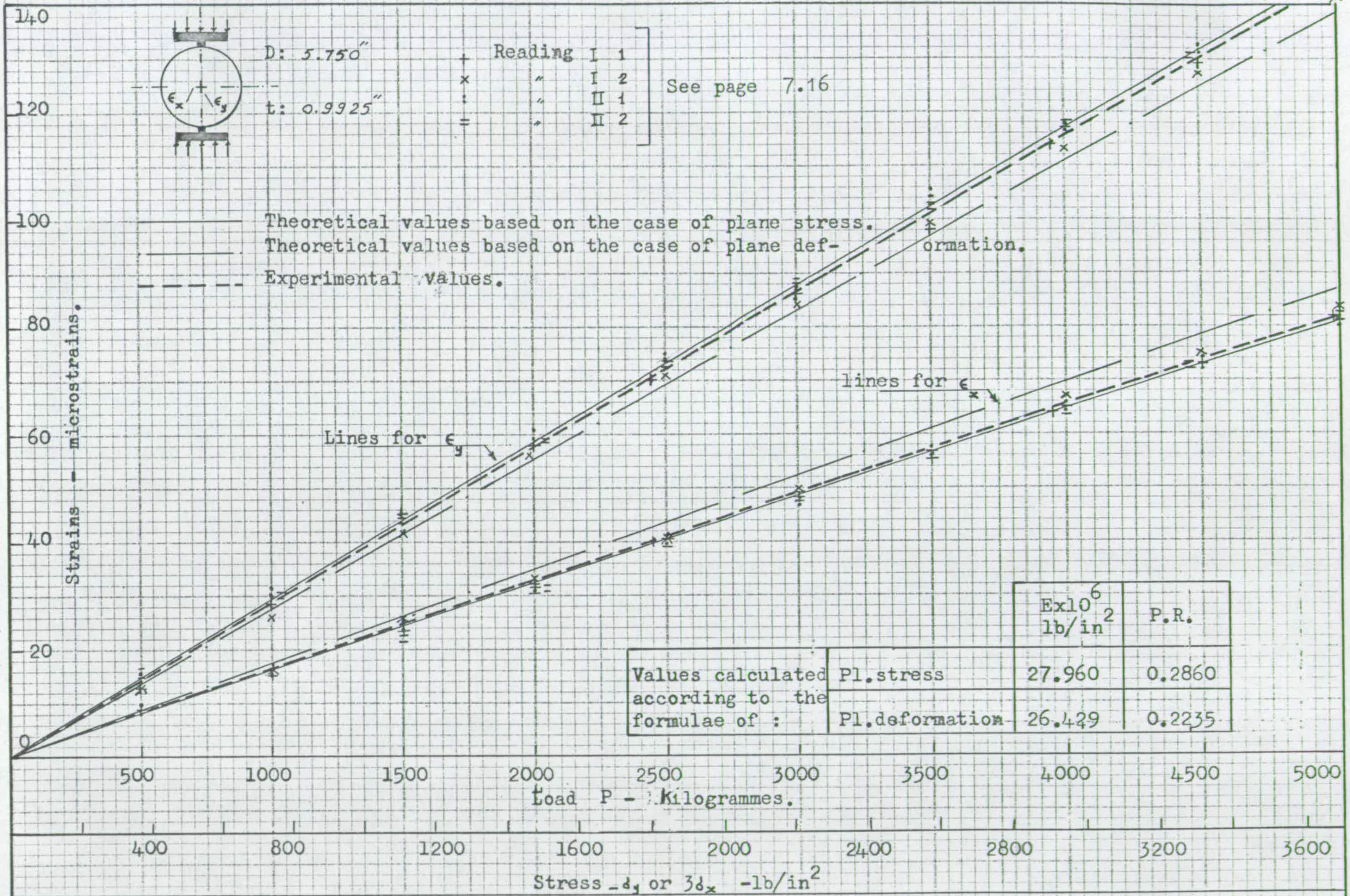
(Equation 5.25)

(Equation 5.26)

The main observations can be summarised as follows:

1. For the disc D_2 (D/t = 5.793, Figure 7.8) the average experimental values are very close to the theoretical values calculated according to the plane stress case. The modulus of elasticity and Poisson's ratio calculated from the lines connecting the different points, and using the plane-stress case, are respectively 27.960×10^6 lb/in^2 and 0.2860. When applying the plane-deformation formulae the calculated values become respectively 26.429 and 0.2235. Here a preliminary conclusion was considered, that the disc D_2 might behave according to the plane stress case.

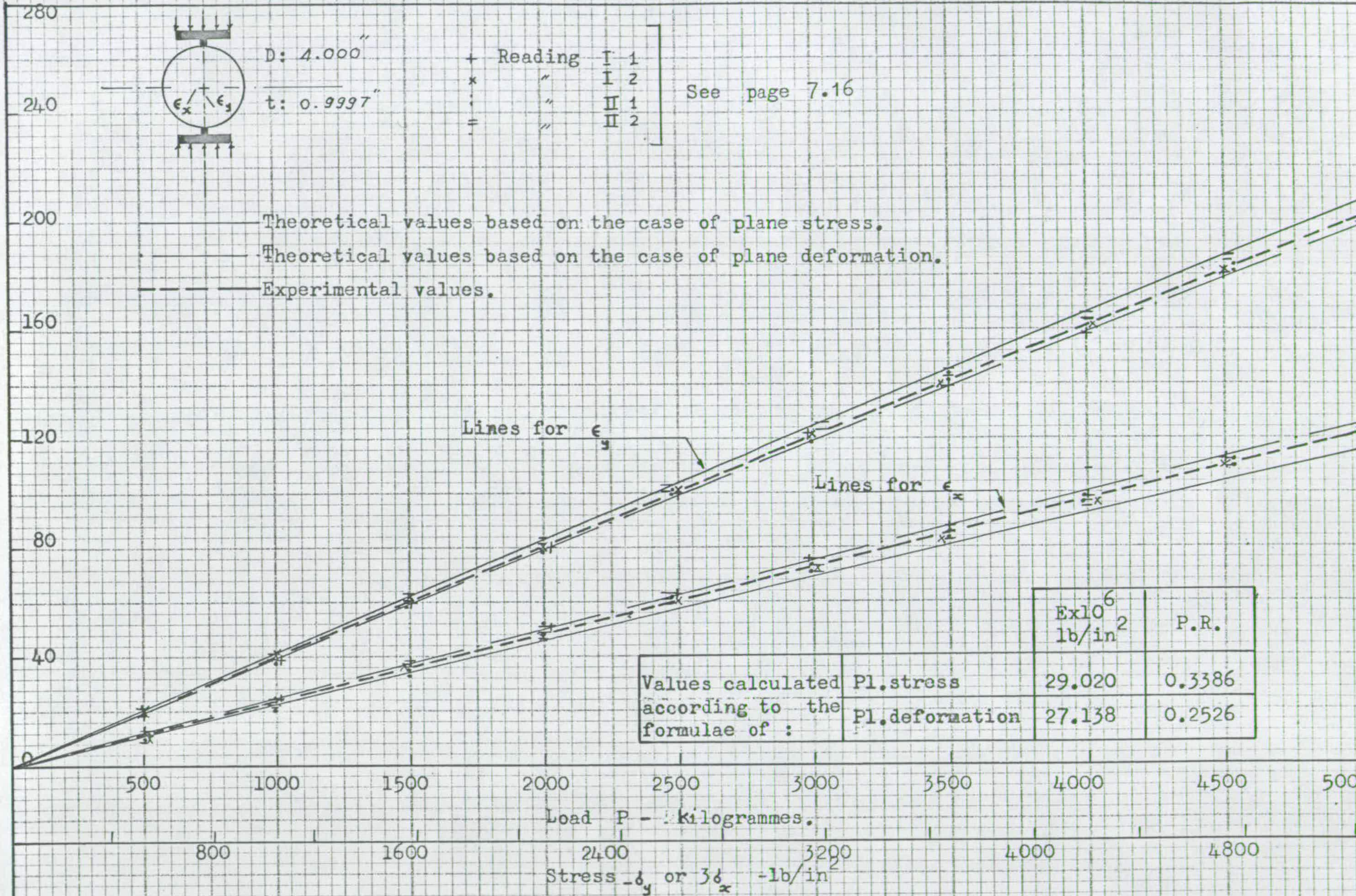
2. For the disc D_4 (D/t = 4,000, Figure 7.9 and 7.11) it is clear that neither the experimental individual values of strains nor the values of modulus of elasticity and



		$E \times 10^6$ lb/in^2	P.R.
Values calculated according to the formulae of :	Pl.stress	27.960	0.2860
	Pl.deformation	26.429	0.2235

Figure 7.8 :

Deformation properties of steel as obtained from D_2. (Testing machine : Instron)



		$E \times 10^6$ lb/in^2	P.R.
Values calculated according to the formulae of :	Pl.stress	29.020	0.3386
	Pl.deformation	27.138	0.2526

Figure 7.9 :

Deformation properties of steel as obtained from D_4. (Testing machine : Instron)

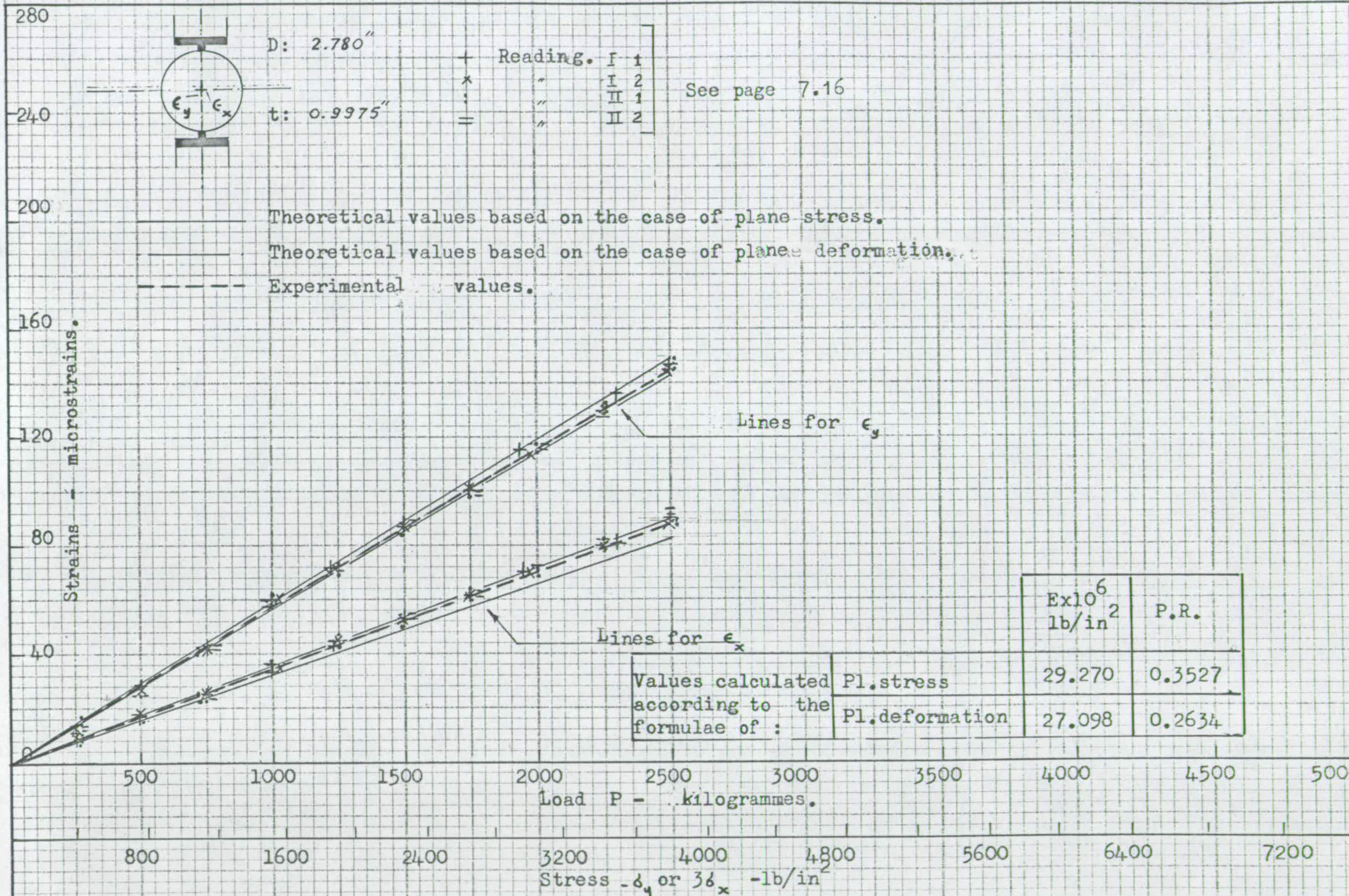


Figure 7.10 :

Deformation properties of steel as obtained from D_6. (Testing machine : Instron)

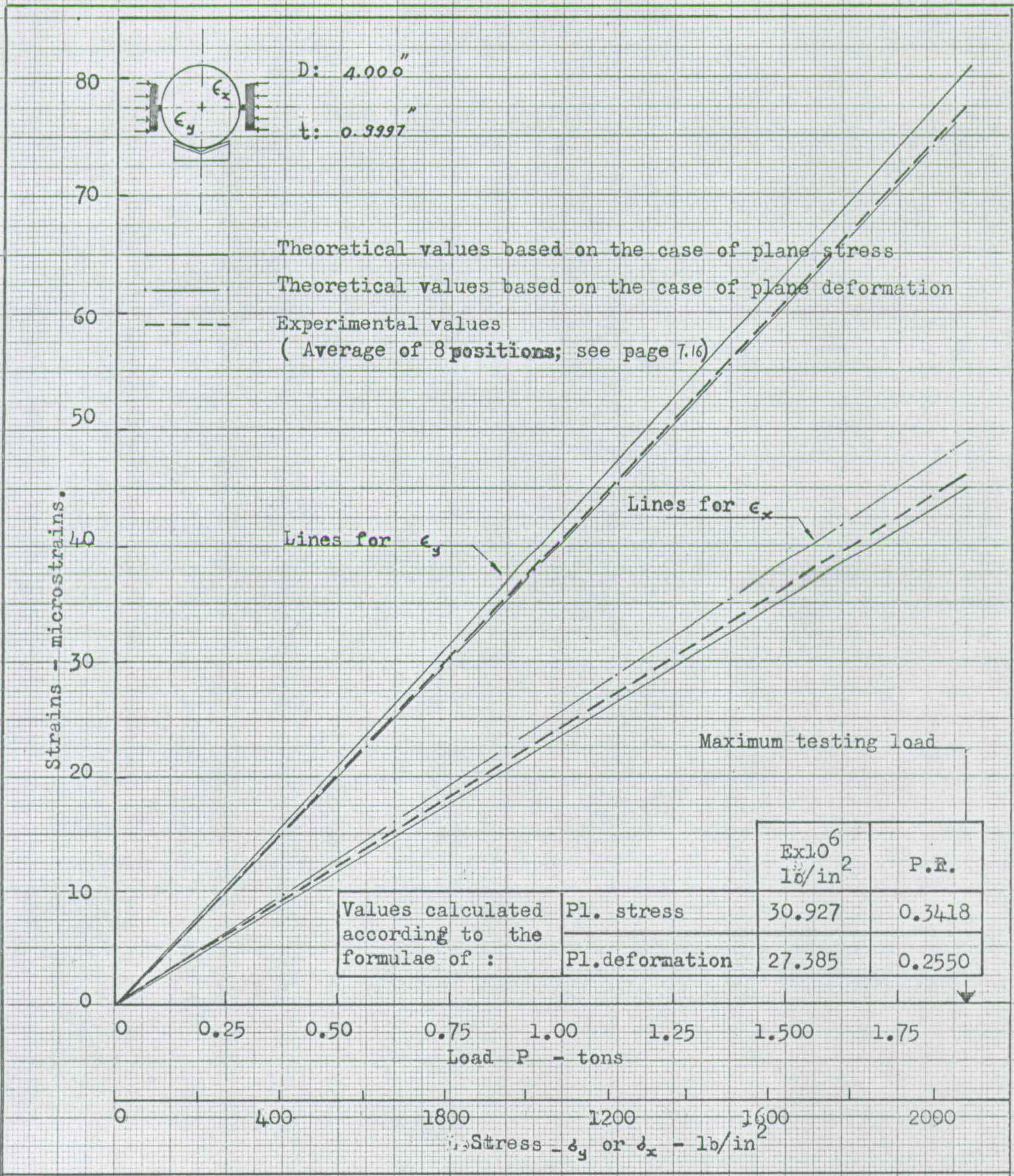


Figure 7.11 :

Deformation properties of steel as determined from D_4. (Loading machine : Hounsfield)

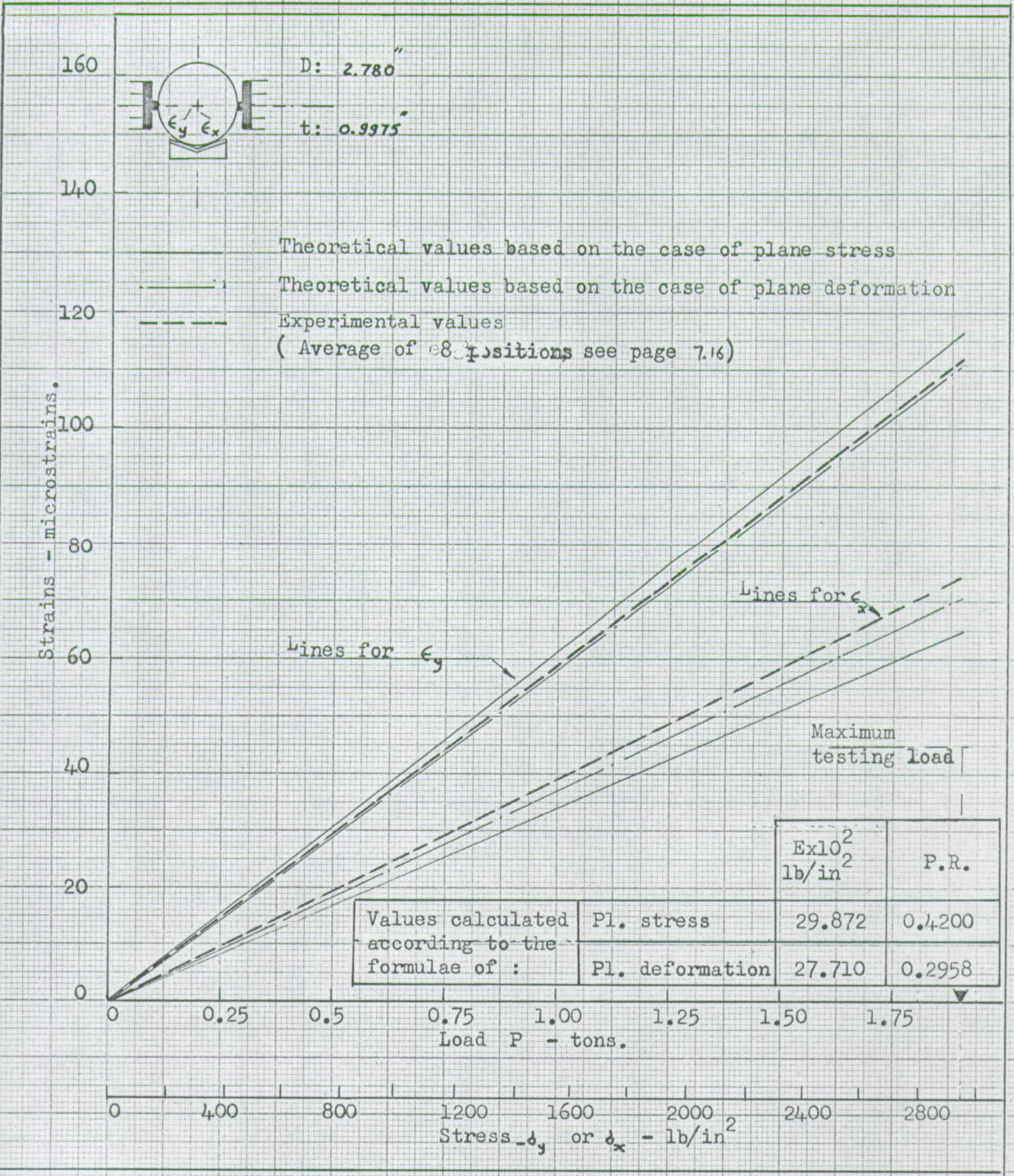


Figure 7.12:
Deformation properties of steel as determined from D_6. (loading machine : Hounsfield)

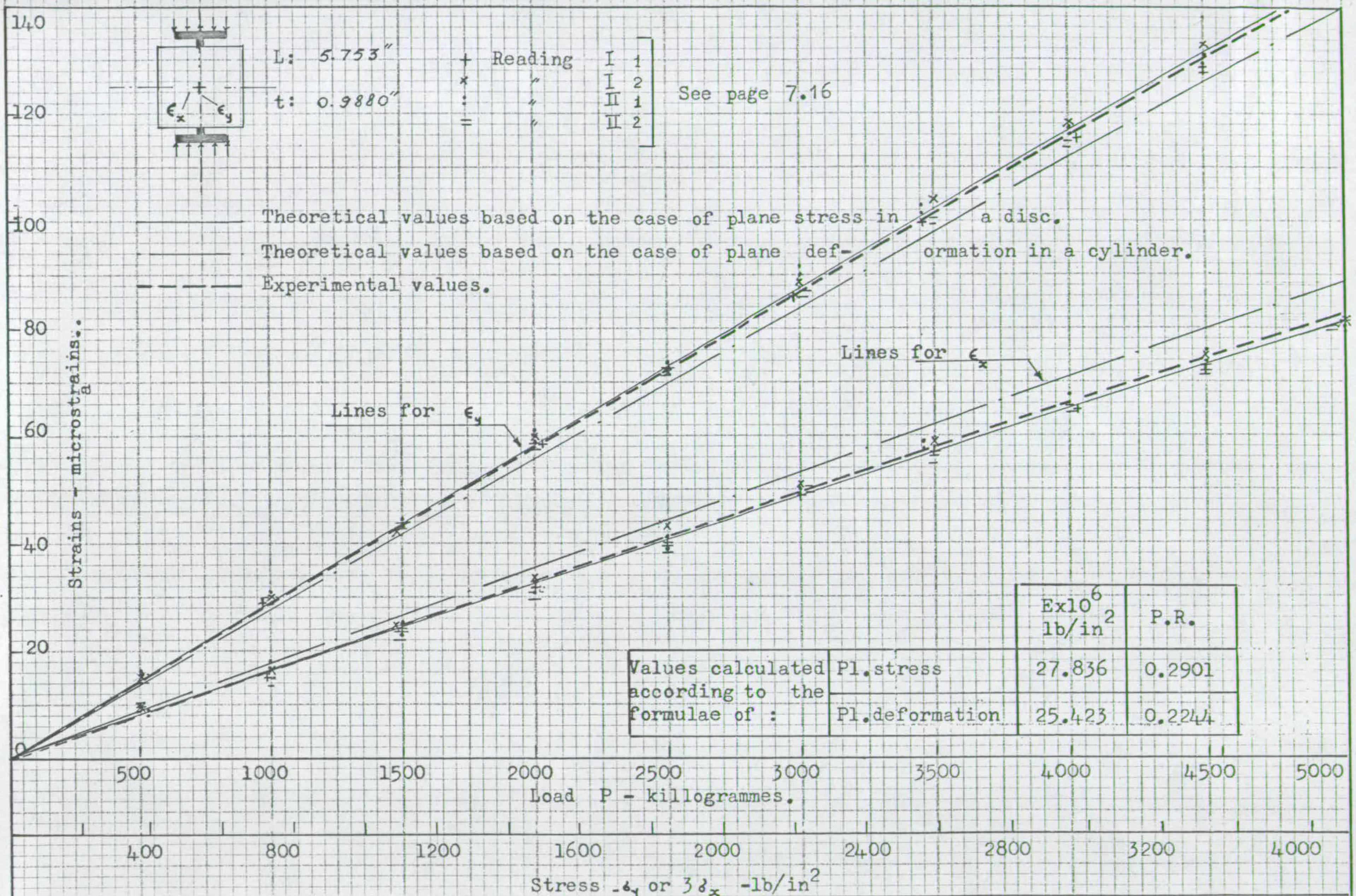
Poisson's ratio calculated according to plane stress and plane deformation cases behave in a defined manner to any of the cases. An explanation for this specimen's behaviour cannot be given now, but will be explained later.

3. For the disc D_6 (D/t = 2.787, Figure 7.10 and 7.12) it appears that both experimental values for strains, and value of the modulus of elasticity and Poisson's ratio calculated according to plane deformation formulae, indicate that the disc behaved according, or close, to the plane deformation case. But for the time being the results were not considered conclusive. A final conclusion would be on a stronger basis after considering the corresponding square specimens.

7.2.7.3. Similarity between square and circular specimens

In Figures 7.13, 14 and 15 the experimental strains are plotted for the square blocks P_1, P_2 and P_3 respectively. The theoretical strains and the experimental values for the modulus of elasticity and Poisson's ratio were calculated on the basis of circular specimens having the same outer dimensions. The following can be noticed:

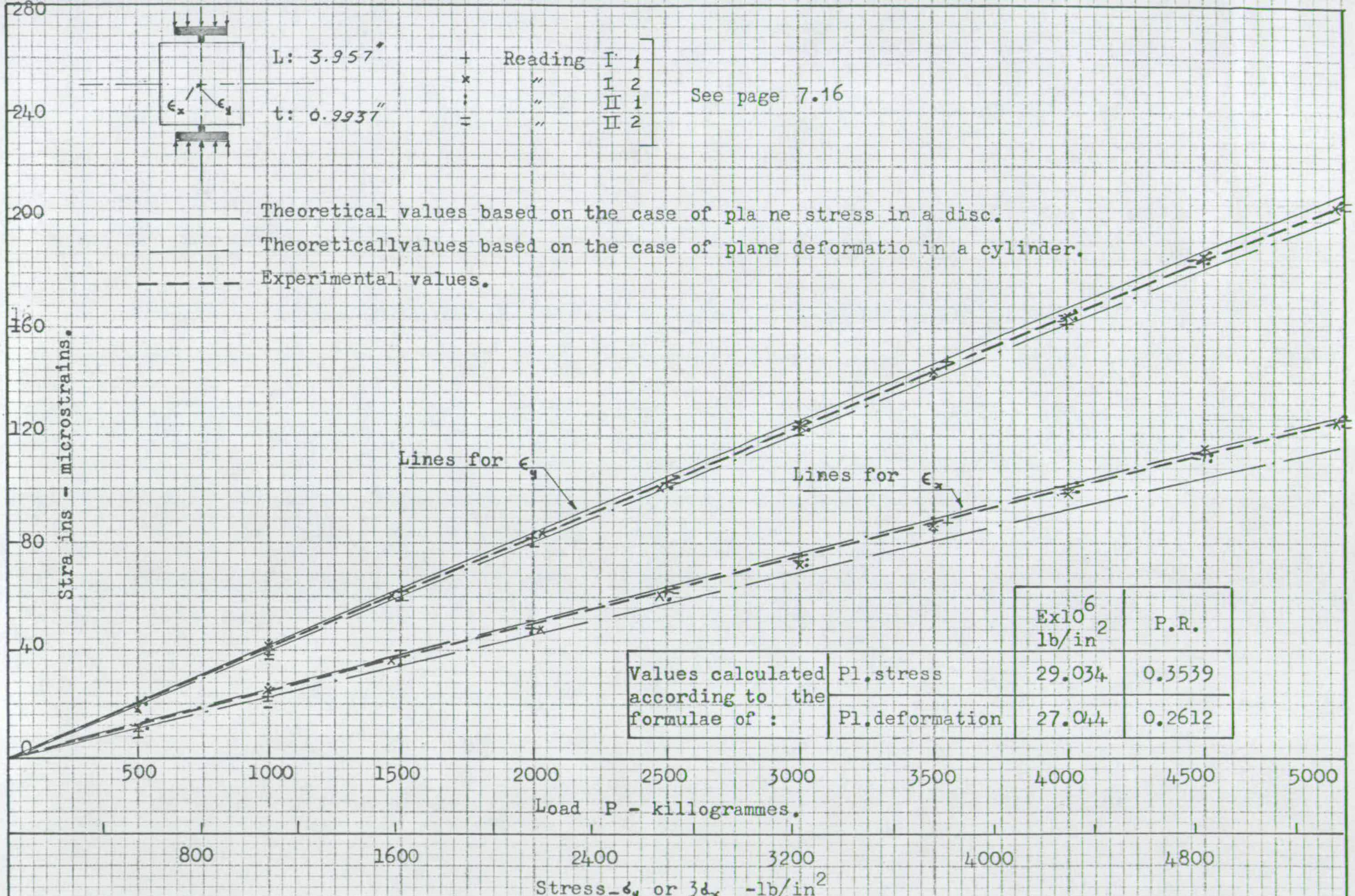
1. Comparing between these square specimens, and the corresponding circular specimens (D_2, D_4 and D_6 respectively), regardless of the state of stress, the strains are found to be more or less similar. Assuming the final values calculated from the circular specimens as the basic ones, the ratios between them and the corresponding values from square blocks, were calculated as shown in Table 7.2. With the exception



		$E \times 10^6$ lb/in^2	P.R.
Values calculated according to the formulae of :	Pl. stress	27.836	0.2901
	Pl. deformation	25.423	0.2244

Figure 7.13 :

Deformation properties of steel as obtained from P_1. (Testing machine: Instron)



		$E \times 10^6$ lb/in^2	P.R.
Values calculated according to the formulae of :	Pl. stress	29.034	0.3539
	Pl. deformation	27.044	0.2612

Figure 7.14:

Deformation properties of steel as obtained from P_2. (Testing machine: Instron)

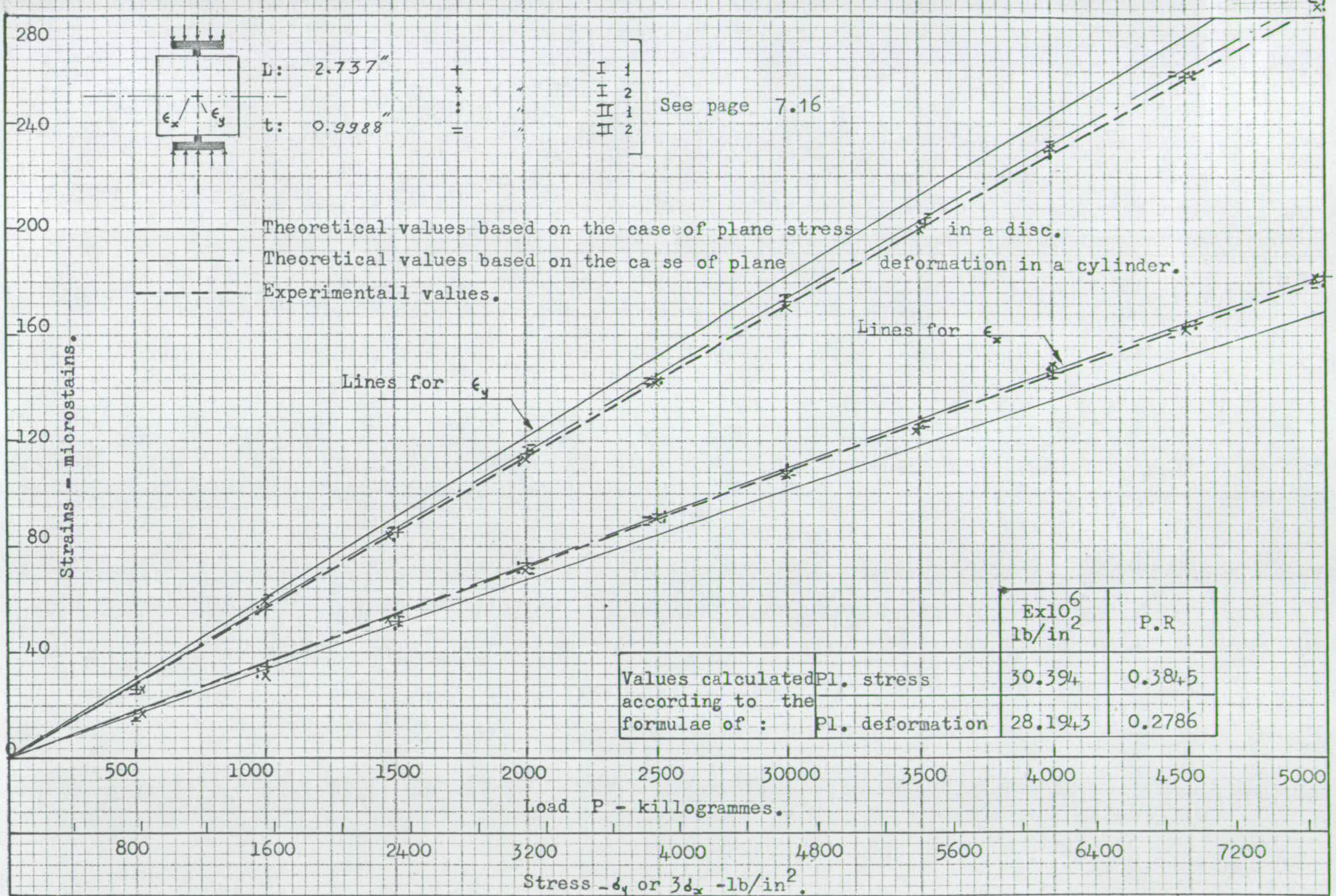
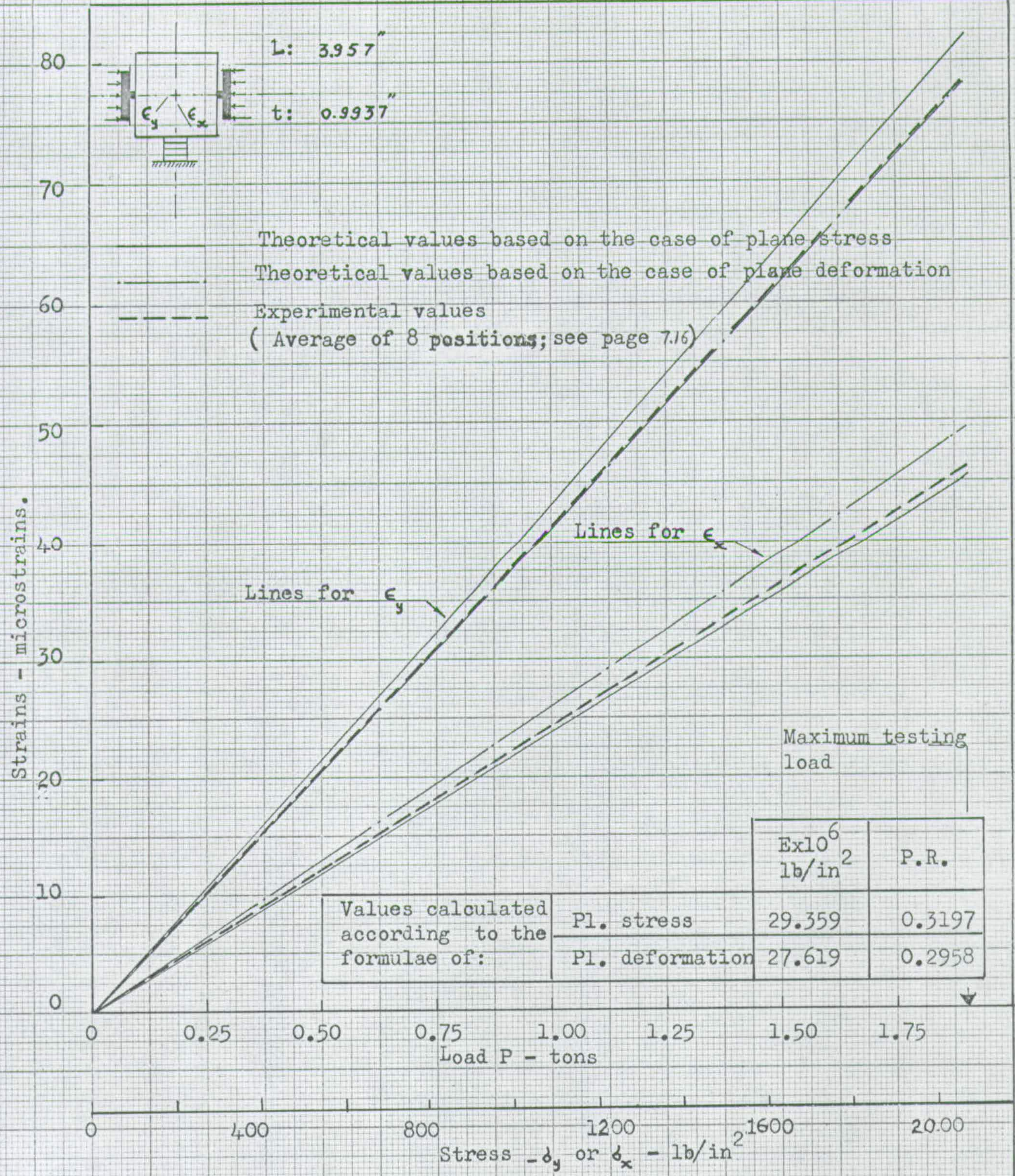


Figure 7.15 :

Deformation properties of steel as obtained from P_3. (Testing machine : Instron)



		$E \times 10^6$ lb/in^2	P.R.
Values calculated according to the formulae of:	Pl. stress	29.359	0.3197
	Pl. deformation	27.619	0.2958

Figure 7.16 :
Deformation properties of steel as determined from P_2. (Loading machine : Hounsfield)

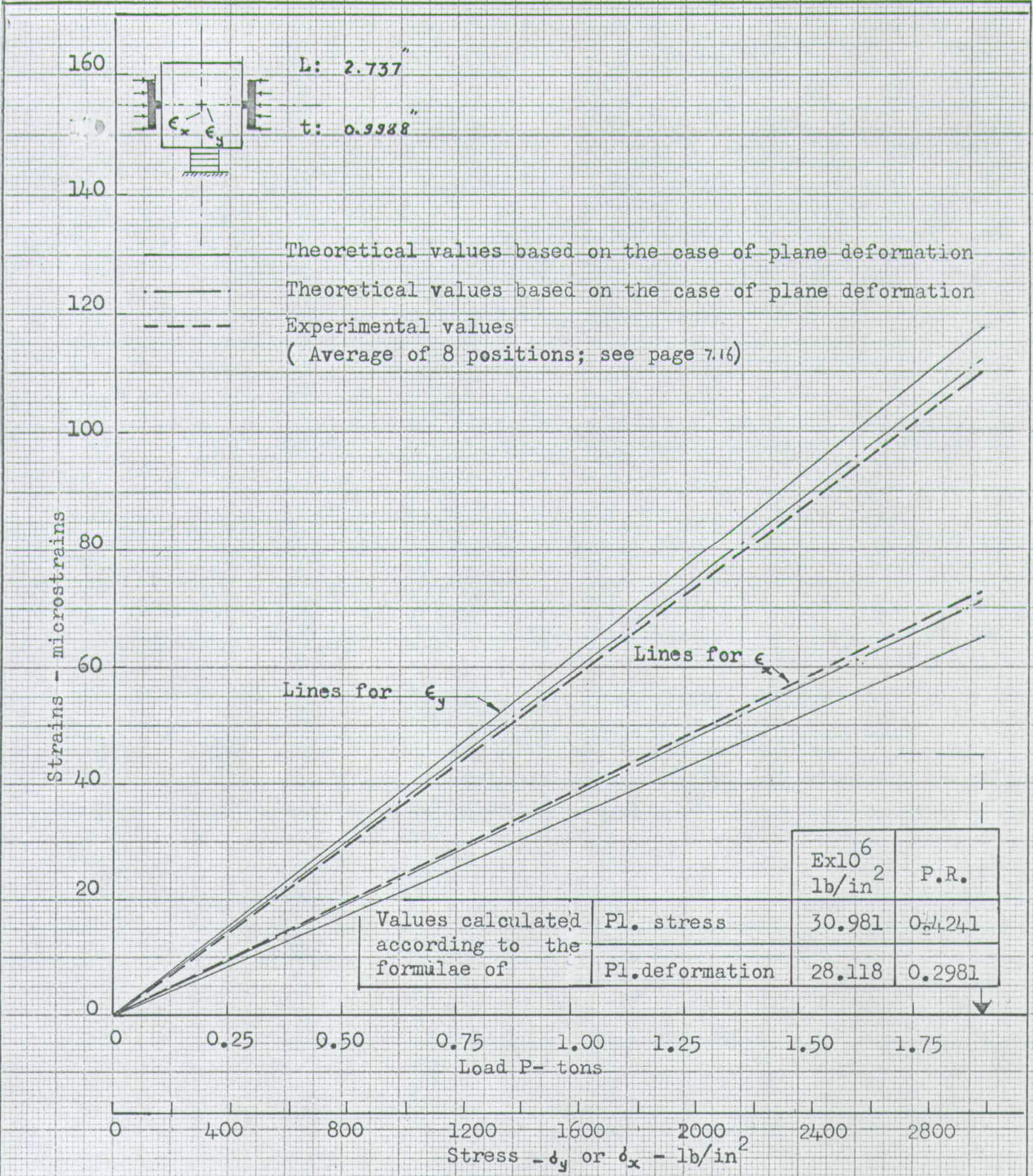


Figure 7.17 :

Deformation properties of steel as determined from P_3. (Loading machine : Hounsfield) of two cases out of twelve, the ratio of the value as assessed from a square block to the one from the corresponding circular specimen was found equal to 1 $\pm$ 0.04. Taking into account the slight difference between each pair, in the dimension-ratio this was considered fairly satisfactory to indicate provisionally the similarity. It should be remembered here that the comparison included both applicable and non-applicable cases, in other words irrespective of whether the case formulae were applied correctly or incorrectly.

2. Examining the values of the modulus of elasticity and Poisson's ratio as obtained from square blocks, when considering the proper formulae with the case of valid stress, the results emerged as more satisfactory. The modulus of elasticity and Poisson's ratio, when applying the plane stress formulae with P_1, deviate from the actual properties by 0.85% and 5.9% respectively. For P_3 and the plane deformation formulae, the deviations were 2.15% and 1.75%. Figure 7.17 shows more or less values of the same order. As regards P_2 the calculated values (Figures 7.14, 7.16) are not close either to the plane-stress or the plane deformation cases.

3. In a similar manner to the corresponding circular specimens, it can be said that the blocks P_1 and P_3 behaved respectively according to the plane stress, and plane-deformation cases, while P_2 was indeterminate.

Although the number of specimens is limited, and with consideration for the possible sources of error listed later, it was concluded that a square block can replace a circular one, for the desired purpose.

Table 7.2

Ratio between square blocks and circular specimens values

Pairs of Specimens	Values calculated according to the formulae			
	Plane Stress		Plane Deformation	
	E	ν	E	ν
D_2, P_1	0.9956	1.0143	0.9619	1.0040
D_4, P_2	1.0005	1.0452	0.9965	1.0340
D_6, P_3	1.0384	1.0902	1.0404	1.0577

7.2.7.4. Dimensional ratio (D/t or L/t) limits for the plane stress and plane deformation cases

The conclusions with the square specimens make it possible to consider here, that the outer dimensions of both a circular specimen and a square one can be employed for determining the state of stress, in terms of the relative dimensions of the specimen. In the upper part of Figure 7.18 two values for the modulus of elasticity for each of the specimens D_2, D_4, D_6, P_1, P_2, and P_3 are plotted. The first value is the one calculated according to the plane stress formulae. These values are represented by the full line. The second is the one calculated according to the plane-deformation formulae, These values are represented by the dotted line. Both lines are plotted with respect to the basic

Table 7.3

Summary of stress-deformation properties as calculated from different specimens.

Group of tests	Specimen	Calculated values $E \times 10^6$ lb/in^2	Calculated values ν	Mean calculated values $E \times 10^6$ lb/in^2	Mean calculated values ν	Deviation from the mean % E	Deviation from the mean % ν
a. Determination of the actual values of E, and ν for the steel used.	D_1	27.837	0.2738	27.600 (Actual value)	0.2738 (actual value)	+0.86	0.00
	D_3	27.423	0.2665			-0.64	-2.70
	D_5	27.541	0.2811			-0.21	+2.70

Group of tests		Specimen	Values calculated according to formulae of Plane stress $E \times 10^6$ lb/in^2	Plane stress ν	Plane deformation $E \times 10^6$ lb/in^2	Plane deformation ν	General behaviour: disc, plate* or cylinder, prism*.
b. Examination of the new proposed dimensions		D_2	27.960	0.2860	26.429	0.2235	disc
		D_4	29.020	0.3386	27.138	0.2526	no precise behaviour
		D_6	29.270	0.3527	27.098	0.2634	nearer to a cylinder
c. Examination of similarity between the circular and square specimens		P_1	27.836	0.2901	25.423	0.2244	plate
		P_2	29.034	0.3539	27.044	0.2612	no precise behaviour
		P_3	30.394	0.3845	28.194	0.2786	nearer to a prism
Subsidiary tests using the Hounsfield.	a. (corresponds to "a" above)	D_3	27.958	0.2769	Case of plane deformation is not applicable		disc
		D_5	27.834	0.2828			disc
	b. (corresponds to "b" above)	D_4	30.927	0.3418	27.385	0.2550	no precise behaviour
		D_6	29.872	0.4200	27.710	0.2958	nearer to a cylinder
	c. (corresponds to "c" above)	P_2	29.359	0.3197	27.619	0.2958	no precise behaviour
		P_3	30.981	0.4241	28.118	0.2981	nearer to a prism

* Disc, plate and cylinder, prism correspond, respectively, to plane-stress and plane-deformation cases.

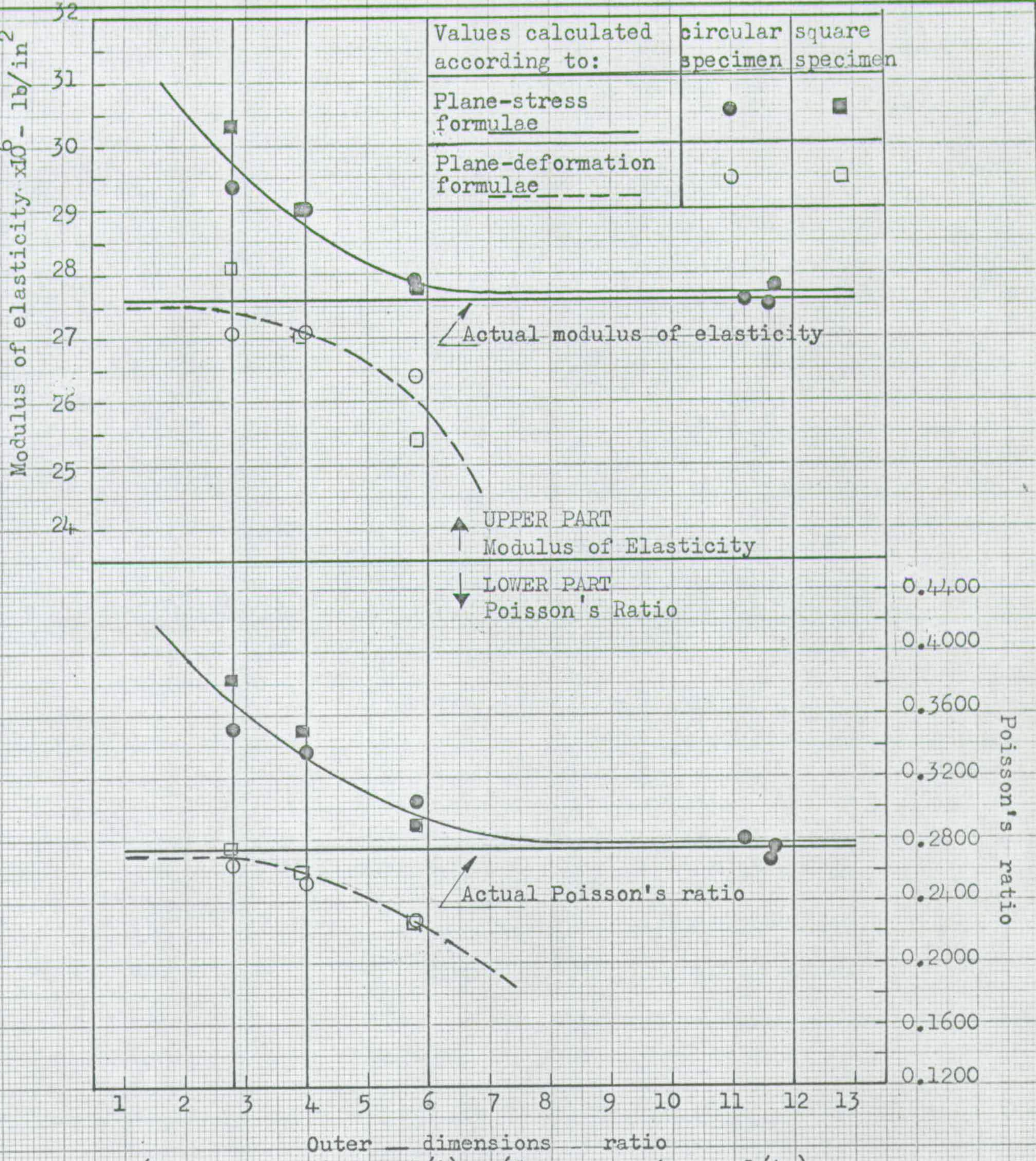


Figure 7.18 :
Determination of the dimensional relations for the cases of plane-stress , and plane-deformation(disc,plate and cylinder,prism)

line which corresponds to the actual value. In spite of the few points, the following two observations can be made with a fair degree of precision:

1. Above a diameter/thickness or side/thickness ratio of 6.5 the full line is an asymptote to the base line. This means that above this value a specimen behaves under the absolute conditions of the plane-stress case. Below this value the curve deviates from the basic line rapidly and with an increasing rate of decrease, as the ratio decreases. This shows that below this dimension ratio a specimen is far away from the case of plane stress, so that the case formulae cannot be applied.

2. Below a dimension ratio value of 2.25 the broken line is an asymptote to the base line, so that it can be said that a disc or square block having a dimensions ratio below 2.25 behaves absolutely under the condition of plane deformation. Above this ratio, and in a more or less similar manner the plane deformation formulae can be absolutely misleading if applied.

3. Between a dimension-ratio of 2.25 and 6.5 neither the plane stress or the plane deformation formulae can be applied. The scattered results around the basic line show this clearly, and this is why D_4, P_2 were described before as behaving according to no precise manner as regards the case of stress.

Shifting now to the lower part of Figure 7.18, the same remarks can be made as regards the behaviour of the specimens, and the calculated values for Poisson's ratio. In fact it can be said that the upper and lower parts of the figure are absolutely identical, with a slight variation in the limits. That is the dimensional-ratio 2.25 and 6.5 are replaced respectively by 2.8 and 7.0. Due to the fact that it cannot be judged which of the parts is responsible for the difference in values, it is conceivable that the average limit should be considered. In other words the lowest dimensional-limit for the plane-stress case, and the highest one for the plane-deformation case are 6.75 and 2.6 respectively. Here it becomes obvious that the ratio used by Hondros (24" diamter, 2" thickness and dimensional ratio 12) was far above the limit.

7.2.7.5. Final choice of mortar specimens

As has been concluded in the above paragraph the dimensional-ratio limits for a mortar specimen can be 2.6 and 6.75 for applying the formulae of plane deformation and plane stress respectively. Considering mortar testing, the 4 inch specimen (dimensional-ratio = 4) was absolutely excluded, and two cases were considered as follows:

1. The first is by employing a specimen of dimensional-ratio = 2.6. This could be achieved either by increasing the thickness of 2.78" - specimen to about 1.1" which makes the dimensional ratio just below 2.6 or by decreasing the side length to 2.6". Due to the fact that the idea was to use the available cube moulds, the latter solution was excluded. Again considering the former carefully it was not found suitable for the proposed mortar testing for one main reason. Referring back to equations 5.32, 533, for calculating the modulus of

elasticity, it can be noticed how the slightest error in the value of Poisson's ratio, can affect the values of modulus of elasticity, and this is why it was hoped at the beginning to avoid the plane-deformation case. Consequently the 2.78 inch specimen was also excluded.

2. The second is employing a specimen having a dimensions-ratio of 6.75. This could be achieved either by decreasing the thickness from 1" to about 0.90" or preparing specimens of 6.7" side. However keeping in mind the same idea of using the available 6" cube moulds and with no desire to reduce the thickness to less than 1", the tolerance in the calculated values allowed these dimensions to be used. From Figure 7.18 this tolerance can be calculated as follows. For the modulus of elasticity it is $\frac{0.2 \times 10^6}{27.6 \times 10^6} \times 100 = 0.725\%$

For Poisson's ratio it is $\frac{0.0194}{0.2738} \times 100 = 7.08\%$

These values, when considered with the sources of error listed in the following section, made it justifiable to conclude that a 6 inch square plate can be used for mortar testing, with the plane-stress state of stress totally applicable.

7.2.8. Sources of Error

Before coming to final conclusions it is worth listing possible sources of error which might be responsible for such deviations. Although every effort was made to minimise the effects of these sources of error, it proved to be impracticable to eliminate them entirely. A summary of them is as follows:

1. The gauge centre lines cannot be said to coincide absolutely with the centre lines of the specimen, because of their relative dimensions, and the use of a hand magnifier during application.

2. The gauge reading is over a length equal to 0.043 - 0.62 of the diameter or side of specimen, while theoretically the reading is at a point.

3. The minimum reading on the scale of the measuring apparatus is 5. Therefore any value less than 5 was subject to visual judgment. This affected the readings, especially at the early stages of loading.

4. Uneven bearing which might have occurred due to slight defects in machining, or the loading from the inserted plywood.

5. The fact that the actual properties are calculated from values obtained experimentally from tests which were subject to the same sources of error mentioned above.

6. The fact that loading a circular specimen is, theoretically, nearer to a line loading than with a square specimen. Although the range 0-D/12 as loading width was shown to be practically the same as regards the centre strains (Figure 5.1.d), it might introduce slight differences expecially with smaller diameters.

7.3. CONCLUSIONS

As a general conclusion, it can be said that the proposed technique has been justified by the experimental examination. On the basis of the present test results, and from theoretical considerations, the following can be summed up:

1. Consideration of the centre conditions, emphasis that a homogeneous and isotropic disc compressed diametrically behaves in accordance with the theory of elasticity.

2. The indirect tension test can be employed for the measurement of stress-deformation properties.

3. Homogeneous and isotropic square specimens compressed along the centre line and perpendicularly to the sides are very similar to circular specimens in both cases of plane-stress and plane deformation, and can be employed for assessing.

a. Tensile strength

b. Modulus of elasticity

c. Poisson's ratio

4. A square specimen allows an easier laboratory technique, but on the whole, the technique is not so easy as was expected.

5. Whether a circular or square specimen is used, consideration of the outer dimensions, expressed in terms of the dimension-ratio, is of vital importance when measuring the deformation properties. For a disc or plate and a cylinder or prism the dimension-ratio should be higher than about 6.7 or less than 2.4 respectively.

6. To minimise the deviation of the average strains given by an electric strain gauge from the actual strains at the centre, care must be taken in the choice of the gauge length in relation to the specimen dimension. In fact this

should be generalised as a rule for any other measuring gauge.

7. Finally from the calculations side the proposed technique was considered more reliable and convenient when applied to the plane-stress conditions. Therefore for assessing the stress-deformation relations and the tensile strength of mortars, the specimen proposed is a square plate 6" x 6" x 1".

C H A P T E R 8

EXPERIMENTAL INVESTIGATION OF THE PROPERTIES OF SOME CONVENTIONAL MORTARS AND THEIR INFLUENCE ON THE STRENGTH OF BRICK MASONRY ASSEMBLAGES

8.1 ORIENTATION

The objects of the experimental work reported in the first section, and analysed in the subsequent sections of the present chapter were:

1. To investigate, comprehensively, the structural properties of some conventional mortars.

2. To give two experimental methods for testing bricks (model bricks) in a more adequate manner, the question posed at the end of chapter 6. At the same time, to find on the scale covering the wide range of different joint materials the location of ordinary mortars.

3. To find experimentally the mortar properties having the greatest influence on the failure characteristics of small masonry wallettes and prisms. In the same time to examine the extent to which the strength of brick masonry may be sucessfully predicted using the theoretically derived formulae.

The chief variable was the type of mortar but the complete set of tests was as follows:

1. Mortar tests: Measurements of deformation and mechanical properties with a total number of about 250 specimens,

2. Brick tests: Measurements of deformation and mechanical properties with a total number of 280 specimens.

3. Assemblage tests: Failure characteristics of brick mortar assemblages (wallettes, and prisms) with a total number of about 50 specimens.

SECTION ONE: DESCRIPTION OF EXPERIMENTAL WORK AND TEST RESULTS

8.2 MORTAR TESTS

8.2.1. General

As the varieties of bonding agents and types of sand, and their possible combinations are beyond the scope of the present project, the materials were limited to one type of each.

8.2.2 Materials

8.2.2.1 Sand

Dry Leighton Buzzard sand was used throughout the tests. The average results of the sieve analyses carried out on three samples of the batch used are summarised in Figure 8.1. The test was carried out in accordance with B.S. 1200: 1944 (15)

8.2.2.2. Bonding Materials

1. The cement used was rapid-hardening Portland cement under the commercial name Ferrocrete. One batch was used throughout the present series.

2. Hydrated lime in the form of powder was used. Its handling and storage followed the same procedure as the cement.

8.2.2.3. Mixing Water

Ordinary water from the tap was used

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(15): British Standard Institution. B.S. 1200: 1944. Natural sands and crushed natural stone sands for brickwork (plain and reinforced) and for masonry.

B.S. Sieve designation	3/ 6"	No.7	No. 14	No.25	No. 52	No.100
Percentage passing by weight (average of 3 samples)	100	100	100	86.66	7.58	1.33

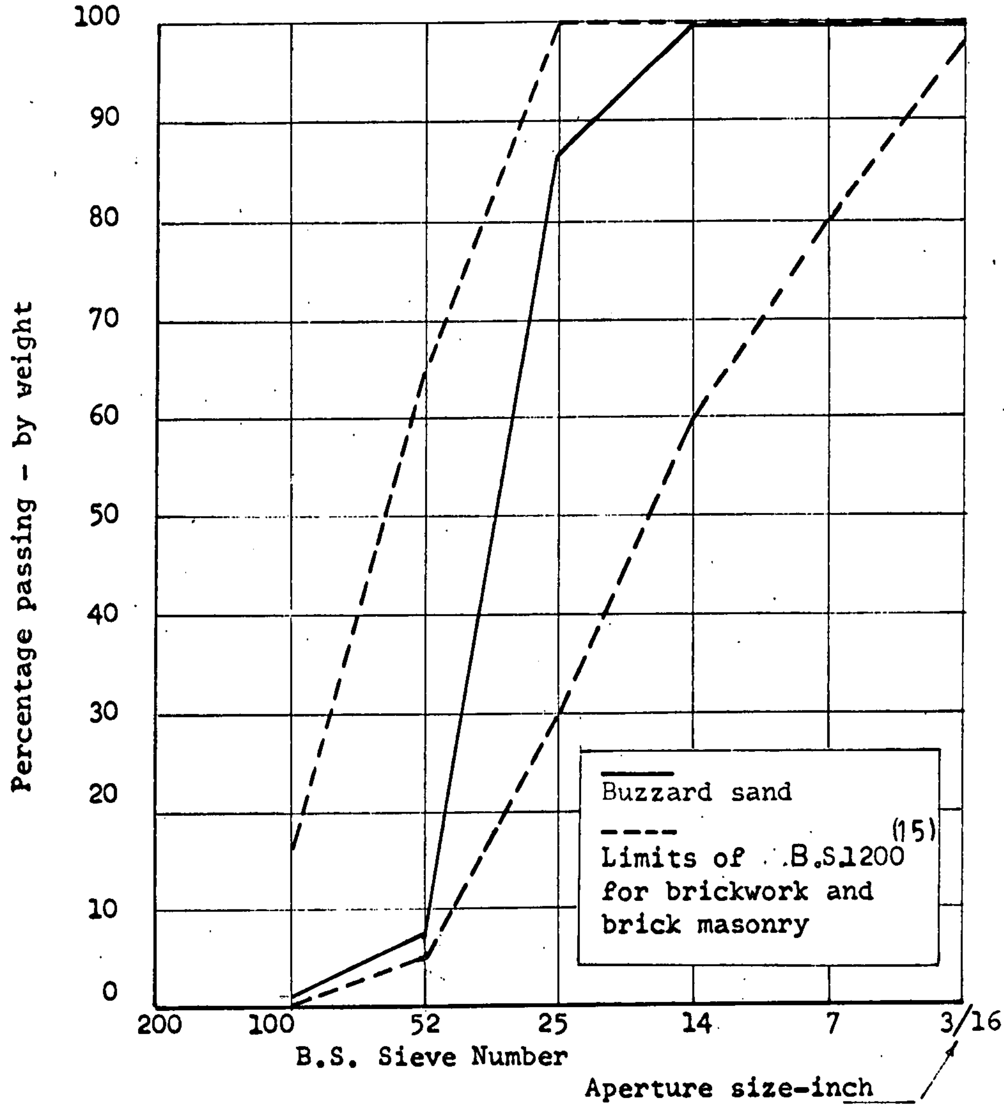

FIGURE 8.1

Grading curve of Leighton Buzzard sand.

for mixing. Water contents of mortars were calculated on the basis of dry sand and saturated bricks.

8.2.3. Mortar Mixes

8.2.3.1. Chosen mixes

In determining mortar mixes it was decided to choose them from among the wide range of mixes usually recommended by scientific or advisory authorities. The five mixes given at the top of Table 8.1 had been used before at the Building Research Station[24], and are among the mixes recommended by both the Station and the Ministry of Works[93].

8.2.3.2. Proportioning of solid materials

For greater accuracy the solid materials for the chosen mixes were converted from volume proportions into weight proportions, on the basis of volume weights given in the B.S. Draft[19] for testing mortar. These weights for sand, cement, and lime are respectively 105, 90 and 36 lb/ft^3. It should be mentioned that when determined in the laboratory for the materials used,the weights were in the same sequence of 103, 92 and 34 lb/ft^3. However, it was thought better to conform to practice in this respect.

8.2.3.3. Mortar mixing and fresh mortar testing

Mixing was done in a mechanical mixer of the vertical axis drum type, mixing procedure being

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(24):Building Research Station Digest. (2nd series) - 58 Mortars for jointing.
(93): Ministry of Works. Advisory Leaflet No. 16: Mortars for brick. H.M.S.Office, 1961.

TABLE 8.1

Mortar mixes and specimens description

a. Mortar mixes

Mortar designation	Mix proportions: By volume C	L	S	By weight C	L	S	Batching weights - lbs C	L	S	W	Flow: % of increase
X_1	1	0	3	1	0	3.5	30	0	105	18.40	-
X_2	1	$\frac{1}{4}$	3	1	0.10	3.5	30	3	105	19.90	14.30
X_3	1	1	6	1	0.40	7.0	15	6	105	19.50	-
X_4	1	2	9	1	0.80	10.5	10	8	105	17.90	15.40
X_5	1	3	12	1	1.2	14.0	7.5	9	105	17.60	17.00

b. Casted mortar specimens

Property		Shape and size of specimen	Specimens	Particulars
Compression		Cylinder: 4" dia. x 8"height	3	
		" " " "	3	Tested with M.G.A. pads
		Cube: 2.78" side	3	
		" : " "	3	Tested with M.G.A. pads
		Cube: 2.0" side	3	
		" : " "	3	Tested with M.G.A. pads
		Cube: 1.5" side	3	
		" : " "	3	Tested with M.G.A. pads
		Cube: 1.0" side	3	
		" : " "	3	Tested with M.G.A. pads
		Prism: 1" x 1" x 3.0"	3	
		Halves of beams: 1" x(≃2") x 1"	2	
		" " " : " " "	2	Tested with M.G.A. pads
Tension	Direct	Briquette: 1" x C.S.	6	
	Indirect	Plate: 6" x 6" x 1"	4	
	Flexural	Beam : 1" x 1" x 4"	3	
Deformation		Plate : 6" x 6" x 1"	3	Tested with M.G.A. pads

rigorously in accordance with B.S.. 1881: 1952[17]. From the mixer used, it was possible to have the whole quantity required, either for mortar specimens or for bricklaying, from one mix. Water was added according to the judgement of a bricklayer, so that the mortar mixes produced were suitable for bricklaying. The quantities of water added are shown in Table 8.1.

Besides this judgement, just after mixing the workability of each mix was measured by the flow test carried out in accordance with the same draft for mortar testing as previously mentioned. Test results are shown in Table 8.1.

8.2.4. Casting

The moulds were assembled and oiled in accordance with the recommendations given in B.S.: 1881 and the draft for testing mortar. Immediately after mixing, the mortar was placed in the mould and compacted in either one or several layers, depending upon the height of the mould. It should be mentioned here that due to the great variety of shapes and sizes of specimens, it was not possible in every case to follow the same steps of mechanical vibration as given in the above-mentioned draft.[19] However, to avoid so far as possible segregation or excessive laitance, all the specimens were compacted on the floor while placed on a wooden sheet, using the Kango hammer. The sizes and number of specimens for each of the mixes were determined as shown in Table 8.1.

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(17): British Standard Institution. B.S. 1881: 1952 Methods of testing concrete.

8.2.5. Curing

Twenty-four hours after mixing, the specimens were removed from the moulds, and immersed in water for two weeks. Then the specimens were taken out of water and left in the laboratory atmosphere for 20 $\pm$ 1 days. There were two main reasons for this peculiar type of curing. The first is that the specimens for the deformation properties were required to be dry before applying the electric strain gauges. Consequently, it was thought better to have all the specimens cured under the same conditions. The second is that the testing period for only one mix was more than two days.

8.2.6. Preparation of Mortar Compression Specimens With M.G.A. Pads

In Chapter 6 the problem of platen friction or end restraint was discussed in detail. In the present chapter this problem is dealt with in a different manner. Compression specimens were tested with the introduction of M.G.A. pads between the specimen and the loading platens. Its function is to minimise the platen restraint, with a resulting strength very near the actual uniaxial compressive strength. The suitability of mortar specimens in both size and shape justified its choice as another attempt.

As regards the first time it was used, and for the extent of its success, reference should be made Hughes and Bahramian(70). It is enough, here, to give its description. The pad consists of a Melinex polyester film, gauge 100, Molyslip grease (containing molybdenums disulphide)

(70): Hughes, B.P. and Bahramian, B. Cube tests and the uniaxial compressive strength of concrete. Magazine of Concrete Research, Vol.17: No. 53. Dec. 1965. pp. 177-182.

and a hardened aluminium sheet, 0.003 in. thick.

The pad was prepared by applying the grease to the aluminium thinly with a paint brush. Then the melinex film was placed on the top of the grease. In use the aluminium sheet was placed against the mortar specimen, and the melinex film against the steel platen. It is clear that any surplus grease is rapidly expelled at the start of the test. Photograph 8.7 shows how the grease fills the voids at the mortar surface.

8.2.7 Application of Electric Strain Gauges on Mortar Specimens

As has already been mentioned, in Chapter 7, the electric resistance strain gauge was the means which had been considered suitable for measuring the strains at the centre. In the majority of tests rosettes comprising two linear strain gauges, perpendicular to each other, were used. Linear strain gauges were used only with few specimens. Both rosettes and linear gauges were supplied from Budd-Instruments Each gauge, either linear or comprised in a rosette had a resistance of 120 ohms, and a gauge factor 2.07.

As regards the techniques and procedures for applying the gauges, they were discussed in detail in Chapter 7, and need not be repeated here. Only one remark may be added about the preparation of the mortar surface. That is done by filling the surface voids with the cement, and moving a circular rod, under pressure on a rubber sheet on the top of sellophane film. By this means the surface produced was smooth and suitable for carrying out all the steps of gauge application from the beginning. With these tests electric terminals for gauges were used between the gauge and the point of glueing the wire on the specimen, as a further precaution.

8.2.8. Testing of Hardened Mortars

8.2.8.1 Strength tests

1. The compressive strength of the specimens Table 8.3-a was determined using the Instron testing machine, with only one exception. The cylinders and 2.78" cubes of the first two mixes (X_1, X_2) were tested in the Instron up to 10 tons which is the maximum capacity of the machine, but less than the ultimate failure strength of these specimens. Subsequently, compression was applied to destruction using the "Denison" compression testing machine. It should be mentioned here that the 10 tons load was applied by the Instron, in the hope of learning something about the stress-strain curve as with all other specimens. This curve could be determined with varying degrees of success from the load-deformation curve plotted automatically in the machine.

2. The indirect tensile and flexural tensile strengths were assessed by testing the briquettes in the Hounsfield Tensometer. Tension was applied using the proper jaws.

3. The indirect tensile strength was assessed by employing the square plate, newly developed, using the Instron machine and the proper platens.

4. The flexural strength test employed the standard beam,

using the Hounsfield and the proper attachments for a third point loading.

8.2.8.2 Stress-deformation tests

At first the indirect tensile strength was measured, then it was decided to measure the stress-deformation properties up to a load not higher than 30% of the failure load in indirect tension. This decision was based on two main reasons. The first was the desire not to cause destruction of the specimens and consequently the rosettes or gauges, which are worth keeping for further tests in the future, such as to investigate the effect of age on the values of the stress deformation properties or the effect of the width of the loading strip. The second is the creep effect, which needs some type of automatic recording equipment. The maximum load having been decided it was divided into 8-12 increments and strains were measured at each of these increments.

8.3 BRICK TESTS

8.3.1. General

Two types of bricks were used in the present tests; one-sixth scale and one-third scale model bricks. The former was the same batch, but the latter were from a different batch to that used before in Chapter 6. As mentioned at the beginning of this chapter, the opportunity was taken to make brick testing more comprehensive, especially as regards the wide range of joint materials.

8.3.2 Strength Tests

8.3.2.1 Compressive strength

With the one-sixth scale bricks, tests were done either on large numbers of specimens or by using only two specimens, depending on whether the influence of the joint material had been investigated before in Chapter 6 or not. For the one-third scale bricks, all tests were carried out on a large number of specimens. As regards the preparation of the materials, assemblages and testing, these were dealt with in detail in Chapter 6, so there is no need to mention them again here. But two main remarks should be mentioned; the first is that, because mortar was considered the main variable, and due to time considerations, the suggested study of the effect of height was not included in the present series. In other words all the bricks in the compression tests were tested flat. The second is that the Instron machine was used in testing instead of the machines mentioned in Chapter 6. This was helpful in emphasising some of the main and important conclusions in Chapter 6.

8.3.2.2 Transverse strength

In the determination of the tensile strength of bricks, two of the methods proposed at the end of Chapter 5 were followed. The indirect tensile strength for bricks tested flat and on edge was determined. Steel platens with the proper dimensions of loading strips were prepared. The width of the loading strip in each case is shown individually in Figures 8.2 and 8.3.

8.3.2.3 Stress-deformation tests

One specimen was prepared for this purpose from each size of bricks. The specimen comprises three super ground bricks, assembled together as shown in Figure 8.4 and Photograph 8.1.

8.4 TESTS ON BRICK-MORTAR ASSEMBLAGES

8.4.1 General

All test specimens were one brick in thickness, but in the choice of the overall dimensions of the specimens the influence of specimen size on the strength, as indicated in Chapter 1, and the case analysed theoretically were taken into consideration. However, two main specimens were chosen as will be shown in the next two paragraphs.

8.4.2 Compressive Strength Wallettes

The details of these specimens are shown in Table 8.2. For both types of brick, this was the basic specimen used in assessing the compressive strength. Three like specimens of this kind were built with each type of brick, and each of the mortar mixes.

8.4.3 Compressive Strength Prisms

In addition to each series of three wallettes, two like specimens of compressive strength prism in stack bond were built. The prisms had the same height as the wallettes. This limitation of the number of prisms was due to the fact that the number of bricks was limited.

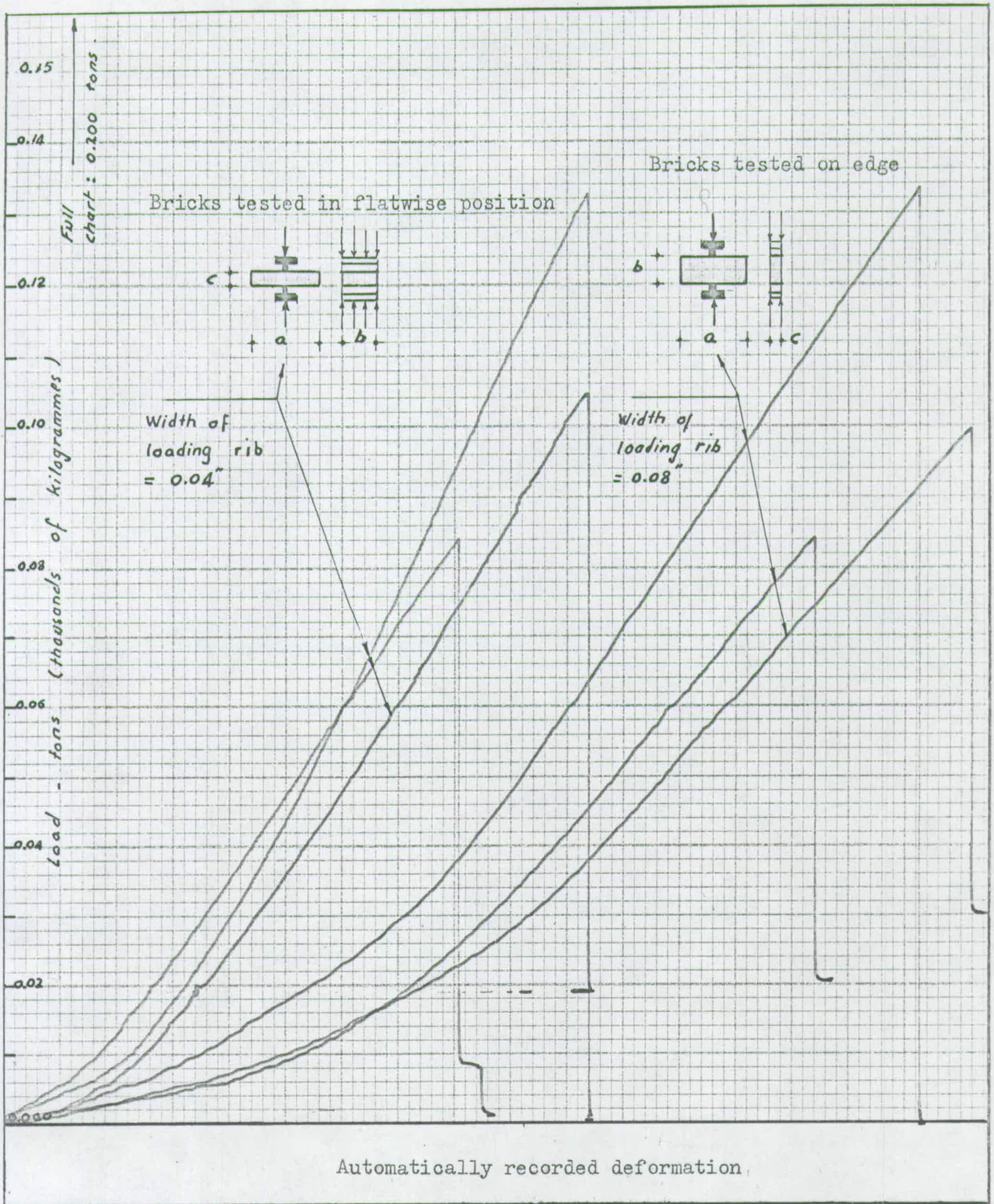


Figure 8.2:
Transverse tensile (indirect tensile) strength of 1/6 scale model bricks.

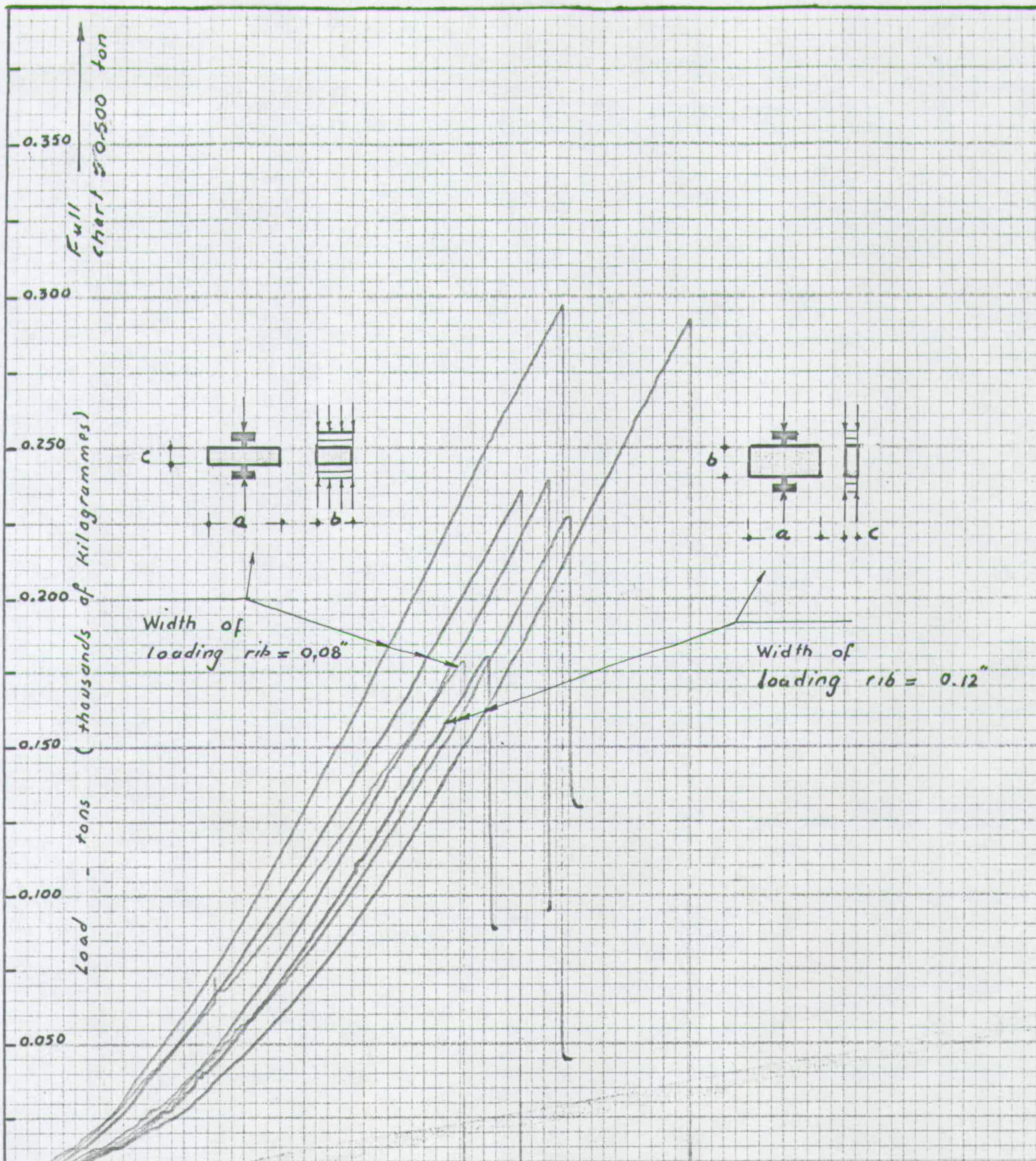


Figure 8.3
Transverse tensile (indirect tensile) strength of 1/3 scale model bricks.

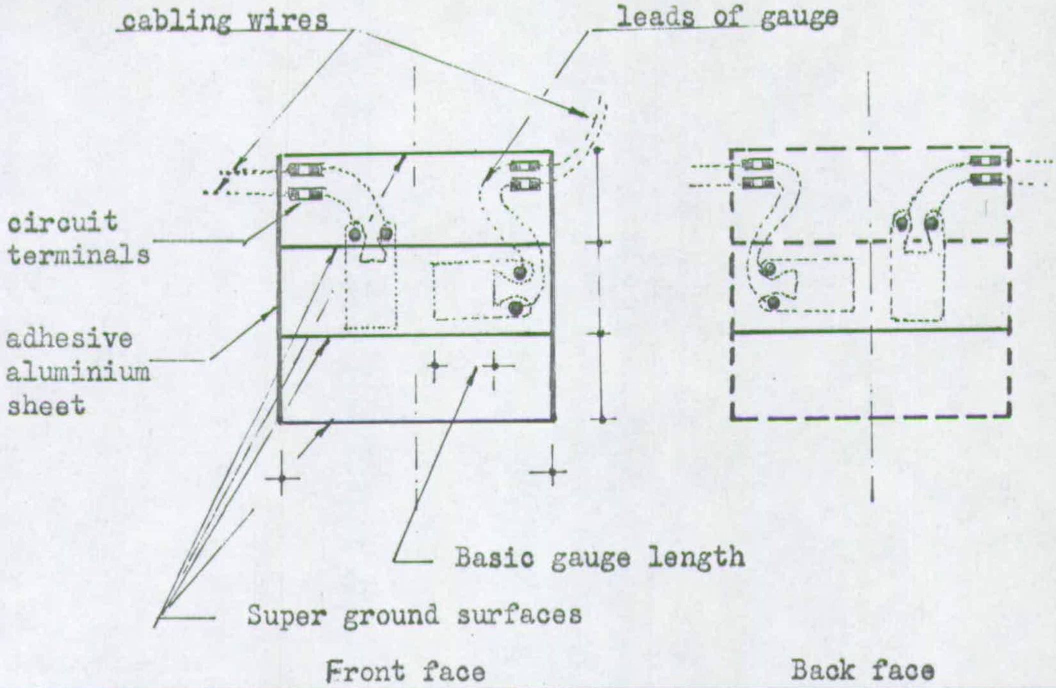
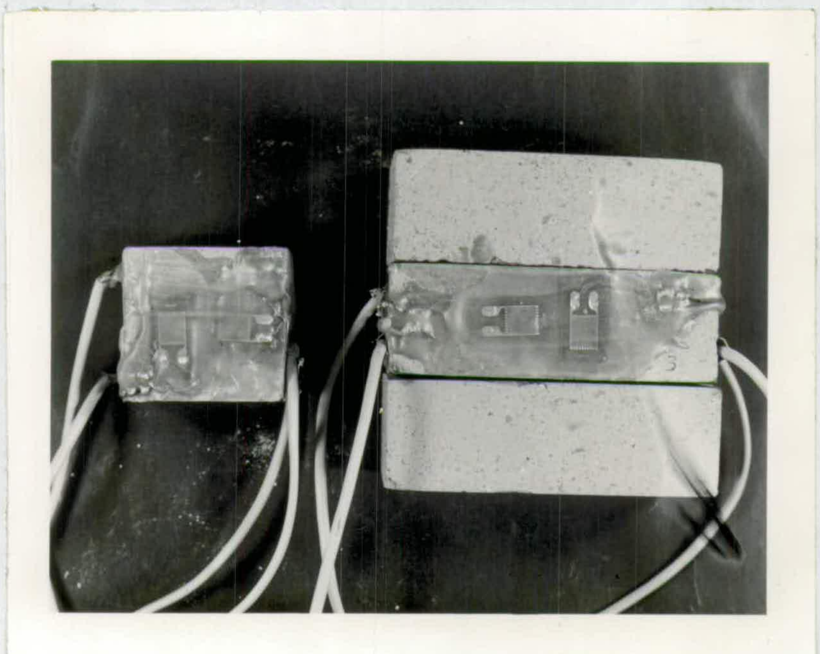


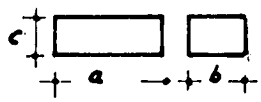
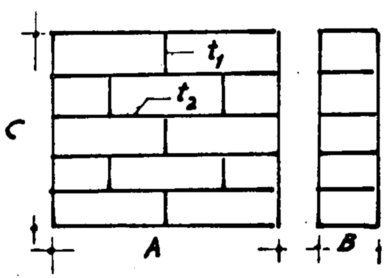
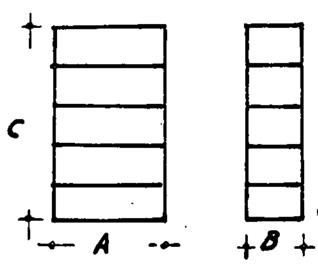
Figure 8.4:
Positioning of electrical resistance strain gauges on one-sixth-scale model bricks. (Scale: full size).



Photograph 8.1:
Brick testing for the determination of the stress-deformation relations.

Table 8.12

Dimensions of bricks and bricks-mortar assemblages

Type of brick / Bricks and specimens		one-sixth scale model bricks	one-third scale model bricks
	a	1.472	2.927
	b	0.691	1.422
	c	0.472	0.979
	A	3.044	6.054
	B	0.691	1.422
	C	2.760	5.695
	t_1	0.1	0.2
	t_2	0.1	0.2
	A	1.472	2.927
	B	0.691	1.422
	C	2.760	5.695
	t_2	0.1	0.2

8.4.4 Bricks embedded in mortar

With some of the mortar mixes (X_3,X_5), in addition to each set of 3 wallettes and 2 prisms, three bricks embedded in mortar were prepared.

8.4.5 Bricks Preparation, Assembling, Workmanship and Curing

Firstly, parts of bricks were cut accurately using a rock cutting saw. The bricks and the parts were immersed in water for 36 hours. Then the specimens were built in specially prepared wooden jigs, which were oiled and coated with melinex having the top face also oiled. The joints were kept to approximately the thicknesses shown in **Table 8.2.** The top end was adjusted so as to be horizontal and parallel to the bottom end using a small steel weight sliding freely between the ribs of the jig. Due to the fact that the ribs of each jig were marked at each course, it was possible to obtain specimens of almost perfect parallelity between the top and bottom ends.

As regards workmanship, the author tried his best to keep it excellent. All end joints were filled and no furrowing of the bed joints occurred. Assembling was done very shortly after mixing, but the period elapsing between mixing and brick laying was kept the same throughout. All specimens were cured under the same conditions as the corresponding mortar specimens.

8.4.6. Preparation and Testing of Brick-Mortar Specimens

Even with the degree of workmanship mentioned above, the required specimen preparation was not tedious, no capping being required. Any non-parallelism of the bearing surfaces was compensated for by introducing M.G.A. pads at the top and bottom of the specimen. As an additional precaution two layers of crepe paper were introduced between the M.G.A. pads and the loading surfaces. From the very beginning with the trial batch, and during all the tests this was found satisfactory. It was noticed clearly when the molyslip grease was squeezed out, more or less uniformly along the long side of the loaded surfaces.

The testing equipment was the Instron machine. Two special platens were prepared for the one-third scale brick wallettes, whose over-all length was slightly more than the diameter of the platen provided with the loading cell. Although all other specimens could have been tested with original platens, they were tested with the new platens in position. This helped to minimise the number of calibration processes. The loading cell was the Instron GRM which was used in mortar testing.

The testing procedure was kept consistent. At first the load was applied to the specimen at a rate of strain of 0.05 cm/min. in low gear, up to a load of a few kilogrammes. The reason for this was to allow slow expulsion of the surplus grease, with better adaptation towards parallelism. Then the load was applied continuously to failure, at a constant rate of strain of 0.1 cm/min. in the same low gear. More detailed curves and accurate readings were achieved by varying the chart speed and the full-scale load from one specimen to another.

8.5 TEST RESULTS

8.5.1 General

As in the previous two chapters, the direct measured results are too numerous to be listed in full. Only a summary of the results in the form of the calculated values will be given here, and in order to facilitate following the discussion the final graphical interpretations will be given later in the proper places.

8.5.2 Strength Properties of Mortars

A summary of the results is given in Table 8.3.

8.5.3 Deformation Properties of Mortars

A summary of the results is given in Figures 8.5 - 8.9.

8.5.4 Strength Properties of Bricks

A summary of the results is given in Table 8.4.

8.5.5 Deformation Properties of Bricks

A summary of the results is given in Figures 8.10, 11.

8.5.6 Compressive Strength of Brick-Mortar Assemblages

A summary of the results is given in Table 8.5.

Table 8.3

Strength Properties of Tested Mortars

(Designation = Cement : Lime : Sand - by volume)

Strength property \ Mortar designation			X_1 1:0:3	X_2 1:$\frac{1}{4}$:3	X_3 1:1:6	X_4 1:2:9	X_5 1:3:12
a. Compressive strength - lb/in^2	Without M.G.A. pads	4" x 8" cylinder +	3412.411	3473.607	839.996	252.652	203.987
		2.78" cube +	3547.214	3717.708	918.771	528.530	348.988
		2.0" cube +	4099.327	4913.704	1358.817	1337.771	786.924
		1.5" cube +	5758.583	5207.091	1369.697	393.666	373.332
		1.0" cube +	5102.202	4494.623	1054.220	574.638	417.253
		1" x (≃2") x 1" half of beam x	6341.152	5890.958	1394.504	823.526	484.233
		1" x 1" x 3" prism	4172.533	2854.891	842.193	464.835	270.117
	With M.G.A. Pads	4" x 8" cylinder +	2884.959	2809.192	916.776	441.486	278.296
		2.78" cube +	3362.513	3212.431	1073.162	552.210	301.205
		2.0" cube +	4227.431	4570.566	1345.092	1207.837	792.415
		1.5" cube +	4717.483	2693.846	1070.381	629.540	300.942
		1.0" cube +	4135.932	2884.172	1068.754	522.655	418.777
		1" x (≃2") x 1" half of beam x	5385.862	5352.921	1357.171	933.330	390.900
b. Tensile strength - lb/in^2 o			548.800	524.533	226.613	137.000	69.200
c. Indirect-tensile strength - lb/in^2 -			470.920	468.589	150.757	74.592	50.706
d. Flexural strength - lb/in^2 +			1267.500	1432.500	435.000	375.000	187.500

"+", "x", "o" and "-" indicate numbers of specimens equal to 3, 2, 6 and 4 respectively.

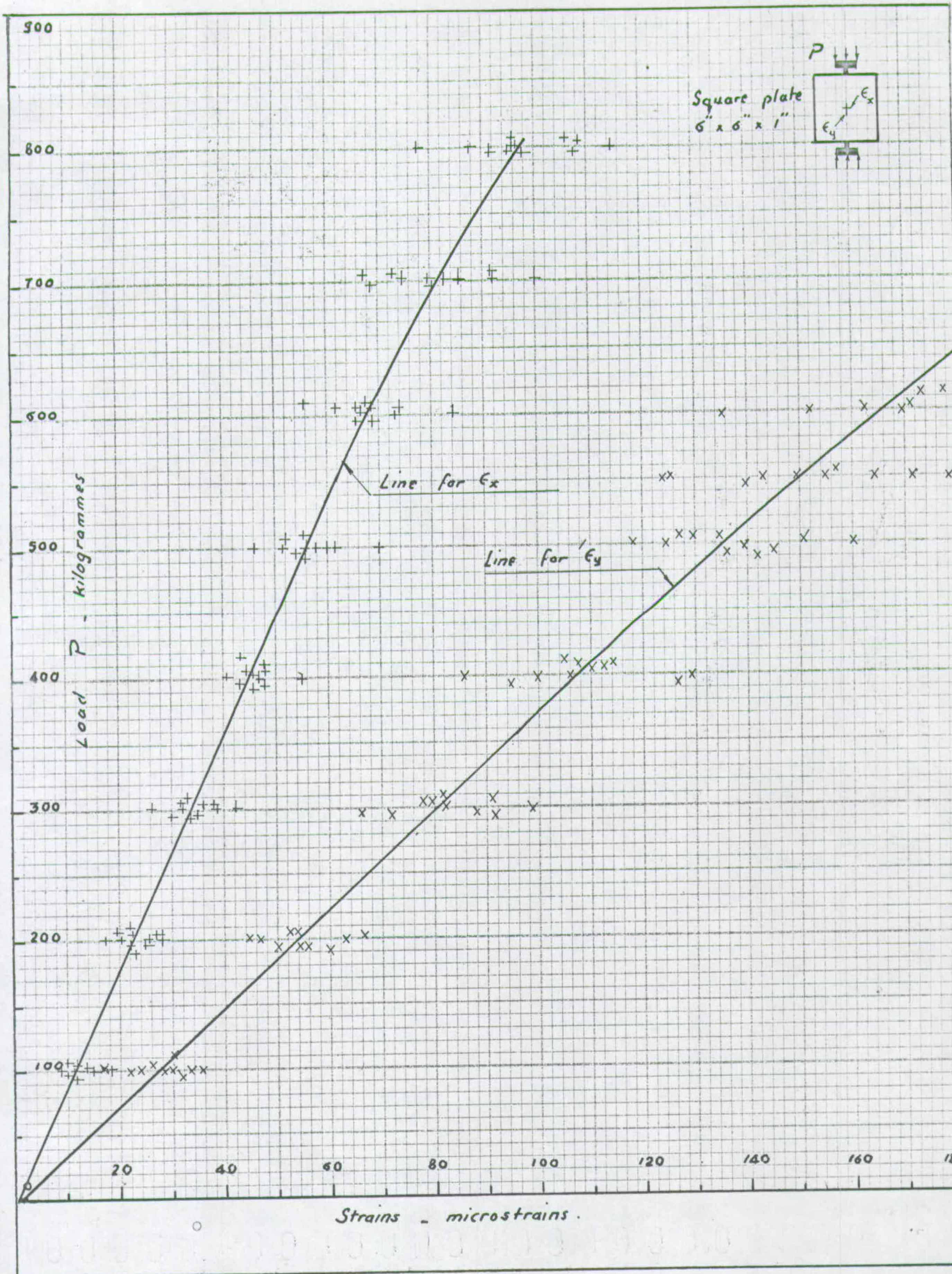


Figure 8.5:
Load-strain relation for mortar mix X_1.

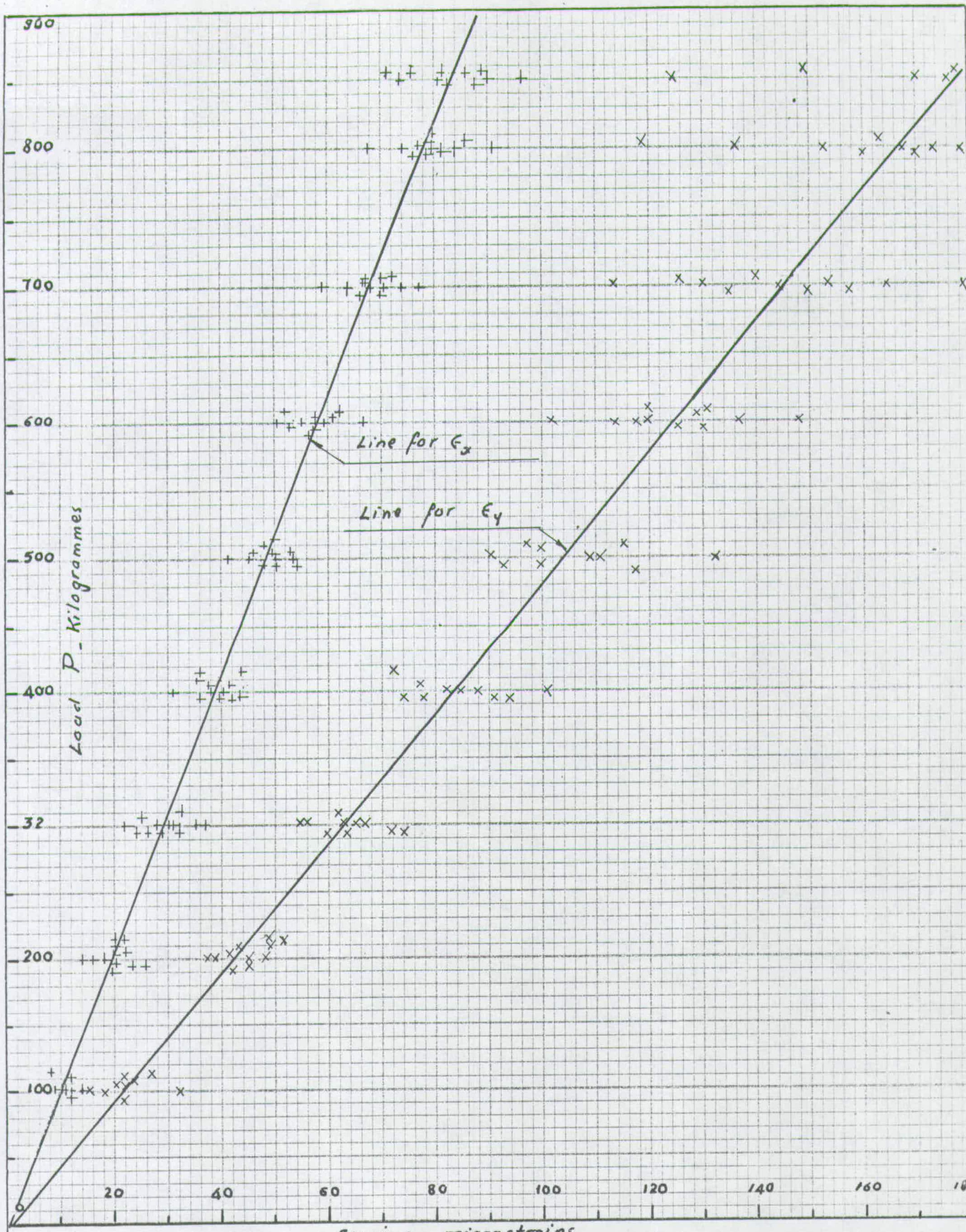


Figure 8.6:
The load-strain relation for mortar mix X_2

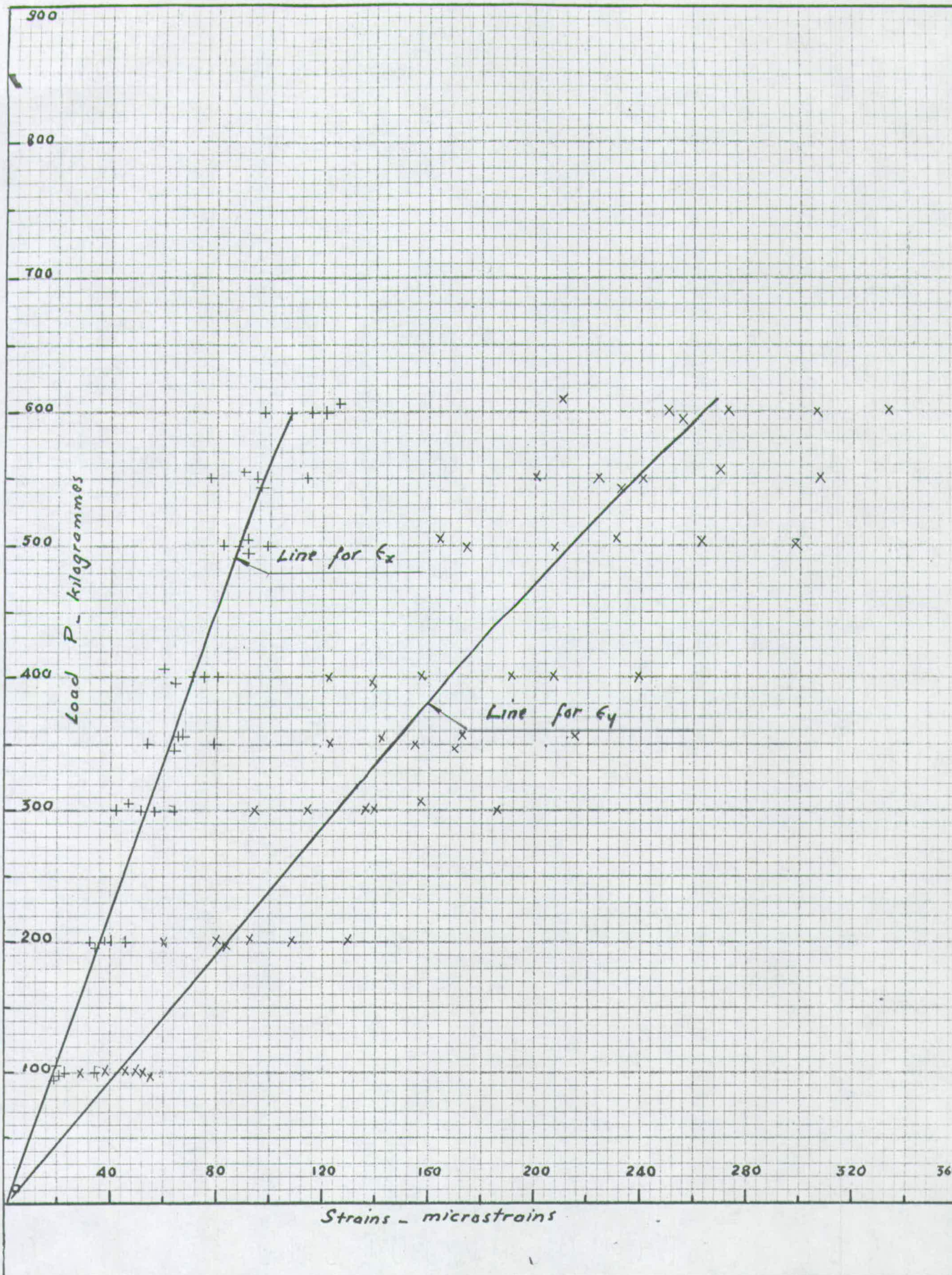


Figure 8.7:
Load-strain relation for mortar mix X_3.

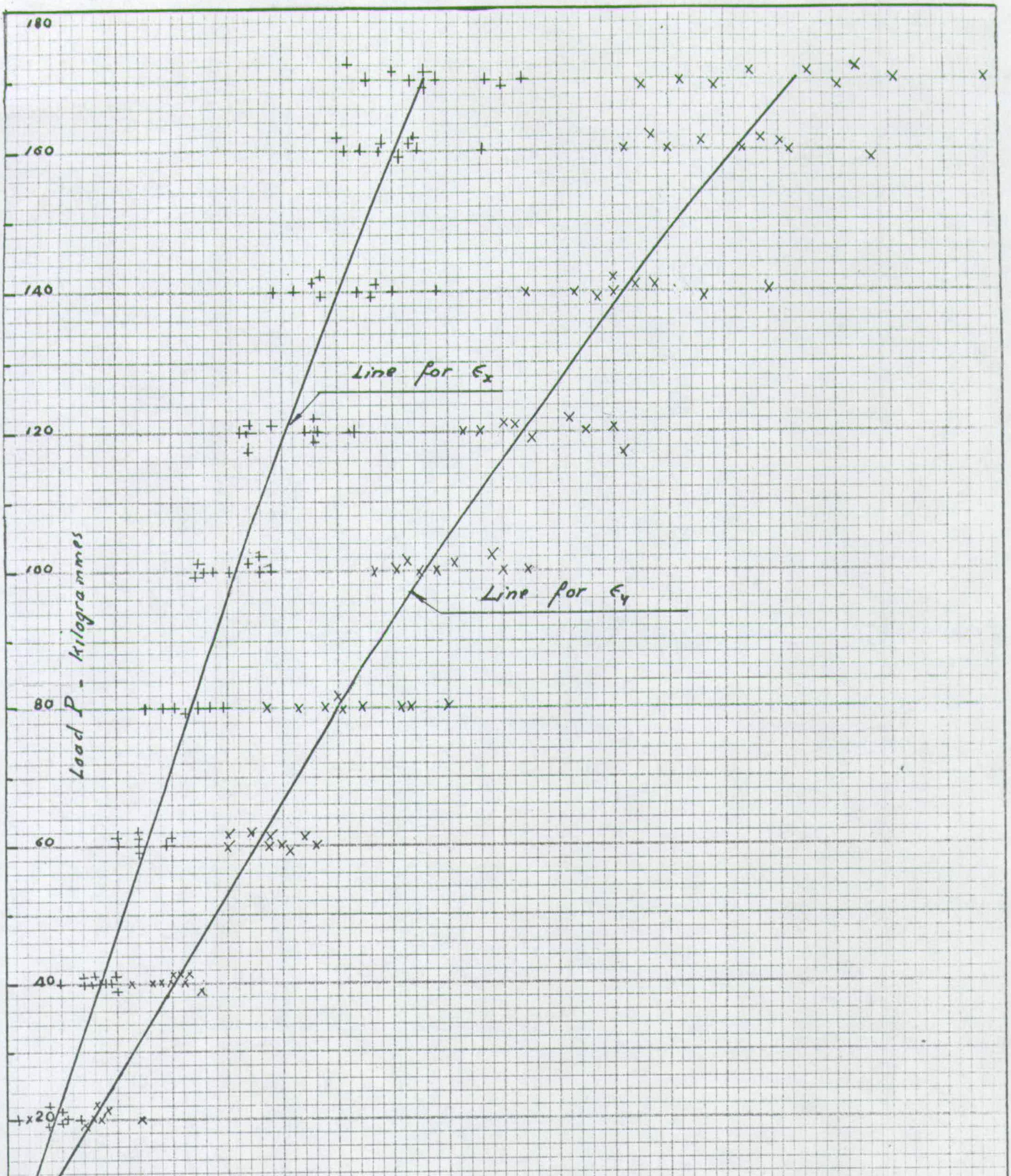


Figure 8.8:
Load-strain relation for mortar mix X_4

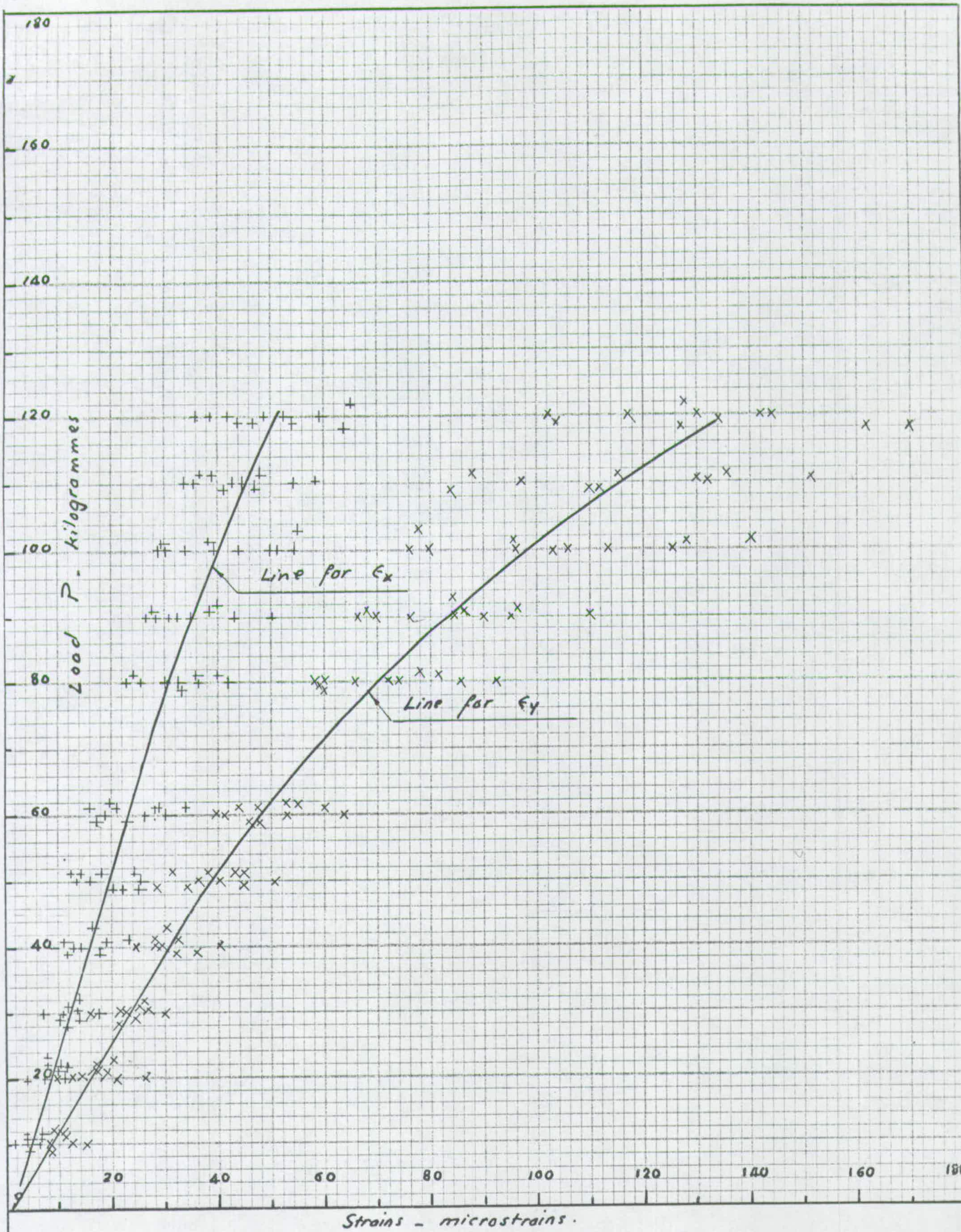
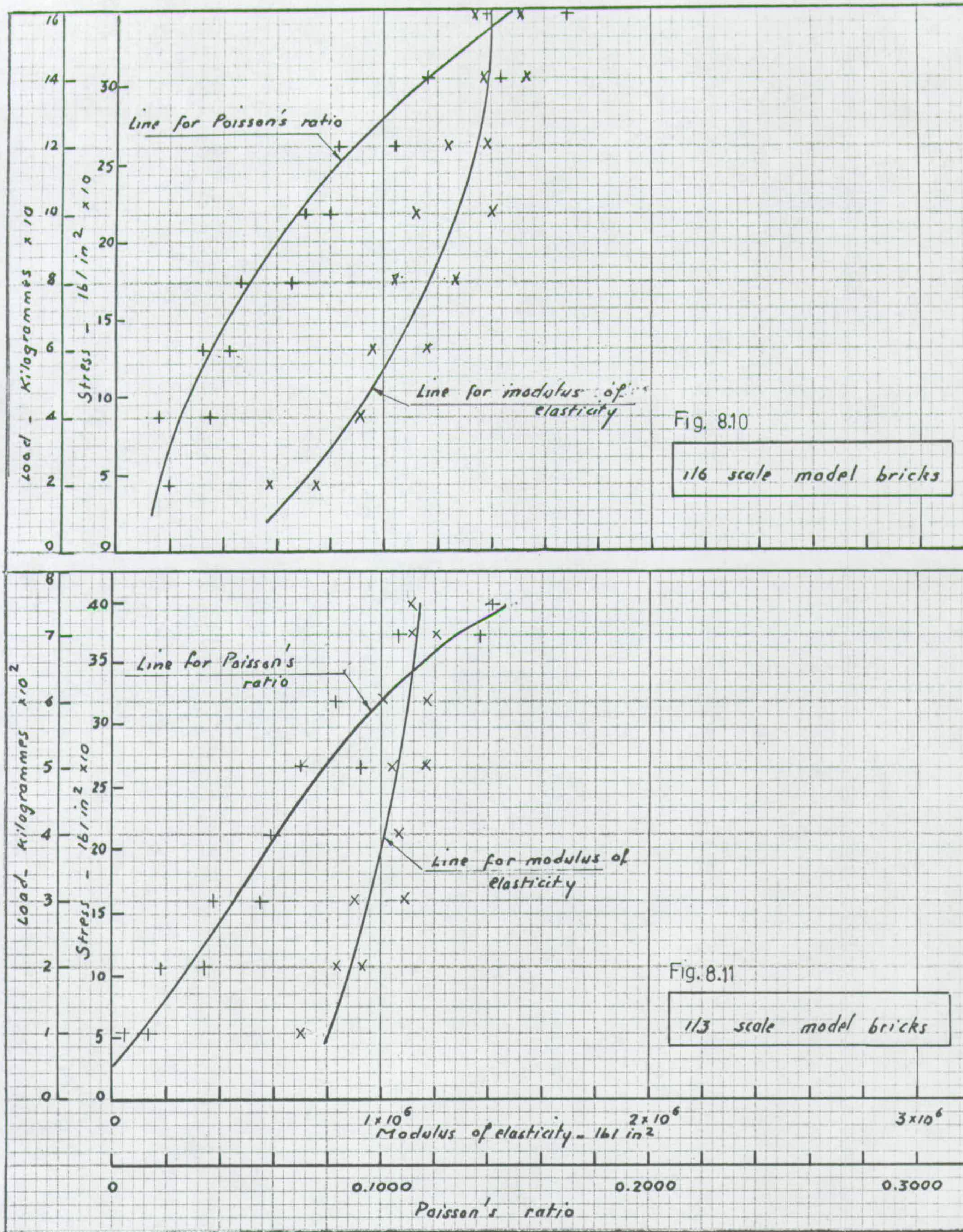


Figure 8.9:
Load-strain relation for mortar mix X_5

TABLE 8.4

Strength properties of Tested Brick

Property	Joint material	Failure strength-lb/in^2		Number of specimens	
		1:6 bricks	1:3 bricks	1:6 bricks	1:3 bricks
Compressive	Steel blocks	7685	5360	7	6
	One sheet of 118" plywood	6040	4854	6	6
	Two sheets of 118" plywood	6183	4891	7	7
	One sheet of 1/8" hard-board	5399	4246	6	7
	Two sheets of 1/8" hard-board	4856	4933	6	7
	One sheet of 1/8" hard-board	5216	3620	6	5
	Two sheets of 1/8"	5180	4565	6	7
	Two sheets of polythene	3950	2907	6	5
	Four " " "	3367	2554	6	5
	Six " " "	3022	2142	6	5
	Eight " " "	2755	2004	6	6
	Ten " " "	2673	1942	6	6
	One sheet of 1/16" rubber	2417	1974	6	6
	Two sheets of 1/16" "	2029	1530	6	6
	One sheet of 1/8" rubber-with-fibres	1727	1634	6	6
	Two sheets of 1/8" " " "	1619	1142	6	6
	One sheet of 1/8" rubber	1230	1049	6	6
	Two sheets of 1/8" rubber	903	692	6	6
	Bricks bedded in X_3(C:L:S 1:1:6)	2612	3004	3	3
	Bricks bedded in X_5(C:L:S 1:3:12)	1018	1916	3	3
Tensile	Bricks tested flat (See Fig. 8.2,3)	453	253	10	11
	Bricks tested on-edge (See Fig.8.2,3	475	248	10	11

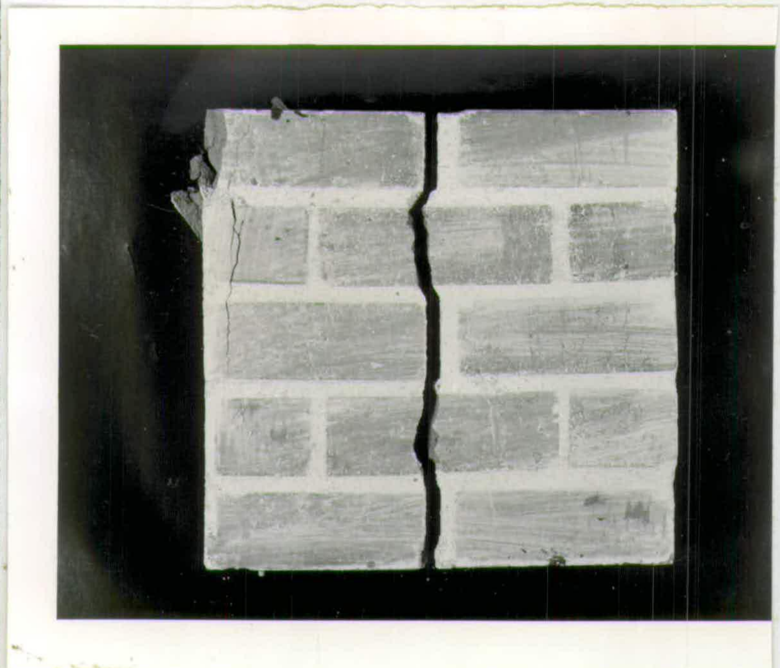


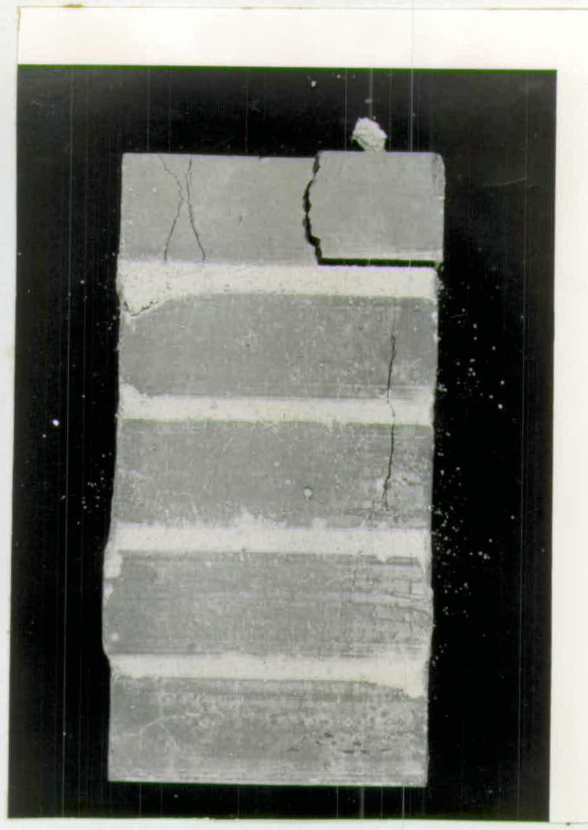
Figures 8.10, 8.11:
Stress-deformation relations of bricks.

Table 8.5

Compressive strength of bricks-mortar assemblages _ lb/in^2

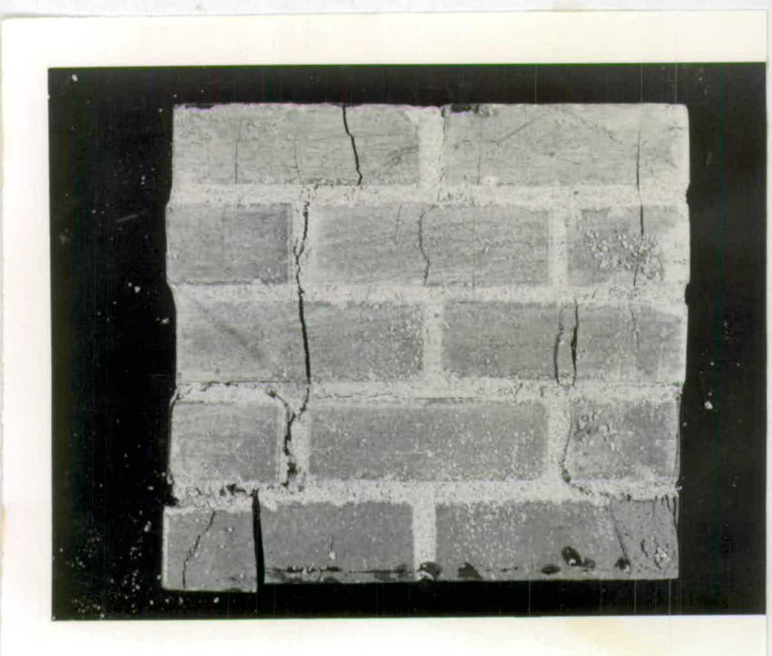
Brick	Mortar designation / Assemblage	X_1 1:0:3	X_2 1:$\frac{1}{4}$:3	X_3 1:1:6	X_4 1:2:9	X_5 1:3:12
One-sixth scale model bricks	Wallette	2923.37	2777.20	2229.08	1600.90	1451.24
	Prism	2052.18	2787.34	2256.21	1770.42	—
One-third scale model bricks.	Wallette	1583.63	1568.05	1286.81	1089.50	895.66
	Prism	2030.42	1438.24	1539.69	950.30	869.21





1/3 scale bricks

1/6 scale bricks



Photographs 8.2 - 8.5:

Bricks - mortar assemblages after testing.

SECTION TWO: AN ANALYSIS OF THE TEST RESULTS OF MORTAR PROPERTIES

8.6 GENERAL

The object of this section is to analyse the data presented in the previous section on mortars. Because of the limitations of space and time a complete analysis which would have entered a new field, "mortar technology", was not possible, but it is hoped that this may be done separately in the near future. The present analysis will be confined to what may be considered to be first concern of the present project.

An important remark should be made here, that, in all the mortar mixes whose test results are discussed, the degree of workability was considered to be roughly constant, being adjusted to approximately the same as would be required in bricklaying practice.

8.7 STRENGTH PROPERTIES

8.7.1 Compressive Strength

8.7.1.1 General observations

From Figure 8.12 some general remarks may be made as follows:

1. Two pairs of mortar mixes can be considered very near each other, "X_1" with "X_2" and "X_4" with "X_5".

2. There is a great drop in strength between "X_1", "X_2" and "X_3", while the drop in strength between "X_3" and "X_4, X_5" is of a lower degree.

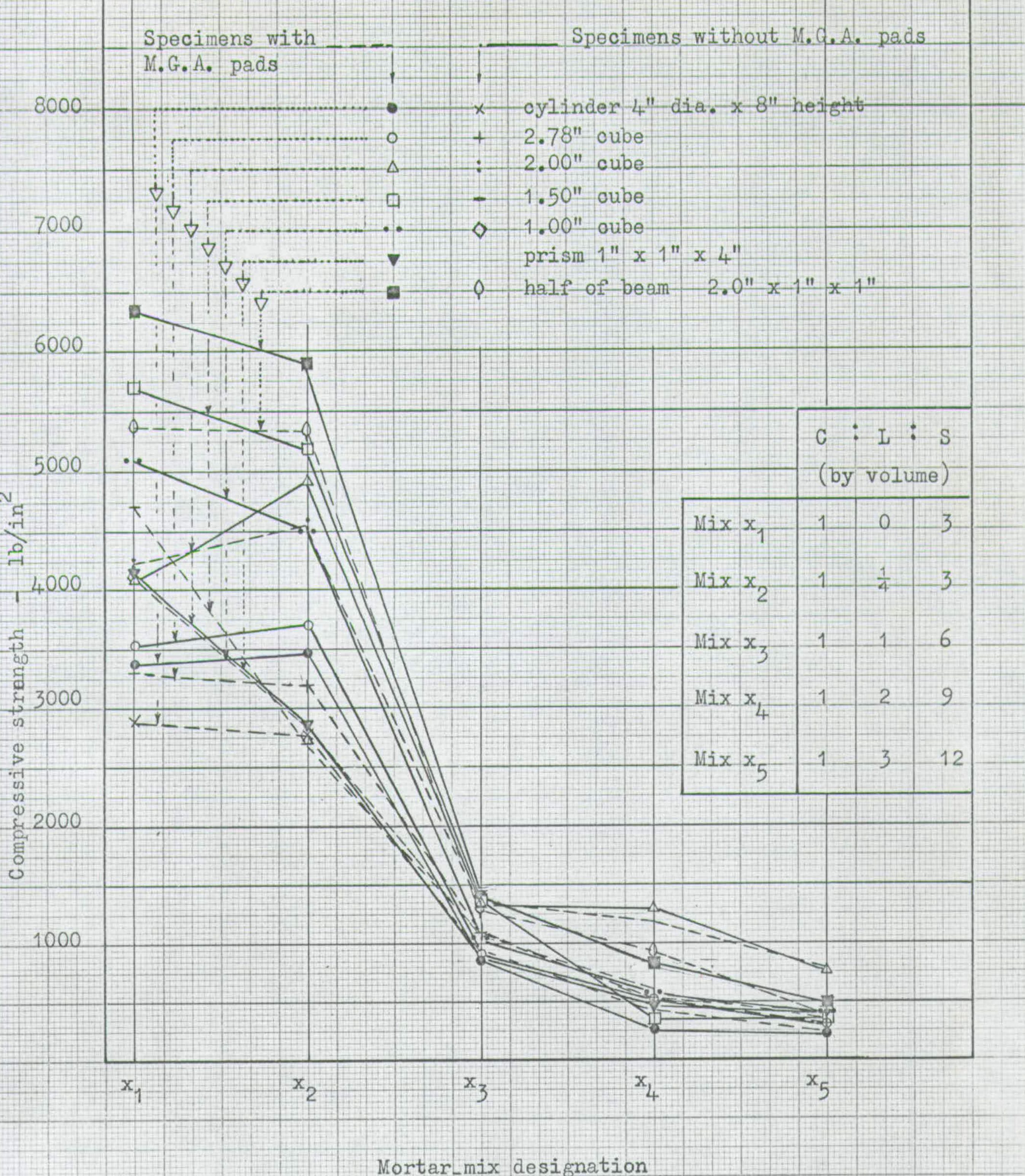


Figure 8.12:

Apparent compressive strength of mortar mixes

3. Considering each mix separately, the variation in strength of mortar specimens, due to the difference in shape and size of specimen is strongly pronounced with "X_1,X_2". With weaker mixes, "X_4" and "X_5", the range is less pronounced, apparently for two main reasons. The first is that the strength values are of a low order, and the second is the congestion of points.

4. Between "X_1", "X_2" and X_2, "X_3" respectively the trend is indicated by the flow of the lines can be considered more or less consistent. Between "X_3", "X_4" and X_4, "X_5" respectively, both the trend and its consistency are to some extent distorted.

8.7.1.2 Effect of shape and size of tested specimens on the apparent compressive strength

It has long been recognised, with materials other than mortar, that variations in size and shape gave different values of compressive strength for the same material. No such information, or very little, is available on mortars. Since the aim of the present section is to examine such influence, all other variables such as mix compaction, testing machine characteristics, and rate of loading that come into play, are eliminated by making them constant. Naturally for the one mix the question of the materials does not arise.

For specimens of different relative shapes it is necessary to have a basic value to which other values should be correlated. Among the specimens tested, the strength of the prism was chosen deliberately as the basic one. At the same time mortar mixes "X_1" and "X_2" were considered enough for the present purpose. Considering the prism strength, (mean value) to be 100, the strength of each type of specimen as represented by the mean value, was correlated to the prism strength. Then the relative strengths were plotted graphically as shown in Figure 8.13. The highest and lowest values were first plotted on the Y-axis, and the corresponding points were connected by a straight line. Each relative strength was plotted on the Y axis and a horizontal line was drawn to meet the line between the extremes at a point (as illustrated by the dotted lines in the same Figure). From this point a vertical line is drawn to determine its projection on the X-axis. This projection was identified by the identification mark of the specimen. The following can be noticed:

1. The congestion of the similar identification marks on the X-axis indicates that the highest and lowest strengths are produced from the halves-of-beams and cylinders respectively.

2. Considering the specimens in order of increasing strength, and apart from the 1" cube, the sequence becomes: cylinder - 4" x 8", 2.78" cube, 2" cube, 1.5" cube, and 1" x 2" x 1" - halves-of-beams. Two remarks can be pointed out about this sequence. The first is that the sequence is obtained for both tests with and without M.G.A. pads. The second is that

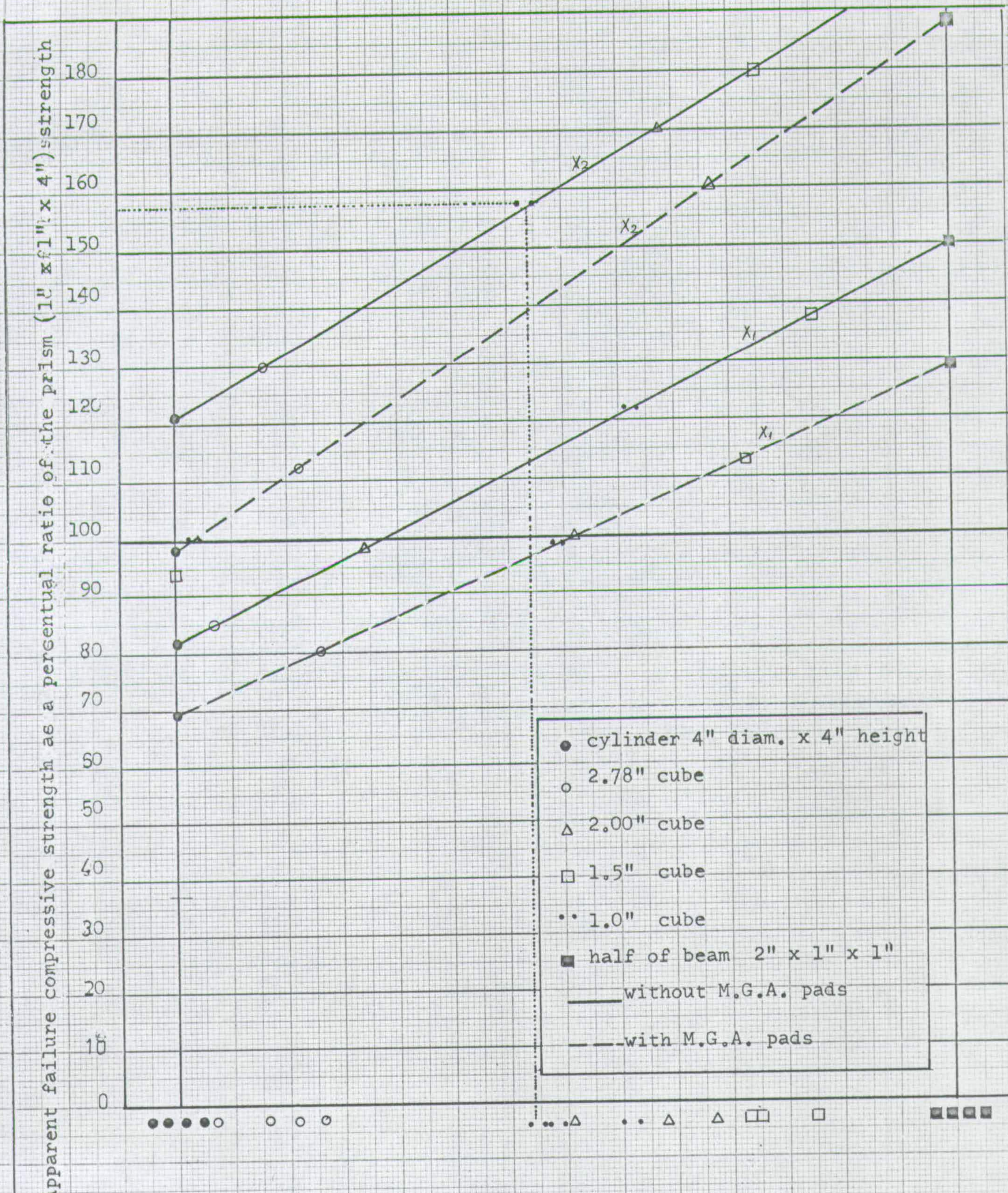


Figure 8.13:

Effect of shape and size of specimen on the compressive strength as a function of prism strength for mortars.

dropping the 1"-cube seemed justifiable, as the general trend indicates that this cube might be situated between the 1.5" cube and half-of-beam. This will be shown later to be inexplicable and due to the small number of specimens, nothing could be confirmed.

3. The slopes of the four lines are very near ear other, so that it can be said that the rate of increase or decrease in strength from a specimen to another, for both mixes, and for both cases, with or without M.G.A. pads, is constant.

4. It is quite clear on Figure 8.12 that prism strength, chosen as the basic one, of "X_2" is slightly lower than the value which can be assumed from the general trend of the flow of the lines. Assuming that this strength was slightly higher, and this is very probable, the calculated relative strengths for "X_2" become of less value. Consequently, the upper two lines of Figure 8.13 might be located at a lower level, to coincide approximately with the lower two lines corresponding to "X_1". Although this is based on mere speculation, the author considers it conceivable. It is unfortunate that the available number of specimens is so small.

As regards specimens of the same relative shape, the cube is the only one available among the specimens tested. It is also well known that the cube is usually considered the easiest one, whatever the available sizes are. From the present mixes an attempt was done with "X_1" - "X_3" to correlate the strength of 2.0" and 1.5" cubes to the strength of the 2.78" cube by considering the latter equal to 100%. The recorded strength of a cube is increased by 20.4% and 19.4% for each $\frac{1}{2}$" when starting from the 2.78" cube and ending with the 2" cube and 1.5" cube respectively. In a simpler manner the general trend between the 2.78" cube and the 1.5" cube is an increase in strength equal to 20% every $\frac{1}{2}$" when starting from the former. But two remarks should be remembered. The first is that this is not valid for "X_4" or "X_5". This might be attributed to the low order of strength, which in such a test shows higher values of the coefficient of variation and again the number of specimens is too small to make any correction. The second concerns the exclusion of the 1" cube from the test results. In the author's opinion these two remarks show the need for further investigation on this point.

A conclusion reached by the American Society for Testing and Materials[111] with concrete is worth mentioning here, because of its suitability for application to mortar. The conclusion was that lower strength concrete requires greater corrections than higher strength concrete. Naturally correction needs a large number of specimens.

8.7.1.3 Effect of the M.G.A. pads on the apparent compressive strength

It has been mentioned in the previous

section that the main object of introducing these pads is to minimize the lateral restraint between the specimen and the machine platens. Due to the many variables in the present series, as regards the shape and size of specimen, and the wide range between the strengths of mortar, the influence of the pads emerged as more significant in some respects, and less significant in others than when it was first used by Hughes and Bahramian(70). From Figure 8.14 the influence of the M.G.A. pads on the strength may be classified into two different types represented in the figure by the upper and lower parts of the graph. This graph may be considered at the moment as a discontinuous one. The remarks can be summarized as follows:

1. All the points of the upper part belong to the mixes "X_1" and "X_2", which, as indicated before, can be considered the strongest of the mixes. With these mixes, it is clear that the pads produce a strength of lower value than the strength obtained from the specimen when tested between the

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(111): Rilem Commission for Concrete. Correction factors between the strengths of different specimen types. Paris, Rilem Bulletin No. 39, 1955 pp. 81-105. Rilem Bulletin No. 12, New Series, 1961. pp. 155-156.

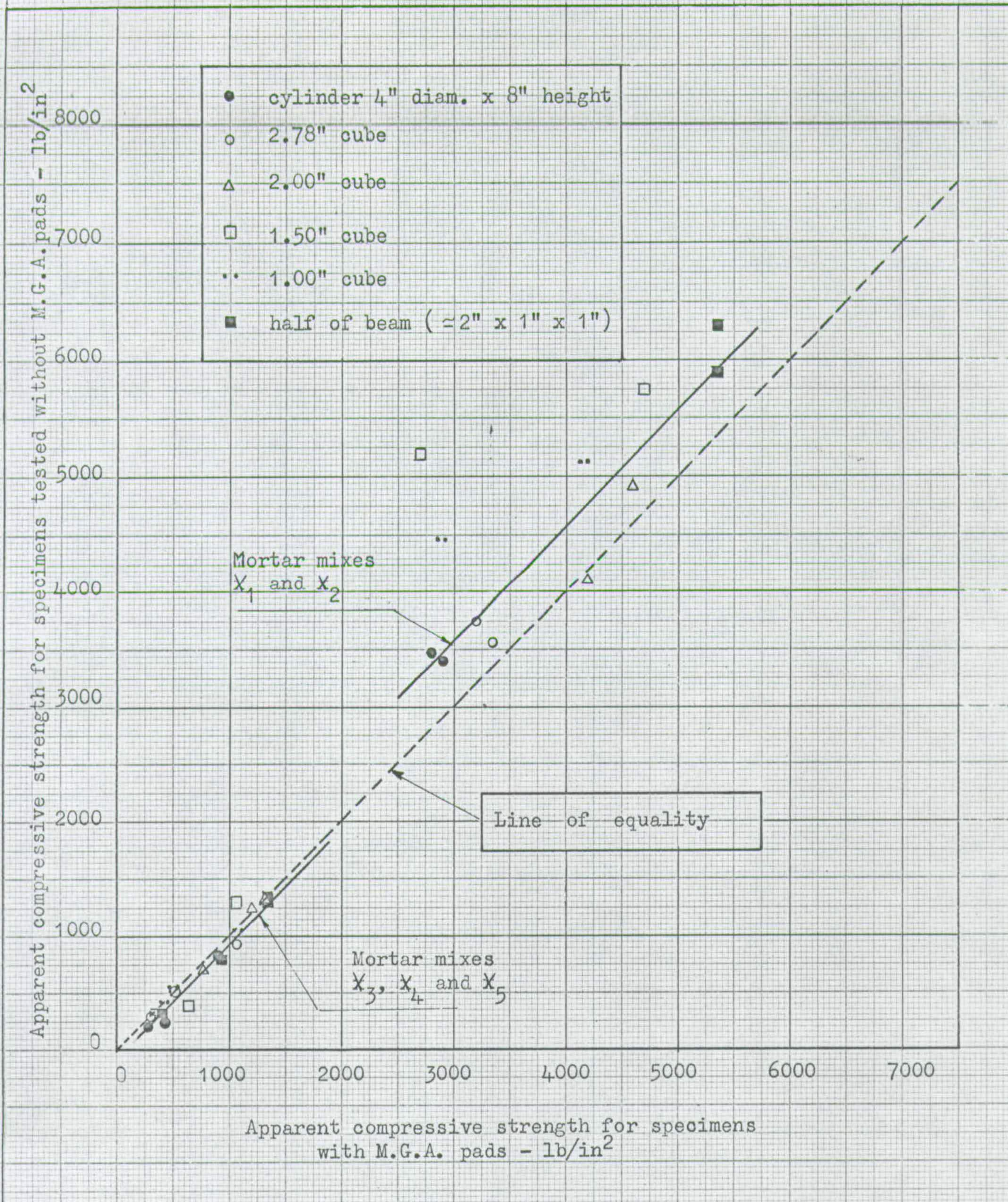


Figure 8.14:
Influence of M.G.A. pads on the compressive strength of mortars.

when tested between the machine platens. This trend complies well with the earlier results of Hughes. It should be mentioned that this can also be noticed from Figure 8.13. In the same part, the influence of the pads, although not definite, appears to vary according to the size of specimen, and especially the cube. As the size of the cube decreases, the percentage decrease in strength with respect to the specimens tested without pads, increases. Considering the state of lateral strains as discussed earlier in Chapter 6 and the influence of insertion of a soft material between the specimen and the machine platens, this influence with its trend seem conceivable.

2. The lower part of the graph illustrates the influence of the pads with the mixes "X_3", "X_4" and "X_5". With these mixes it can be said that the pads may have little or no influence towards increasing the strength. This is indicated by the line being very close to the line of equality. Having no influence might be reasonable, but having the influence of increasing the strength seems peculiar. At the present stage, and in the light of the discussions in Chapter 6, one explanation might be given. That is, with these mixes, the pads acted either as a medium end material, or a hard end material with slight differences in the relative rigidity.

3. Considering both parts of the graph, it may be expected that if the tests cover a wide range of mixes with small

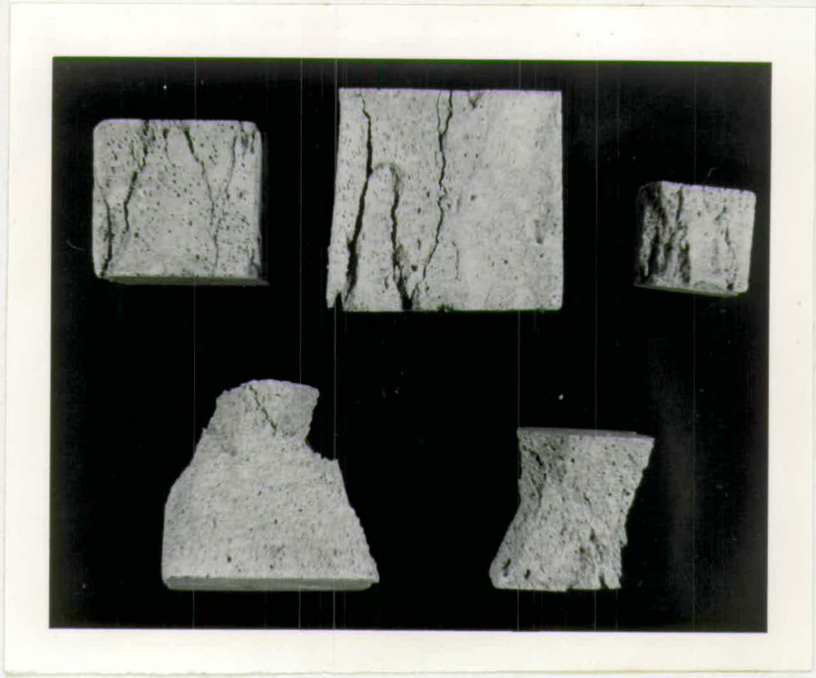
gaps between them, the parts should join each other forming a continuous graph with an inflexion point, probably near the line of equality.

4. Apart from the variation in strength, the mode of failure when using pads was consistently, but sometimes only slightly, different from the mode of failure of specimens without pads. Excluding the cylinder, the form of failure as shown in Photograph 8.6. clearly indicates that cracking occurs in vertical planes parallel to the direction of loading. In fact some similarity exists between the mode of failure of specimens with pads and that of the prism or cylinder either with or without pads. Photograph 8.7 shows how the grease slips to fill the voids.

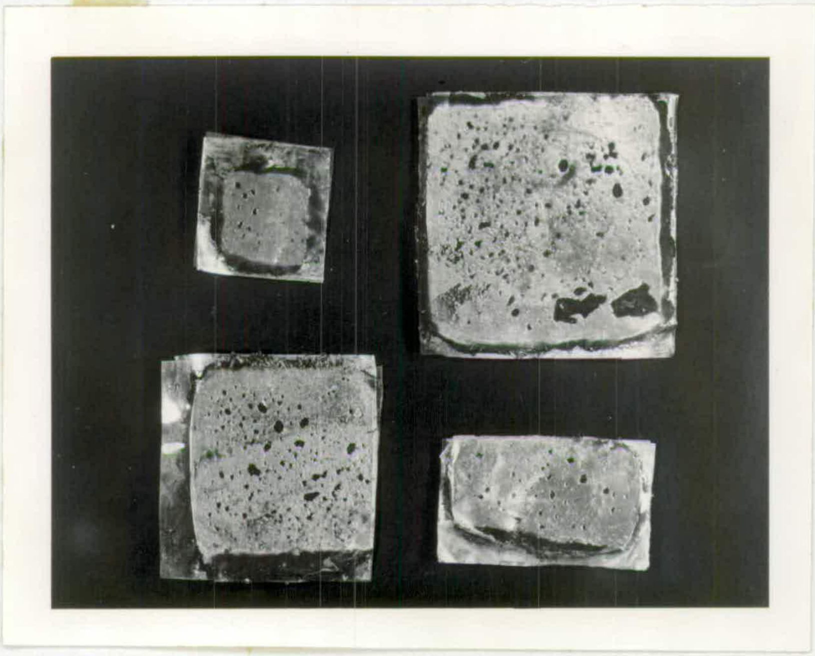
5. As regards the general use of the pads, the previous results of Hughes and the present tests make it justifiable to say that the pads work effectively with mixes of high strength, and not with all grades of strength as was implied before. In other words no general use can be made before further clarification of the matter.

8.7.1.4 Representative values of compressive strength for the mortar mixes tested.

On the basis of the recorded values, and with the object of making the main study conclusive, each of the mortar mixes tested has been considered to have three possible values for its apparent compressive strength. These values are a medium, a higher, and a lower value, and are



Photograph 8.6:
Mode of failure obtained by M.G.A. pads, almost vertical cracking.



Photograph 8.7:
M.G.A. pads after testing. (From the Melinex side).

indicated respectively by "M", "H" and "L". The higher and lower values represent the extreme readings as deduced from the flow of the lines between the points (Figure 8.12) but with a slight modification for the weak mixes. The medium value represents the middle reading between the corresponding extremes. It should be pointed out that from now on these values will be representative of the apparent strength regardless of the shape and size of specimens or the condition of loading (with or without M.G.A. pads). These values, as deduced from Figure 8.12 are considered as (6400- 4650 - 2900), (5900 - 4300 - 2700), (1450 - 1150 - 850), (850 - 550 - 250) and (500 - 350 - 200 lb/in^2) for mortar mixes X_1, X_2, X_3, X_4 and X_5 respectively .

8.7.2. Tensile Strength

8.7.2.1 General observations

Generally speaking the groups of mortars as regards the trend in tensile strengths are exactly the same as in the case of the compressive strength. This is quite obvious in comparing Figure 8.15 with Figure 8.12.

8.7.2.2 The new proposed method and test results

Three topics of importance can be put under this heading. The first is to evaluate the new technique as a method for determining the tensile strength. The second is to study the mode of failure and see how far it is comparable with the modes produced from circular specimens,

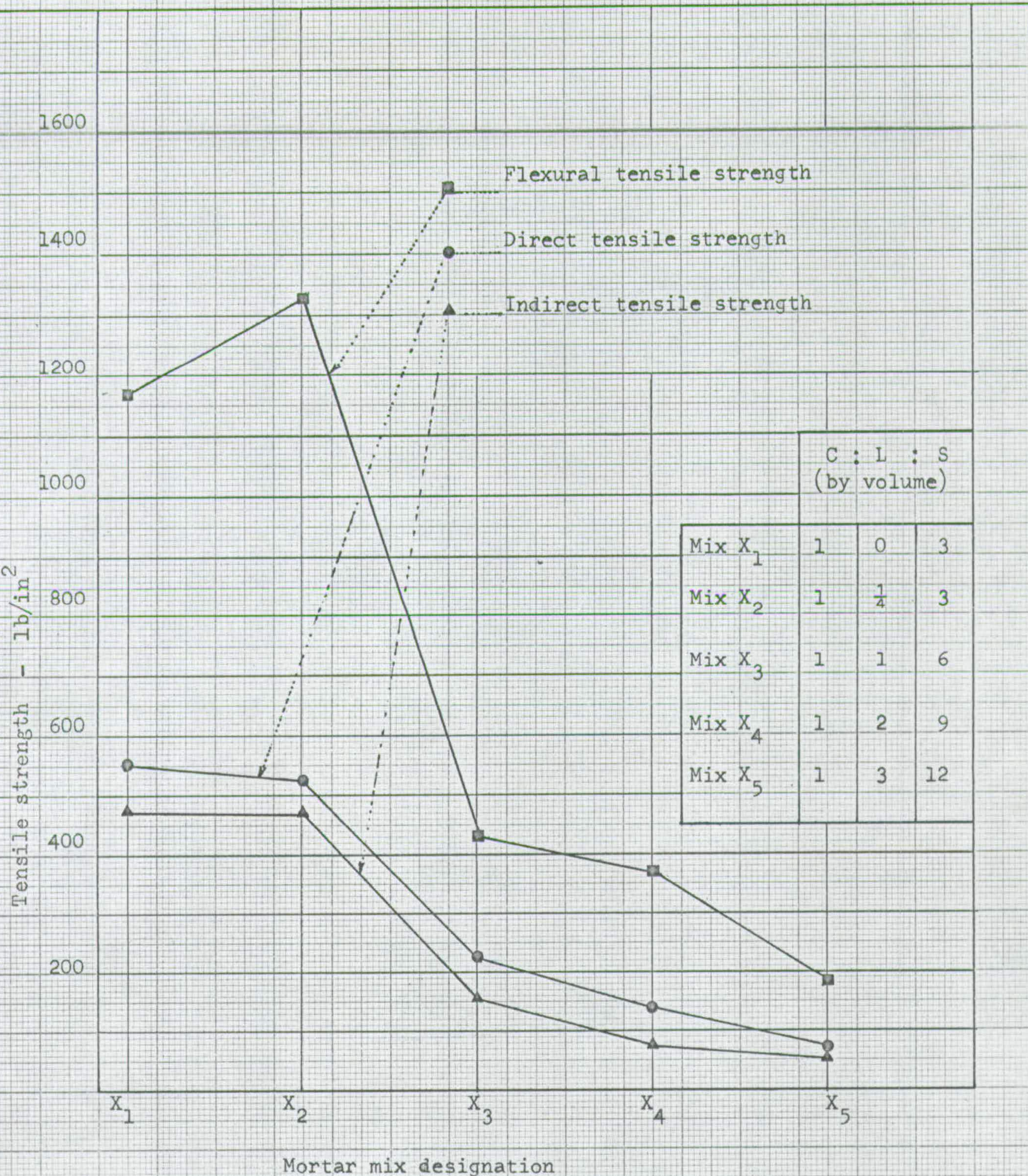


Figure 8.15:

Tensile strength of mortar mixes

The third is to see how far the recorded values can be reliable.

Starting with the first topic, it would have been advantageous if the test results included the indirect tensile strength assessed from cylinders prepared and treated under the same conditions as the square plates. But this was not possible because of the limited number of moulds, quite apart from the great quantities of materials required. Therefore, it was decided to carry out the examination by comparing the relation between the indirect-tensile-strength and the compressive strength as obtained from the present tests, and the same relation as obtained from previous investigations on concrete, when employing cylinders.

From the present tests the relation is illustrated by the three graphs indicated in Figure 8.16 by I_L, I_M, and I_H. Three graphs are produced for the one relation, resulting from the fact that three values for the compressive strength were considered. The three graphs are of the same trend, so that the relation may be defined by the range between I_L and I_H. The same relation as obtained by previous investigators using concrete cylinders is shown in Figure 8.17. Due to the fact that each investigator used his own method for assessing the compressive strength, with the great possibility that each value for this strength might have been subject to doubt as regards its location between high and low values, defining this relation from the previous results

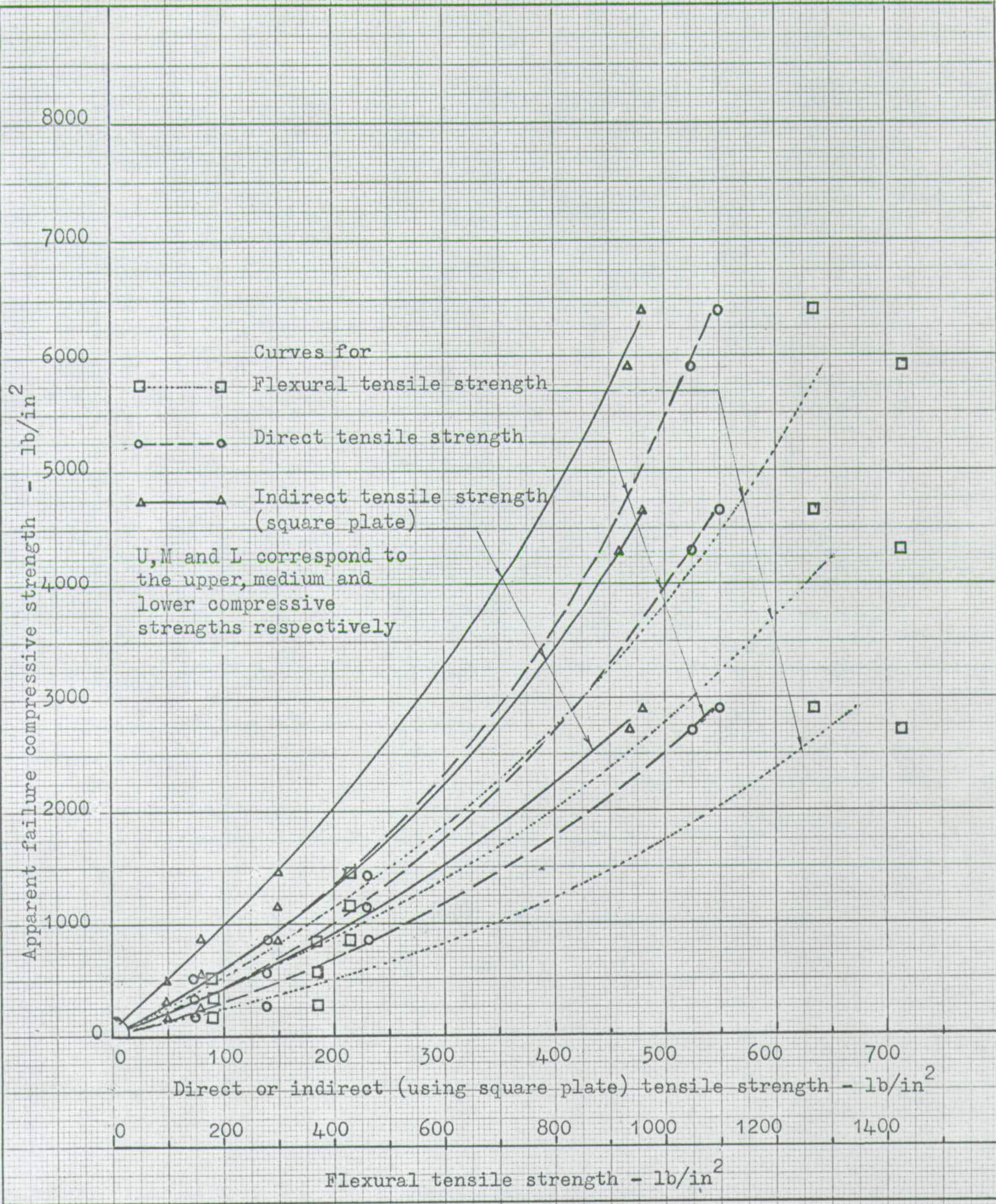


Figure 8.16:

Relation between tensile strength and compressive strength for mortars.

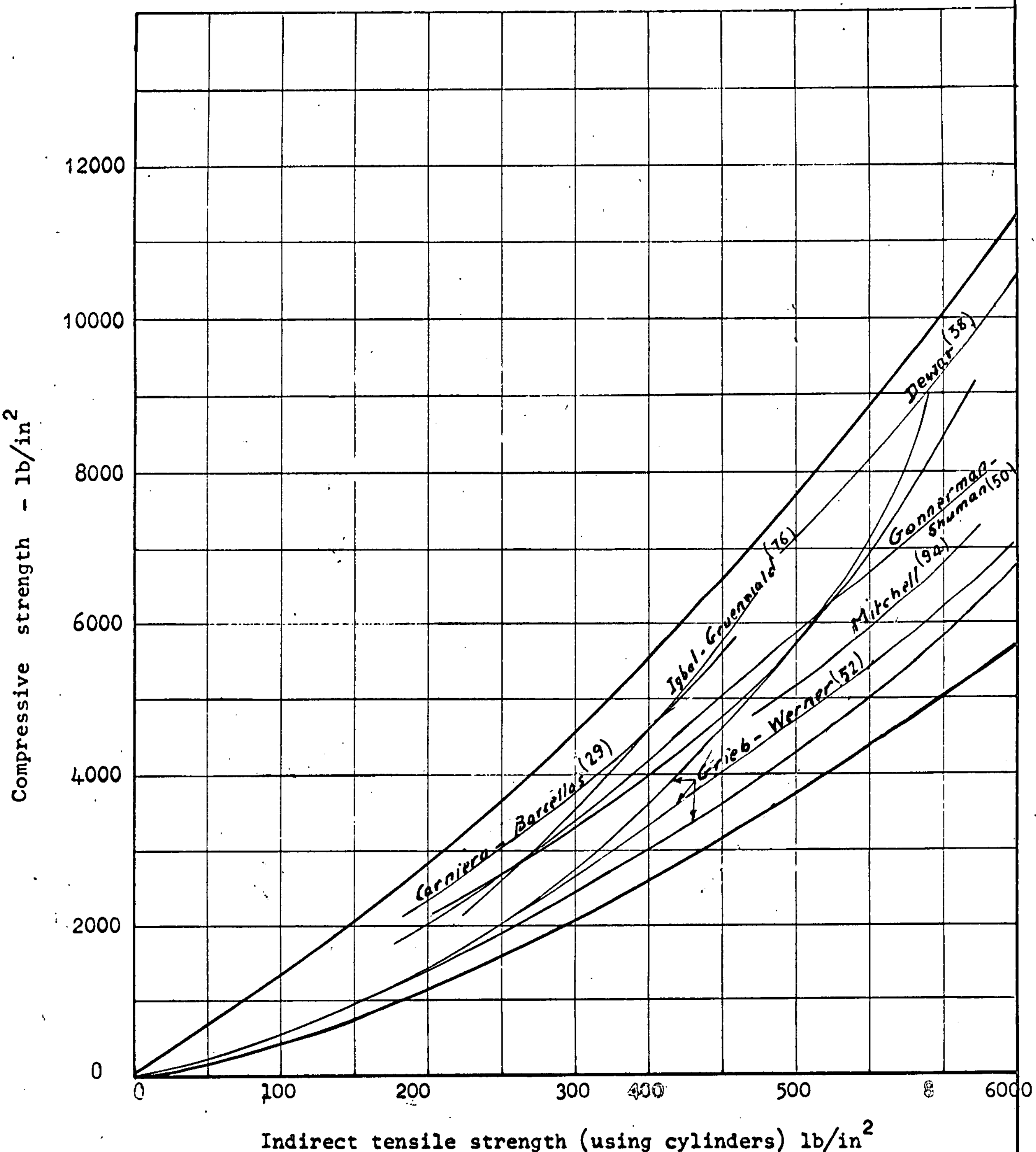


Figure 8.17 :
Relation between the compressive strength and the indirect tensile strength using cylinders for concrete as obtained by previous investigators.

by a range seemed reasonable. This latter range is shown in Figure 8.17 by the upper and lower curves starting from the origin, which were drawn by the author.

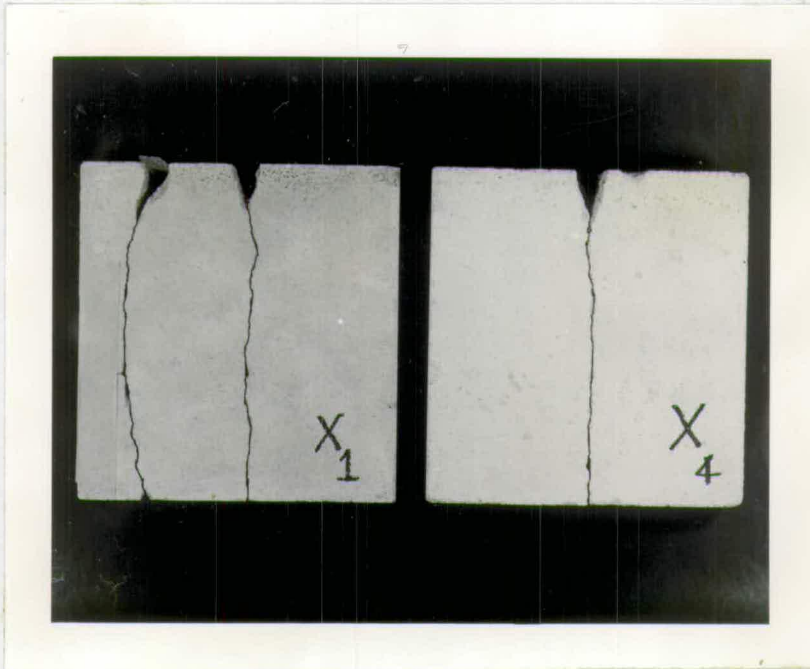
Now comparing the two ranges, produced from the new technique and the previous tests, they were found in good agreement as regards the trend. The general scatter of differences between the separate curves may be attributed mainly to the compressive strength considered, and partially to testing variables, rather than to differences between methods of testing. Such variables may include curing and its relative influence on different strengths, the maximum nominal size, degree of workability etc. Therefore, the range covers all these variables except the basic compressive strength, and forms a better basis for comparison. The main difference between the two ranges from the quantitative point of view is that the upper extreme of range produced from the present tests is considerably higher than the corresponding one from previous tests. This can be attributed to the fact that the upper limit of the present tests corresponds to a specimen 1" x 2" x 1", one half of a beam). Such a specimen, undoubtedly, has not been used previously when working with concrete.

One variable may be claimed here to have some influence, namely the thickness. This effect can be referred to Grieb and Werner[52], who found no influence of the thickness on the value of the indirect tensile strength.

Finally, with respect to the first topic, it can be said that the present results form a satisfactory support to the proposal to determine the indirect-tensile strength not only of mortar, and possibly other brittle materials, by employing square specimens rather than circular ones.

The second topic is concerned with the mode of failure. Photograph 8.8 of the fractures has been included to illustrate the type of failure obtained. It is very remarkable that the single cleft failure always started at the top of the specimen, which was loaded with the moving platen of the machine. The same phenomenon was obtained before by Mitchell(94) from tests on concrete cylinders with different conditions of loading, small plates, no plates at all, and with masonite plates.

As regards indications of tensile failure from the mode of failure, they can be seen in the same photograph, which shows that the specimens had fractured perfectly along the centre line. Other fractures at the quarter lines were obtained when repeating the tests on the half-specimens produced after the first failure. The fractures are quite satisfactory for indicating tensile failure. But at this stage the author feels strongly, that in spite of the minimized area in the test between the specimen and the loading platens, still the relative rigidity



The photograph illustrates the types of failure obtained. Attention is called to the fact that the single cleft failure always started at the top of the specimen, usually loaded.with the moving platen.

Photograph 8.8:

Mode of failure of square plates of mortar - Indirect tensilt test.

between them plays a considerable role. Exact tensile strength determinations for mortars and other brittle materials, using the new technique, will be worthy of additional research on the most suitable material to put between the platen and the specimen.

The third topic is concerned with the calibration of the indirect-tensile strength as determined from the new technique with values from other methods. The answer to this question belongs to the next paragraph. In the meantime it is enough to say here that there is a great portion of area overlapping between the ranges of the direct and indirect tensile strengths as shown in Figure 8.16. This made it possible to consider the results from the new technique as satisfactorily reliable.

8.7.2.3 Comparison between the proposed new method and other methods used for determining the tensile strength.

Both Figures 8.15, 18 show that the values reported from the indirect tensile test are lower than the flexural tensile strength Considering what has been pointed out in Chapter 5, this may agree qualitatively with the previous results reported with concrete. From the quantitative point of view, the present tests show a percentage increase in the latter over the former, of a value higher than the one reported from previous tests on concrete. Here, among the test variables, the one which emerges as most responsible for this difference may be the flexural strength is not a single value, but varies greatly with the size of specimen.

An attempt to establish a relationship between the flexural strength and the indirect-tensile strength from the present tests was not successful, but the general trend complies with the previous contributions on concrete. As in those contributions, this factor is disputed, and it seems to be a variable that decreases as the tensile strength increases. In other words, it may not be assumed that the indirect-tensile-strength is a constant proportion part of the flexural tensile strength, independent of the mortar or the size of the flexural specimen.

As regards the relation between the indirect-tensile and the direct-tensile strengths, it is also illustrated in Figure 8.18. The present results differ slightly from the previous results reported with concrete. Here the former is slightly less than the latter, while the contrary was previously found. Neither an explanation nor a correction has been possible, because of two main reasons. The first is that previous data about the difference between the strengths due to the test variables is scanty: the second is that the number of points, as well as the number of specimens of the present tests, is small. However, the difference between these two strengths, either in previous or present work, is little.

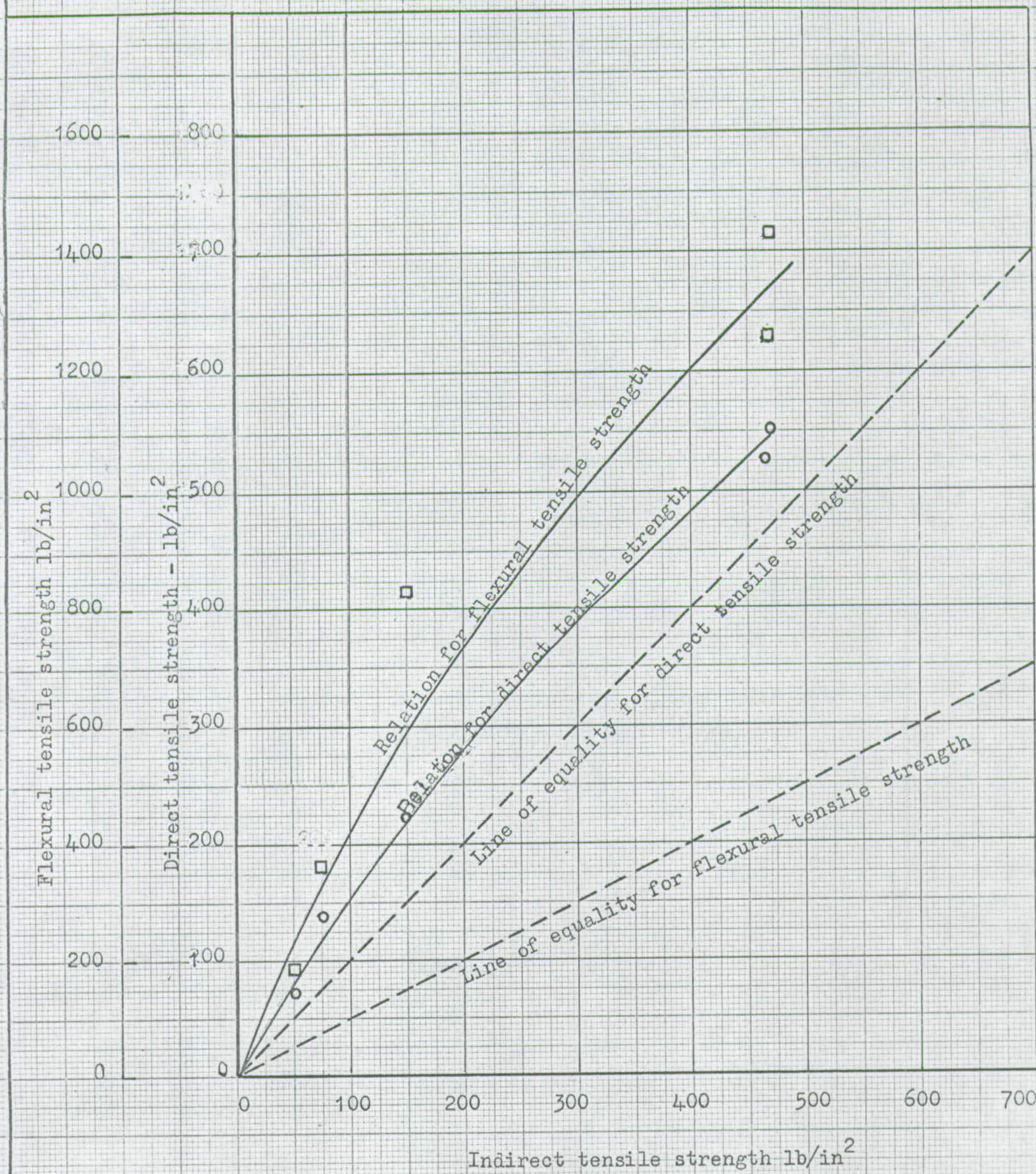


Figure 8.18:
Comparison between different methods for measuring the tensile strength

8.7.2.4 General relation between the tensile strength and the compressive strength.

On the same basis discussed before for the representative compressive strength, and similarly for the indirect-tensile-strength, the relations for the direct-tensile and flexural-tensile strengths were illustrated in Figure 8.16. The relation for the indirect-tensile strength has already been discussed. Similarly to it, the relation for each of the other two is represented by three graphs ("H", "L", "M"), and two ranges. Two main remarks can be made about them. The first is that the graphs and the ranges are of the same general form. The second, although partially repeated, is the great portion of area overlapping between the ranges of the direct and indirect tensile strengths. Considering the scale factor it is hard to find an area of both the ranges overlapping with the range corresponding to the flexural-tensile strength. Naturally, this is due to the difference between the two forms tensile strength.

It is unfortunate, due to the wide range of values of compressive strength, and the small number of points in each case, that establishing a mathematical relation for any of the cases has not been possible. Such a relation can be looked upon as an advantageous one, especially if it becomes necessary to establish a link between the tensile strength and the compressive strength which is usually considered more convenient in the laboratory.

However, on the basis of the present test results one might conclude that with present-day mortars, and with mixes encountered in practice, most of which are within the range of tested mixes, the relation between the compressive strength and the tensile strength is as shown in Figure 8.16 by the three indicated zones. But for any prediction using this figure, consideration should be given to the method of testing, size of specimen, and the most precise location of the value of the compressive strength.

8.7.2.5 Representative values for the tensile strength of the mortar mixes tested.

Considering the unreliability of the flexural strength, discussed before, the tensile strength from this test was discarded. Then due to the slight difference between the indirect and direct tests in the present series, the average was taken as the representative tensile strength. Thus, the tensile strength for mortar mixes X_1, X_2, X_3, X_4 and X_5 were considered equal to 510, 497, 189, 106 and 55 lb/in^2 respectively.

8.7.3 <u>Contribution to the common test methods for determining the strength properties of mortars.</u>

The methods incorporated in specifications and codes for assessing mortar strength are usually one or both of the compressive strength and the tensile strength (direct or flexural). The preceding discussions on the strength properties may make it very easy for differences of opinion to arise in connection with the wide range of readings for what is considered to have a single value, and consequently the relations expressed in the form of wide ranges. Such relations have never been dealt with in any of the specified tests.

Considering broadly the laboratory testing of mortars in the context of structural materials, it becomes a difficult question to think about in the present chapter. Similarly, when thinking within the sphere of brick masonry, the use of either a compression or a tension test can hardly be justified. In compression, there is a considerable doubt as to whether the compressive strength is the most influential property. Not only this, but the actual quantitative measure is far from being perfectly known. As regards the tensile tests, the number of test variables has not been considered in an adequate manner in specifications. Even if considered, a situation very similar to that of compression would exist.

8.8. DEFORMATION PROPERTIES

8.8.1 General Observations on the Load-Strain Relations

It was not possible to take all the readings expected from the rosettes, but the number of the recorded readings for each mix was more than sufficient to determine the load-strain relations for both the "X" and "Y" directions as was shown before in Figures 8.5 - 8.9. On the basis of these relations, and considering the common straight portions of ϵ_y and ϵ_x graphs, the stress-deformation relations, according to the case of plane-stress, were calculated in the same manner described in Chapter 7 with steel specimens (Equation, 5.22 Table 5.1 or Figure 5.2 and Equation 5.24 were employed for Poisson's ratio and modulus of elasticity respectively).

One remark should be mentioned in regard to the strains plotted in the above-mentioned figures. In spite of the great precautions taken to avoid any eccentricity of loading, it can be seen that there is a considerable scatter in the readings, especially for those of "ϵ_y". But the variation in general appears to be random and unrelated to the proportions of the mixes, or to the properties of the hardened mortar. A subsidiary test which was carried out during the analysis of results, has thrown light on a possible explanation for this scatter. The test was carried out on a spare specimen from the trial mix. Readings were taken during loading and unloading for a time and the reproducibility of the readings was, in general, excellent. This evidence from cyclic loading implied that the scatter may be attributable to hysteresis and creep effects. Considering the fact that each specimen was tested at four positions, and bearing in mind the hystersis effect, together with Glanville's observations[43,49] on the effect of creep in different directions (see Chapter 5), this scatter appears acceptable and the relationships established of reasonable reliability. This also, has led to another remark. Due to the fact that this reproducibility happened before with steel, it seems that this may be expected with this type of test, in which the specimen is tested in a state of ease.

8.8.2 Modulus of Elasticity

8.8.2.1 General discussion

1. As might be expected the graphs (Figure 8.19) representing the stress-deformation relations have a common feature if they are plotted completely. However, all the specimens without exception showed a pronounced elastic behaviour giving a constant modulus of elasticity. If the formulae were applied beyond this range each line would have started to deviate giving a lower value for "E" and a higher value for "γ".

2. A general observation which can be made regarding the results is that the values of "E" for the different mixes showed a tendency towards classifying the mixes in the same manner as the strength properties, "X_1" with "X_2", and "X_4" with "X_5" being similar.

3. The mixes which produced the nearest approach to a constant modulus of elasticity were "X_1", "X_2". Following

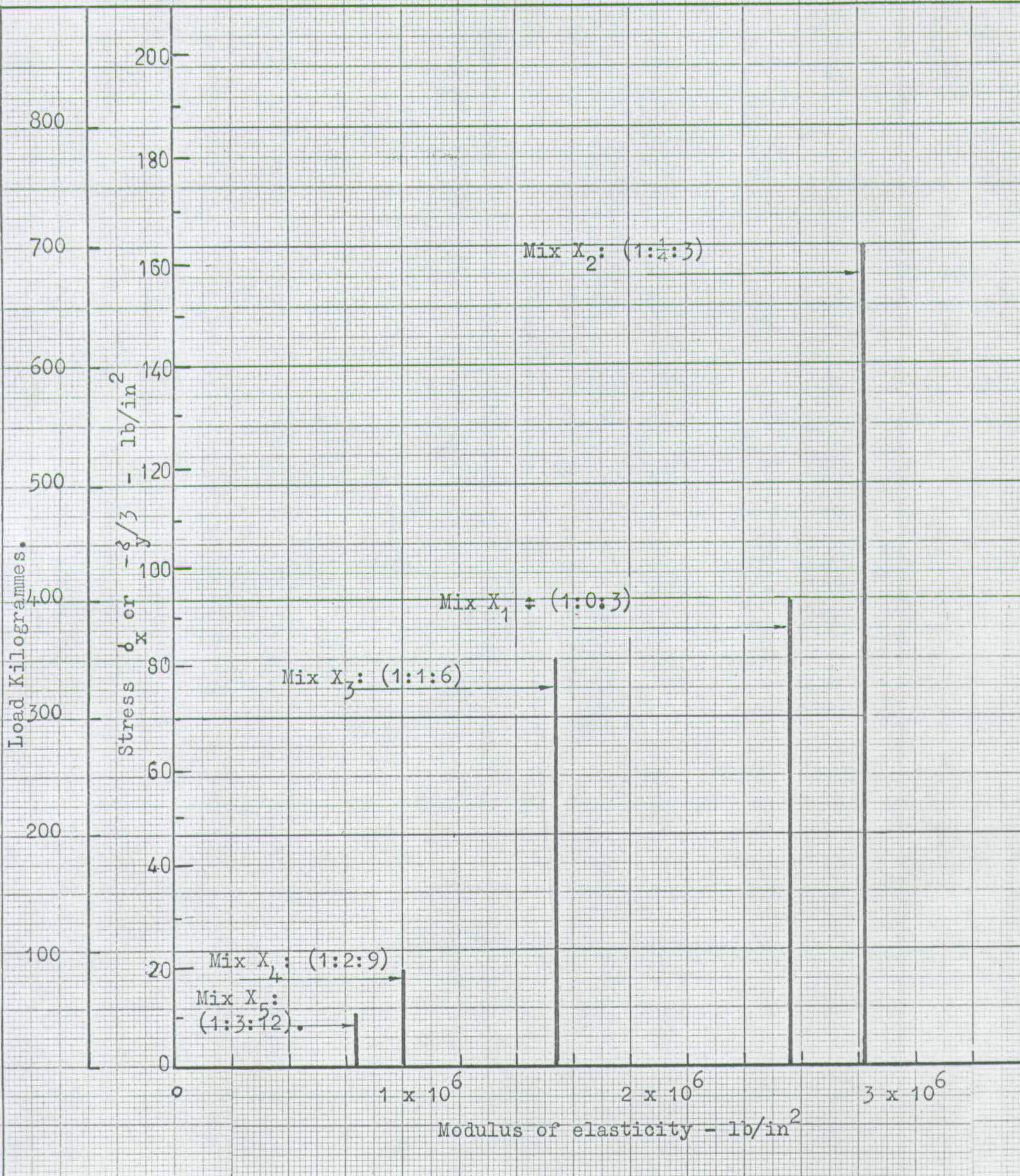


Figure 8.19:
Modulus of elasticity of mortar mixes.

the general trend from "X_5" to "X_1" the curve for "X_1" seems to be inconsistent when compared with that of "X_2" and vice versa. It seems to the author that a possible explanation may lie in the composition of the mixes. The main difference between "X_1" (1:3) and "X_2" (1:1/4:3) is that the latter contains a small proportion of lime. This might have given mix "X_2" better workability with the result of better compaction. Due to the fact that compaction was carried out without the occurrence of segregation, this could lead to a higher value for the modulus of elasticity.

4. The amount of strain recorded when testing mixes "X_4", "X_5" was particularly small and for low loads difficulty was experienced in reading just at the exact loading. It was necessary to apply a slightly higher load to allow for the time elapsing between stopping the loading machine and taking the reading from the measuring apparatus.

5. The relation between the modulus of elasticity and the compressive strength as shown in Figure 8.20 compared favourably with some previous investigations. For example, it can be compared with Lenczner[87], Morsy[97] and Walker[140]. The first gave test results on mortar and the second and the third gave it on concrete.

8.8.2.2 Representative values of the modulus of elasticity for the mixes tested.

Comparing the values of modulus of elasticity for each mix, the choice of the load at which the

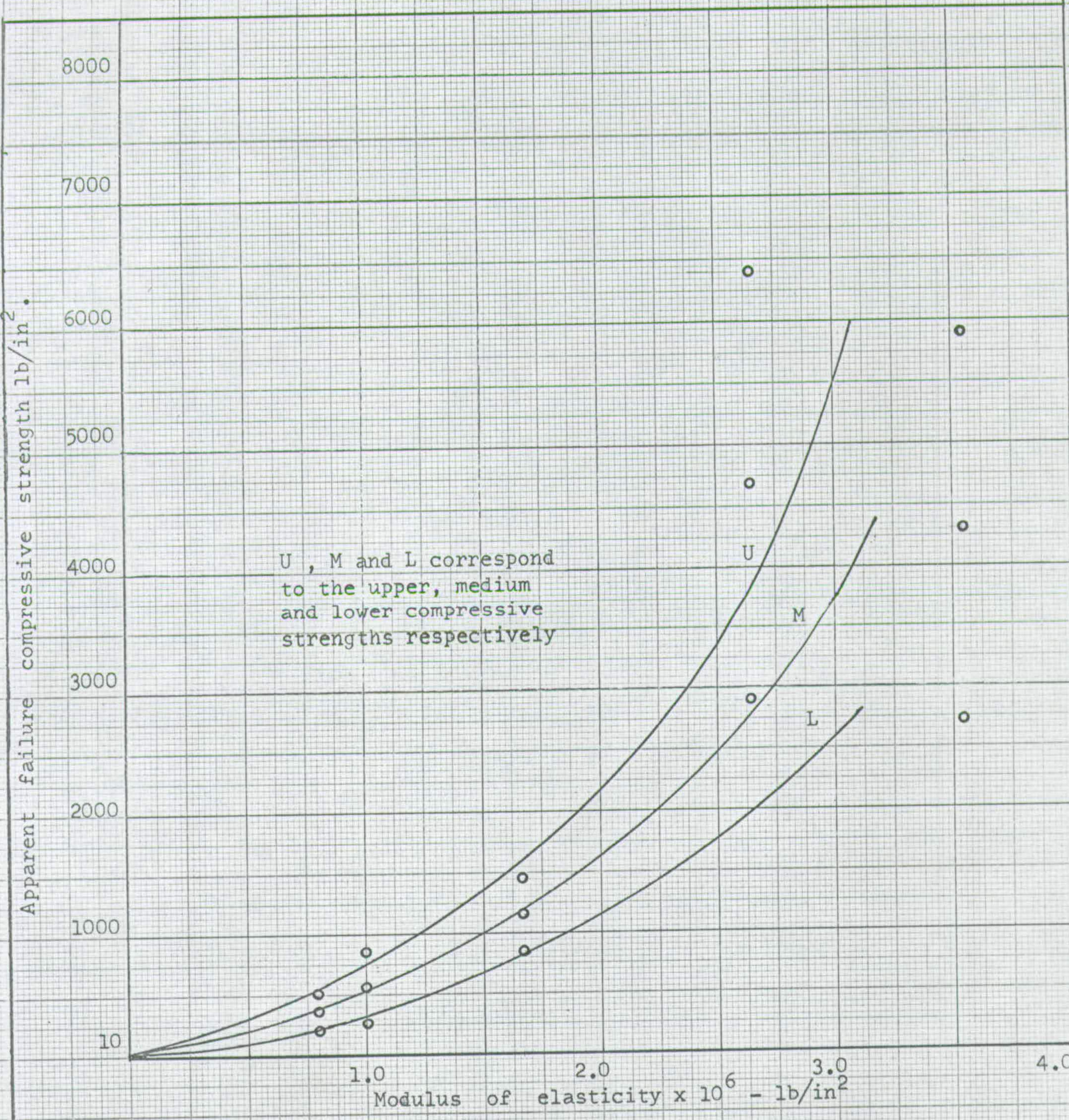


Figure 8.20 :

Relation between modulus of elasticity and compressive strength for mortars.

representative modulus may be taken was governed by the maximum load applied. However, it was thought reasonable to consider one value as representative of the modulus. The first is one calculated from the strains at the end of the common two straight portions of the ϵ_y and ϵ_x relations. Therefore, the values of modulus of elasticity for the mortar mixes X_1, X_2, X_3, X_4 and X_5 were considered respectively equal to 2.69×10^6, 3.53×10^6, 1.67×10^6, 1.01×10^6 and 0.8×10^6 lb/in^2.

8.8.3 Poisson's Ratio

8.8.3.1 General observations

1. Although the values of Poisson's ratio were calculated from the same values used for the modulus of elasticity, it can be seen that the measured values were not so conclusive as the values of the modulus. Probably the main reason, as mentioned before, is the sensitivity of equation 5.22, to the slightest experimental error. At the same time, it is felt that it is unlikely that the error for every point of the curve will always be in the same direction. Therefore, it is very

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continued:
(140): Walker, S. Discussion on a paper by Williams, G.M. Some determinations of stress deformation relations for concrete under repeated and continuous loading. Proc. A.S.T.M., Vol.20, Part 2, 1920.

probable that the curves plotted give representative values of Poisson's Ratio.

2. The value of Poisson's ratio is constant over the same ranges of stress which produce a constant modulus of elasticity. The mixes producing the nearest approaches to a constant ratio are the same mixes "X_1", "X_2".

3. It can be noticed that the same mixes X_1, X_2 gave remarkably low values for Poisson's Ratio. A hypothesis for this phenomenon will be given later.

4. Mixes X_3, X_4 and X_5 showed higher values for Poisson's ratio, but no clear trend emerged.

5. It appears that the present results give support to the opinion that Poisson's Ratio is constant over a wide range of stress rather, than the view that it becomes variable after being zero for a considerable load. In the author's opinion the latter case may be due to the effect of strain-lag at low stresses, as was found to exist with brick testing.

6. Apart from "X_4" it appears that a mortar mix with a low cement/sand ratio will have a higher value of Poisson's ratio, and vice cersa. This means that in the range covered by the present tests a high strength mortar tends to have a lower Poisson's ratio than a low strength mortar.

8.8.3.2 Representative values of Poisson's ratio for the mortars tested.

The mortar mixes X_1, X_2, X_3, X_4 and X_5 were considered to have values of Poisson's ratio equal to 0.097, 0.156, 0.109, 0.264 and 0.238 respectively.

SECTION THREE: AN ANALYSIS OF THE TEST RESULTS OF BRICK PROPERTIES

8.9 GENERAL OBSERVATIONS AND CONCLUSIONS

8.9.1 Compressive Strength

The tests on the individual bricks described above, emphasised some of the previous important conclusions given in Chapter 6, and added unmistakeably new aspects in the area of brick testing. They can be summarised as follows:

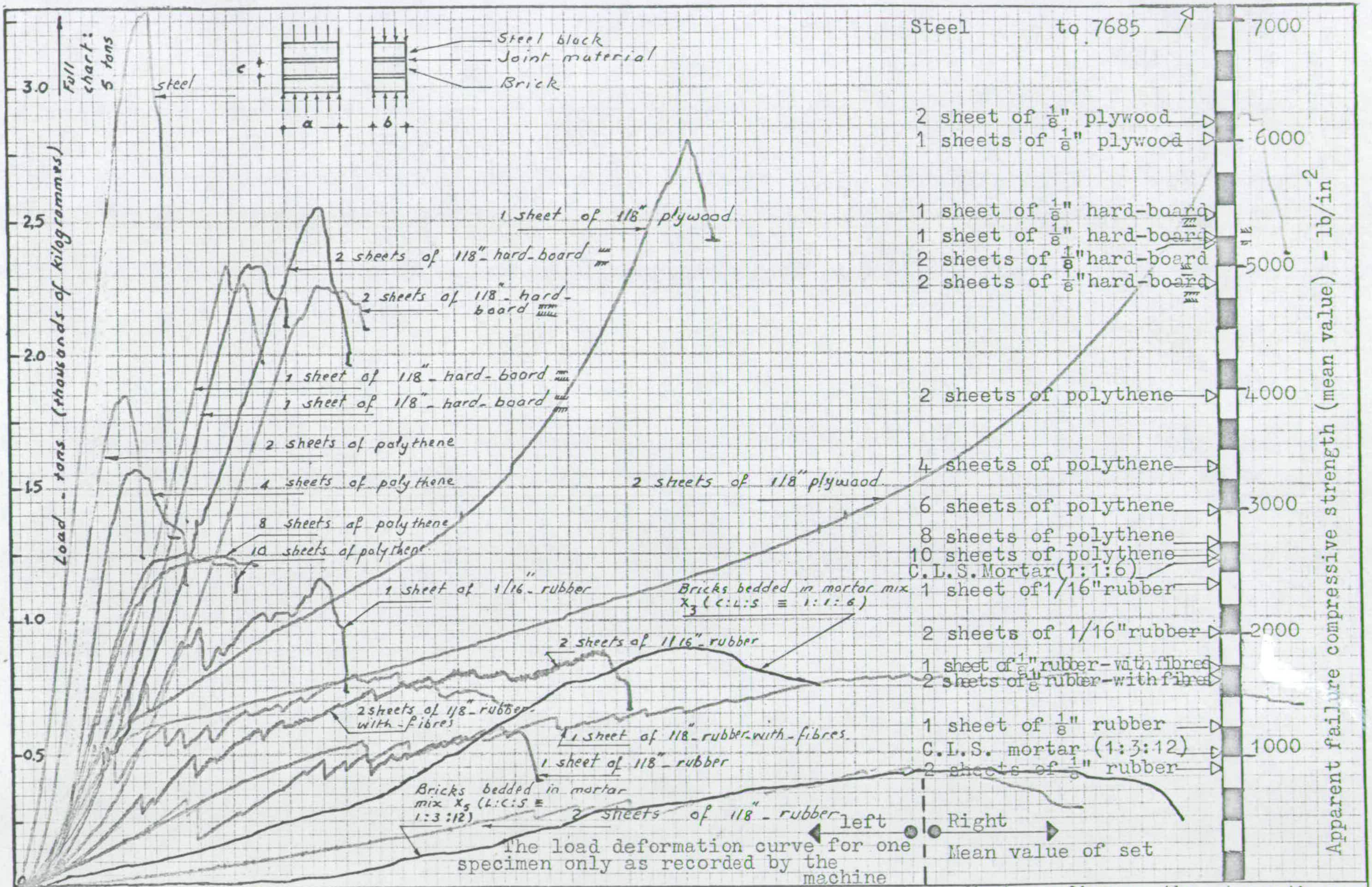
1. Due to the wider range of joint materials used, a wider range of values for the apparent compressive strength of bricks was obtained, 903-7685, and 692-5360 lb/in^2 for one-sixth and one-third scale bricks respectively.

2. The relative dimensions of the brick and the end-material in the vertical direction is emphasised, again to be more pronounced in the case of soft end-materials than in the case of the soft ones. This can be seen in Table 8.4 by comparing the variation in strength when two sheets are used instead of one in the cases of hard joint-materials (plywood, hard-board), and soft joint materials (rubber, rubber with fibres). While no great change in strength is obtained in the former case, there is a considerable drop in the latter.

3. Also the reduction in compressive strength with increasing soft joint thickness was shown in Chapter 6 (Figure 6.24) to be nearly linear, when examined in terms of the ratio of thickness of the polythene sheets (constant) to the height of the brick

(variable). The present results with polythene sheets help to establish this phenomenon in terms of the same ratio between the number or thickness of the polythene sheets (variable), and the height of the brick (constant). Comparing Table 8.4 and Figure 6.24 a similar linear effect can be easily seen. It was unfortunate that bricks embedded in mortar were not tested for different thicknesses of the latter, but previously discussed the graphs at the top of Figure 6.24 emphasise the phenomenon with brick masonry and real mortars.

4. Continuing the discussion to the present codes of practice, it can be emphasised again, that plywood is the wrong material for brick testing. In Chapter 6, the discussion was based only on other investigator's previous results and the failure characteristics of the assemblages tested in that chapter. Now the graphs recorded automatically by the testing machine (not available previously) give strong support in a more comprehensive manner. In Figures 8.21, 22 the graphs relating to plywood show that it is a highly compressible material, as indicated by the greater deformation of the specimen tested between two double sheets of plywood. In spite of this high compressibility, plywood ultimately introduces restraint very similar to the steel platen restraint. What seems to be strange is that the two-sheets of plywood gave a strength slightly higher than the strength of the one sheet.



Automatically recorded deformation

Each reading on the strength scale (right) corresponds to the reading on the load scale (left) of the same vertical ordinate.

FIGURE 8.21: Apparent failure compressive strength of one-sixth scale model bricks, tested between different materials.

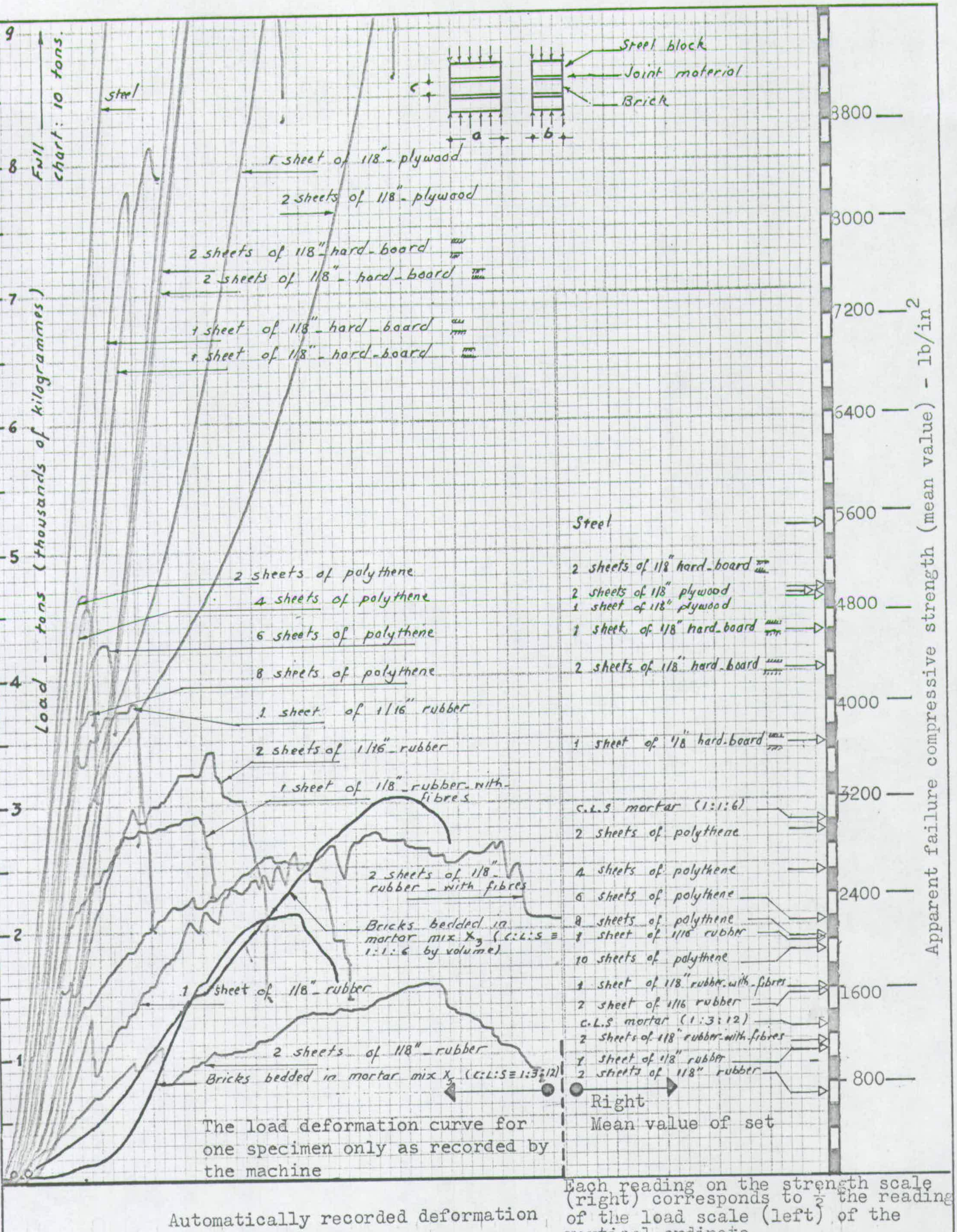


Figure 8.22: Apparent failure compressive strength of $\frac{1}{3}$ scale model bricks tested between different materials.

5. In a more or less similar manner to plywood, hardboard showed the same trend as regards the type of restraint.

6. The best method giving the nearest apparent strength to the assemblage strength, which is one of our chief concerns in the present analysis, will be discussed later along with simulation of mortar.

7. Due to the fact that the actual compressive strength of bricks is not a limiting factor its discussion in Chapter 6 was considered enough. At the same time the compression test giving the nearest strength to that in masonry will be discussed later.

8.9.2 Tensile Strength

The proposed new method for assessing the tensile strength of bricks (see Figure 5.4) appears to be successful from both considerations of convenience and reliability. Both Figures 8.2,3 illustrate typical examples of the load vertical-deformation graphs as recorded automatically by the testing machine. The following can be noted:

1. Bricks tested on edge undergo more vertical deformation than bricks tested flat. Probably this can be attributed to the greater height of the brick between the loading platens in the case of tests on edge. A careful inspection of the specimens recorded in the figures justifies this. It can be seen that the average ratio (based on both figures) between the deformation of specimens tested on edge and those tested flat is 1.23:1. This equals approximately the average ratio

of "b/c", for both types of bricks used, which is equal to 1.455.

2. Regarding the ultimate strength which is the point of chief concern in the present test, it can be noticed that in the same figures 8.2,3 (illustrating only some examples), and Table 8.4 (giving the mean of all the specimens tested) there is a slight difference between the mean values. In the author's opinion this order of difference gives an experimental proof that the proposed test can be used successfully with all brittle materials, even those which cannot be cast in the laboratory.

3. As for the representative value for the tensile strength, it was thought to be better if the average value of the means is considered. Therefore, the representative values for the tensile or transverse strength were considered to be as follows (see the bottom of Table 8.4):

a. For one-sixth-scale bricks = $(453 + 475)/2 =$ 464 lb/in^2

b. For one-third-scale bricks = $(253 + 248)/2 =$ 250.5 lb/in^2

8.9.3 Deformation Properties

Because bricks were not the main variables the value of E and ν were calculated for each brick from one single test. Figures 8.10, 11. E and ν were (1.4×10^6, 0.148) and (1.13, 0.140) for one-sixth and one-third scale model bricks respectively.

SECTION FOUR: AN ANALYSIS OF THE TEST RESULTS FOR BRICKs MORTAR ASSEMBLAGES

8.10 GENERAL

In this section an attempt is made to interpret the output of the values measured on individual components in the form of practical tools for studying the assemblage failure characteristics, and how they can be influenced by these measured values and other factors. At this stage of study more factors had been recognized during the sequence of steps incorporated in the previous chapters. Accordingly, the attempt has been made along two separate lines, the first by considering only the experimental values, and the second by examining the extent to which the theoretical prediction of brick masonry behaviour can be successful.

8.11 ASSEMBLAGE COMPRESSIVE STRENGTH AS A FUNCTION OF BRICK STRENGTH PROPERTIES

1. Comparing Table 8.4 and Figures 8.23, 24 it can be seen that the present tests are within the range in which a higher masonry compressive strength is associated with a higher compressive strength. Naturally, this is apart from the fact that other factors as the size, relative dimensions in the vertical direction are not equal.

2. As regards relating the assemblage strength to the brick strength, both being in compression, it will be shown in the next section, how this can be achieved by careful simulation of mortar in brick testing. For the sake of completness

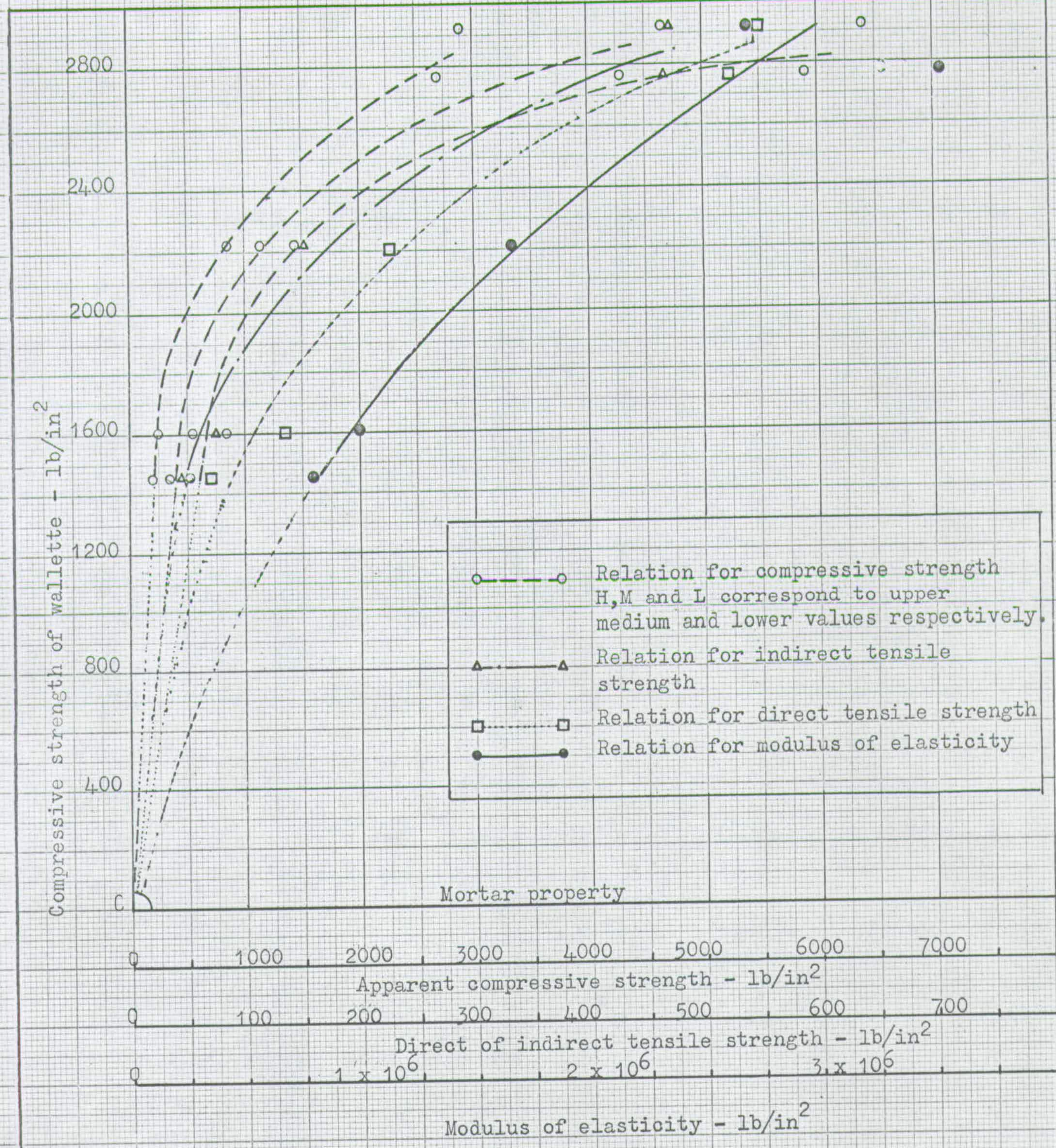


Figure 8.23:

Compressive strength of wallette as a function of mortar properties - bricks used were one-sixth scale model bricks.

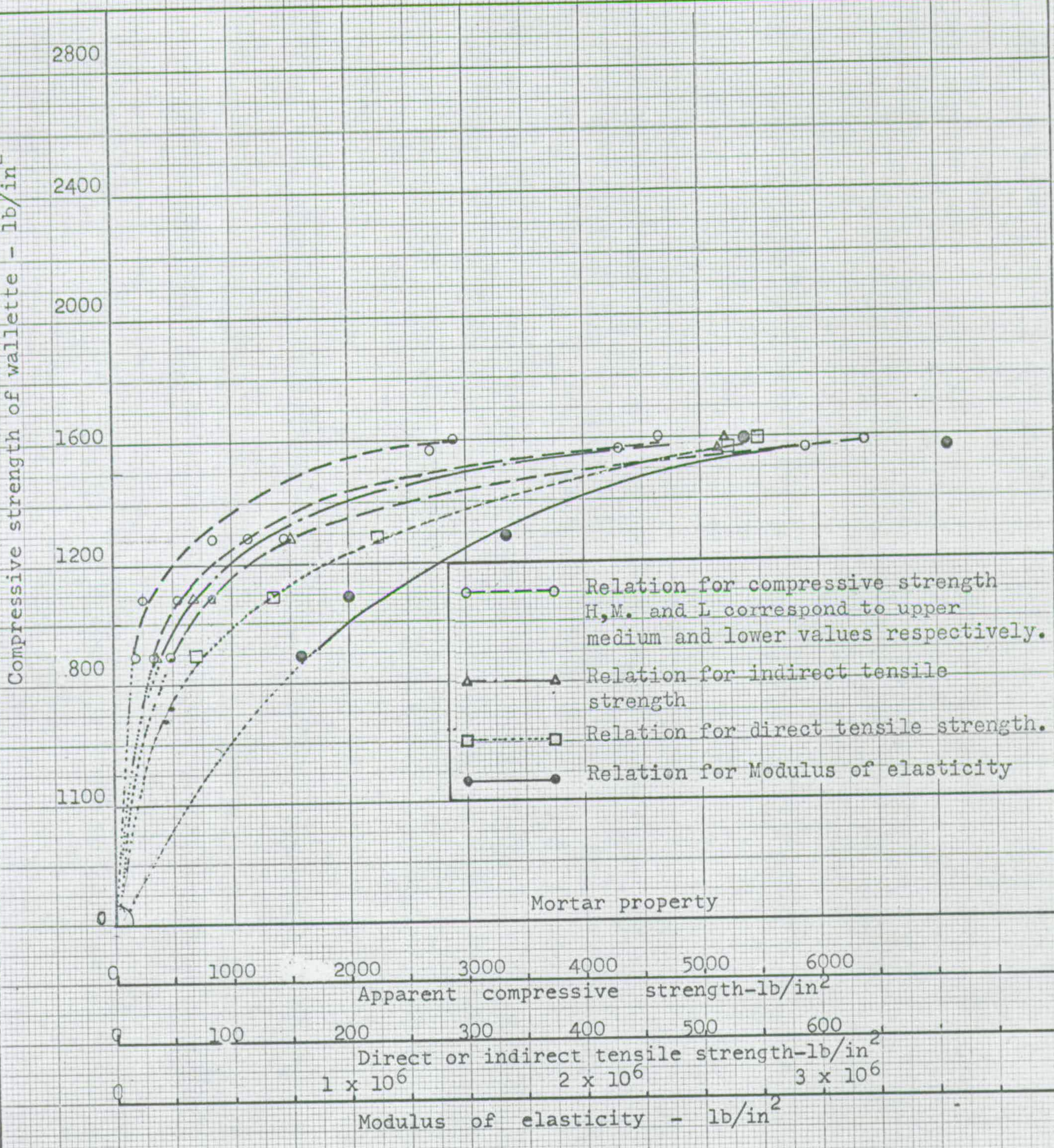


Figure 8.24: Compressive strength of wallette as a function of mortar properties - bricks used were one-third-scale model bricks.

under the present heading it may be added here that a discrepancy of not more than 10% can be achieved when required.

3. The present results indicate that the strength quality of a brick may be assessed by its being embedded between mortar layers rather than sheets of plywood. But it should be remembered that this method still gives a higher value than that of the assemblage. This may be attributed to the fact that the strength of brick masonry elements decreases, down to a certain limit, as the number of the horizontal joints increases. Naturally, this is up to a certain limit of this number where buckling starts to take place.

4. From 2 and 3 it can be added to the conclusions given in Chapter 6 that, the contrary to the conclusions of the tests carried eut at the Building Research Station[36], the compression test of a brick can be designed to give reliable values. As indicated above this will be dealt with quantitatively later.

5. Regarding the influence of the deformation properties of bricks on the assemblage strength, the tests carried out were very few. The values obtained may be regarded as a tool which will be used later when verifying the mathematical formula for strength.

8.12 ASSEMBLAGE COMPRESSIVE STRENGTH AS A FUNCTION OF HARDENED-MORTAR PROPERTIES

The effect of conventional mortar types which were shown to have distinct properties, upon the compressive strength of assemblages is illustrated in Figures 8.23, 24. For each type of brick and assemblage mortar was the only variable. The following remarks may be made:

1. Assemblages built with mortar mixes X_1, X_2 yielded the highest compressive strength. Shifting from X_5 to X_1 shows that a sharp decrease in mortar compressive strength is not accompanied by a similar one in the assemblage strength in some regions of the curves. Generally speaking, this complies with previous results e.g. (Figures 1.2,3 and 5).

2. Although the effect of mortar strength on the assemblage strength may be defined at some portions of the graphs, an important fact emerges clearly from the present results. The graphs show that it is not only the compressive strength of mortar which is, as usually known, responsible for the flattening out of the curve plotting the masonry compressive strength against the mortar compressive strength, but also the method of determining the latter. The latter appears to be a real limiting factor which undoubtedly accounts for a part of the flattening.

3. It seems probable that, within the range of mortars tested, the higher the compressive strength of bricks the

later will be the start of the flattening out of the curve.

4. Predicting the influence of conventional mortars by employing equivalent joint materials seem to be most interesting and profitable especially from the practical point of view. A comparison between the assemblage test results and those of bricks between simulated mortars, in terms of the ratio between assemblage strength and brick strength is shown in Figures 8.25,26. Considering the present bricks as building units, the figures suggest that the conventional mortars used may be simulated in the brick test so that they give reliable predictions for the assemblage strength as given in Table 8.6.

As the table shows, it does not seem an extensive task from the experimental point of view for a team of workers to simulate all the conventional mortars with respect to different types and grades of bricks. At the same time it is emphasized once more that the inevitable variations in the experimental results due to a considerable tolerance, does not seem greatly misleading. This is shown clearly from the scatter of the curves on both sides of the line representing a ratio equals one (thick horizontal line).

5. The comparison in "4" above gives an example of the work required on full scale bricks, which arose before at the end of chapter 6. Whatever this work is, it will not be by any means more tedious than the work done at the B.R. Station and listed by Davey and Thomas[36].

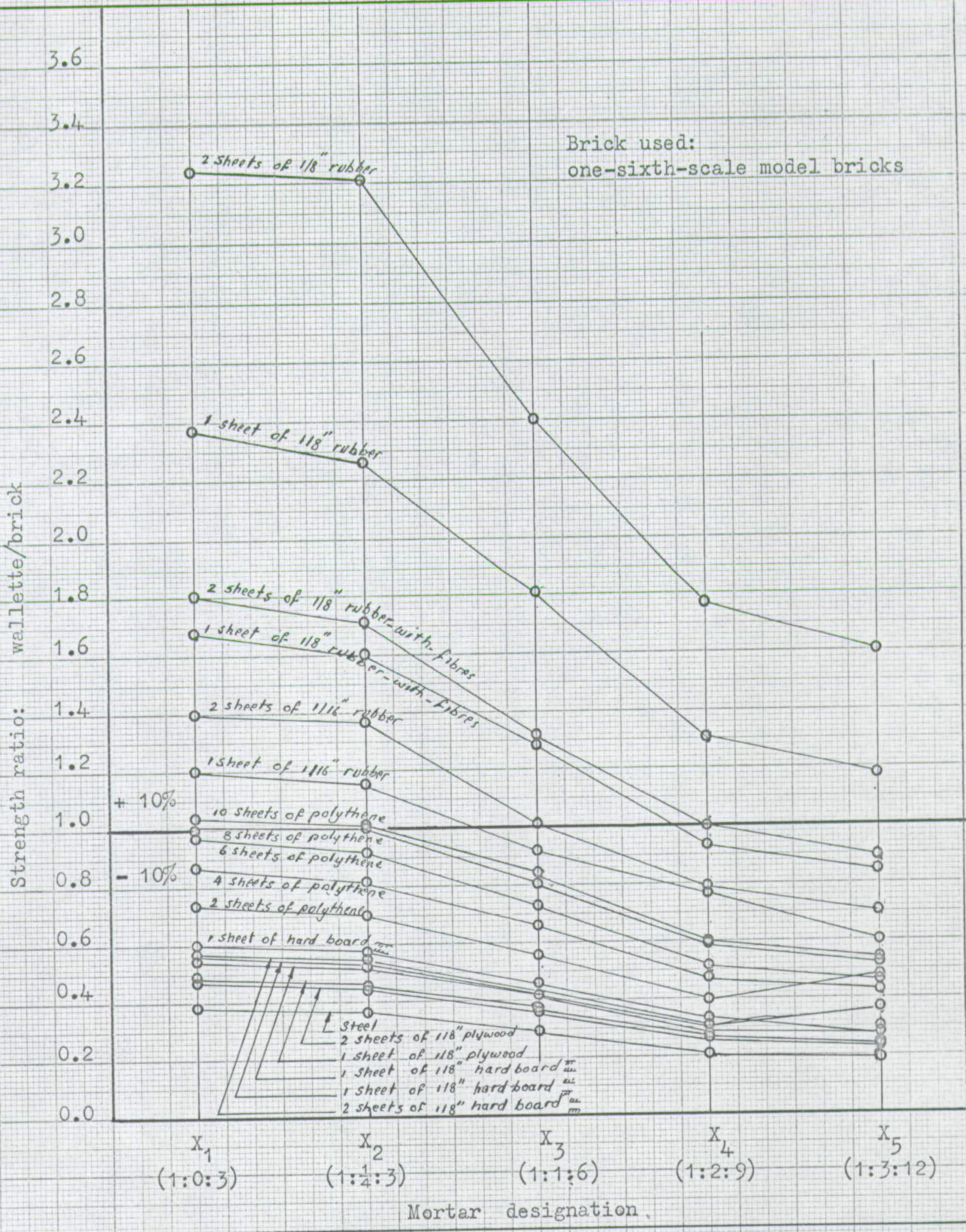


Figure 8.25:
Simulation of some conventional mortars in brick testing as a function of strength ratio: wallette/brick.

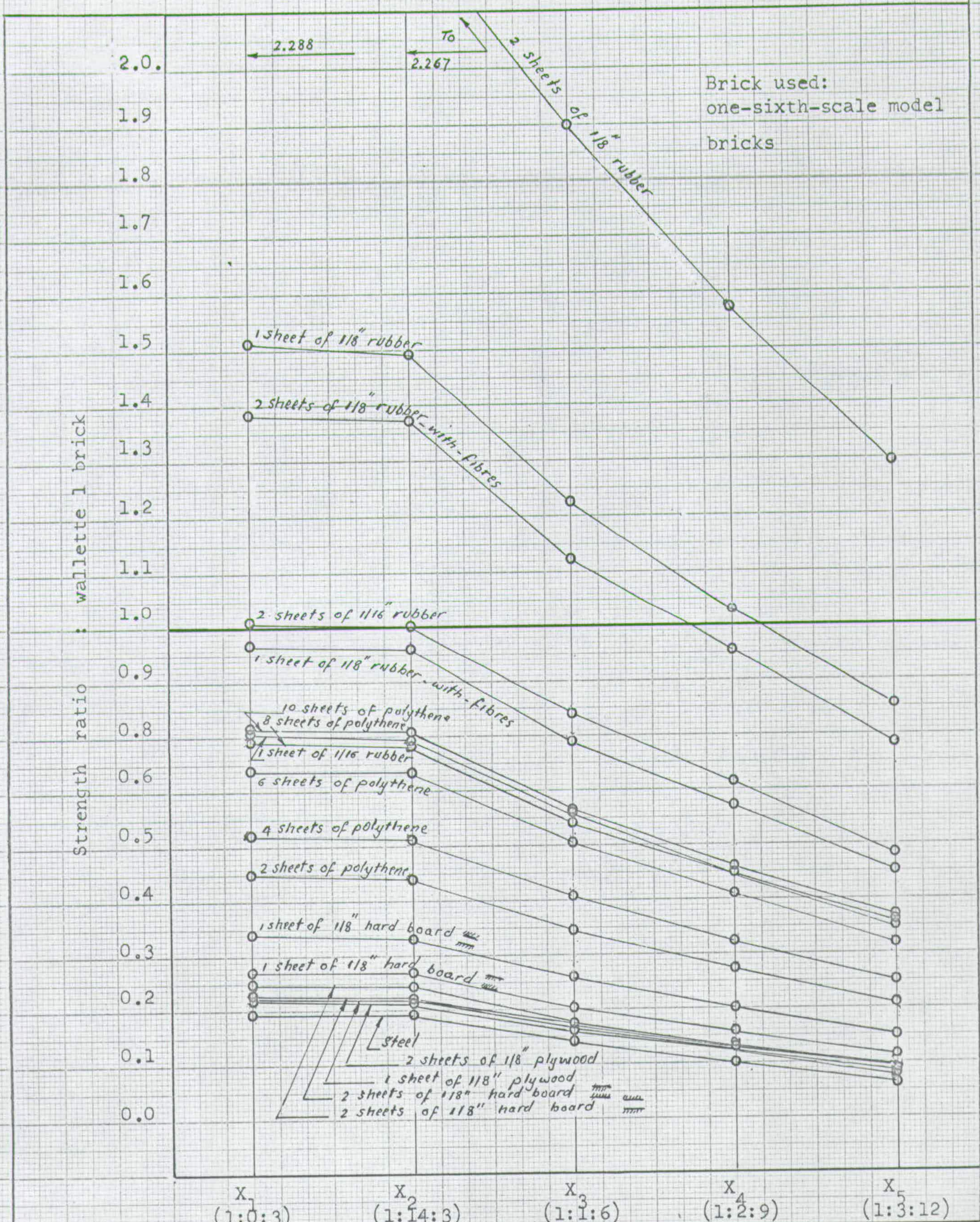


Figure 8.26:
Simulation of some conventional mortars in brick testing as a function of strength ratio: wallette/brick.

TABLE 8.6

Examples showing how simulation of mortars in brick testing can give more reliable predictions for brick masonry strength

Bricks	Mortar proportions	Materials giving a $\frac{\text{wallette strength}}{\text{brick strength}}$ ratio = 1 ± 10%		$\frac{\text{wallette strength}}{\text{brick strength}}$ according to the standard loading test (using plywood)
One-sixth-scale model brick.	1:0:3	6 sheets of polythene	0.97	0.48
		8 " " "	1.06	
		10 " " "	1.09	
	1:¼:3	6 sheets of polythene	0.92	0.46
		8 " " "	1.01	
		10 " " "	1.04	
	1:1:6	1 sheet of 1/16" rubber	0.92	0.37
		2 sheets " "	1.10	
	1:2:9	1 sheet of 1/8" rubber-with-fibres	0.937	0.26
		2 sheets " " " "	1.00	
	1:3:12	2 sheets of 1/8" rubber-with-fibres	0.90+	0.24
On-third scale model bricks	1:0:3	2 sheets of 1/16" rubber	1.03	0.33
		1 sheet of 1/8" rubber-with fibres	0.97	
	1:¼:3	2 sheets of 1/16" rubber	1.02	0.32
		1 sheet of 1/8" rubber-with fibres	0.96	
	1:1:6	the nearest material was two sheets of 1/8" rubber with fibres (1.1268)	-	0.26
	1:2:9	2 sheets of 1/8" rubber-with fibres	0.95	0.22
		1 " " " " "	1.04	
	1:3:12	the nearest material was one sheet of 1/8" rubber (0.8538)	-	0.18

+ approximated from 0.8964

6. As was expected, the tensile strength of mortar may have a better defined effect upon the assemblage strength. In some cases it may be possible to obtain the graph plotting the masonry strength against the mortar strength in the form of a straight line.

7. Shifting to the effect of the modulus of elasticity of mortar on the assemblage strength, the figures show that it is more influencial and may be easier to define. In fact, the sharp change in both the assemblage strength and the mortar modulus disappeared. It appears that within the present results and apart from Poisson's ratio of mortar, the modulus of elasticity of mortar determines its strength better than any other measured property. Investigation of more mortar mixes inside the present range of mixes tested is required for clarification of this point.

8. The remaining property if Poisson's ratio. A consistent trend between the assemblage strength and the mortar Poisson's ratio was not achieved. But in the light of the hypothesis given in section two of the present chapter (8.8.3) concerning the decrease in the value of Poisson's ratio with the increase of cement content something might be added here. At the same time it is worth referring here to Davies and Thomas's conclusions. They mentioned that it is useful to keep the cement content in mortar within certain limits, suggested by them, because of the improved resistance of brickwork to cracking when a weak mortar is used. The present author suggests that it is not the weakness

of mortar which determines this phenomenon. Mortar can have a high compressive strength, but that is of less importance than its deformation properties, in that the values of "E" and "ν" together enable it to give slightly to take up the differential movements either inside the unit (masonry assemblage) or in the whole unit.

8.13 THE VALIDITY OF THEORETICAL APPROACHES TO THE DETERMINATION OF COMPRESSIVE STRENGTH OF BRICK MASONRY

8.13.1 Conditions for Verification

It was shown in Section 4 of the present chapter that appropriate simulation of conventional mortars can lead to an easy and reliable prediction of the masonry compressive strength. But still the question of theoretical prediction of strength remains important, especially after this effort to measure the data which were said to be lacking. Therefore, it is time, now, to examine the expression derived theoretically in the light of the assessed properties.

The formula*, derived in Chapter 2, for the ultimate failure compressive strength in terms of the individual properties was given as:

$$P_{ultimate} = \left[\frac{\left(\frac{\alpha}{\alpha + t_1}\right)\left(\frac{f_{tb}}{E_{tb}}\right)}{\left(\frac{\nu_m}{E_m}\right) - \left(\frac{\nu_b}{E_b}\right)} \right]$$

* Reference to be made to the notation of Chapter 3 (3.2).

Before starting the verification two points should be noted. The first is that all the properties included in the Formula were measured except one, that is E. (the modulus of bricks in tension). This would have needed much work to achieve a proper technique with the brick sizes used. Eventually this might be possible with full-scale bricks. However, this modulus will be considered equal to the modulus in compression. The second point is that the assumptions for deriving the formula should be remembered. It was assumed that bricks are rigid when compared with mortar, having much higher modulus of elasticity. In the present tests this assumption is roughly valid for mortar mixes X_4 and X_5 with both types of bricks.

8.13.2 Specimens with Bricks Having More Rigidity Than Mortar

Table 8.7 shows a comparison between the calculated and measured values. The following remarks can be made.

1. The values of strength obtained experimentally did agree to a small extent with theoretical values, as can be seen from the degree of closeness of most of the ratios to one, and the average deviation percent of the former values from the latter ones.

2. Apart from the scatter between individual values, the average ratio appears to indicate experimental values higher than the theoretical values.

Table 8.7

Comparison between computed (from theoretical formula) and actual (from experimental tests) values of the ultimate compressive strength of wallettes

Mortar	Bricks	Computed value lb/in^2	Actual value lb/in^2	$\frac{\text{Computed value}}{\text{Actual value}}$	Deviation of calculated value from actual value
X_4 (1:2:9): $E = 1.008 \times 10^6$ lb/in^2 $\nu = 0.264$	1/6-scale model bricks $E = 1.4 \times 10^6$ lb/in^2 $\nu = 0.148$	1985	1601	1.240	+24%
	$\frac{1}{3}$-scale model bricks $E = 1.13 \times 10^6$ lb/in^2 $\nu = 0.140$	1553	1089	1.428	+ 43%
X_5 (1:3:12): $E = 0.796 \times 10^6$ lb/in^2 $\nu = 0.238$	1/6-scale model bricks	1602	1451	1.104	+ 10%
	$\frac{1}{3}$-scale model bricks	1214	896	1.355	+35.5%
Average ratio of computed value/actual value = 1.283					
Average deviation of computed value from the actual value = + 28%					

3. Searching for an explanation of this general trend in "2", either one or both of the following possibilities might be thought responsible:

a. using measured values for individual components was not adequate.

b. some of the assumptions incorporated in the theoretical analysis are not valid.

4. Penetrating deeply into this point, the author has been inclined to conclude that, even though the present measurements were extensive and more adequate than in previous work with conventional mortars, the tests could have been carried out in a more comprehensive manner. In other words neither "a" nor "b" appeared to be valid for a great part of the responsibility for this deviation. The following discussion shows how this may be justified.

a. Considering the mortar, all the mixes were tested not far beyond the recoverable or elastic limit. The result is that the measured deformation properties, in spite of giving successful relative measurements, do not represent the actual values near failure. As the load approaches failure E decreases. At the same time, according to the previous investigations and present work's indications ν increases. In other words, the numerical value of the ratio ν_m/E_m, most probably, increases with increase of load. Substituting a higher value for this ratio in the above formula gives a calculated strength of lower value than the one obtained when applying the values corresponding to the present tests. Thus the higher the load at which E and ν are measured, the more correct will be the predicted values for strength.

b. Considering the bricks, the measured deformation properties have already been discussed. As bricks are of higher rigidity than mortar the present results may indicate that the numerical value of the ratio ν/E_m is less affected.

5. It might be claimed that assuming E_{tb} equal to E_{cb} may be responsible. But the author believes that although this might be true the above factors appear much more influencial.

6. It is felt now that measuring the stress-deformation relations up to the highest possible load, using automatic recording for both mortar and brick would appear very profitable in this respect.

8.13.3 Specimens With Stiff Mortar

With these specimens the basic assumption in deriving the strength formula does not exist. A careful inspection of the formula shows that it cannot be applicable unless the numerical value of ν_m/E_m is greater than ν_b/E. Therefore, comparing actual values with calculated ones, using this formula was out of the question. The following is an attempt to explain these results.

Referring back to Chapter 6 it was shown clearly that testing of bricks between hard materials had given a strength not only higher than those tested between soft materials, but also higher than the actual compressive strength. With the present tests mortar mixes X_1 and X_3 have both the compressive strength (based on the upper and medium limits) and the modulus of elasticity were greater than bricks. At the same time Poisson's ratio is less. With these conditions one would have expected an assemblage failure strength higher than the brick strength. This did not happen with either type of brick used. Here near the end of the project, in a similar manner to the beginning, a big question arises again. That is: Why has the strength of assemblage in this region failed to be eludicated with the properties of the components while the present measurements can be considered of a fairly high degree of adequacy.

To answer this question it was necessary to consider the parameters which may govern the assemblage strength under the present conditions of relative rigidity. Before this, two important points should be stated.

a. It is not only the present tests (with X_1 and X_2) which gave an assemblage strength smaller than the strengths of the components, but also previous tests showed the same phenomenon. (See Figure 1.2 - middle graph).

b. Attempting a theoretical analysis based on mortar having more rigidity than bricks would be expected to fail in advance for two main reasons. The first is that using the same unit shown in Figure 3.1 with this relative rigidity would lead to a shear failure (see Photograph 6.8 - second to the left) and not a splitting one. With proper specimens (enough height for diminishing the loading platens effect) shear failure has never been obtained with conventional mortars. In fact this was the main basic point when contributing to the current specifications as regards brick testing. The second is that the masonry compressive strength was shown to be lower than the strength of the components. These two points were considered enough to expect no profit from such analysis with respect to the point considered.

At this point the author was inclined to suggest that the only parameter responsible for this reduction in assemblage strength is the non-uniformity of vertical stresses along one of the horizontal courses. It appears that not withstanding that the workmanship is excellent and the loading ends are parallel, irregularity inside the wall is created. In fact, many previous contributors mentioned this but it was only speculations during the analysis of the ultimate-strength values and not detailed tests on properties of constituents. However, assuming that the prime mode of compression failure in brick masonry is vertical splitting it is now in order to summarize this phenomenon.

It is well known in the field of concrete technology that within the plastic range of workability (starting from the lean range) the modulus of elasticity increases as the compressive strength increases. In the same way the latter increases as the water content or degree of workability decreases. Due to the fact that the suitable workability for bricklaying lies within this range the relation between the compressive strength and modulus of elasticity was shown to exist with the tested mixes. Therefore, it can be said that the result of using a cement mortar is meant that its strength and rigidity in the hardened state increase while its workability in the fresh state decreases. The latter is already known in the field of brick masonry. The result is that even with best care in laying the bricks to produce non-uniform and imperfectly full joints, some of the bricks are evenly supported by their mortar bed. Also mortar supports become of rigid type. This produces, whenever the external load is applied uniformly, concentrations of stresses at some portions of any arbitrary horizontal section. Consequently, in addition to the transverse-tensile stresses in bricks, it may be subject to flexural and shear stresses. Keeping in mind that brick is a brittle material an early failure due to all these factors may be justified. In the author's opinion this may explain the fact that superior workmanship is of more influence when using low strength bricks and stiff mortar. (Reference may be made to Chapter 1, Parson's observations).

It was fortunate, very near the end of the present work, that a new term called " the coefficient of uniformity" was introduced by Hilsdorf(63). The present author agrees to a great extent with a part of his hypothesis, and desagrees with the other part. This will be discussed in the following paragraphs.

Hilsdorf defined the non-uniformity coefficient as the ratio between the maximum normal stress observed within the brick to the average stress acting on masonry. Quoting from him, the coefficient of non-uniformity is a function of the applied load. At low stresses as the external load increases local yielding or crushing of mortar at points of high stress concentration results in a more even distribution of stresses. As failure approaches, the non-uniformity coefficient rapidly increases. In the present author's belief this may be conceivable.

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(63): Hilsdorf, H.K. An investigation into the failure mechanism of brick masonry loaded in axial compression. A paper presented to the International Conference on masonry structural systems, Austin, Texas, 1967.

Then he added that the coefficient of non-uniformity at failure is a function of the strength and workability of the mortar. As the mortar strength increases the coefficient decreases. The coefficient should be larger for mortars with low workability than with mortars of high workability. Also according to him, this tendency could not be clearly deduced from the available data. The present author believes that these are contradictory ideas. It is well known that within the range discussed, which most probably covers our case, the mortar strength increases as the workability decreases. Therefore it is impossible that the coefficient decreases (due to the increasing strength) and increases (due to the decreasing workability) at the same time. It appears that Hilsdorf's tests did not cover the practical range for bricklaying.

Another factor which may affect the strength of brick masonry markedly is the irregularity of the bed faces of the bricks, even though they are well shaped. An example can be given from a recent paper by Astbury and West(7). and assembled without mortar had a crushing strength of 14500 lb/in^2. The mean crushing strength of unground bricks assembled as cubes without mortar was 8274 lb/in^2 and with 3/8" mortar joints 9111 lb/in^2.

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(7) Astbury, W.A. and West, H.W.H. Tests on storey-height brickwork panels and developments of a site control test for brickwork. British Cer.Res. Association, Stoke-on-Trent, England. Session IX - Construction. August 1967.

In the light of the observations on the present tests the author is inclined to attribute the troubles resulting from cement mortar mainly to its stiffness in the hardened state. An increase in stiffness means no "give" in the mortars, an increase in the degree of non-uniformity, creation of rigid supports, and bricks subject to transverse, shear and flexural stresses.

In order to make the discussion conclusive the question with which any engineer is concerned arises. That is: If these sources of troubles are inevitable, how will it be possible to predict, and increase the strength of brick masonry under these conditions? The author's reply in short is in two parts based on some of the present work as follows:

a. In addition to all the traditional recommendations, the height of the brick should be increased. Such increase gives better resistance to shear and flexural stresses as well as the transverse stresses. This can be easily proved from the strength of materials.

b. Predicting the strength from the properties of individual components may be very difficult, even impossible, if prediction is based on theoretical basis. Simulating conventional mortars in brick testing appears the quickest and preparation of data or charts for this object may be practically the least tedious for research workers.

CHAPTER 9

SUMMARY, PRINCIPAL CONCLUSIONS AND SUGGESTIONS FOR FUTURE RESEARCH

9.1 ORIENTATION

Three statements can be made about the present project:

1. It does not claim to be more than an introduction to the fundamentals of composite action in assemblages of two-phase materials having different structural properties, together with some emphasis on the properties of the individual phases. The difference between properties was produced by the use of bricks and conventional mortars, or bricks and other materials replacing mortars.

2. The scope of the investigation might appear to have been extended from its topic into various subjects, but this was inevitable. The author's belief is that a two-phase material such as masonry could never be understood on the fundamental level until the structural properties of its components are adequately understood.

3. Because the study was concerned with fundamentals, the use of scale model bricks and simulated mortars proved to be profitable in producing, besides the experimental data, a considerable number of basic ideas.

However the purpose of the present chapter is to conclude the work described in this thesis in terms of a summary of the important points, principal conclusions, and suggestions for future research. In this respect repetition is inevitable for the sake of completeness, and it was thought better to group the text under general headings relevant to different aspects of the work rather than to summarize the individual chapters. However under each heading the chapters which make up the references to it are given.

9.2 SUMMARY AND CONCLUSIONS

9.2.1. Failure Criteria of Brick Masonry

(Ref: Chapters 2,3,4 and 8).

9.2.1.1. Brick masonry with bricks having more rigidity than mortar.

On the basis of complete heterogeneity in a brick masonry assemblage under ideal conditions two theoretical analyses were developed.

a. The first is a failure hypothesis for the masonry assemblage when loaded in axial compression, showing that the internal structure of a single leaf wall may be represented by different series of three-space-mesh frames or one-space lattices with ties. Each of these series introduces a useful mechanical-model by which the failure criteria and the most influential factors on the ultimate strength of brick masonry can be successfully visualized.

b. The second is based on action and interaction between the components of the wall. Considering their individual properties together with the internal stresses and strains, expressions were derived for the determination of: modulus of elasticity within the uncracked limit, critical cracking load (a new concept)and ultimate failure load.

Tension was shown to be the primary cause of failure. The direction of cracks at failure follows the lines of maximum tension in both idealized and actual units, vertical splitting being a succession of tensile failures. Structural properties having greatest influence may be grouped as follows:

a. Mortar properties (E_m, ν_m, f_{tm})

b. Brick properties (E_{tb}, ν_b, f_{tb})

c. Relative physical properties (height of brick in the vertical direction). Variables relevant to "a" and "c" were chosen as objects of primary concern in the later stages.

9.2.1.2. Brick masonry with stiff mortar

With stiff mortar, even with the best workmanship and the greatest possible uniformity in the dimensions of the bricks and the bed surface, there will occur under loading progressive non-uniform of vertical forces acting on the bricks on either side of the vertical joints. Therefore in this case, in addition to tension, bricks may be subject to flexural and shear stresses, and the coefficient of non-uniformity of vertical stresses along one or more of the horizontal courses may be the most influential parameter . Due to the fact that such a coefficient is a function of many factors, including workmanship and degree of irregularity in the bed faces of the brick, which cannot be measured, developing expressions for strength is not likely to be based on more than hypothetical assumptions.

9.2.2. Testing Techniques

(Ref.: Chapters 6,7 and 8)

9.2.2.1. Deformation properties

The stress analysis of a circular specimen in an indirect tensile test was described and developed to yield new expressions for evaluating the stress-deformation relations of a mortar-like material. Charts and tables were also developed. On the same basis of calculation the specimen was adopted to a new technique for determining the deformation properties of mortar.

With the objects of assessing experimentally the validity of the technique itself and of selecting the most suitable specimen for the present tests a series of steel specimens was tested. It was shown that:

a. Homogeneous and isotropic square specimens compressed along the centre line and perpendicularly to the sides are very similar to circular specimens in both cases of plane stress (disc or plate) and plane-deformation (cylinder or block) and can be employed for assessing modulus of elasticity and Poisson's ratio.

b. A square specimen allows an easier laboratory technique, but on the whole, the technique is not so easy as was expected.

c. Whether a circular or square specimen is used, consideration of the outer dimensions expressed in terms of the dimension ratio, is of vital importance when measuring the deformation properties. For a disc or plate and a cylinder or prism the dimension ratio (height / width along the centre line of action of the load) should be higher than about 6.7 or less than 2.4 respectively.

d. From the calculation side the proposed technique was considered more reliable and convenient when applied to the plane-stress conditions.

9.2.2.2. Tensile strength

The proposed test was successfully used for the determination of the mortar tensile strength. Also for the same purposes it was possible to develop and use the test for bricks (or any other material which cannot be casted in the laboratory).

9.2.3. Structural Properties of Some Conventional Mortars

(Ref.: Chapters 5,7 and 8)

Only five conventional mortars were investigated. The tested mixes were approximately of a workability suitable for practical bricklaying. Their properties according to the techniques used are summarized in the following table.

Property	Mortar - C:L:S by volume			1:0:3	1:$\frac{1}{4}$:3	1:1:6	1:2:9	1:3:12
Strength Properties	Compressive strength limit	Upper	lb/in^2	6400	5900	1450	850	500
		Medium		4650	4300	1150	550	350
		Lower		2900	2700	850	250	200
	Tensile strength	Direct	lb/in^2	548	524	227	137	69
		Indirect		471	467	151	76	51
		Flexural		1267	1432	435	375	187
Deformation properties	Modulus of elasticity x 10^6		lb/in^2	2.69	3.53	1.67	1.01	0.80
	Poisson's ratio		ν	0.10	0.16	0.11	0.26	0.24

General remarks are briefly as follows:

1. Upper and lower limits do not refer to the range in a set of readings; the values are concerned with different methods of testing (different shapes and sizes of specimen, with or without using M.G.A. pads, etc.)

2. The identical form of relation was found for the tensile strength and the compressive strength.

3. Because of the limited number of mixes, the relation between the tensile strengths was not found definite. Direct and indirect tensile strengths were approximately equal, each of them roughly half the flexural tensile strength.

4. The range for the relation between the compressive strength and the tensile strength as obtained from the square

plate specimens compared well with the same range obtained by previous investigators when a cylinder was used for the tensile test.

5. Applying the load up to 20%-54% (differing according to the mix) of the ultimate failure load in an indirect tensile test produced a linear load-deformation relation. In other words the mortar obeyed the elastic theory within certain limits.

6. The relation between the compressive strength and modulus of elasticity compared well with previous investigators on concrete.

7. The change of Poisson's ratio with respect to different mixes was much less consistent than that of any other measured property.

8. No relation could be found between Poisson's ratio and any other measured property.

9. The range of loading used was not enough to provide strong evidence of the trend of change in the values of E and ν. It is felt, however, from the limited number of readings that E decreases and ν increases, probably due to the formation of fine longitudinal cracks.

10. Generally speaking it appears that mortar in brick masonry research is quite different from mortar in the area of concrete technology.

9.2.4. Properties of Conventional Mortars Influencing Brickwork Strength of Brickwork on Compression
(Ref.: Chapter 8.)

On the basis of the test results the conclusions are:

1. Modulus of elasticity and Poisson's ratio of mortar emerged as the most influential properties of the mortar affecting the compressive strength of the assemblages.

2. When bricks were rigid compared with mortar, compensation between them has the leading role in determining the strength.

3. When the contrary case exists the mortar's modulus of elasticity may be the sole influential factor, and on the whole mortar in this case tends to create inequality of inside loading.

4. Although it was not very clear the mortar tensile strength becomes the third in sequence. In the meantime, if it is to be defined consideration must be given to the method of measurement.

5. The traditional view of the influence of mortar compressive strength on the compressive strength of brick masonry appears to be unchanged. For the same group of materials more than one traditional curve may be obtained depending on the method (shape, size of specimen and condition of loading) of mortar testing.

9.2.5. Prediction of Failure Compressive Strength of Brick Masonry
(Ref.: Chapters 6 and 8)

It cannot be claimed that the present work has succeeded in absolute terms in offering a method for predicting the compressive strength of brick masonry in terms of the measured properties of the individual components. But it can be said that, for the conditions analysed, the calculated values agreed approximately with the actual values even though there was some scatter. At the same time the project has suggested a new general method which, although it is empirical, has proved its success. At this point it can be said that by the end of the project the way of looking at brick masonry and its components had undergone many developments. However, the test results suggested the following methods of predicting the compressive strength of brick masonry.

1. Theoretical-experimental approaches

a. Brick masonry with mortar having less rigidity than the bricks. Predicting the strength by mathematically derived formulae has been successful. The general deviation between calculated and actual values appeared to be attributable to the order of load at which the properties were measured rather than to any other factor. The present work has already shown how extensive is the work still required before the method can be put forward in the form of a practical tool, covering all conventional mortars, grades of bricks, different types of bonding and shapes of structural members. By covering all these factors comprehensively, general applicablity of the present strength formula, or any similar one, could be achieved.

b. Brick masonry having stiff or high strength mortar: To achieve agreement between calculated and actual values appears, if not impossible, extremely difficult. Formulation of an expression which can be generally applied requires investigation of other factors besides those previously mentioned. To realize the difficulty it is enough to consider the degree of irregularity expressed by the coefficient of non-uniformity The latter was suggested by Hilsdorf[63] as a function of five variables, including the quality of workmanship. The author believes that this is more likely to be defined on the basis of hypothetical assumptions than on experimental values.

2. Experimental approach based on a new method for brick testing: As mentioned above the present work has already given successful examples of the use of other materials in place of the conventional plywood sheets in brick testing, thus achieving prediction of the corresponding brickwork strength with an accuracy of ± 10%. With the bricks used, the materials which gave these results were polythene sheets, rubber and rubber-with-fibres. It follows that if the standard method of brick testing incorporated in B.S. 3921:1965[18] were to be amended by the substitution of these soft materials for the plywood, it is very likely that the same degree of accuracy would be achieved.

In the present work, the soft materials used were rubbers (with and without reinforcement), and polythene, having different grades and thicknesses. Undoubtedly, the thickness required with full scale bricks to produce the same degree of accuracy (± 10% of brickwork strength) will be greater than those relevant to scale model bricks, by an uncertain amount which can only be determined by further experimental work. As far as the author can judge, from the examples given, this approach appears to be the most profitable

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(18): British Standard Institution. B.S. 3921: 1965. Bricks and blocks of fired brick earth, clay or shale.

and efficient for covering all the influential factors: types of mortars, grades of bricks, thickness of joints, and patterns or bonding systems.

9.2.6. Possibilities of Increasing the Compressive Strength of Brick Masonry in Compression.

(Ref.: Chapters, 1,2,3,4,6 and 8)

9.2.6.1. General

The author suggests that these possibilities can be classified into three groups; traditional, easy-and-economic, and difficult-and-expensive. The former, although listed, have been suggested either in whole or in part by many authorities. The latter two groups are based on the present project.

9.2.6.2. Traditionally recommended possibilities

In short, they are: using bricks of high compressive and tensile strengths; ensuring minimum coefficient of variation for both strength and dimensions; increasing cross-sections under tension; keeping perforations to a minimum; level bed surface; uniform joints; improving the kind of cement; controlling quantity of cement; improving sand-grading; adding a certain amount of lime for improving the workability.

The author does not seek to minimize in any way the importance of these recommendations, especially after many of them are fully justified theoretically and experimentally. It must, however, be said that they have been known for such a long time, that the ultimate benefit from them should by now have been achieved.

9.2.6.3. Suggested easy and economic possibilities

One of the main results yielded by the present work is the increase in strength achieved by increasing the brick height, which was demonstrated in both theoretical and experimental analyses. Assuming that it will be difficult, as described later, to produce bricks of greater height this can be achieved in one of the following ways:

1. The use on a wider scale of blocks which are already commercially available in a variety of sizes. The Code of Practice C.P.111, based on limited evidence, allows the greatest increases in permissible compressive stress for walls built with blocks, up to a certain maximum crushing strength having a height to thickness ratio of 2 to 3(this ratio for a common brick is 3/4).

2. With present clay standard bricks of reasonably uniform properties an increase in brickwork strength may be expected of roughly 20% when the bricks are laid on edge compared to the conventional (flat) method of bricklaying. This would inevitably meet with difficulties in persuading people even to contemplate changing the deep-rooted traditions of building in brick, as well as the practical problems of bonding and the thickness of joints and of the walls themselves.

9.2.6.4. Difficult and expensive possibilities

1. Producing bricks of greater height

2. Improving mortar properties so as to achieve compensating values of modulus-of-elasticity and Poisson's ratio.

In the author's opinion it might be easier to make changes in the physical properties rather than the structural properties. On that basis it would be a very profitable solution to adapt production lines for bricks of greater height. As has been proved, a brick of a greater height will give the most efficient state of balance between bricks and mortar. What may be expected in addition is that the trouble taken with mortar properties in the hardened state would decrease to a minimum. This is of course apart from the failure compressive strength which may occur in the horizontal mortar joint between the vertical joints, which might be the optimum strength.(Figure 1.8) But the economic **implications** of adaptation of production lines is a big question, and far beyond the author's ambitions to answer.

9.3 SUGGESTIONS FOR FUTURE RESEARCH

9.3.1. Mortar Technology

1. Though the mixes tested were within a considerable practical range the results obtained cannot be claimed to cover this range adequately. Much useful confirmatory data for the present results could be gained from a further series of tests on the same lines, covering mortar mixes mainly within the present range, with a few mixes outside it. All the tests included in the present work could profitably be repeated as well so that the gradual change in the properties can be observed, more accurately than with the present mixes.

2. The low values of Poisson's ratio for mixes having higher cement content (mixes of C:L:S = 1:0:3 and 1:$\frac{1}{4}$:3) have been explained on a speculative basis, but the phenomenon suggests that the fundamental structure of mortars within the range of mixes used in bricklaying should be carefully examined. This might give a better understanding of mortar's deformation and failure characteristics. The author believes that defining the range, as stated, is a point of vital importance. To clarify this an important fact should be remembered, namely that when mortar is investigated in the area of concrete technology, it is usually looked upon as concrete with fine aggregate, and no lime is included in the bonding materials. Therefore the conclusions yielded do not necessarily apply to mortars used with brick masonry. Adding lime to cement may be responsible for producing considerable changes in the mechanism of failure. This requires a comprehensive investigation and the present

author hopes to achieve a part of the work relevant to this point in the near future.

3. Since the beginning, the present work has been carried out on the basis of complete heterogeneity in brick masonry. Now it can be realized how much difference it makes to assume heterogeneity instead of homogeneity. In a similar manner mortar may be looked upon as a two-phase material, and for this reason a sand investigation should be carried out along two lines as follows:

a. The limits incorporated in B.S:1200 represent a wide range and there is a strong probability that dividing this range into three or four zones may have a great influence on the properties. Therefore a study is required to show the effect of variation of sand grading on the structural properties of mortar mixes within the practical range of workability for bricklaying.

b. The quality of sand needs a more precise definition than is generally realized. Parameters should include the average particle size, fineness modulus and shape of particle. Therefore the same tests suggested with sand gradings should be repeated using different types of sands.

9.3.2. Tests on Scale-Model and Full-Scale Bricks and Brick Masonry Assemblages

A long-term test programme of scale-model and full-scale bricks and assemblages is required as further research projects with a view to making the new contributions obtained here, more meaningful. As far as the author conceives priorities, they may be listed as follows:

1. Repeating some of the work included in Chapter 6, on the influence of end and joint conditions of different rigidities on the failure characteristics of brick having the same horizontal cross section, and different heights.

2. Carrying out the same tests, suggested in "1", with brick masonry assemblages using conventional mortars.

3. In the light of "1" and "2" together with the examples given in Chapter 9 for introducing end materials in brick testing, a third series which should be as comprehensive as possible with full scale bricks is required. The latter may form the basis for a new test to replace the standard test for bricks incorporated in B.S.:1257.

4. An investigation is required to study how far the strength of brick masonry units can be increased by laying bricks on edge with all possible methods of bonding.

9.3.3. Testing Techniques

9.3.3.1. Deformation properties

As noted before, the new technique for measuring "E" and "ν", althoughbasically sound needs further assessment as regards the size of specimen, rate of loading and reproducibility of the results. Consequently large test series with castable materials like, mortar and uncastable materials, like bricks, would be useful both as a research tool, and for measuring such basic properties. It would also be useful if, parallel to these tests, others were carried out using some conventional methods for comparison.

9.3.3.2. Strength properties

1. As has already been described the standard test for assessing the performance of bricks in brick masonry should be urgently changed by replacing plywood sheets usually recommended in B.S.:1257 by soft materials, as rubber, polythene or other similar materials. The specification writers should start thinking of doing this by calibrating mortars against other soft materials. This may be carried out on the same lines as the corresponding part of the present work (Figures 8.25,26) and Table 8.6).

2. Employing cubes for assessing compressive strength of mortar needs a more precise definition as regards size.

3. The function of the M.G.A. pads with respect to different grades of strength needs more clarification, before their general use can be recommended.

4. The influence of loading conditions on the type of fracture of a square plate in an indirect tensile test, and the phenomenon of single cleft need further clarification (See Photograph 8.8)

9.3.4. Author's Conclusive Comment

Finally, the author is conscious of the difficulty which lies ahead in the task of changing some of the various factors and techniques well established by custom, and carrying out the suggested future research. Nevertheless, he believes that the views and opinions incorporated in the present thesis are supported by limited, though consistent results. This, together with the data, previously known about failure characteristics, the failure of attempts to elucidate these characteristics in terms of traditional properties of components, and the increasing interest in the use of load-bearing brickwork for high buildings, encourage investigation of all aspects of change. The author feels confident that this change in both research and practical fields will help brick masonry to be successful in economical use and reliable prediction of its strength.

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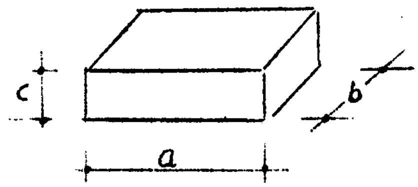
Appendix 6.1
(See page 6.103)

Dimensions and cross-sectional areas of $\frac{1}{3}$ scale model brick.



Property		Dimensions			Cross-sectional areas		
		a inch	b inch	c inch	a x b $inch^2$	a x c $inch^2$	b x c $inch^2$
Mean value		2.790	1.489	1.040	4.153	2.903	1.549
Range	Minimum value	2.750	1.478	1.022	4.150	2.840	1.526
	Maximum value	2.807	1.497	1.052	4.195	2.948	1.567
Standard deviation		0.0895	0.0043	0.0088	0.0353	0.0359	0.0320
Coefficient of variation %		3.208	0.292	0.846	0.850	1.236	2.06
Number of specimens		8	8	8	8	8	8

Appendix 6.2
(See page 6.104)

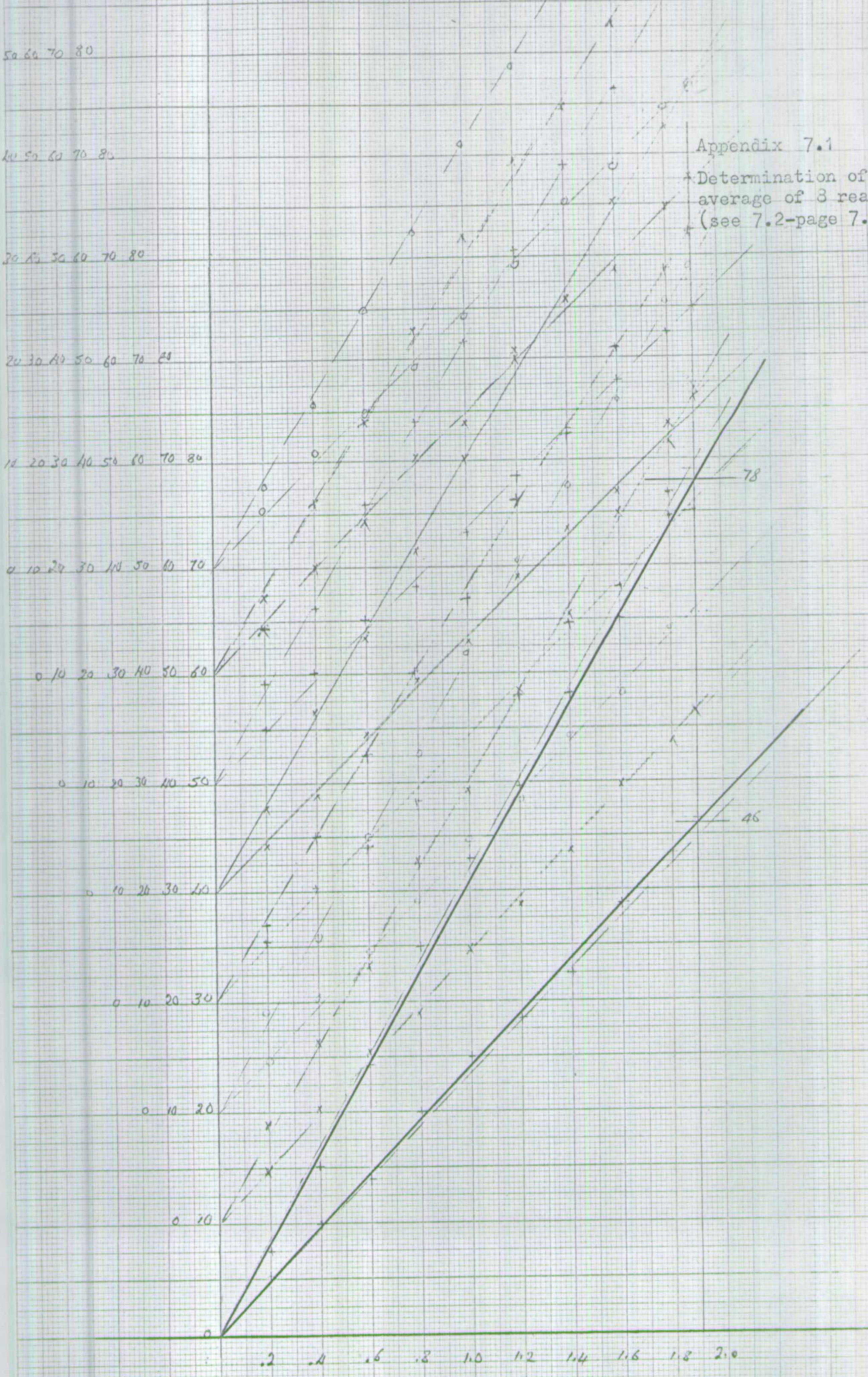
Failure characteristics of one-third-scale model bricks with and without end bricks. (all bricks were super ground).

Characteristic	Joint material and direction of loading		Range: Min. lb/in^2	Range: Max. lb/in^2	Mean lb/in^2	Standard deviation	Co-efficient of variation	Number of specimens
Apparent failure compressive strength lb/in^2	$=$	Plywood	3803	4018	3938	120.0	3.05	3
		Rubber with fibres	1321	1456	1375	58.4	4.24	3
	$\neq$	Plywood	4099	4639	4424	246.0	5.56	3
		Rubber with fibres	1564	1591	1582	102.6	6.48	3
	$\equiv$	Plywood	1966	3009	2404	700.0	29.12	3
		Rubber with fibres	1235	1427	1331	84.5	6.32	3
	$\not\equiv$	Plywood	3488	4151	3755	286.0	7.61	3
		Rubber with fibres	1466	1582	1517	65.8	4.33	3
	$\|\|\|$	Plywood	2603	3269	2834	313.0	11.04	3
		Rubber with fibres	1547	1808	1644	116.0	7.00	3
	$\not\|\|\|$	Plywood	3832	4989	4579	526	11.48	3
		Rubber with fibres	1301	1377	1336	32.4	2.40	3

Characteristic	Direction of loading	$=$	$\neq$	$\equiv$	$\not\equiv$	$\|\|\|$	$\not\|\|\|$
Influence of height	Plywood	3803	4099	1966	3488	2603	3832
	Rubber with fibres	1321	1564	1235	1466	1547	1301
Mode of Failure	Plywood	Shear and crushing failure	Shear failure	No distinctive mode of failure	Shear failure	Shear and splitting failure	Shear failure
	Rubber with fibres	Splitting in middle brick and few crushing	Splitting and few crushing	Splitting in end or end brick and few crushing	Splitting and few crushing	Splitting and few crushing	Splitting and few crushing

Appendix 7.1

Determination of the average of 8 readings (see 7.2-page 7.16)



Appendix 7.2
Reproducibility of readings
(See 7.2.52 - page 7.14)

1	4th loading	+	0	0	4
0		-	0	1	1
1		+	0	0	0
0		+	0	0	0
1	3rd loading	+	0	5	3
0		-	0	9	7
1		+	0	0	0
0		+	0	0	0
1	2nd loading	+	0	5	4
0		-	0	9	8
1		+	0	0	0
0		+	0	0	0
1	1st loading	+	0	5	4
0		-	0	9	8
1		+	0	0	0
0		-	0	0	0
1		+	0	0	0
0		-	0	0	1
1		+	0	0	0
0		-	0	0	1
1	Instrument adjustment	+	0	0	0
1		+	0	0	0
1		+	0	0	0
0		+	0	0	0
0		+	0	0	0
0		+	2	0	0
0		+	2	9	0
0		+	2	9	0
0		+	2	9	0

Start point

0		-	0	8	8
1	5th loading	+	0	5	4
0		-	0	9	8
1		+	0	4	7
0		-	0	8	8
1		+	0	4	1
0		-	0	7	8
1		+	0	3	7
0		-	0	6	9
1		+	0	3	1
0		-	0	5	9
1		+	0	2	5
0		-	0	4	9
1		+	0	2	0
0		-	0	4	0
1		+	0	1	4
0		-	0	3	0
1		+	0	0	9
0		-	0	2	0
1		+	0	0	4
0		-	0	1	1
1		+	0	0	0
0		+	0	0	0
1	4th loading	+	0	5	3
0		-	0	9	8
1		+	0	4	7
0		-	0	8	8
1		+	0	4	2
0		-	0	7	8
1		+	0	3	6
0		-	0	6	9
1		+	0	3	1
0		-	0	5	9
1		+	0	2	5
0		-	0	5	0
1		+	0	2	0
0		-	0	4	0
1		+	0	1	4
0		-	0	3	0
1		+	0	0	9
0		-	0	2	0

1		+	0	4	7
0		-	0	8	7
1	6th loading (6th cycle)	+	0	5	3
0		-	0	9	6
1		+	0	4	7
0		-	0	8	6
1		+	0	4	2
0		-	0	7	6
1		+	0	3	6
0		-	0	6	7
1		+	0	3	0
0		-	0	5	7
1		+	0	2	5
0		-	0	4	8
1		+	0	2	0
0		-	0	3	8
1		+	0	1	4
0		-	0	2	8
1		+	0	0	9
0		-	0	1	8
1		+	0	0	4
0		-	0	0	9
1	Unloading (5th cycle)	+	0	0	0
0		-	0	0	1
1		+	0	0	4
0		-	0	1	0
1		+	0	0	9
0		-	0	2	0
1		+	0	1	4
0		-	0	3	0
1		+	0	2	0
0		-	0	3	9
1		+	0	2	5
0		-	0	4	9
1		+	0	3	1
0		-	0	6	0
1		+	0	3	7
0		-	0	6	9
1		+	0	4	2
0		-	0	7	8
1		+	0	4	7

End point

1	Unloading (6th cycle)	+	0	0	0
0		+	0	0	0
1		+	0	0	3
0		-	0	0	9
1		+	0	0	9
0		-	0	1	8
1		+	0	1	4
0		-	0	2	8
1		+	0	1	9
0		-	0	3	8
1		+	0	2	5
0		-	0	4	8
1		+	0	3	1
0		-	0	5	8
1		+	0	3	6
0		-	0	6	7
1		+	0	4	2
0		-	0	7	7