BEHAVIOUR OF CONCRETE

UNDER SIMPLE AND COMBINED STRESSES

by

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ABSTRACT

This thesis examines the problems associated with defining the deformation and failure of plain concrete under simple and combined states of stress including the philosophies of strength involved and the work done in this field of research.

A combined stress testing machine was designed and built to yield the experimental data required for failure criteria both in terms of stress and strain measurements and also mode of failure. The machine was a basically biaxial stress machine with facilities for compression, tension and torsional stresses and capable of extension to triaxial testing. It had a maximum working load capacity of 200 ton compression or 30 ton tension and applied static loads at constant rates of stress.

Detailed proving trials were carried out on the behaviour of the testing machine and the states of stress actually achieved in the test specimens prior to the main series of tests.

A range of thirteen mixes including mortars, normal aggregate concretes, and lightweight concretes were tested under biaxial compression and six mixes under compression-tension. The tests were all short-term static loading tests on saturated specimens at 28 days old. As a result biaxial ultimate and discontinuity stress envelopes were obtained and, together with the results of a complementary series of uniaxial tests, the elastic behaviour of the mixes was examined. It was recognised that two basically different classes of behaviour existed. In general, all the mortar and lightweight aggregate concrete mixes were elastic in behaviour below the discontinuity point with constant elastic modulus whilst the normal aggregate concretes under biaxial compression exhibited a marked increase in elastic modulus.

This distinction was also observed in the biaxial stress envelopes where the mortars and lightweight concretes approximated to a modified Coulomb-Mohr theory of failure whilst the normal-aggregate concretes gave a complex mechanism of failure not explained by any classical failure theory. Failure criteria were proposed and mechanisms of crack propagation examined.

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CHAPTER 1

INTRODUCTION

1.1. Introduction

This thesis describes a research project into the strength and deformation of plain concrete primarily under biaxial stress states in an attempt to establish failure criteria for these states of stress. The project is the latest in a series of research projects into testing techniques and failure criteria for concrete co-ordinated by Mr. K. Newman and executed by the Concrete Materials Research Group at Imperial College under the overall supervision of Professor A. L. L. Baker.

Commencing with the simple uniaxial states of stress and the development of suitable testing machines and experimental techniques, the programme has been extended progressively into more complex states of stress. Additional studies have concentrated on failure theories and the various models designed to simulate concrete behaviour.

Work by Lachance[1, 2, 3] showed the effect of the geometrical proportions of uniaxial compression test specimens on the results obtained in short-term tests and established a prism test as a standard laboratory test for uniaxial compressive strength. The state of stress in the centre third of the prism was shown to be uniform compression and the effects on the stress pattern of various packing materials placed between the loading plattens and specimen were demonstrated.

The Lattice Theory of concrete deformation and failure proposed by Professor Baker[4] was investigated by Anson[5, 6] in a general study into the heterogeneous nature of concrete and, in particular, two-phase approximations to it.

An investigation into the uniaxial tensile strength of concrete by Ward[7] followed in which a standard laboratory test

to obtain the direct tensile strength of concrete was developed. In this project, the various indirect tension tests for concrete were compared with the results from the new direct or uhiaxial test and with the results of uniaxial compression tests on prisms.

In the Autumn of 1961, a comprehensive series of tests was carried out on slab and disc specimens subjected to certain types of flexural stress using testing techniques similar to those proposed by Blakey and Beresford[8]. However a recent investigation by Sigvaldason[9] has shown that the testing techniques did not induce the state of stress assumed in the elastic analysis.

A detailed survey was then made by Robinson[10] into theories of failure in which he compared the theoretical predictions of both stress characteristics and failure mode with the experimental results of other workers. The effects of heterogeneities in concrete were then considered and Anson's work on the Lattice Theory extended into biaxial states of stress. Crack detection techniques were developed[11] and the project was completed with some biaxial compression tests on mortars and concretes.

During the same period, Sigvaldason[9,12,13,14,15] was investigating testing machine characteristics and their effect on the experimental results obtained. He also conducted a detailed study on the slab and disc flexural tests referred to above and developed new testing techniques for these tests to obtain reliable states of biaxial tension and compression-tension flexural stress.

The results of Robinson's work on failure theories and Sigvaldason's on testing machine characteristics indicated that a detailed investigation into biaxial compression and possibly other biaxial stress states might yield valuable experimental data upon which to build a failure criteria.

Accordingly the following terms of reference for the project were drawn up by Mr. K. Newman:-

"Aim to establish the criteria which determine the conditions of equilibrium, cracking and failure of plain concrete subjected to biaxial states of stress. An attempt will be made to define the permissible working stresses that concrete can withstand under all states of biaxial stress, in the form of an envelope."

"Examine all previous work on uniaxial and biaxial stress states, with special reference to:-

(1) the loading systems used to achieve the desired stress state.

(2) the results obtained and the definition of "failure"."

"To design and construct a biaxial testing machine to enable more refined tests in all states of biaxial stress to be made, giving results over any part of the envelope."

Following negotiations with these terms of reference as a basis, the Civil Engineering Research Association agreed to give the project financial support and a two year contract for this work commenced on the 16th September, 1965.

This introductory chapter deals with the examination of previous work and the philosophical considerations which led up to the plan of the research project finally adopted.

1.2. Factors affecting the Mechanical Behaviour of Concrete

Newman[2,3,15,16] and Price[17] have listed and described the factors affecting the mechanical behaviour of concrete. However, it is worth summarising those factors here together with definitions and conventions peculiar to this particular research project. They will serve as an introduction to the review of past research contained in the following section (1.3.) and avoid repetition of definitions in the ensuing text.

Definitions and Conventions

1) Principal stresses are those stresses acting on planes of zero shear stress; simple stress occurs when only one principal stress acts on the value under consideration whilst combined stress occurs when more than one principal stress acts upon the volume on mutually orthoganal planes.

2) The sign convention adopted for principal stresses throughout is tension positive and $\sigma_1 > \sigma_2 > \sigma_3$.

3) As the methods of applying stress uniformly to test specimens are imperfect, shear stresses are often transferred to the test specimen together with the applied stress, especially in the case of uniform tests (defined below). These shear stresses are defined as primary when they are due to machine deflections and secondary when they are due to friction between the test specimen and loading devices.

4) Types of stress are classified in three groups for convenience as follows:-

a) uniform tests - sometimes referred to as direct stress tests - in which the stresses on a plane orthoganal to the direction of principal stress are uniform (18). This research project is concerned with tests of this kind.

b) flexural tests in which there is a stress gradient on the plane orthoganal to the direction of the principal stress. This type of test was investigated by Sigvaldason[9] for the Concrete Materials Research Group.

c) calculated stress tests in which an elastic stress analysis is necessary to obtain the stresses acting upon the critical portion of the test specimen.

This classification includes hollow cylinders but excludes the flexural tests defined above. This type of test is especially vulnerable to serious error when the ultimate stress is calculated using elastic analysis in a non-elastic specimen condition.

5) As has been suggested by both Newman[16] and Shah and Sturman[19] different failure criteria will be required for uniform and flexural states of stress. In this thesis the failure criterion sought is that under uniform states of stress.

6) Concrete mix designation for the mixes used in this research project is summarised in fig.6.4.1. but for mixes used by other investigators the following convention and short-hand description is used, e.g. (1:2.4:5.1:0.55/0.51:$\frac{3}{4}$").

1 - cement/cement ratio by weight
2.4 - sand (3/16")/Cement ratio by weight
5.1 - aggregate (3/16")/cement ratio by weight
0.55 - water/cement ratio by weight
0.51 - volume fraction of coarse aggregate (often approximate)
$\frac{3}{4}$" - maximum aggregate size.

7) Mortar is defined as having a maximum aggregate size of 3/16" or less whilst concrete is defined as having a maximum aggregate size greater than 3/16".

8) Order of loading or loading sequence in biaxial tests can be divided into -

a) "sequence" loading in which one principal stress is applied until its final value is reached and then the other stress is applied
b) "proportionate" loading in which the two stresses are applied together in a fixed proportion to each other.

Factors affecting Behaviour

1) Volume fraction of sand and coarse aggregate in mix. This affects the strains at discontinuity stress (defined by Newman[20]) as demonstrated by Kaplan[21].

2) Maximum aggregate size, as will be shown by the results of this research, determines the biaxial stress behaviour of the mix. The mixes can be classified as mortars and concretes with pastes as a third classification.

3) Specimen shape and size yield statistical effects (16.).

4) Specimen condition in terms of its temperature, humidity and moisture content (16).

5) Rates and patterns of loading. i.e. static or dynamic loading over short or long periods of time (16).

6) Modes of cracking - these determine both the failure criteria for discontinuity stresses and also the inelastic behaviour and ultimate load above the discontinuity stress.

7) Degree of particle interaction will affect the elastic constants and failure criteria (10).

8) Testing and machine effects as investigated by Sigvaldason[9,12-15] i.e., longitudinal stiffness of machine frame, lateral stiffness of machine frame, spherical seating effects, etc. The spherical seating effects include the degrees of freedom of movement of the test specimen at the loaded faces.

1.3. Review of Past Research

The intention of this section of the thesis is to make a brief review and assessment of the principal contributions to the study of the biaxial strength and deformation of concrete. The contributors will be considered in chronological order and attention will be paid to the factors listed in section 1.2. with particular reference the loading system used, investigations into the actual state of stress achieved, and the test results obtained.

An exhaustive examination of all previous work is vital in any field of research prior to any planning or preparation for a research programme. It will be seen in the course of this review that much work has been executed in apparent ignorance of earlier, and in some cases, better, contributions.

1.3.1. Foppl (1900)[22]

The earliest reported work on the biaxial strength of concrete appears to be that carried out by Foppl in 1896 and published in Munich in 1900[22]. It was later reported by Mesnager[23].

He designed an ingenious lever mechanism (fig.1.3.1.1.) which he names a "pressure cross" (literal translation). This device fitted into a uniaxial compression machine and applied equal biaxial compression to the test specimen. The device, however, appears to lack lateral stiffness and the deflection of the frame in the longitudinal direction, shown in an exaggerated form in fig.1.3.1.2., would alter the alignment of the principal externally applied loads on the test specimen inducing primary shear stresses within the specimen.

Cubes of mortar (7 cm.) were tested with stearin and tallow pads between the platens and the test specimen to eliminate secondary shear stresses. A small reduction in strength of about 10% was observed under equal biaxial compression as compared with the uniaxial compressive strength (fig.1.3.1.3.). However, as originally pointed out by Ros and Eichinger[24] and recently confirmed by Robinson and Vile[25], the stearin and tallow pads would tend to squeeze out under pressure and initiate a tensile splitting failure in the specimen.

With cubes tested without the stearin and tallow pads, an increase in strength of 87% was obtained under equal biaxial compression. Though some of this increase can be attributed to the

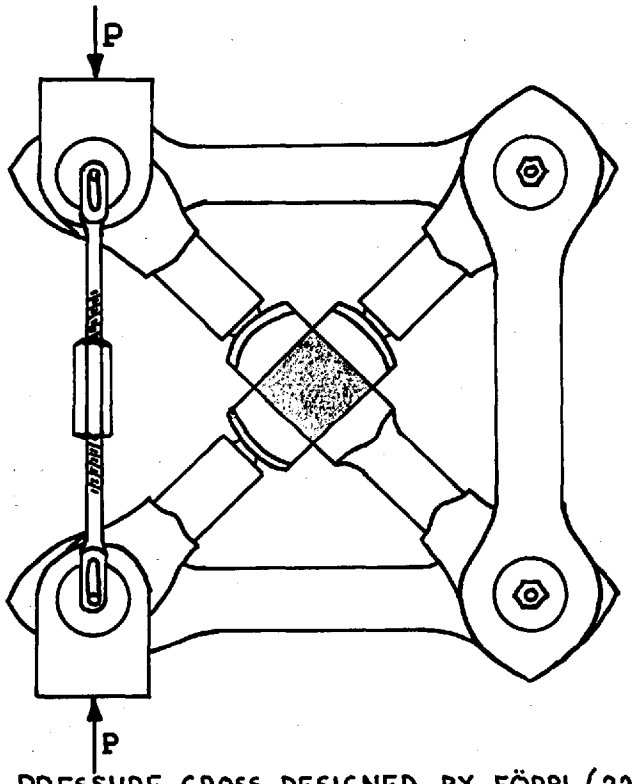


FIG. 1.3.1.1. PRESSURE CROSS DESIGNED BY FÖPPL(22).

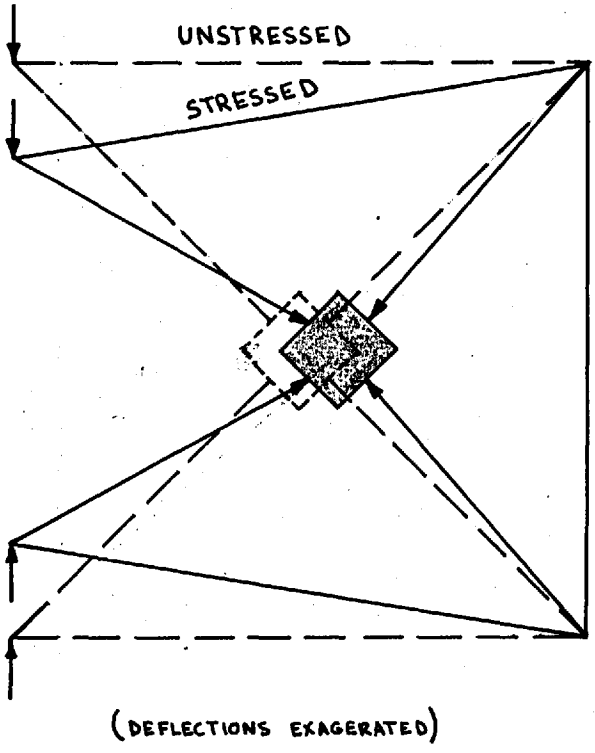


FIG. 1.3.1.2. DEFLECTION OF PRESSURE CROSS DURING TEST.

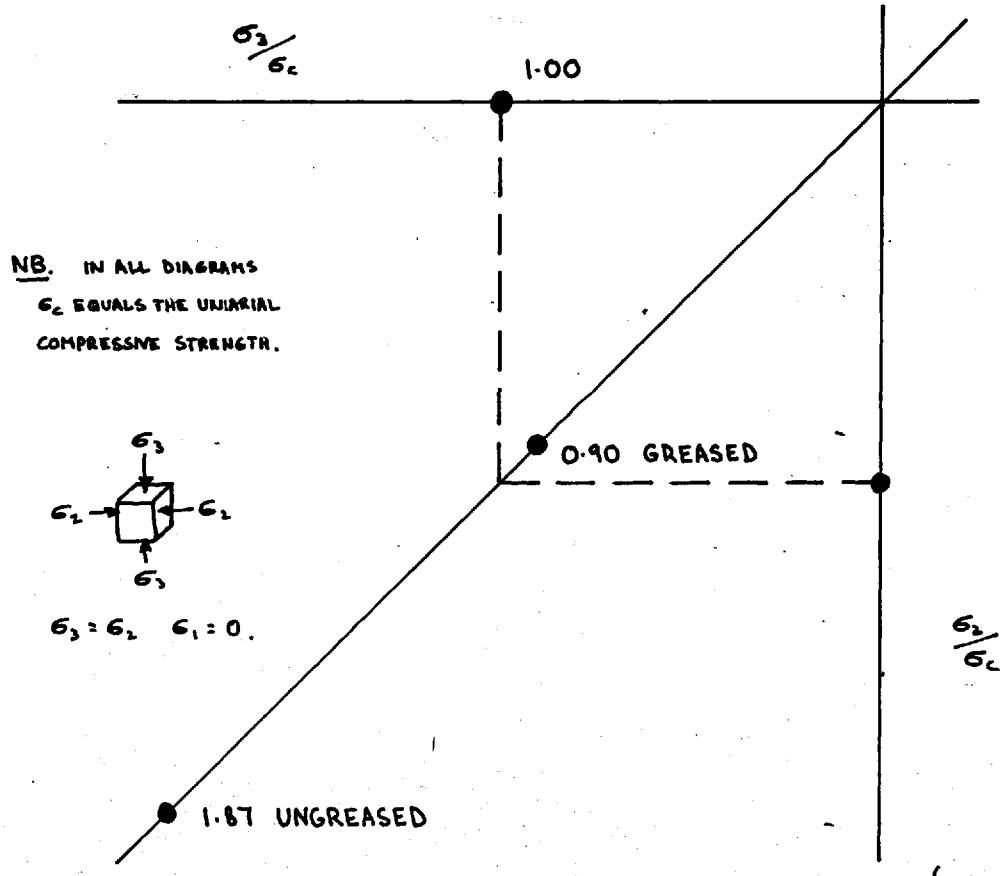


FIG. I.3.I.3 BIAXIAL COMPRESSION STRENGTH OF MORTAR (FÖPPL(22))

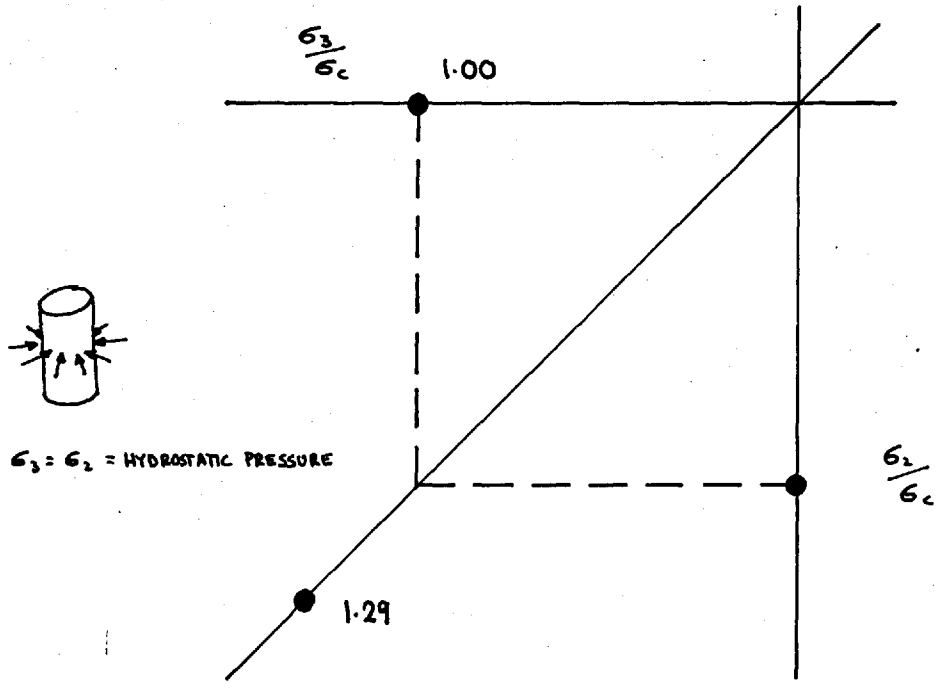


FIG. I.3.3.I BIAXIAL COMPRESSION STRENGTH OF MARBLE (KARMAN(30) + BOKER(31))

absence of the tensile stress action of the stearin and tallow pads, a substantial percentage of the increase would be due to the frictional restraint effect of the loading platens on the cube producing secondary shear stresses. These secondary shear stresses tend to produce a state of triaxial compression in the specimen and hence have a reinforcing effect[1-3,26].

1.3.2. Considere (1904-1907)[27-29]

The work of Considere was also reported by Mesnager[23] in 1923.

Considere conducted experiments on mortar and concrete prisms containing helical binders and compared the results with those obtained on 3 cm. diameter, 8 cm. long solid cylinders subjected to axial compression and radial hydrostatic pressure.

He obtained the formula $\sigma_u = 1.5\sigma' + 4.8\sigma_r$ where σ_u was the ultimate axial strength under a radial pressure of σ_r and σ' was the uniaxial compressive strength (fig.1.3.2.1.). This empirical formula was claimed to be valid for both reinforced prisms and plain cylinders under radial hydrostatic pressure. Furthermore the mathematical analysis of A. Pourcel was cited as corroborating evidence.

Few details of the testing technique were given and so it is difficult to criticise this work adequately.

1.3.3. Karman(1912)[30] and Baker (1915)[31]

The work of Karman and Baker has also been reported by Ros and Eichinger[32], and by Nadai[33].

These two workers carried out mainly triaxial compressive tests on cylindrical marble specimens in which an axial compressive load was combined with radial hydrostatic pressure. It was observed that the marble became almost ductile under triaxial compression and

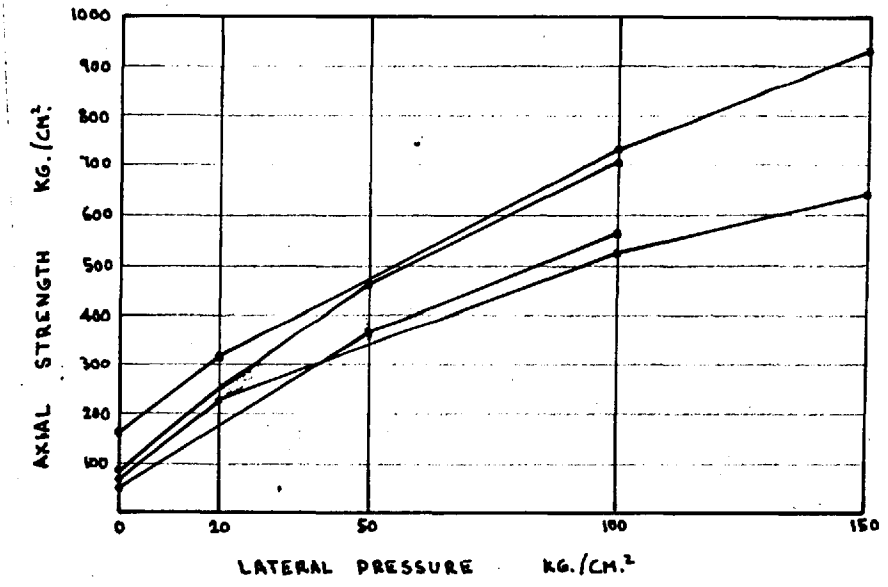


FIG. 1.3.2.1. VARIATION OF AXIAL STRENGTH WITH LATERAL PRESSURE (CONSIDÉRE (23)).

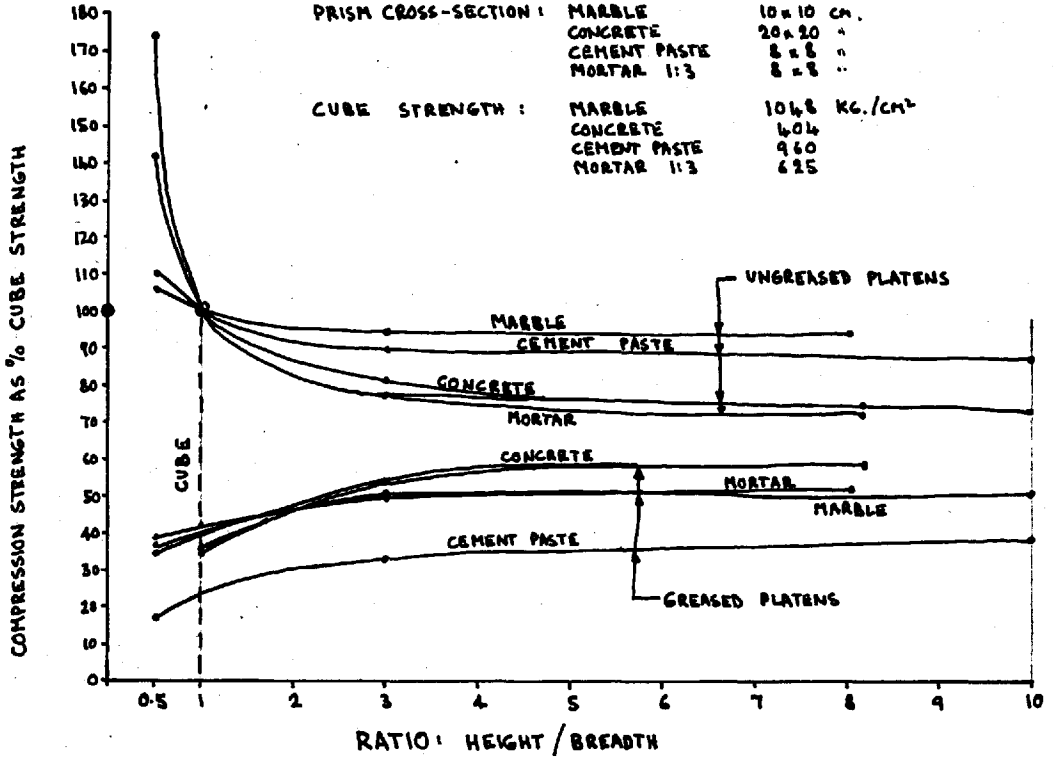


FIG. 1.3.4.1. EFFECT OF HEIGHT / BREATH RATIO AND GREASED PLATENS ON COMPRESSION STRENGTH (ROŠ (35)).

a similar behaviour was noted in specimens of cement paste and mortar. (See later in Chapter 10).

With marble under equal biaxial compression (achieved by applying hydrostatic pressure only) an increase of strength was observed of 29% compared with the uniaxial strength and it was implied that a similar increase might be expected from cement paste and mortar. (fig.1.3.3.1.).

1.3.4. Ros (1925)[34]

This work, first published in 1925, has subsequently been republished with amendments and additions in collaboration with Eichinger in 1928[35] and again in 1949[32] as part of a general investigation into failure criteria for all materials.

Some uniaxial tests were carried out on 12 x 12 x 36 cm. concrete and 4 x 4 x 16 cm. cement paste prisms. An interesting study was made on the effect of specimen height to lateral dimension ratio and greased platens on the strength of prisms for uniaxial compression tests as a result of this a 3:1 ratio of height to width was adopted (fig.1.3.4.1.). Another result of these tests, that has a bearing on the biaxial strength work in this thesis, was the increase in elastic modulus observed with increasing compressive strength (fig.1.3.4.2.).

Unfortunately, no biaxial strength tests as such were carried out on cement paste or mortar, the apparatus used providing axial compression and radial hydrostatic pressure on solid cylinders of 2 cm. diameter, 5 cm. long of cement pastes and mortars.

The choice of test reflects the confidence placed in shear stress failure theories in which the intermediate principal stress was of no significance.

However, a most interesting result which parallels biaxial compression results obtained in this research project, was the variation of overall strain expressed as a comparative stress with the mean normal stress as shown in fig.1.3.4.3.

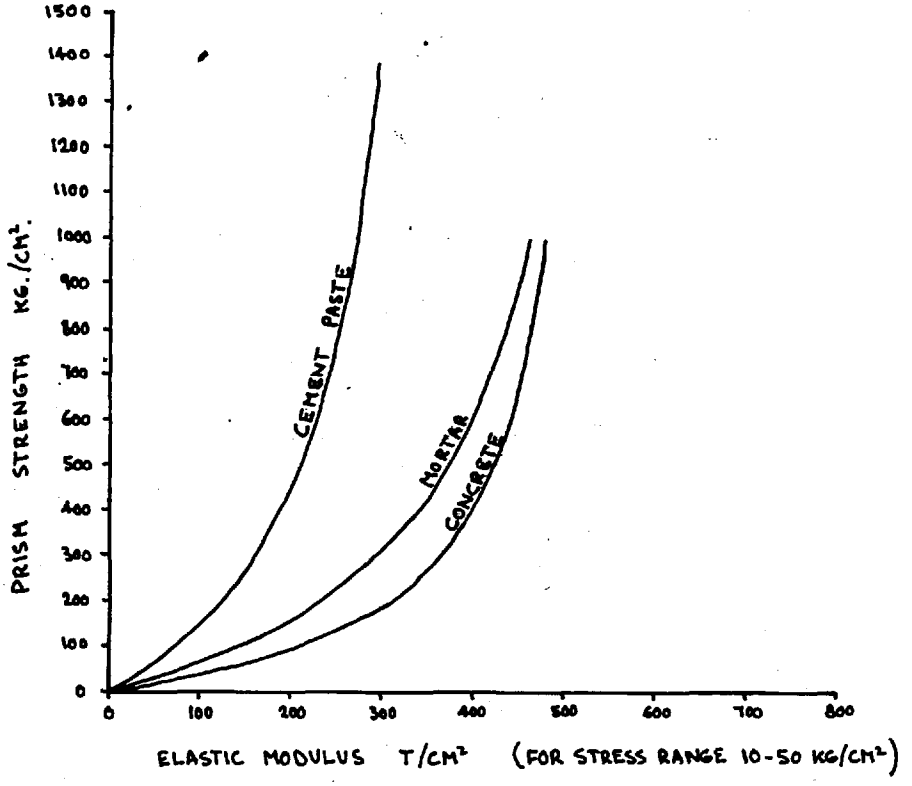


FIG. 1.3.4.2. VARIATION OF ELASTIC MODULUS WITH STRENGTH (ROŠ(35)).

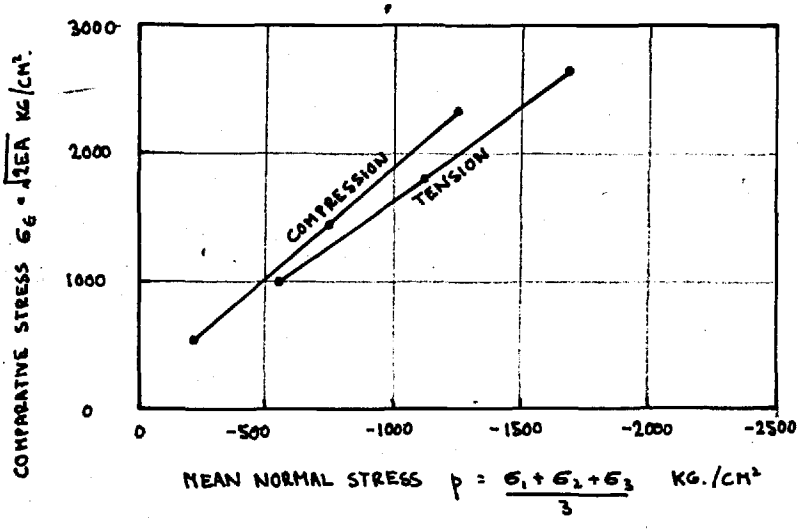


FIG. 1.3.4.3. VARIATION OF STRAIN IN TERMS OF COMPARATIVE STRESS WITH MEAN NORMAL STRESS (ROŠ(35)).

1.3.5. Richart, Brandtzaeg and Brown (1928)[35]

Though primarily an investigation into the triaxial strength of mortars and concretes, this research did include a test series concerned with biaxial compressive strength only.

The tests were carried out on $3^{29}/32$" dia. solid cylinders 22" long overall but with a reduced effective length (about 10") in which equal biaxial compression was applied by radial hydrostatic pressure. Three mortars and five $\frac{3}{4}$" aggregate concretes were tested and six specimens were case from each mix, uniaxial compression tests being carried out on 4" dia. by 8" long cylinders.

A deviation from elastic behaviour was observed at stresses ranging from 67% of ultimate load in lean mixes to 75% with rich mixes. The ratio of equal biaxial strength to uniaxial strength varied from 0.91 to 1.25 with one extreme case of 1.67 but the authors adequately criticised their own testing technique and suggested that the true ratio was about 1.20 to 1.50 for the leaner mixes and 1.10 to 1.35 for the rich mixes (see figs.1.3.5.1. and table 1.3.5.1.).

In addition, a variation in strength was found depending on the direction of loading relative to the laws of compaction.

A cleavage mode of failure parallel to the compressive stress direction was noted under biaxial compression.

A short reference was made to work by Millard at Illinois (1912)[36] in which equal uniaxial and biaxial strengths were obtained using greezed concrete cubes.

The three authors also carried out a comparative study on plain and spirally reinforced concrete under compressive loads (37) similar to the work of Considere.

MIX No.	PROPORTIONS	RATIO σ_B/σ_u
A	1:1	1·00
B	1:2	1·09
C	1:3	0·91
D	1:1:2	1·15
E	1:2·1:2·5	1·00
F	1:2:4	1·25
G	1:3:5	1·24
H	1:4:7	1·67

TABLE. 1.3.5.1. BIAXIAL COMPRESSION STRENGTH RESULTS. (RICHART, BRANDTZAEG, and BROWN (35)).

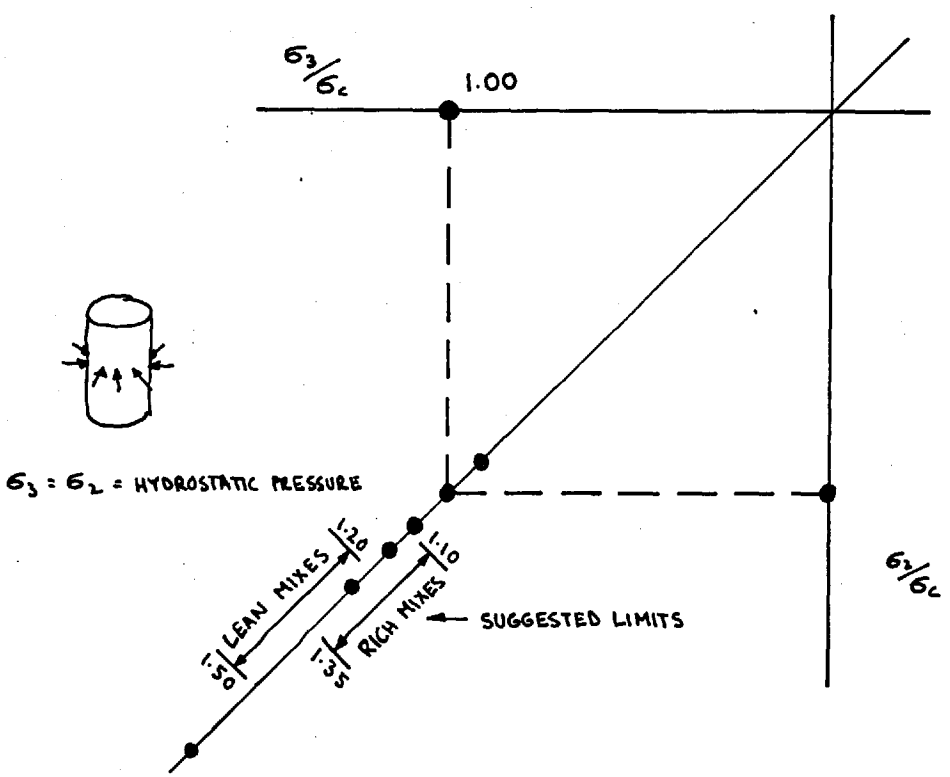


FIG. 1.3.5.1. BIAXIAL COMPRESSION STRENGTH (RICHART, BRANDTZAEG, and BROWN (35)).

1.3.6. Wastlund (1937)[38-40]

The assumption in the German Code of Practice of Mohr's theory for the failure of plain concrete was questioned by Wastlund who set out to demonstrate the influence of the intermediate principal stress and examine the failure modes obtained.

Wastlund developed a biaxial compression testing device in which a cube was held at varying angles in a uniaxial compression machine to vary the ratio of the two principal compression stresses (fig.1.3.6.1.). The manner in which this was achieved can be seen from fig.1.3.6.1. which shows the flat bearing faces on the two "half-moon" castings for different ratios of compressive loads. A variation of this basic principal was suggested for triaxial compression.

Sheets of rubber were interposed between the platen and specimen, the required thickness being determined by obtaining equal lateral strain at the centre of the cube and at a point near the edge; the purpose being to eliminate platen restraint effects.

As has been pointed out by Robinson[10] this procedure contains two fallacies.

1) Only one measurement of lateral expansion near the edge of the specimen was taken which does not guarantee a uniform expansion of the whole specimen as can readily be seen from the results obtained by Robinson and Vile[25]. (see section 2.5.).

2) At much higher loads the rubber will squeeze out precipitating a tension failure and producing a strength lower than the true one.

However, as the rubber pads were common to all uniaxial and biaxial tests, the relative values may be of significance.

The test specimens were 15 cm. cubes and the mix used was a mortar (1:2.0:-: 1/0:3/16"). Four cubes, two for biaxial compression and two for uniaxial compression, were taken from each mix and, with

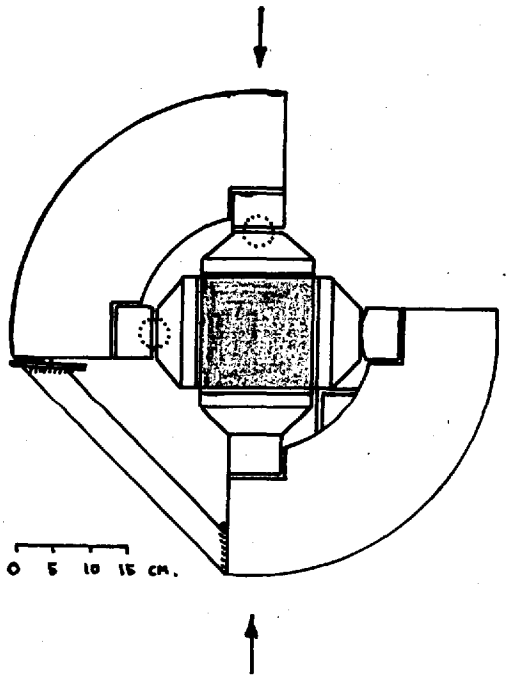


FIG. I.3.6.1. BIAXIAL COMPRESSION DEVICE (WÄSTLUND (40)).

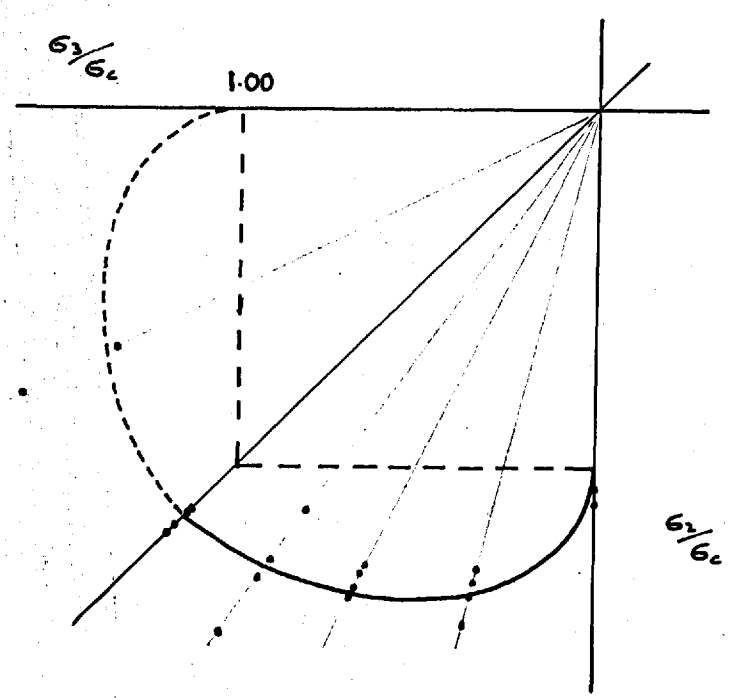


FIG. I.3.6.2. BIAXIAL COMPRESSION STRENGTH (WÄSTLUND (40))

four results on each of four σ_3/σ_2 ratios and two on another, a total of 36 cubes were tested. The resulting ultimate strength envelope which clearly shows the influence of the intermediate principal stress is given in fig.1.3.6.2. Wastlund notes especially the rapid increase of strength at low lateral loads.

An extensometer frame was used in these tests to measure the lateral strains but, probably due to its being mounted on the machine rather than the specimen, unreliable results were obtained. A solution to this problem was found by Robinson and Vile[25] and is described in section 2.5. of this thesis in which the frame was supported on the specimen.

A cleavage mode of failure in the third unstressed direction was sheared and evidence of internal disintegration in the specimen prior to ultimate load was noted. Wastlund suggested that the commencement of this disintegration process was related to the fatigue capacity of concrete and referred to work by Graf[41].

1.3.7. Smith (1953)[42]

In a very short series of tests, on unspecified mortar mix was tested under biaxial compression-tension stresses by pre-loading the centre section of a 3 x 4 x 16 in. beam with a lateral compression and then loading it to failure (fig.1.3.7.1.). A total of eighteen beams were divided into six groups of three and the envelope of the averaged results is shown in fig.1.3.7.2. From the definitions of section 1.2., this test is a combination of uniform and flexural stresses. In this sense it stands in a category of its own but is included in this review, along with some calculated stress tests described later, for completeness.

This testing technique is somewhat crude in that the effect of the lateral compression platens will tend to reinforce the beam against flexural failure and will also interfere with the strain distribution due to flexure which in any case is not uniform (178).

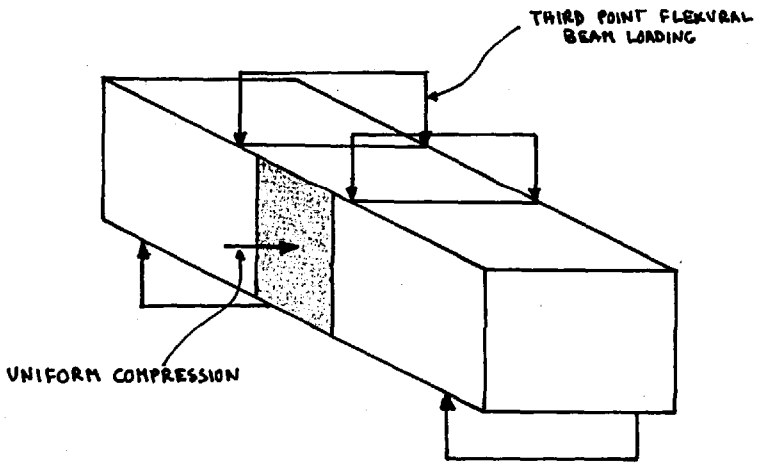


FIG. 1.3.7.1. LINE DIAGRAM OF BIAXIAL COMPRESSION-TENSION MACHINE BY SMITH (42).

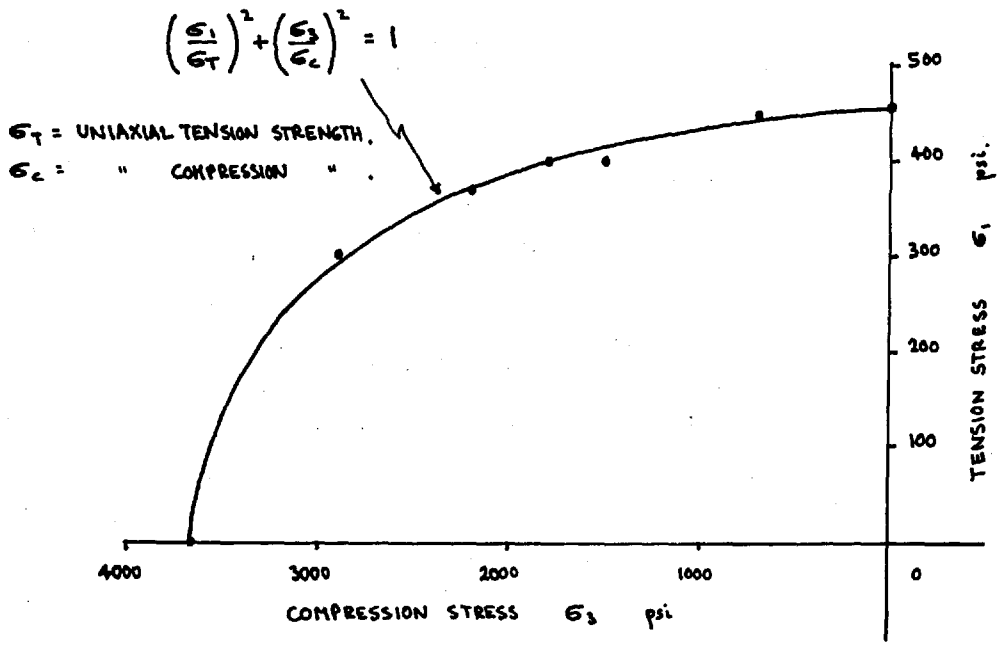


FIG. 1.3.7.2. BIAXIAL COMPRESSION-TENSION STRENGTH (SMITH (42)).

The ASTM modified cube was used as the basis for the uniaxial compressive strength whereas the beam itself should have yielded this ordinate. Failure appeared to be a tensile crack normal to the principal tensile stress.

1.3.8. Reinius (1955)[43]

To yield data for use in the development of a model theory for concrete, Reinius devised the biaxial compression machine shown in fig.1.3.8.1. in which an axially loaded 7 x 7 x 20 cm. prisms received a lateral compression on two opposite faces from the hydraulically filled rubber bags. The friction in the axial direction was reduced by a layer of porous wood-fibre board. However, the rubber bags had a maximum capacity of 40-50 kg/cm^2 only and so the scope of the tests was limited.

Mortar prisms (1:3.75:-0.73/0:3/16") were tested; each point on the resulting curve in fig.1.3.8.2. being the average of 12 prisms except for the point at which the lateral pressure is $0.15\,\sigma_u$ which was the avarage of 15 prisms.

The biaxial ultimate strength envelope is very limited in extent (see fig.1.3.8.2.) but does show the initially large increase of strength at low lateral loads observed by Wastlund. The scatter of results obtained does seem rather large for the number of specimens tested and thus the experimental technique must be suspect, though with the few details published, it is difficult to suggest what could be at fault.

1.3.9. Bresler and Pister (1957)[44,45]

Bresler and Pister, with the specific aim of establishing a failure criterion for concrete, carried out a series of tests on tubular concrete and mortar specimens measuring 9" O.D., 6" I.D. and 30" long. Compression was applied axially in a uniaxial compression machine and shear applied by a torsion rig which acted through clamps at each end of the cylinder as shown in the line diagram in fig.1.3.9.1.

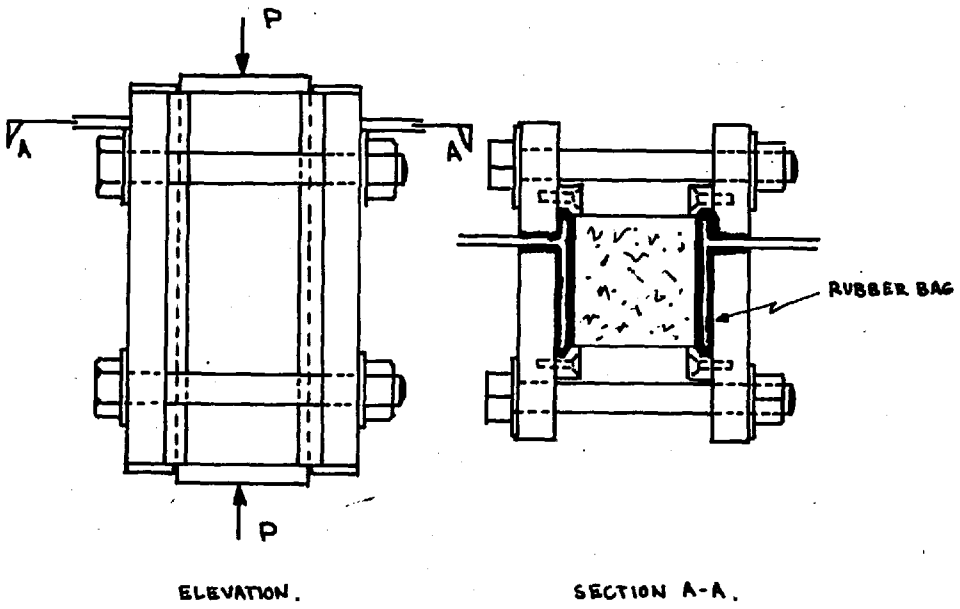


FIG. 1.3.8.1. BIAXIAL COMPRESSION MACHINE BY REINIUS (43).

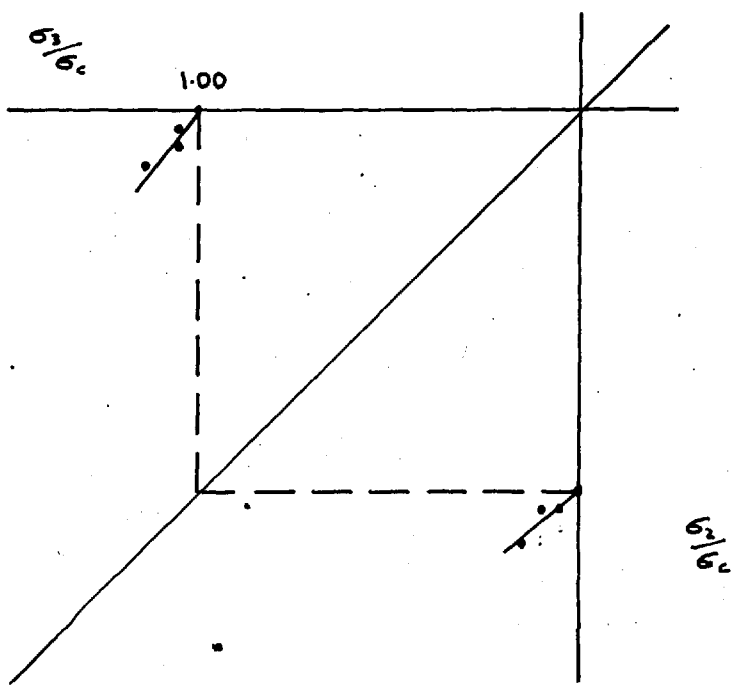


FIG. 1.3.8.2. BIAXIAL COMPRESSION STRENGTH (REINIUS (43))

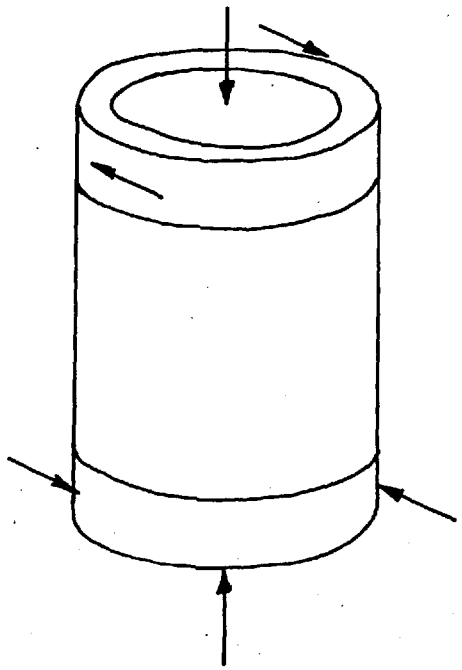


FIG. 1.3.9.1. LINE DIAGRAM OF BIAXIAL COMPRESSION-TORSION TEST BY BRESLER AND PISTER (44,45).

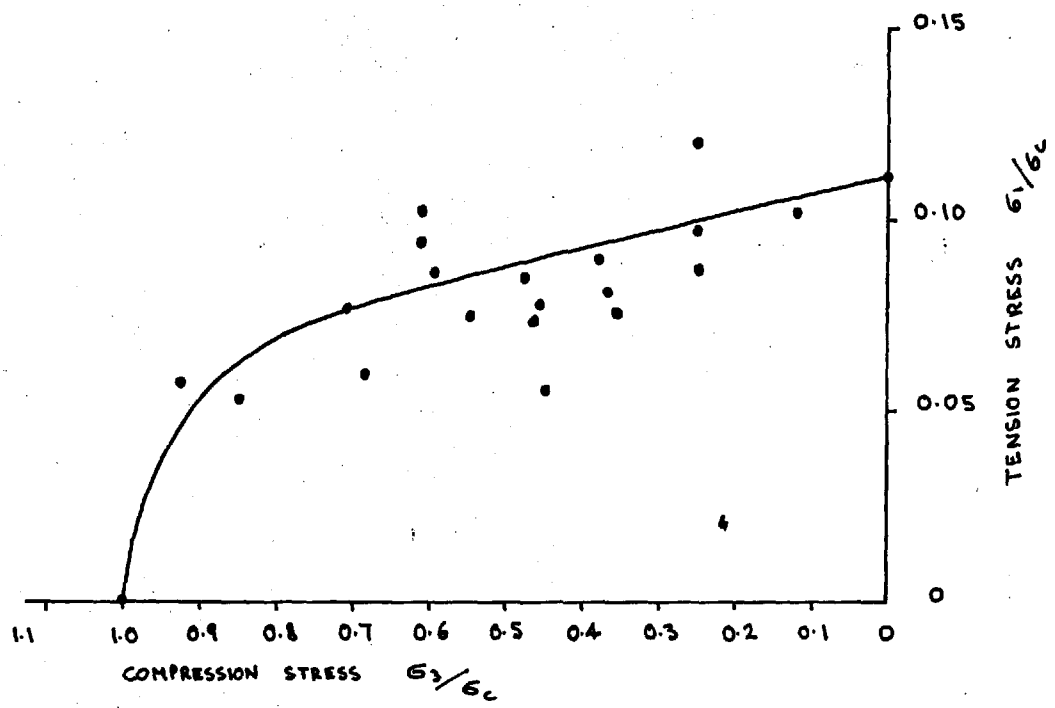


FIG. 1.3.9.2 BIAXIAL COMPRESSION-TENSION STRENGTH (BRESLER AND PISTER (44))

In all 65 tubular specimens were tested and the results are shown in fig.1.3.9.2. Basically three $\frac{1}{2}$" aggregate concrete mixes were used - their proportions being tied to the code design strengths of 3,000; 4,500; and 6,000 p.s.i. compression - and the volume fraction of coarse aggregate varied from 40% to 45%. When all the ultimate strength results are plotted in terms of σ_c (uniaxial compressive strength) as shown in fig.1.3.9.2. little difference was observed between the results from different mixes. All test specimens were tested in the same saturated condition as they enjoyed in the curing tank.

Sequence loading, in which torsion was applied to a pre-compressed cylinder, was used for most of the tests but in the last series proportional loading was shown to give the same ultimate strengths.

This is, of course, a calculated stress method of test for biaxial compression-tension and this state of stress is achieved only on the outside surface of a thick-walled cylinder across which a considerable stress gradient exists. Although the axial compression ball races were calibrated and the effects of the torque bands carefully checked in a series of uniaxial compression tests, no direct check was made on the state of stress achieved in the test specimen during an actual test.

However, there have been few investigations in the field of the biaxial compression-tension strength of concrete as this state of stress is so difficult to achieve. The testing technique has been carefully developed and the convex shape of the biaxial compression-tension envelope agrees with the predictions of plastic theories. Furthermore the actual shape of the envelope reflects the two failure modes observed of cleavage and crushing. Hence the results can be regarded with some confidence though, as mentioned above, no state of stress investigation was carried out on the test specimens and hence the absolute reliability of the results cannot be established.

Bresler and Pister proposed a failure criterion based on stress invariants to explain their results but the results of this mathematical approach are strictly limited to the range of their own tests.

Unfortunately no strain measurements were made on the specimen; the only observations being ultimate strength and crack pattern.

As pointed out in section 1.2. the application of an elastic analysis to a material in an inelastic state can lead to errors and this criticism is applicable to these tests.

1.3.10. McHenry and Karni (1958)[124]

In their series of tests on thick-walled tubular specimens, McHenry and Karni used an internal hydrostatic pressure to produce the tensile stresses in an axially compressed specimen in contrast to Bresler and Pister who applied torsion to the ends of the specimen. The desired state of stress of direct biaxial compression-tension was achieved on the outer surface of the specimen (fig.1.3.10.1.).

The cylinders measured 14" O.D., 10" I.D., and 24" long and 91 specimens were tested in all. Three mixes of $\frac{3}{4}$" aggregate concretes were tested having nominal cylinder compressive strength of 3,000; 5,000 and 6,500 p.s.i. at 28 days. The specimens were listed in a fully saturated condition.

The order of loading was to apply the hydrostatic pressure first and then maintain it a constant value whilst the axial compressive load was applied. A series of tests was carried out in which this order of loading was reversed but no significant differences in the ultimate load strengths obtained were observed.

Again no strain measurements were made: the ultimate strength results are shown in fig.1.3.10.2.

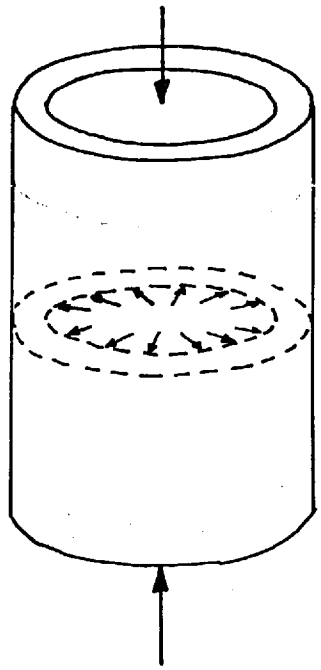


FIG. 1.3.10.1. LINE DIAGRAM OF BIAXIAL COMPRESSION-TENSION TEST BY McHENRY AND KARNI (124).

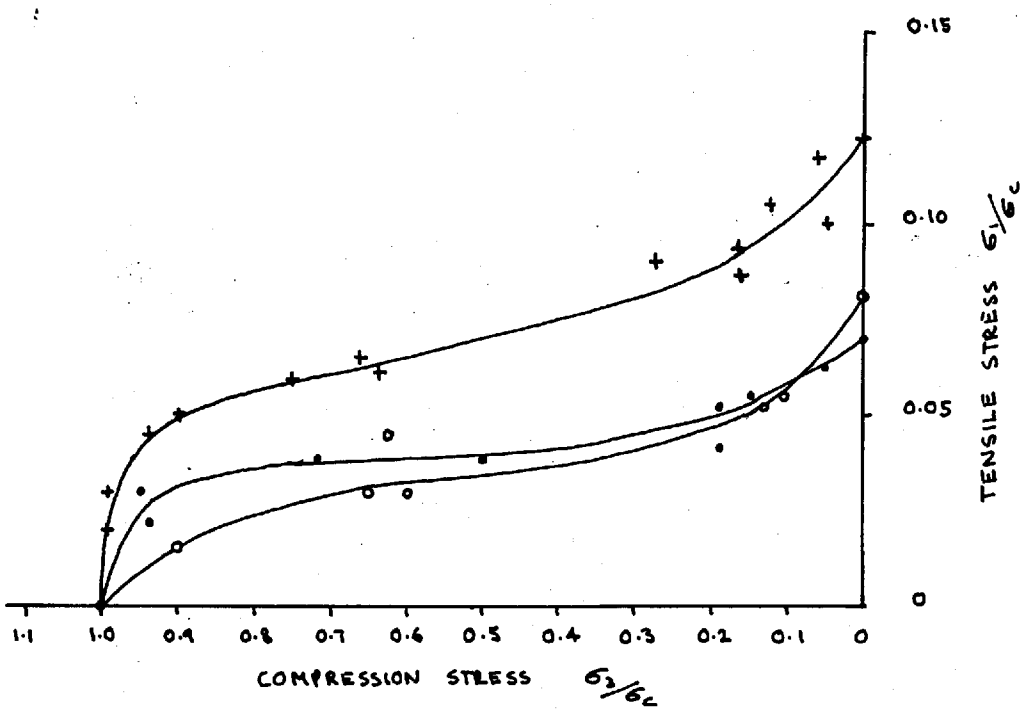


FIG. 1.3.10.2. BIAXIAL COMPRESSION-TENSION STRENGTH (McHENRY AND KARNI (124)).

The strength loop is of a different shape to that obtained by Bresler and Pister[45] for concretes with about the same range of coarse aggregate volume concentrations. It may be that the high tensile strengths obtained can be attributed to the stress gradient across the cylinder and the reinforcing effect of the friction on the faces of the end platens.

A theory suggested by the authors is the relating of the octahedral shear stress as a function of the octahedral normal stress. Robinson[10] in his review has commented on this theory and shown how large variations in the tensile stress have very little effect on the octahedral shear stress whilst it is very sensitive to changes in the compressive stress. This theory will therefore give a good plot for the data in that little scatter will be observed and yet it suppresses the tensile stress which is the vital quantity in this section of the biaxial stress loop.

1.3.11. Glomb (1958)[46]

In apparent ignorance of both the machine and results of Wastlund[40], Glomb developed an octagonal machine frame with pairs of opposed jacks to test concrete cubes under equal biaxial compression (fig.1.3.11.1).

The purpose of this research project was to determine the bonus of strength, if any, that existed in concrete under biaxial compression with a view to obtaining economies in the design of two-way spanning slabs.

Two sizes of concrete cubes, 10 cm. and 20 cm., were used in the investigation and the ultimate strength results are shown in figs.1.3.11.2. and 1.3.11.3. in which an average bonus of strength of 38.8% is claimed. The concrete mixes used were specified only by strength and were within a range of uniaxial strengths: 200-600 kg/cm^2.

Lontidudinal and lateral strain readings were taken and this appears to be the first investigation in which the increase of elastic modulus under biaxial compression was observed (fig.1.3.11.4.).

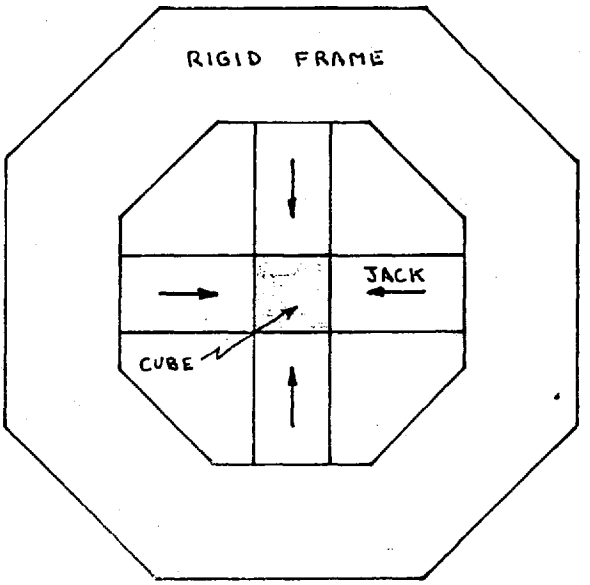


FIG. 1.3.11.1. LINE DIAGRAM OF BIAXIAL COMPRESSION MACHINE BY GLOMB (46).

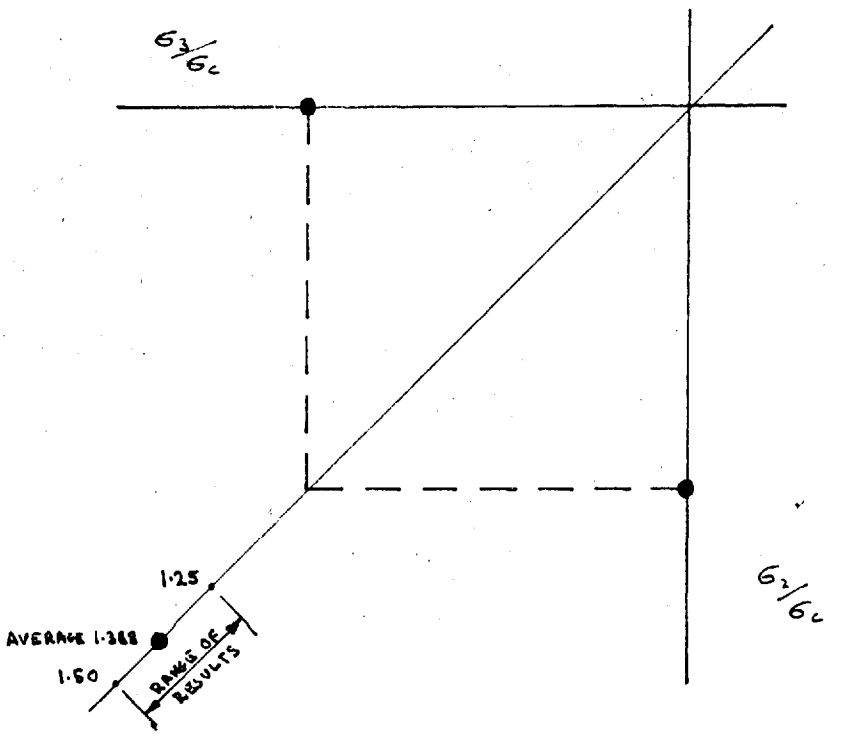


FIG. 1.3.11.2 BIAXIAL COMPRESSION STRENGTH (GLOMB (46))

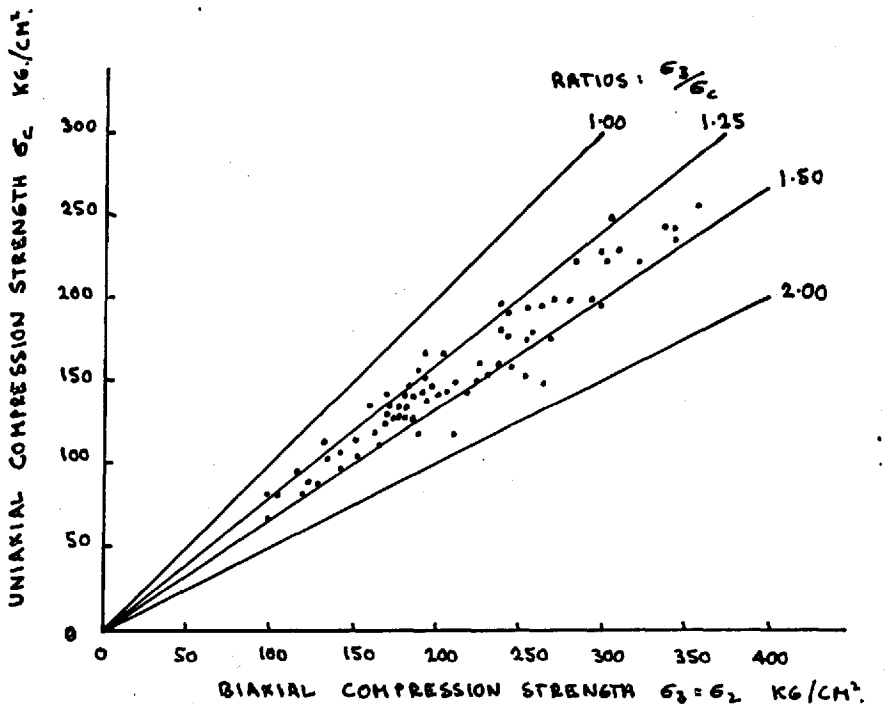


FIG. I.3.H.3 RATIO OF BIAXIAL TO UNIAXIAL COMPRESSION STRENGTH (GLOMB(46)).

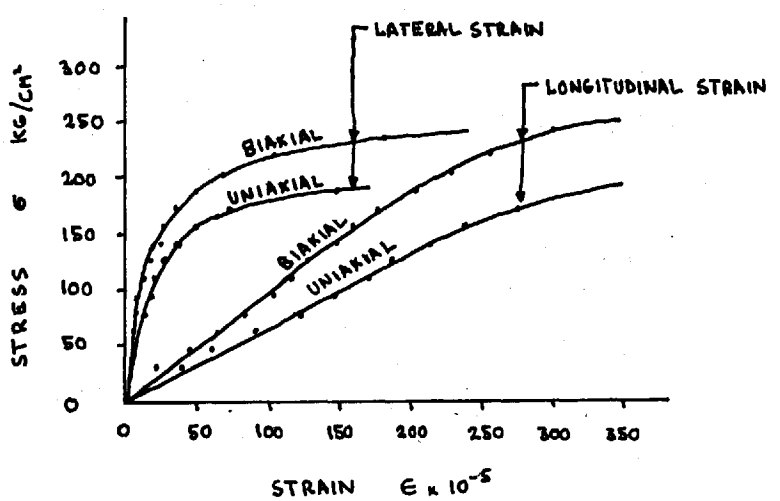


FIG. I.3.II.4 COMPARISON OF BIAXIAL AND UNIAXIAL STRESS-STRAIN CURVES (GLOMB (46)).

In all 223 specimens were used in the investigation; 136 No. 20 cm. cubes, 42 No. 10 cm. cubes, and 45 No. 16 cm. dia. cylinders. Paraffin wax was used in an effort to reduce platen restraint and definite cleavage failures were observed under uniaxial and biaxial compression.

Unfortunately the testing machine and concrete mixes used were inadequately described and therefore few useful comments can be made on this contribution.

1.3.12. Tsuboi and Suenaga (1960)[47]

Tsuboi and Suenaga conducted a series of tests on concrete under biaxial compression-tension and compression-torsion using equipment and testing techniques remarkably similar to those used by McHenry and Karni[124] and Bresler and Pister[44].

The stress results obtained were for ultimate strength only and exhibited considerable scatter (see fit.1.3.12.1. and 1.3.12.2.).

Strain readings were taken from which stress-strain curves and histories could be plotted. However, these results showed only the expected behaviour of the material and no attempt was made to obtain a failure criterion in terms of strain.

The checks made on the state of stress in the specimen was not really adequate though some of the strain readings appear to indicate uniform loading. However, the considerable scatter of test results tends to contradict this.

Too few details were given on the concrete mixes and the curing procedure used to allow comment.

Some very interesting biaxial tension tests were mentioned but not unfortunately reported.

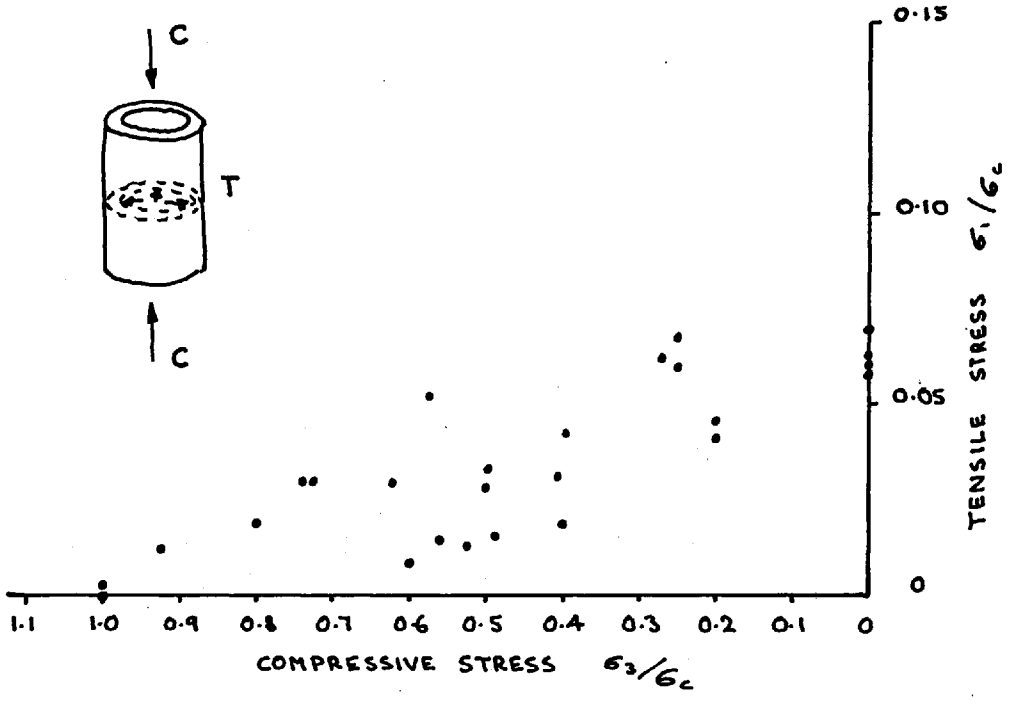


FIG. 1.3.12.1. BIAXIAL COMPRESSION - TENSION STRENGTH FROM COMPRESSION - TENSION TESTS (TSUBOI AND SUENAGA (47)).

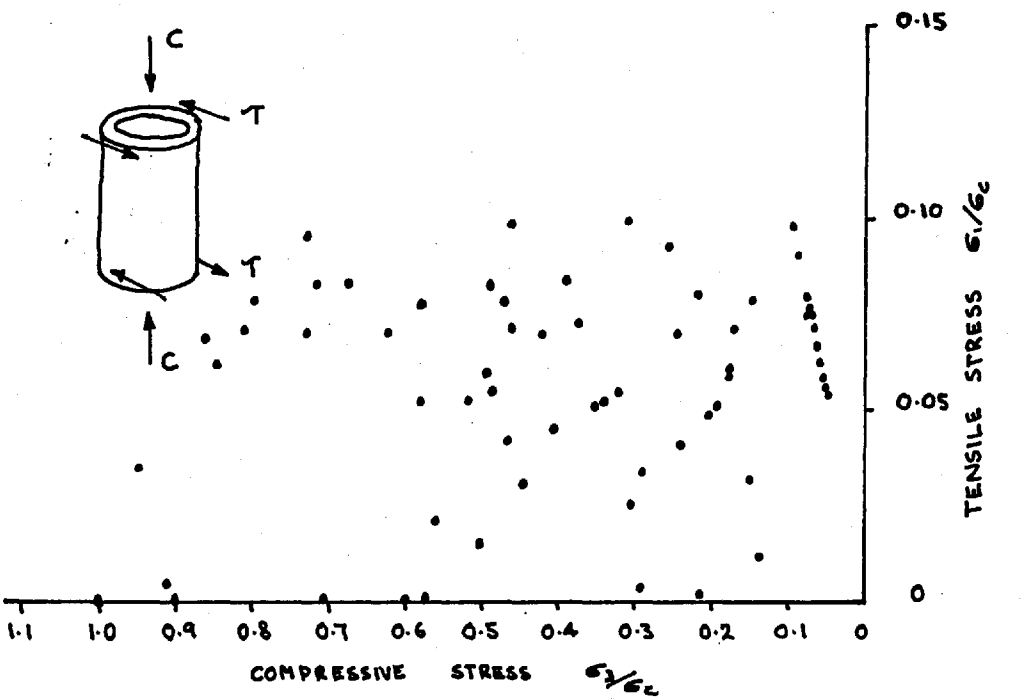


FIG. 1.3.12.2. BIAXIAL COMPRESSION - TENSION STRENGTH FROM COMPRESSION - TORSION TESTS (TSUBOI AND SUENAGA (47)).

1.3.13. Bellamy (1961)[48]

A method of biaxial compression testing was presented by Bellamy in 1961 which consisted of applying external hydrostatic pressure to an axially loaded hollow cylinder. Biaxial compressive stresses were achieved on the inside surface of the cylinder, where there was access for fitting gauges, etc. (fig.1.3.13.1.), though triaxial stresses exist on the outside surface.

The cylinder dimensions were 6" O.D., 2.96" I.D., 12" long and the sole mix used was a mortar (1:2.46:-:0.55/-:3/16"). Only eight specimens were tested under biaxial compression and after curing for seven days, these were allowed to dry out at average laboratory temperature and humidity for 200 days.

The rate of loading was 2,3000 p.s.i./min. which is very high and the order of loading in all cases was to apply the confining hydraulic pressure first and then the axial load. No checks were made on the effect of order of loading.

The cylinders failed by a shear cone forming at one or both ends of the specimen and causing longitudinal splitting. The ultimate strength results are shown in fig.1.3.13.2.

Strain measurements were taken only on the movement of the axial compression piston and these were with a dial gauge. No useful results emerged.

No checks were carried out on the state of stress and the tests are subject to several major criticisms. The cylinders are thick walled with a sharp stress gradient across them. As Robinson[10] has pointed out, even with a 2 in. wall thickness on a 40 in. diameter, a strain difference of 10% will exist between the inside and outside walls.

Failure under biaxial compressive stresses initiated on the inside surface of the cylinder but no satisfactory method of determining the stress at which this occurred was developed.

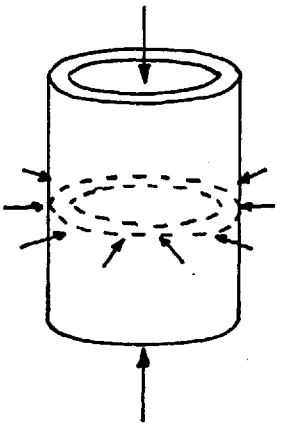


FIG. 1.3.13.1. LINE DIAGRAM OF BIAXIAL COMPRESSION TEST BY BELLAMY (48).

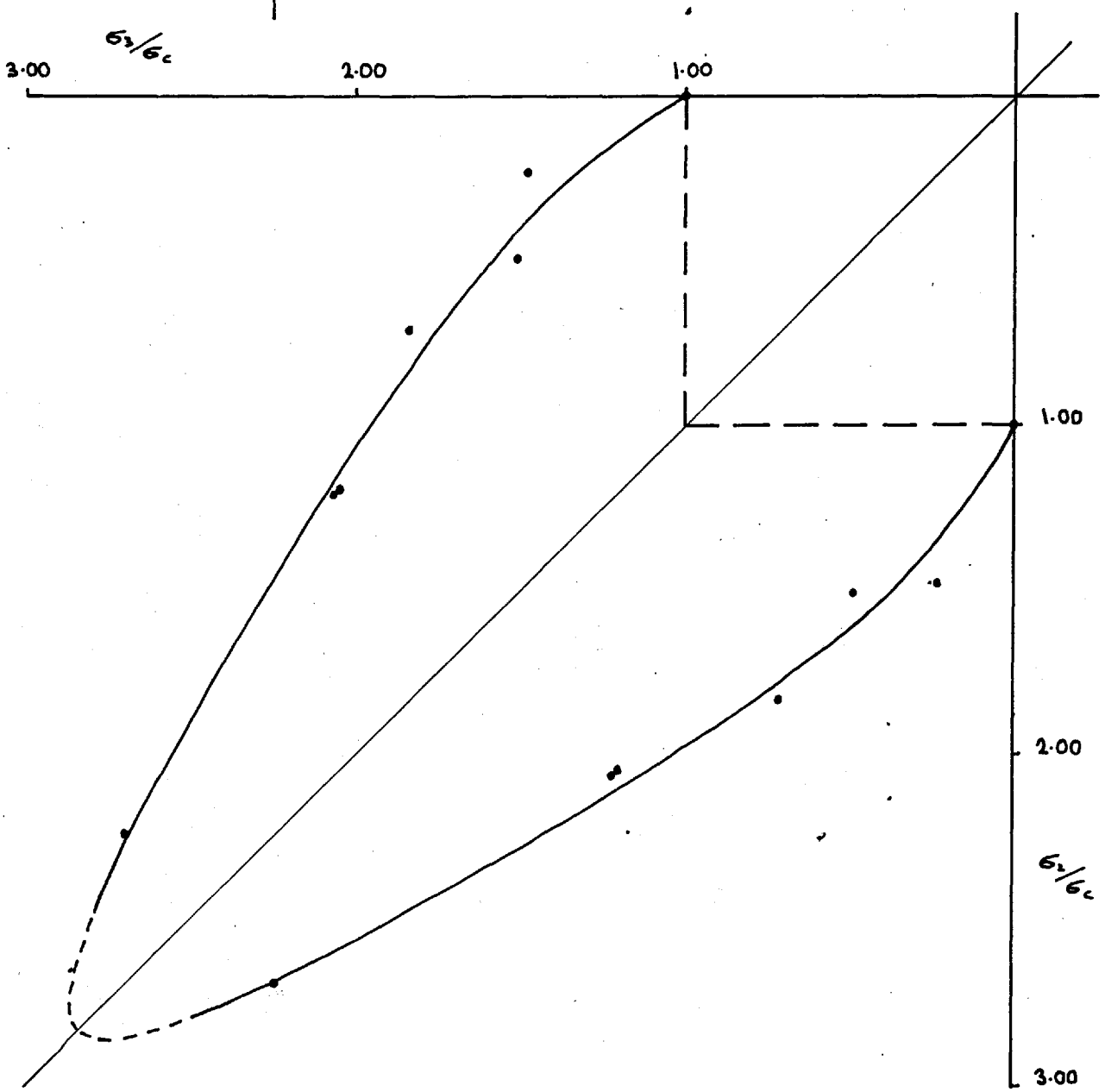


FIG. 1.3.13.2. BIAXIAL COMPRESSION STRENGTH (BELLAMY (48)).

The ultimate load under biaxial compression was taken as the load at which the whole specimen collapsed. This point was probably the result of triaxial stress and this is reflected in the very high strength results he obtained.

Furthermore, the failure crack patterns obtained indicate considerable platen restraint from the axial compression facility. The shear cases observed may be indicative of this.

Finally it cannot be overlooked that Bellamy compares his results not with previous biaxial compression work by Wastlund[40], Reinius[43], and Glomb[46] of which he appears to be ignorant, but with the triaxial compression work of Balmer[49] and Richart, Brandtzaeg and Brown[35]. In view of the tendencies towards triaxial compression in his testing technique, this may in fact be a better comparison.

1.3.14. Weigler and Becker (1961)[50,51]

Recognising the lack of information on concrete strength under biaxial stress and its obvious usefulness in structural design, Weigler and Becker refined the testing apparatus used by Wastlund[40] by introducing rollers and ball joints to eliminate the primary shear forces of the original design and adopting a slab instead of the cube as a test specimen (fig.1.3.14.1.). The adoption of the slab reduces the platen restraint effects which are the cause of the secondary shear on the test specimen; the proportions of the slab were selected using the Ros and Eichinger[24] method of assessing the variation of compressive strength with specimen shape. The test specimen size selected was 10 x 10 x 2.5 cm.

Cement paste and mortar specimens were tested having a maximum aggregate size of 7 mm. (0.276 in.) and the ultimate strength results are shown in fig.1.3.14.2. General cleavage crack patterns were obtained especially with the cement pastes in which the planes of cleavage were parallel to the third unstressed direction. All the

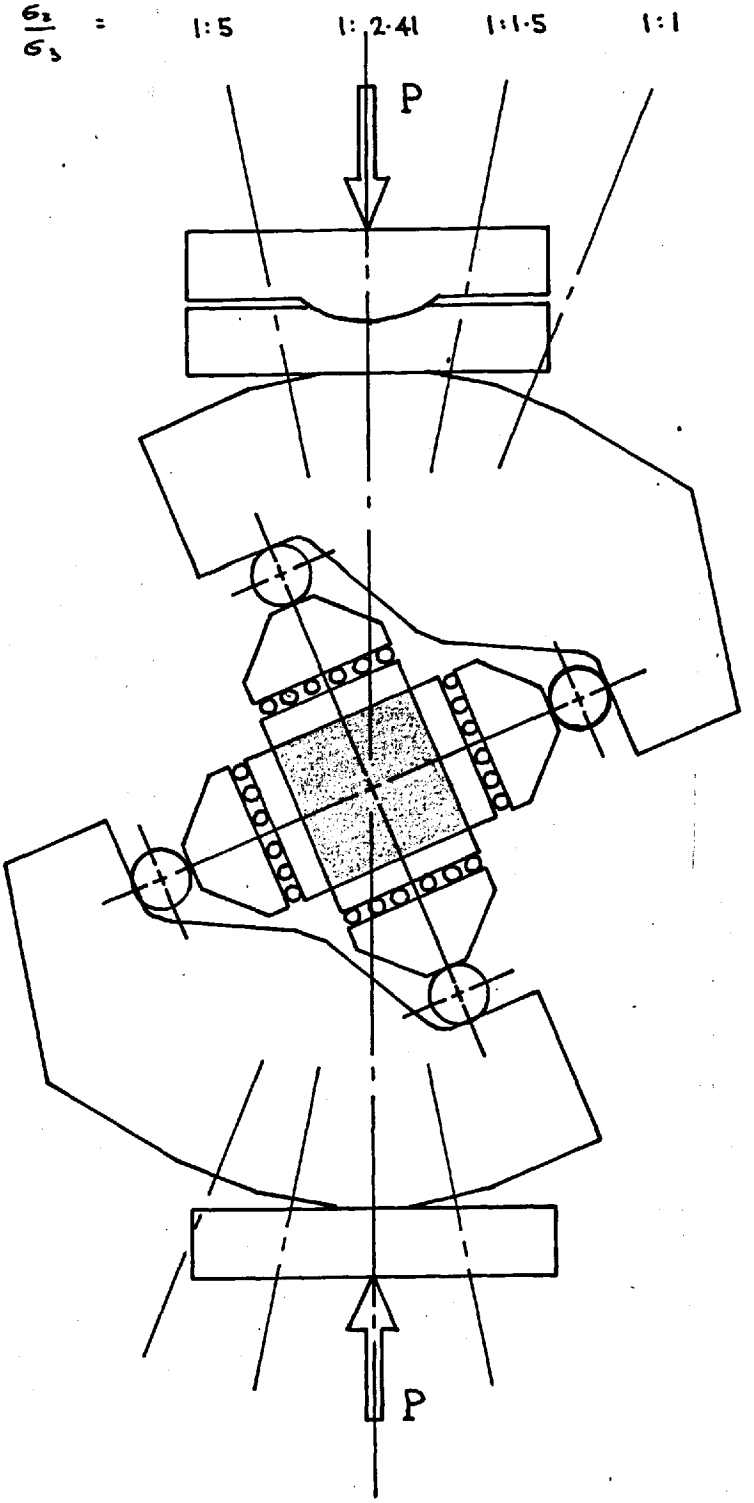


FIG. 1.2.14.1. BIAXIAL COMPRESSION DEVICE (WEIGLER AND BECKER (50,51)).

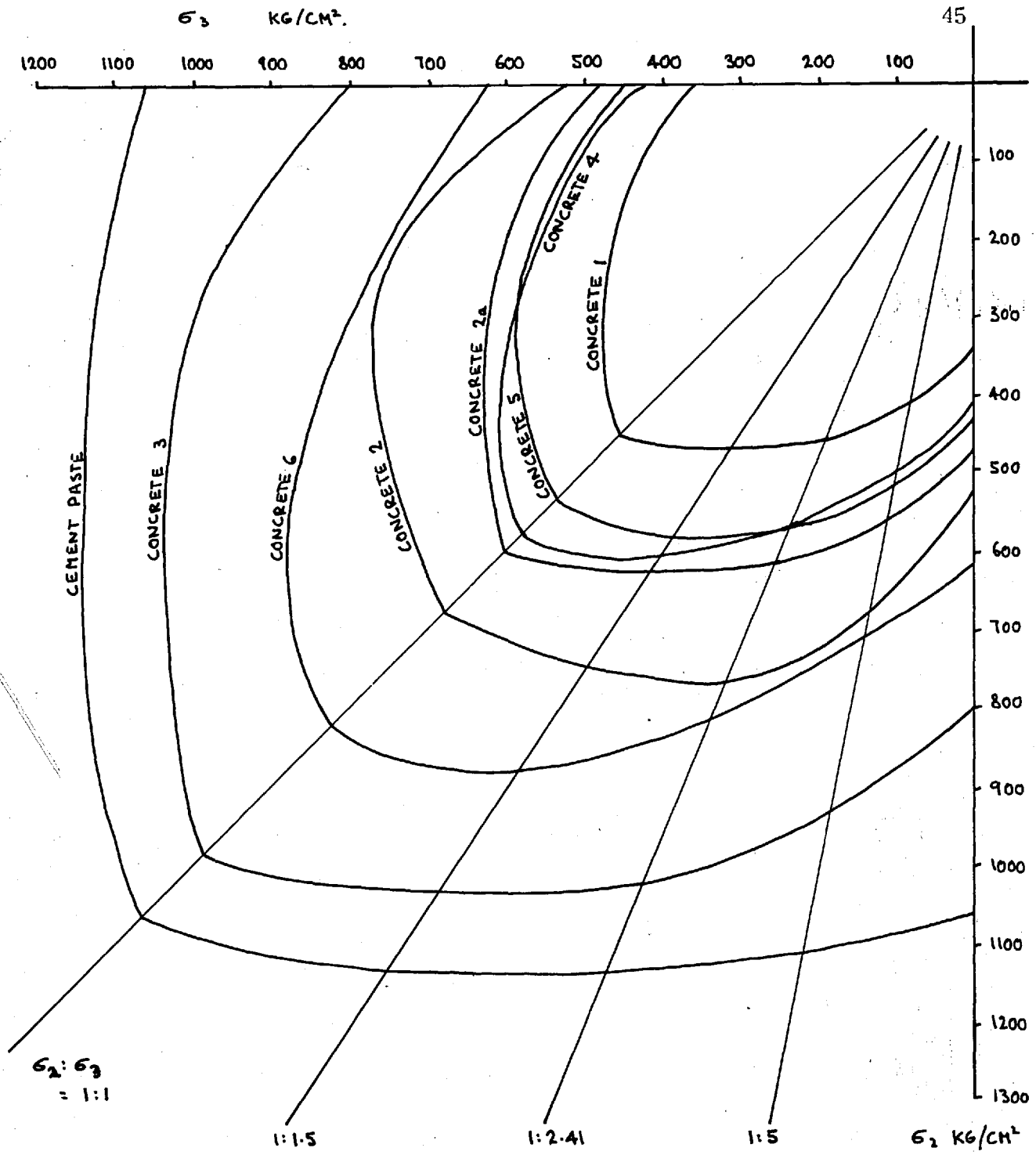


FIG. 1.3.14.2. BIAXIAL COMPRESSION STRENGTH (WEIGLER AND BECKER (50,51)).

specimens were tested in a partially dry state in that, after they had been cured in water, they were stored in an air conditioned room (19°-20°C; 50%-60% relative humidity) for between two and six weeks.

Strain measurements were taken in all three dimensions but these and other techniques employed in the project have been criticised by Robinson[10] and are examined below.

In addition to the compression strength trials referred to above, proving trials on the state of stress in the specimen were carried out using a thin sheet of araldite in place of the test specimen. This, of course, is no indication of the state of stress in an actual specimen. In view of the compression strength trials, the ultimate strength characteristics are probably satisfactory. However, the biaxial compression device is unable to apply uniaxial compression and hence there may be discrepancies between these two sets of data.

The strain readings are probably unreliable as the gauges spread into the zones of non-uniform stress. Certainly the Poisson ratio variations must be suspect as Weigler and Becker show significant changes in Poisson's ratio where all other workers (5,7,10) have found it to be effectively constant prior to discontinuity defined in section 1.5. Furthermore the use of mechanical dial gauges to measure such small strains cannot be entirely satisfactory.

From the machine diagram fig.1.3.14.1. it is clear that for different ratios of load, the arms of the profiled block will be spanning different distances from the centre of load application and hence their deflection under load will vary. This will in turn alter the alignment on the specimen of the loading platens and may vary the state of stress to give strength curves which are due more to this machine effect rather than to the intrinsic structural properties of concrete.

1.3.15. Nishizawa (1961)[52]

The first direct biaxial compression-tension tests on concrete were carried out by Nishizawa using the test set-up shown in fig.1.3.15.1. No friction reducing packings were used between the compression platens and the test specimen, and no proving trials on the state of stress achieved were reported.

The main problem with this type of test is the strain concentrations which occur at the edges of the compression platens due to the frictional restraint exerted by the platens on the test specimen. There is also the problem of alignment of compression and tension facility for which no provision appears to have been made in these tests.

Two concrete mixes (designated A and B) were used in these tests but the mix proportions were not stated.

No strain readings were taken; the only results being the ultimate strength values given in fig.1.3.15.2.

An attempt was made to obtain a failure criterion in mathematical terms via the octahedral shear and normal stresses but the testing technique used is so open to criticism that the results are unreliable. Indeed the degree of scatter obtained is in excess of normal experimental error.

1.13.6. Campbell-Allen (1962)[53]

Campbell-Allen, who supervised Bellamy's[48] work in biaxial compression described in section 1.3.13, then used the same apparatus and specimen size to extend the work into biaxial compression-tension by applying internal hydrostatic pressure to the hollow cylinder (fig.1.3.16.1.).

A further modification was the use of a $\frac{3}{8}$" aggregate concrete, in place of the mortar mix previously used, of proportions 1:1.24:1.56:0.40/0.44:$\frac{3}{8}$", the coarse aggregate being a crushed basalt.

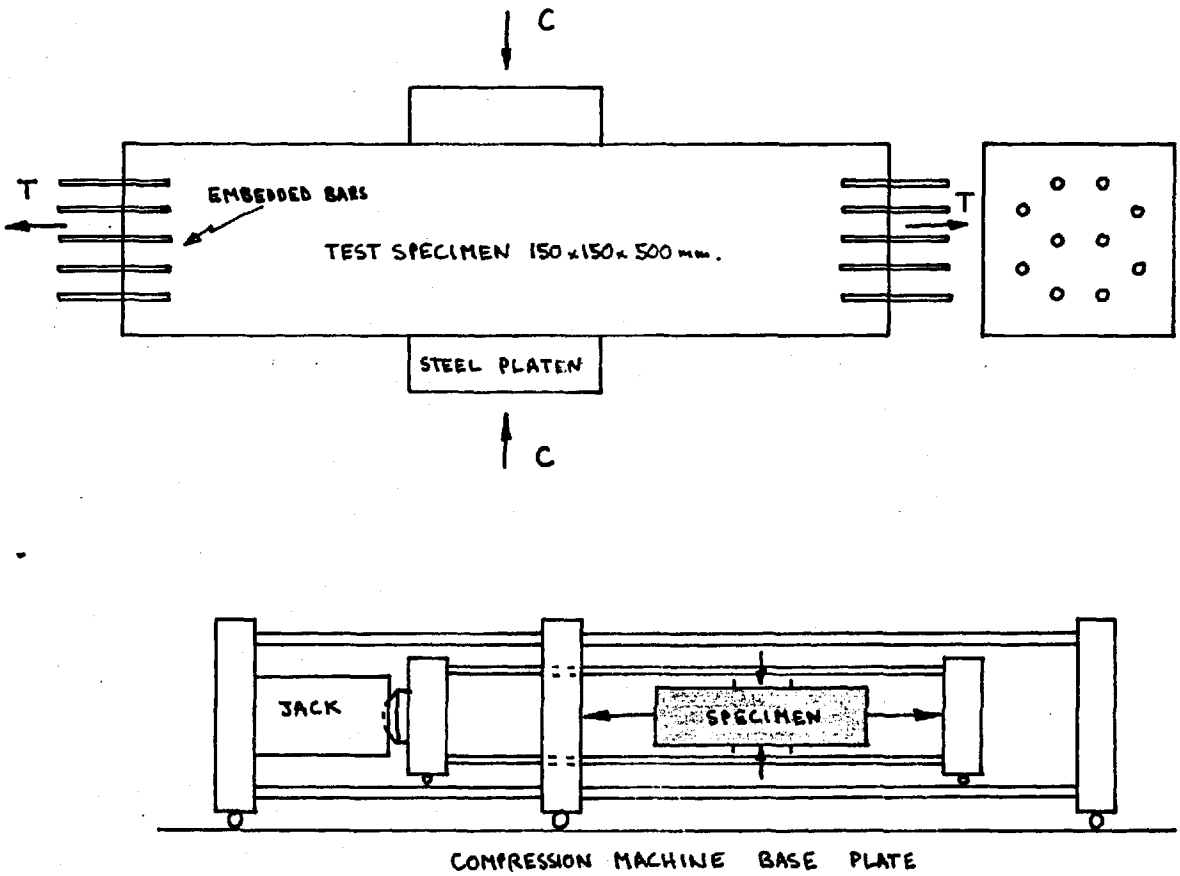


FIG. 1.3.15.1. LINE DIAGRAM OF BIAXIAL COMPRESSION-TENSION MACHINE AND TEST SPECIMEN BY NISHIZAWA (52).

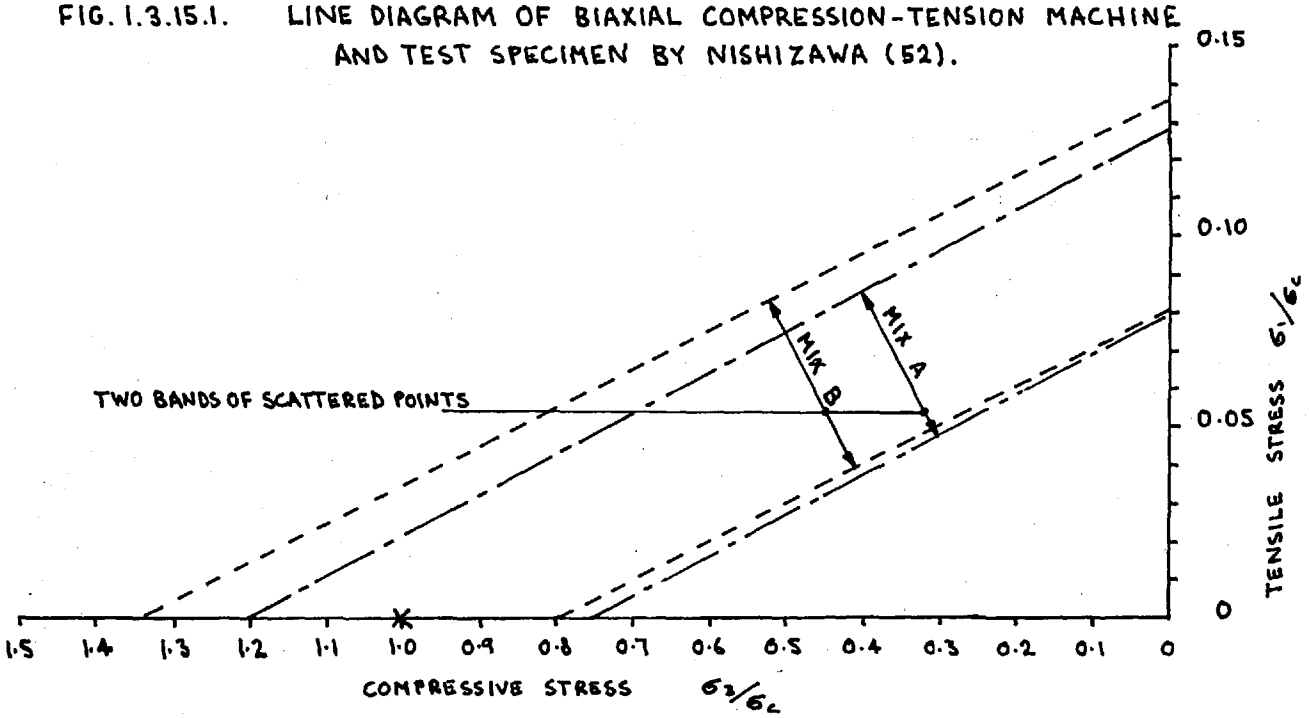


FIG. 1.3.15.2. BIAXIAL COMPRESSION-TENSION STRENGTH (NISHIZAWA (52)).

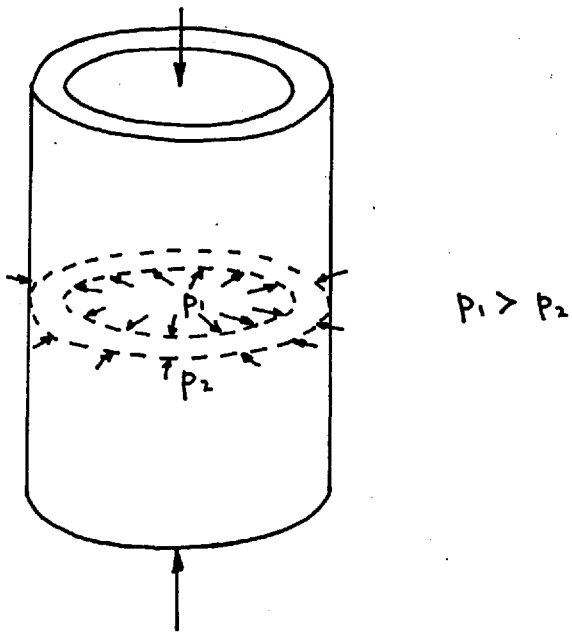


FIG. 1.3.16.1. LINE DIAGRAM OF BIAXIAL COMPRESSION-TENSION TEST BY CAMPBELL-ALLEN (53).

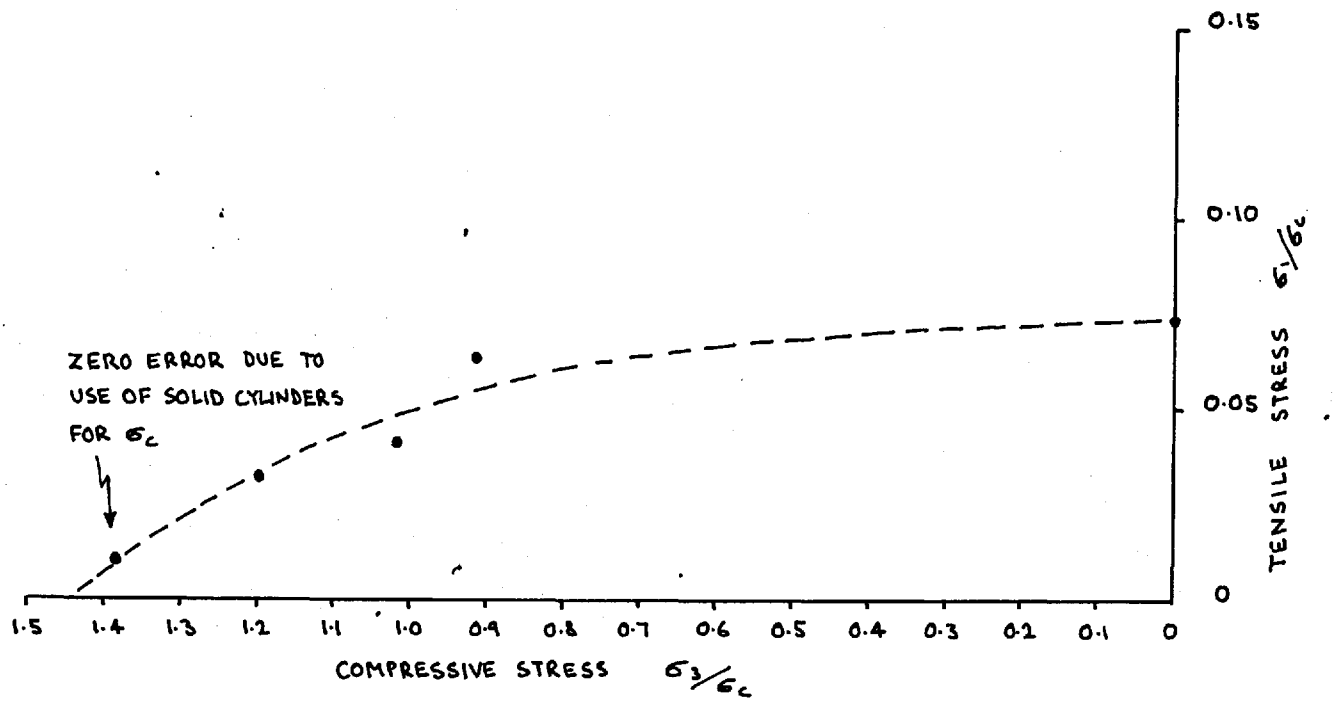


FIG. 1.3.16.2. BIAXIAL COMPRESSION-TENSION STRENGTH (CAMPBELL-ALLEN (53)).

All the previous criticisms of this method of test are pertinent with the additional problem of radial stress due to differential pressures being applied to the inside and outside of the cylinder. This renders the material to a complex state of triaxial stress (C-C-T) reaching critical proportions on the inside face of the cylinder.

A cleavage mechanism of failure was observed in these tests, with the cleavage occurring in radial planes parallel to the axis of the applied compression, and the ultimate strength results are shown in fig.1.3.16.2.

Stress invariants are offerred as a possible approach to failure criteria.

1.3.17. Reeves (1962)[125]

A limited series of tests on ten thin walled plain concrete hollow cylinders subjected to torsion and end compression was conducted by Reeves as an adjunct to work on prestressed concrete beams under combined bending and torsion. Only one concrete mix was used (1:1.5:2.5:0.4/0.42:$\frac{1}{4}$") and, as the test specimens were manufactured by a spinning process, the mix tended to segregate with the larger aggregate particles on the outside face.

The testing machine used was very flexible in that the compression stresses were applied using a central prestressing tendon but surface strain readings showed that uniform states of strain were being achieved. This appears to be the first work in which the state of strain was adequately checked.

Sequence loading was used with the compressive stress being applied first but no stress rates were given.

A line diagram of the testing machine is shown in fig. 1.3.17.1. and the ultimate strength results in fig.1.3.17.2.

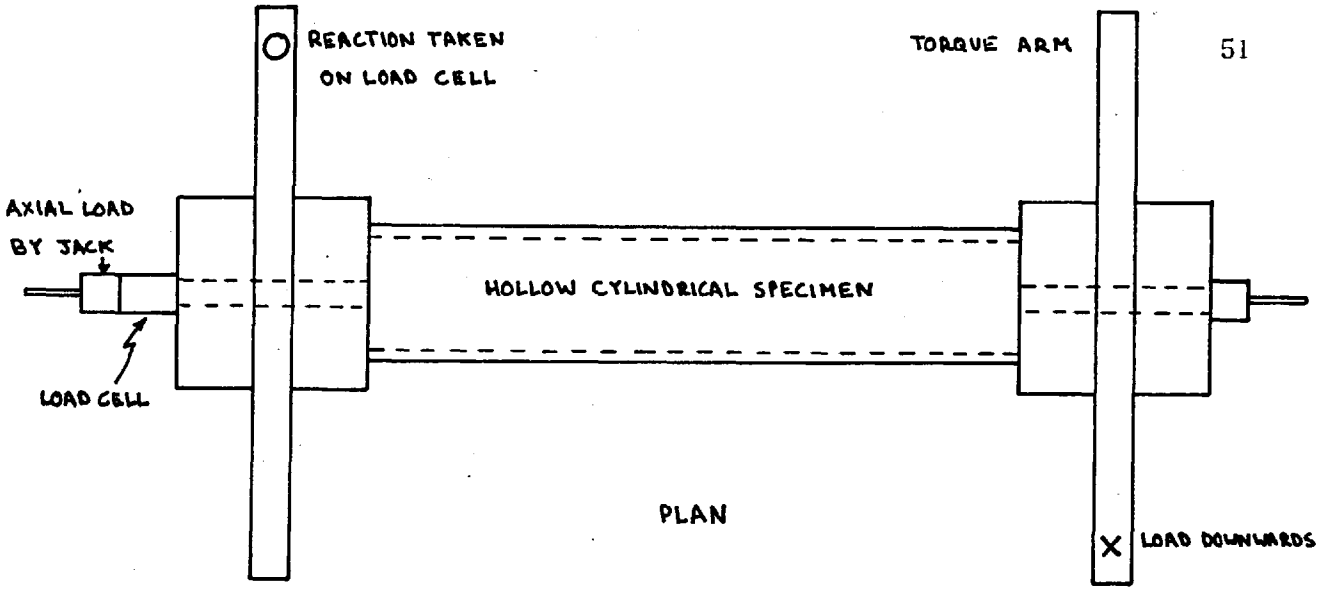


FIG. 1.3.17.1. LINE DIAGRAM OF BIAXIAL COMPRESSION - TORSION MACHINE BY REEVES (125).

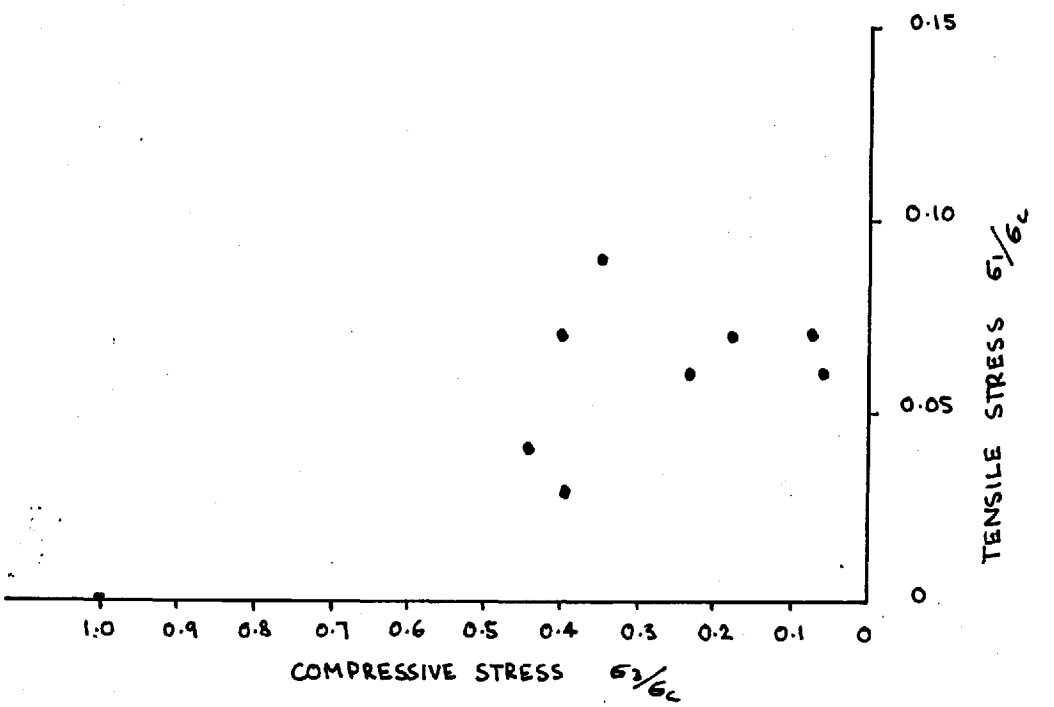


FIG. 1.3.17.2. BIAXIAL COMPRESSION - TENSION STRENGTH (REEVES (125)).

1.3.18. Sundara Raja Iyengar, et al. (1963)[54]

This contribution is included in this review as it involves an interesting attempt using simple direct and indirect tests to interpolate the ultimate strength behaviour of concrete in a biaxial compression-tension stress field.

The tests finally adopted for this exercise were the uniaxial compression test on a 6 in. cube, the splitting test on a 4 in. dia., 8 in. long solid cylindrical specimen, and a torsional test on a 4 in. dia., 16 in. long solid cylindrical specimen. Using the results of these three tests and assuming a ratio of 1:3, tensile to compressive stress in the splitting test, three points were plotted on a graph relating octahedral shear stress to octahedral normal stress as shown in fig.1.3.18.1. A straight line was drawn through the three points with the justification that McHenry and Karni[124], and Bresler and Pister[145] had obtained a straight line relationship from their tests which is not entirely valid (see references 45 and 124).

Six mixes of unspecified aggregate size were used ranging in richness from 1:1.2:2:0.42 to 1:3:5:0.60 and and in compression strength from 6850 p.s.i. to 2200 p.s.i. at 28 days.

Using the straight line graph of fig.1.3.1., the six curves shown in fig.1.3.18.2. were derived mathematically by selecting ar random values of τ_o/σ_c and σ_o/σ_c from fig.1.3.18.1. and knowing the value of σ_c (the uniaxial compressive strength), the two equations for the octahedral shear stress τ_o and normal stress σ_o could be solved for the two unknowns σ_2 and σ_2.

However, it is clear that the assumptions made regarding the state of stress in the simple compression, indirect tension, and torsional tests performed are not exact and the straight line characteristic between octahedral shear and normal stress is open to question. Hence, no great reliance should be placed on these results but the approach to the problem is interesting.

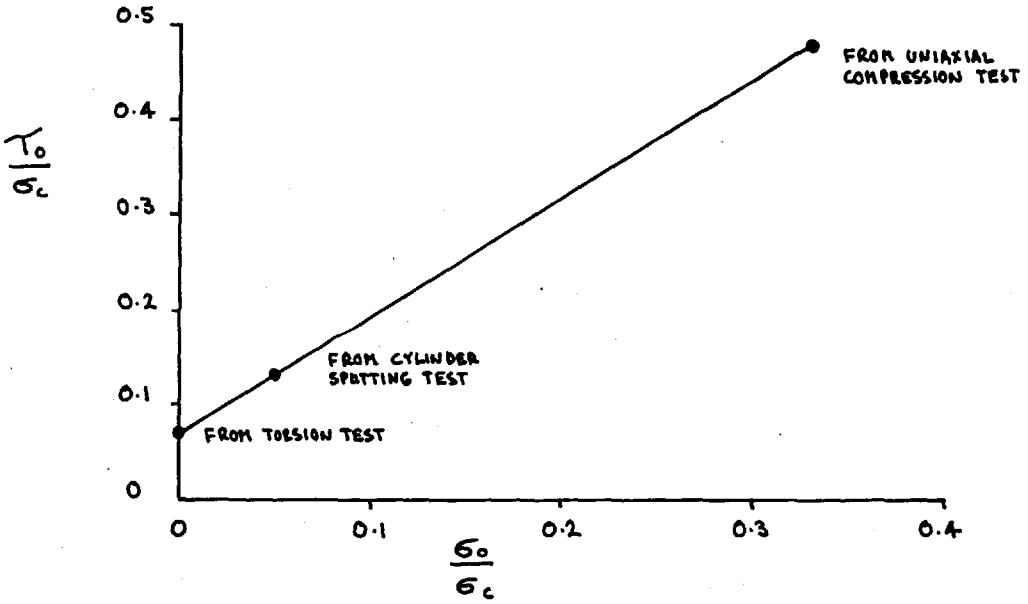


FIG. I.3.18.1. RELATIONSHIP BETWEEN OCTAHEDRAL NORMAL AND SHEAR STRESS PARAMETERS (SUNDARA RAJA IYENGAR & A. (54)).

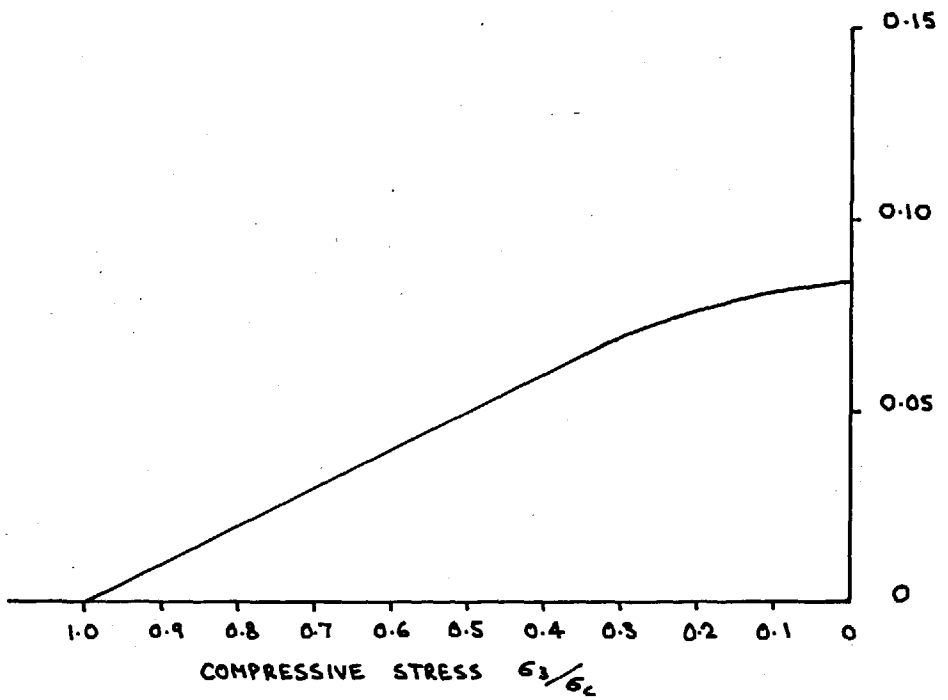


FIG. I.3.18.2. BIAXIAL COMPRESSION-TENSION STRENGTH (SUNDARA RAJA IYENGAR & A. (54)).

1.3.19. Robinson (1964)[10]

The work by Robinson on the biaxial compression of concretes and mortars formed a section of his work in the Concrete Materials Research Group at Imperial College.

From his fundamental work on concrete failure he was especially interested in the effects of the intermediate principal stress and the strain behaviour of the specimen up to failure.

He designed a biaxial compression machine from two existing single ram compression machines by suspending the smaller machine between the columns of the larger machine and counterweighting it so that alignment on the test specimen could be maintained. The load was applied to the specimen through steel platens (fig.1.3.19.1.).

From a series of preliminary tests on platen restraint effects by Robinson and Vile[25], he chose a test specimen size of 10" x 10" x 4" and then carried out extensive proving trials on the state of stress in the specimen and showed that, from measurements of surface strain on a mortar specimen, a central zone of about 4 in. square was in a uniform state of strain.

A series of biaxial compression tests was then carried out on three mortars and three $\frac{3}{4}$" aggregate concretes (see table 1.3.19.1.) and the ultimate and discontinuity strength envelopes are shown in figs.1.3.19.2. and 3. The limitations of machine capacity prevented complete strength envelopes being obtained. All specimens were tested fully saturated after 28 days curing in water.

Detailed strain measurements were made but the lateral strain measuring device apparently did not operate satisfactorily and hence the results of this device are not quoted here.

Robinson noted that the Poisson's ratio remained effectively constant up to the 'elastic' or discontinuity limit but that the elastic modulus tended to increase. However, much of this increase he attributed to loss of load due to the restraint effects of the side platens and

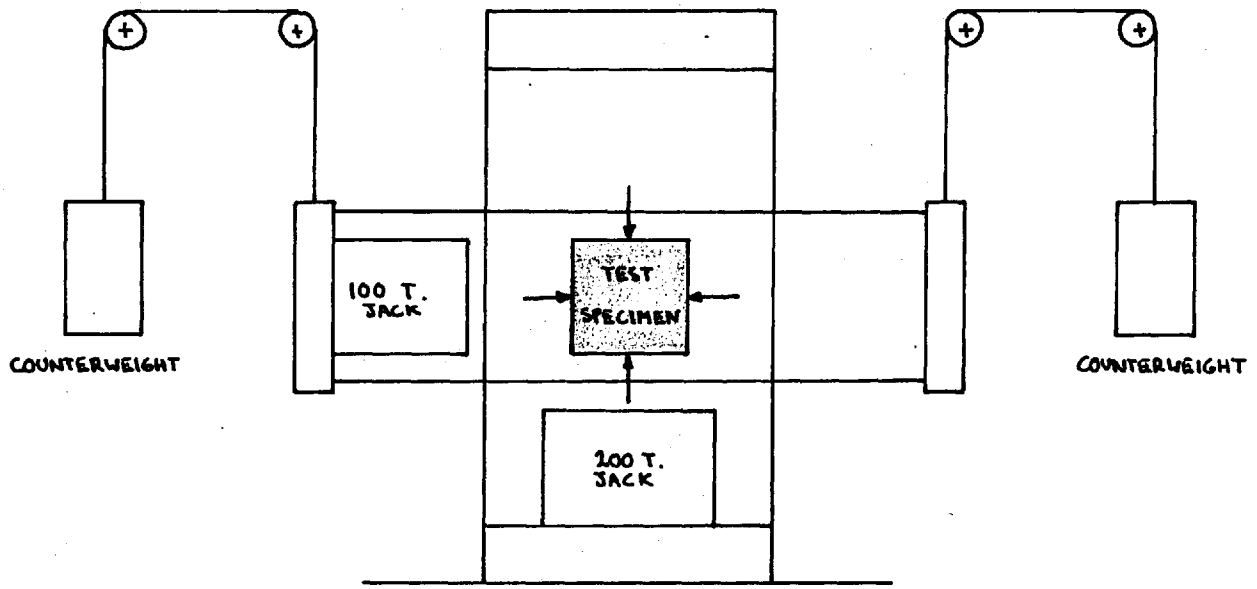


FIG. 1.3.19.1. LINE DIAGRAM OF BIAXIAL COMPRESSION MACHINE BY ROBINSON (10).

MIX No.	W/C	S/C	CA/C	A/C	$V_{f\,c.a.}$
1	0.45	1.8	–	1.8	0
2	0.45	1.8	2.7	4.5	0.41
3	0.45	2.4	–	2.4	0
4	0.45	2.4	3.6	6.0	0.45
5	0.60	2.4	–	2.4	0
6	0.60	2.4	3.6	6.0	0.45

TABLE. 1.3.19.1. MIX PROPORTIONS USED BY ROBINSON (10).

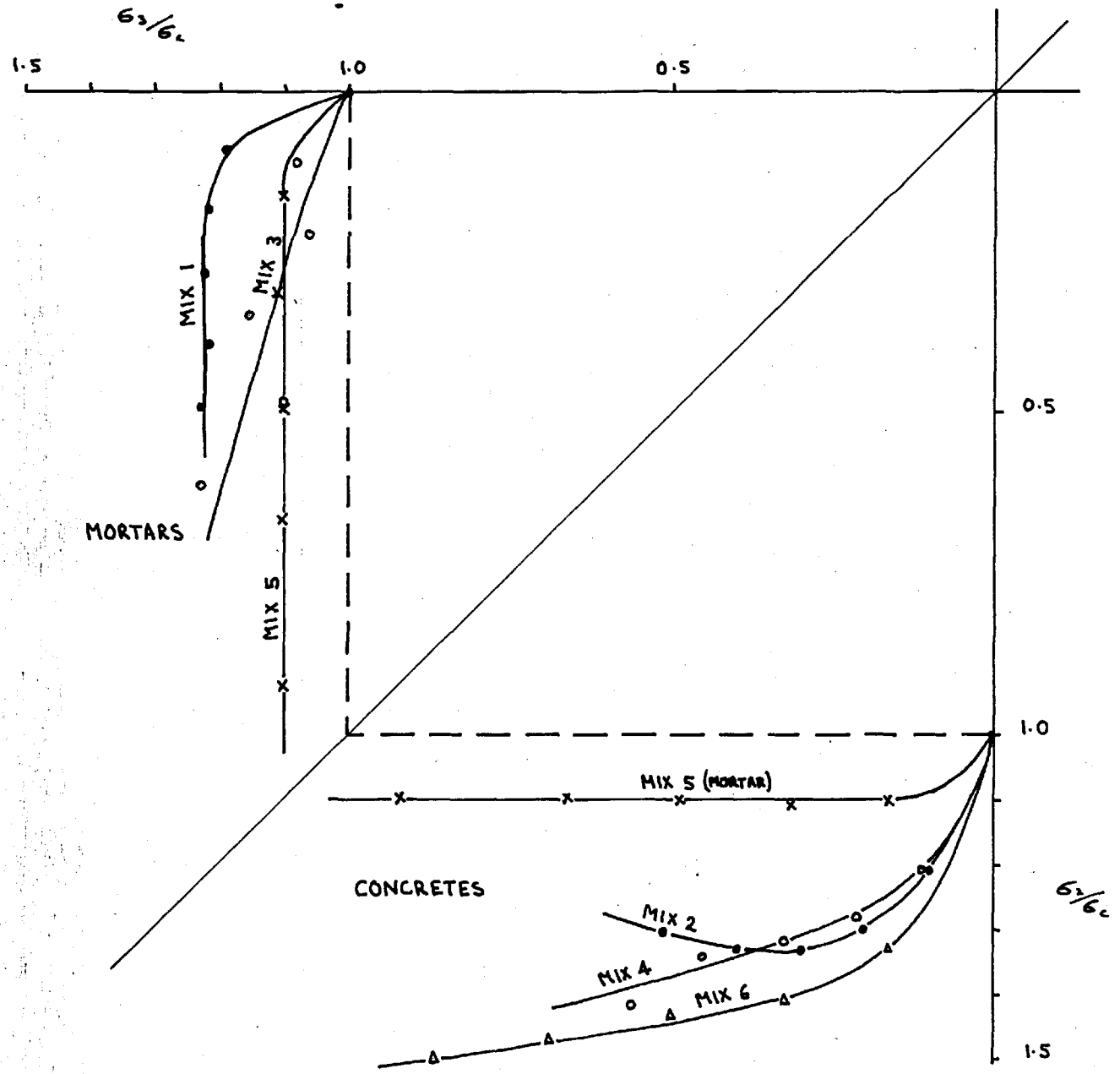


FIG. 1.3.19.2. BIAXIAL COMPRESSION ULTIMATE STRESS (ROBINSON (10)).

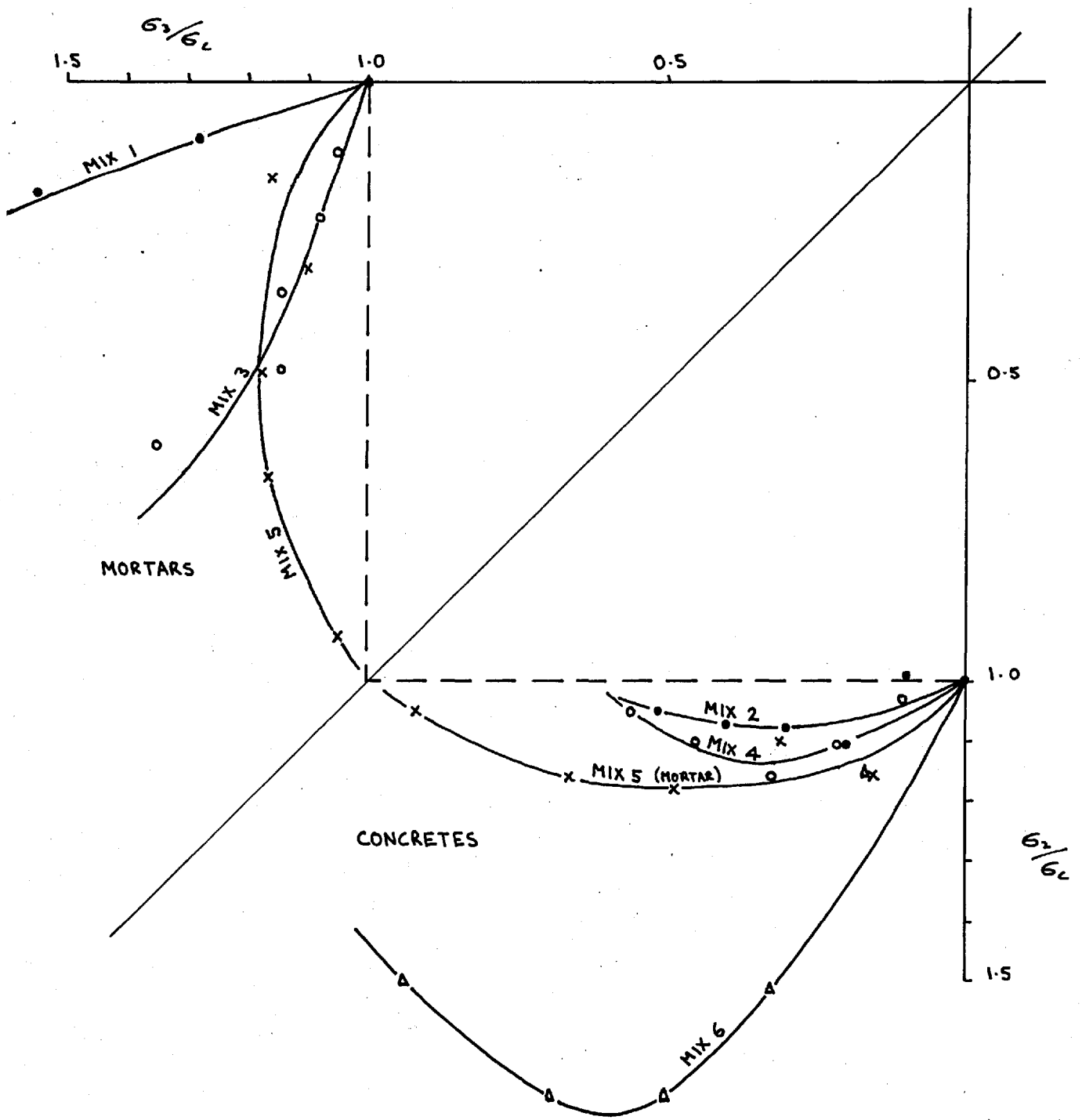


FIG. 1.3.19.3. BIAXIAL COMPRESSION DISCONTINUITY STRESS (ROBINSON (10)).

hence this result was a little inconclusive though the associated particle interaction effect was acknowledged. This conclusion was reacked without apparently being aware of Glomb's work (46).

He concluded that a limiting tensile strain and the volumetric stress were significant in the failure of concrete but did not propose any failure criterion.

1.3.20. Sundara Raja Iyengar, et at (1965)[55]

This work was similar to Robinson's in that two single ram frames were used to apply the comprehensive loads to the test specimen through steel platens. However, the testing machine did not appear to be of such a high standard of precision and no proving trials were reported to discover the state of stress achieved.

These workers appeared to be ignorant of the work by Föppl[22], Glomb[46] and Weigler and Becker[51] on cubical and prismatic specimens. The test specimens were 4 in. and 6 in. cubes and no attempt was made to reduce friction at the platen/specimen interface. As a result, a complex state of triaxial compression was obtained in the specimen as pointed out by Vile and Sigvaldason[56].

Twenty three concrete mixes and four mortar mixes were tested under unstated conditions of saturation and the numbers of specimens from each mix was unknown.

The results obtained are shown in fig.1.3.20.1. and a correlation with the results of Bellamy was claimed. It should be noted that Bellamy's results discussed in section 1.3.13 were probably the result of an undefined state of triaxial compression. As noted above these results are also due to a complex biaxial compressive stress state and hence it is probable that the correlation of results obtained is coincidental.

The fact that these workers dismissed Wastlund's contribution (40) is unfortunate as his results were obtained using what is probably a better testing technique.

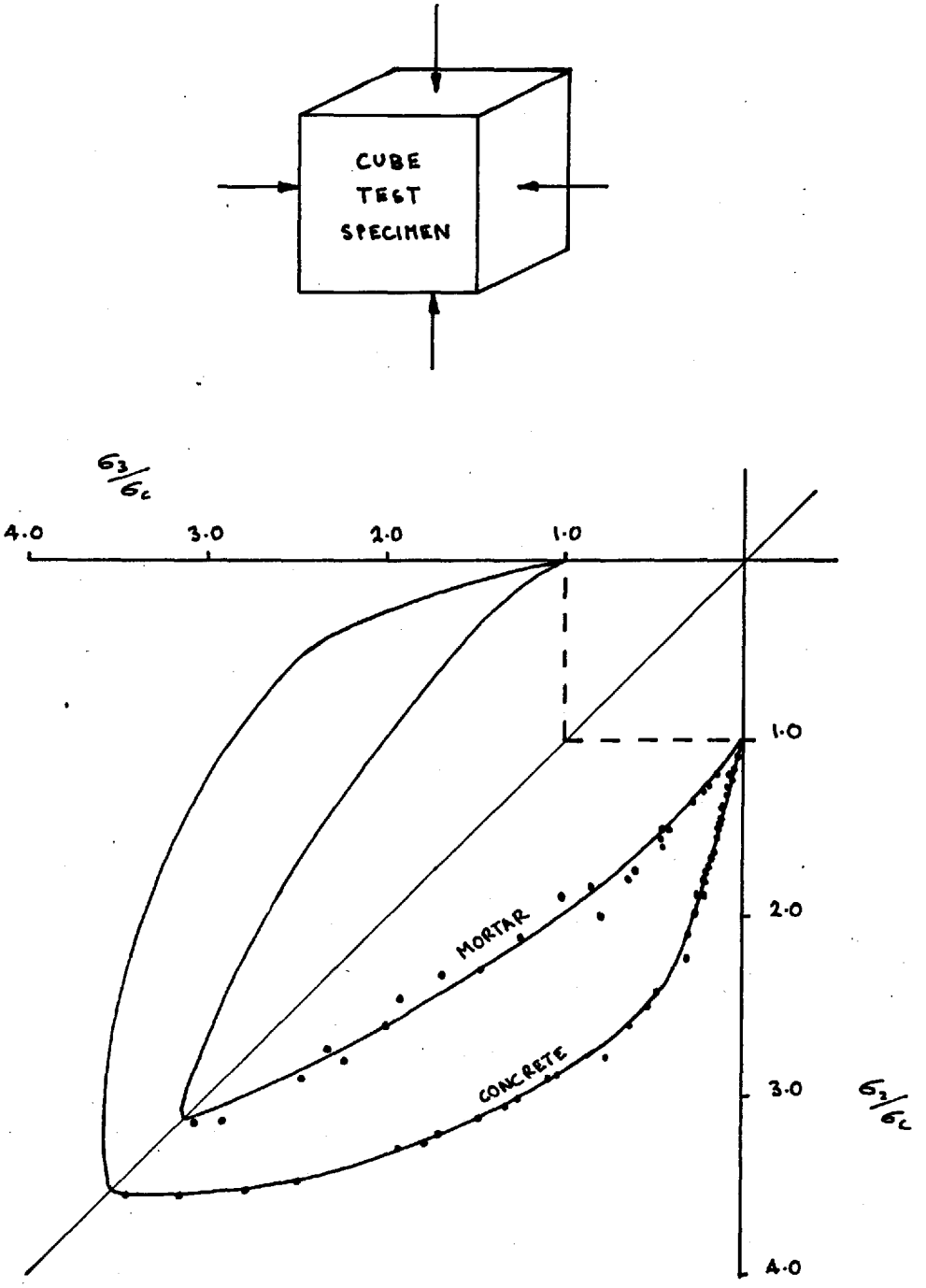


FIG. I.3.20.1. BIAXIAL COMPRESSION STRENGTH (SUNDARA RAJA IYENGAR & A (65)).

Again the octahedral shear and normal stresses were used in an attempt to produce a failure theory and equations governing these variables were produced. However, in view of the testing techniques used, little reliance can be placed in the results.

A more detailed discussion on this work has been contributed by Vile and Sigvaldason[56].

1.3.21. Hilsdorf (1965)[57]

The first detailed analysis of biaxial stress testing machines has recently been completed by Hilsdorf. Not all the contributors in this review were included by Hilsdorf but his general conclusions reached are of importance.

He claims that the solid steel platen method of loading a specimen should never be used as a reliable test method because of load shedding into the side platens (fig.1.3.21.1.). This effect is claimed to spread right across the test specimen and hence the actual stress at failure would be less than that calculated by dividing the externally applied load by the loaded area. This effect is discussed in detail in section 4.4.6. following on the writer's speciment state of stress proving trials and this conclusion of Hilsdorf's is disputed.

To support this contention, he has developed a compression platen in which, to eliminate platen restraint, he has clamped a set of thin plates with low stiffness along the long side of the slab specimen but with high stiffness across the slab (fig.1.3.21.2.) With this device he reports an increase in biaxial compression strength of only 1.12 compared with 1.37 obtained on the same concrete using unlubricated solid platens.

However, this steel brush device will probably have the same effect as the soft packings investigated by Robinson and Vile[25] in that their inadequate stiffness will cause tension stresses and precipitate failure below the true compressive strength in uniaxial

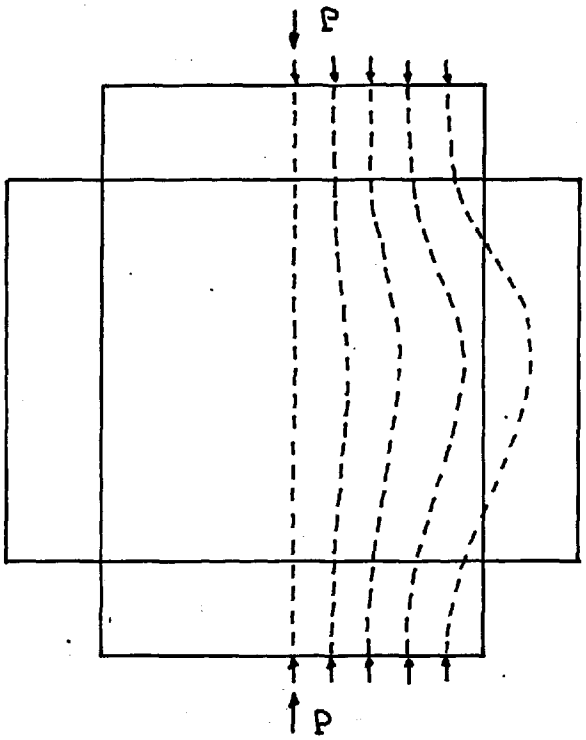


FIG. 1.3.21.1. DEFLECTION OF STRESS TRAJECTORIES DUE TO FRICTIONAL RESTRAINT OF SIDE PLATENS (HILSDORF (57)).

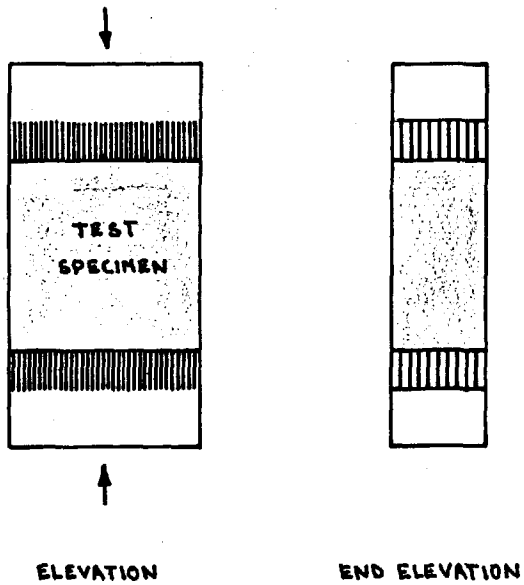


FIG. 1.3.21.2. STEEL BRUSH COMPRESSION AND TENSION PLATEN BY HILSDORF (57).

compression. The mode of failure in biaxial compression has yet to be disclosed by Hilsdorf.

1.3.22. Conclusions

From this review it is clear that certain factors common to any experimental work of this kind must receive special attention because of the complexities of combined stress testing (see section 1.2.).

In all the work reviewed, the attainment of a uniform state of stress of known principal directions and magnitude has been a major problem. The geometric problems with hollow cylinders regarding the ratio of wall thickness to diameter are basic and unalterable, together with the problems associated with the use of an elastic analysis in an inelastic circumstance. Therefore, it would seem that the direction application of loads will give the best possibilities for development. Yet the problems of removing platen restraint and the difficulties of applying tensile stresses this way have proved to be very difficult to overcome and the failure to solve them has invalidated, to a large extent, many of the results achieved to date. Robinsons work[10] shows signs of hope though even this only yielded a small zone of uniform stress but of unknown magnitude. Clearly, as recognised by Hilsdorf[57], the solution of this problem is of major importance in any experimental work under combined states of stress.

The method of testing also effects the mode of failure observed in the test specimens. In this context not only is the state of stress in the test specimen important but also the whole machine/specimen interaction[12]. From the modes of failure reported by the researchers, some have been due to a splitting action caused by a soft packing material between the loading platen and the specimen whilst others have shown clearly the effects of platen restraint or machine instability.

The interpretations of the results especially of strength have not been satisfactory as in most cases they are based on ultimate strengths only, with no record of discontinuity (see section 1.5.) stresses and are often coloured by the failure theory favoured by the researcher. Robinson[10] has shown that the approaches adopted to date do not take account of the heterogeneous nature of concrete and, until some attempt is made to take this into account, few valuable interpretations will result.

Concrete mix selection and test specimen conditions are of secondary importance as problems but clearly, if dry or partially dry test specimens are to be used, then warping and general shrinkage effects will provide extra difficulties.

1.4. Review of Failure Theories

A detailed review of failure theories will not be attempted in this section as this has already been done in numerous publications. Reviews of failure theories and comparisons with experimental results have already been made for concretes and mortars by Ros and Eichinger[32], Cowan[58], Swamy[59] Anson[5] and Robinson[10]; the latter contribution being the most extensive to date and especially recommended for further study.

For the purposes of this thesis, it is thought to be sufficient to list the basic theories in table 1.4.1. and include a special bibliography with the references listed under the main classes of theory. A useful general reference for classical failure theories can be found in the work of Morin[60,61,62], Jaeger[79] and Westergaard[88,89].

The theories themselves can be classified in general terms under the headings of classical, concrete, and models. This is a very approximate division as all theories use elements of the classical theories in their composition. By classical theories

FAILURE THEORY	ASSUMED MATERIAL	ADAPTATIONS OR MODIFICATIONS BY
1. MAXIMUM STRESS (LAME, CLAPEYRON, RANKINE)	HOMOGENEOUS ISOTROPIC	
2. MAXIMUM STRAIN (ST. VENANT)	— " —	
3. MAXIMUM SHEAR STRESS (GUEST) (63,64)	— " —	
4. OCTAHEDRAL SHEAR STRESS (33)	— " —	a) HUBER-HENKY-MISES (79)
5. MAXIMUM STRAIN ENERGY (HAIGH) (65,66)	— " —	
6. INTERNAL FRICTION THEORY (COULOMB) (67)	— " —	a) RŐS (32) b) COUARD (68) c) COWAN (58,69,70) d) HARRIS (71) e) PAUL (72) f) BLEICH (73)
7. MOHR'S THEORY - GENERAL. (74,75)	— " —	a) PALOTAS (76) b) ACKROYD (77)
8. BRANDTZAEG	HOMOGENEOUS NON-ISOTROPIC	
9. BRESLER AND PISTER (44,45)	HOMOGENEOUS ISOTROPIC	a) McHENRY AND KARNI (46)
10. GRIFFITH-IRWIN (80,81,90,91)	CRACKED ISOTROPIC	a) KAPLAN (82) b) GLUCKLICH (83,84)
11. MODELS.		
① REINIUS (43,85)	2-PHASE	
② BAKER (4)	2-PHASE LATTICE	
③ THURSTON AND DERESIEWICZ (86)	LATTICE	
④ ROY AND SOZEN (87)	BOX LATTICE	

TABLE. 1.4.1. SUMMARY OF CLASSICAL FAILURE THEORIES (Nos. 1-7) AND CONCRETE THEORIES (8-11).

is meant those theories proposed direct from hypothesis of stress, strain and energy in an ideal homogeneous isotropic material whereas the concrete theories are those especially developed for concrete to take into account its heterogeneous anisotropic nature to some degree. The model theories are those structural simulations of concrete behaviour which have been proposed as alternatives to the above theories.

The assumption of homogeneous nature in the majority of the theories is a serious drawback to their application to concrete, a material which is clearly heterogeneous at all levels (concrete, mortar, and paste). To the naked eye. The interaction between the phases in concrete is a complicated matter in which the varying stiffnesses and shrinkage properties all play dominant and complex roles. Those theories and models proposed specifically for concrete try in their various ways to simulate this interaction but with only limited success.

Probably the most serious failing of all reviews of failure theories and especially when comparing theory with experimental results has been the neglect of the reviewers to make adequate distinction between mortar and concrete; or, to be more specific, to specify particle size and making separate comparisons depending on the maximum particle size. This error is surprising on several counts:-

a) At the macroscopic level, mortar is effectively homogeneous (though heterogeneous relative to paste) whereas concrete is not.

b) In any composite material subject to drying shrinkage, the non-shrinking elements are bound to play a leading role in determing deformation and failure characteristics depending on their size.

The same illusion of a single theory to cover all 'concrete' behaviour seems to penetrate much of the experimental work where the term concrete is often used to refer to mortar.

This research will show later how basic this general error may be.

From the review of past research on concrete, it is clear that the intermediate principal stress has an influence on the behaviour of concrete under biaxial compressive stresses. In comparing the theories discussed above with experiment it is vital that this influence be known so that a suitable theory may be selected from existing theories or a new theory proposed. Therefore biaxial compression tests as noted by Robinson[10] are of unique value in the search for a failure criterion for concrete.

1.5. Philosophy of Strength

A point of contention which occurs in both reviewing previous work and examining the failure theories proposed is the parameter of strength and what exactly it means. For such a basic parameter to be questioned requires that failure must be defined.

The ultimate strength of a test specimen ss measured in a short-term static load test, by custom, has become to be regarded by engineers as the "strength of concrete". However, it is known that the value obtained from this type of test is time dependent (17,93-101) and therefore by no means the final quantitative result. Nevertheless the test has the virtue of involving only very simple testing techniques and machines. Furthermore there can be no doubt about the value of the load at which the ultimate load is reached; it being simply the maximum load recorded by the testing machine.

However, over the last ten years, interest in the local structural breakdown in concrete has increased significantly. Local ultimate strengths are exceeded long before the overall ultimate strength is reached and the long-term strength of concrete is known to be substantially less than the short term (96,100-102). The

differing elastic moduli of the concrete constituents and their shrinkage characteristics inevitably produce local strain concentrations both prior to loading and under load (8,10,103-114). Recent work in this field has been particularly concerned with the local tensile and shear bond strengths between the aggregate particles and surrounding mortar or paste (115-118).

All these observations have built up a picture of gradual structural breakdown consisting of several stages from which it is apparent that concrete has "failed" for some purposes prior to attaining its short-term ultimate strength. The problem this poses is which strength or strengths do we need to know at the phenomenological level to safeguard ourselves in design. The answer lies in the degree of structural deterioration that we are prepared to accept having regard to such factors as long-term performance, durability, permeability, stiffness, etc.

In the restricted field of mechanical strength properties as applied to engineering, the most vital factor is probably deterioration of the load carrying capacity and the effect of this deterioration on the long-term ultimate strength.

One obvious point of interest emerges from the stress-strain curves of concrete and this is the load stage at which the behaviour of the material ceases to be "elastic" as indicated by a change in shape of the stress-strain curve. Work by Jones[104,109], Kaplan[21], Hsu[110,111,117], Robinson[10,11] and others[112], has shown that at this stage, termed "discontinuity" by Newman[113], the slow extension of what are mainly aggregate/paste band cracks is replaced by the relatively rapid extension of cracks through the surrounding matrix. The volumetric changes in the material and also a change in the ultrasonic pulse velocity through the concrete also indicate a basic change in the nature of the material and recently Robinson[11] was able to show that all these definitions gave effectively the same point. He concluded that this point was so vital that it might well replace the ultimate strength as a measure of the quality of concrete.

The significance of this point of discontinuity has been further emphasised by results obtained during this research project in which the rate of strain increase was measured on concrete prisms subjected to constant loads. The rate of increase was found to increase only very gradually with load below the point of discontinuity but to increase very rapidly above it (fig.7.5.7.1.). This result indicates a positive link between the point of discontinuity and the long-term strength as has been suggested by Rusch[101].

Hence it may be concluded that this discontinuity stress is more significant than the ultimate stress, particularly as it appears to be time independent, and therefore should be measured in all tests of concrete strength. To date, the only researcher to have measured this strength under combined states of stress is Robinson under biaxial compression.

Many other factors effect the strength of concrete (see section 1.2.). The type of loading, i.e. dynamic, static etc., and the previous strain history are subjects on which little work has been done and for which there will be no time in this research project.

Statistical considerations enter in at two levels. The most well known is the common problem of a representative number of specimens to obtain a statistically significant result[119,120]. The less well known which is especially critical for a heterogeneous material is the zone over which the critical failure stress field is spread. That is to say, will the initial fracture occur somewhere within a specified volume, surface, along a critical line, or at a peak point; and what are the dimensions of these critical zones. For complex loading systems the problem of freedom of crack propagation once the initial fracture has occurred also arises.

These statistical factors are further qualified if the alignment of potentially critical flaws are significant. The importance of this factor depends on the directions in which the specimen is allowed to fail and whether the number or orientation of these directions vary

according to that state of stress. This factor has been recognised by Neville[107], and Lichtenberg[121] and Weibull[122] have both proposed statistically based failure theories. Use is made of this statistical phenomenon in Chapter 12 in proposing a failure theory for mortar.

Furthermore, the testing machine/specimen reactions are important and, in any test, the degrees of freedom allowed to the loaded faces of the test specimen must be closely specified as discussed in section 1.2.

All these considerations introduce parameters which affect the strength of concrete to some degree. It will be shown later in the thesis which of these factors are being varied in the various test series.

1.6. Philosophy of This Project

The state of affairs in the field of failure criterion for concrete, as has been seen above, is unsatisfactory and stems from the complex structure of the material. It is clear that these complexities cannot be ignored and yet attempts to take them into account have so far failed to yield satisfactory predictions of behaviour. It therefore seems unlikely that any immediate breakthrough can be achieved on the theoretical side until much more is known about the structure of the material.

From the discussion of previous experimental work it was clear that most contributions were subject to criticism on major points of testing technique and from this review, a need for more reliable experimental work at a phenomenological level in the first instance arose. However, the execution of phenomenological tests alone is insufficient especially if these are conducted from purely an overall stress point of view since strain may be the more convenient criterion of failure. Furthermore, many structural characteristics of concrete might be marked by a purely phenomenological approach.

Hence the general aim of this project will be phenomenological tests on concrete under simple and combined stresses with a definite structural bias. With this in mind, the project will concentrate on such parameters as volume fraction and size of coarse aggregate which are important from the structural point of view as they define the inter-particle structure. These tests will have the dual purpose of yielding a more complete picture of the mechanical behaviour of concrete with a view to suggesting a failure criterion and of indicating the types of more fundamental tests that might be profitable in studying the structure of concrete.

An additional function of the tests would be to provide data for application to design problems involving concrete under combined stresses. The resulting design curves should be useful even in the absence of a mathematical form of failure criterion.

It should be pointed out that these objects are consistent with the terms of reference stated in section 1.1.

1.7. Testing Facilities Available

A survey of testing facilities available in January 1964 at the time when this project was finally planned showed the following:-

1.7.1. Uniaxial Compression

1) 300 ton. Amsler. - Excellent machine but not due for delivery until May 1965; hence of no consequence to this project.

2) 200 ton. Robinson. - Designed by Lachance[1] and being adapted by Robinson[10] for biaxial compression by replacing two 100 ton rams with a single 200 ton ram. This machine had proved suitable for compression tests.

3) 150 ton. Demion. - An old machine with load stiffness characteristics (9). Not to be relied upon for precision work.

1.7.2. Uniaxial Tension

1) 30 ton Ward. - Developed at Imperial College by Ward[7] and operating satisfactorily - sub-standard control valves.

1.7.3. Flexural Stress

1) 5 ton beam machine. - Developed at Imperial College and described by Ward[7]. Tends to be very flexibie frame and control valves are poor.

2) Slab and Disc machine. - Developed at Imperial College by Sigvaldason[9]. Good features include stiffness, control valves, and load cell.

1.7.4. Uniform Biaxial Stress

1) 200/100 ton Robinson[10]. - Still in development stage. Limited load capacity and only one facility of biaxial compression.

1.7.5. Selection of Machines

As a result of the survey, the 200 ton Robinson compression machine was adopted for uniaxial compression tests, the 30 ton Ward tension machine for uniaxial tension tests, and the Slab and Disc machine with attachments for the flexural beam test.

Robinson's biaxial compression machine was intended for only the one stress facility and the load capacity of 100 ton. in the horizontal direction was insufficient for most concretes. Hence a new and more versatile machine was necessary as anticipated in the terms of reference (section 1.1.).

1.8. Outline of Research Project

The philosophy of this research project has already been established and so, having regard to the laboratory equipment available, a series of logical steps followed.

Firstly, the design and development of a new combined stress testing machine was essential in view of the limitations of existing equipment. The need for this had been forseen in the terms of reference for this project (1.1.). Such a machine would be intended primarily for biaxial compression testing but several other facilities would be desirable especially biaxial compression-tension. The experience and ideas of previous workers in this field was to be drawn on as well as capitalising on much recent work on testing machine design.by the Concrete Materials Research Group (1-3,7,9,10). This section of the work is dealt with in Chapters 2-4.

Secondly, having successfully developed this testing machine, a comprehensive series of tests would be apanned to study the mechanical deformation and failure characteristics of a representative range of mortars and concretes (cube strength of 3,000-1,500 p.s.i. of CP.114 and CP.115) under biaxial compression with a few tested under biaxial compression-tension if this second facility could be developed in time. This work is described in Chapters 6, 8 and 9 whilst a uniaxial series of tests to act as an adjunct to this work is described in Chapter 7.

In view of the problems associated with the philosophy of strength (1.4.), the USPV technique as recommended by Reiding[123] and as used by Robinson[10] was adopted. This equipment plus all the strain measurement equipment and associated techniques are dealt with in Chapter 5.

From the results of these tests an attempt was to be made to establish failure criteria or at least failure mechanism for concretes and mortars and this process is covered in Chapters 11 and 12.

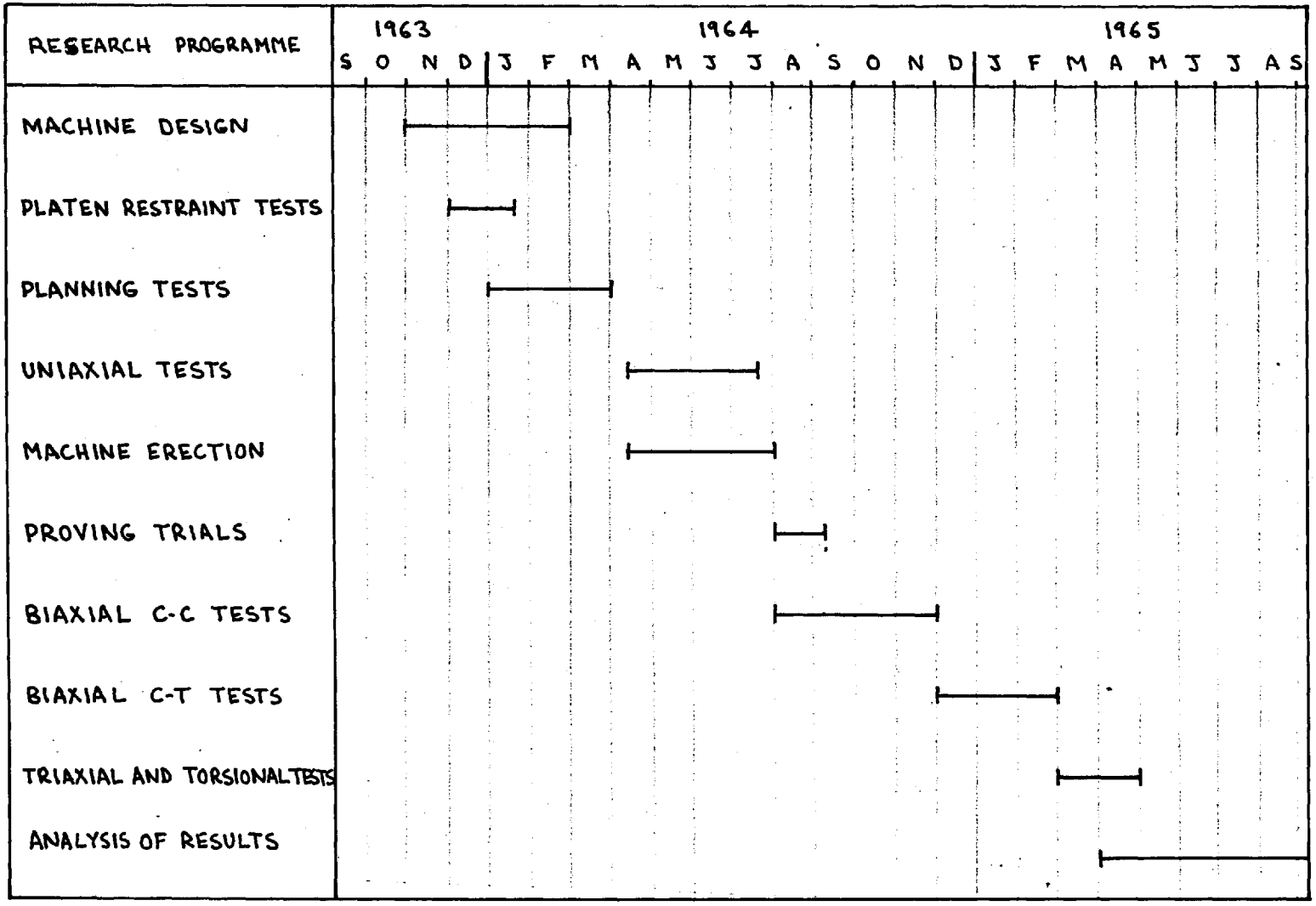


FIG. 1.8.1. ACTUAL RESEACH TIMETABLE.

Additional work included preliminary trials on the biaxial compression-torsion and triaxial compression facilities and these are described in Chapter 10. Chapter 13 concludes the thesis and Chapter 14 suggests some future research projects.

An outline programme of the research work as actually carried out on this project is shown in fig.1.8.1.

1.9. Summary

Commencing with the given problem of failure criteria for concrete and the terms of reference for this research project, a review of past research (section 1.3.) brought the basic problems into focus. The complex behaviour of mortars and concretes, and the difficulties with various testing techniques were emphasised.

A brief examination of failure theories (section 1.4.) revealed the short comings of their basic assumptions when compared with the structure of concrete and explained the gap that exists between theoretical predictions and experimental results.

The philosophy of concrete strength (section 1.5.) revealed another aspect of the overall problem of specifying failure criteria for concrete.

A philosophy for this research project was then developed (section 1.6.) from the foregoing and, from a brief review of available testing facilities (section 1.7.), an outline for the research project was established (section 1.8.).

CHAPTER 2

SELECTION OF TYPE AND GENERAL ARRANGEMENT OF COMBINED STRESS TESTING MACHINE

2.1 Introduction

The first task to be tackled is the design and development of a new combined stress testing machine as forseen in the terms of reference (section 1.1) and confirmed in the outline of the research project (section 1.7). This chapter deals with all the factors that led up to the choice of type and general arrangement finally selected.

During this research project several visits were made to other research establishments to discuss various ideas on machine design and to inspect existing machines. A preliminary to any discussion on possible machine types is a thorough survey of existing machines and the results of this survey are recorded in the next section (2.2).

The general requirements of a combined stress testing machine have been listed in section 2.3 to form a basis for the machine selection and as guides at the detail design stage.

The most likely possible types of machine are then examined (section 2.4) and the problem of selection of type is finally resolved by the platen restraint trials conducted by Robinson and Vile[25] and reported in section 2.5.

Finally, having established the type of machine to be used, the general arrangement of the machine is selected (section 2.6) and certain outstanding features of its arrangement are discussed (section 2.7).

2.2 Review of Previous Designs

In this review, only uniform stress and some calculated stress machines or loading devices are considered (see section 1.2 for definitions). Flexural and semi-flexural machines such as those

used by Smith[42] and Bresler and Pister[45] are not considered as the research in this project is restricted to uniform states of stress only.

The three machines[124, 47, 48], included in section 2.2.2., in which stress gradients exist in the test specimen illustrate the problems associated with combining hydraulic and mechanical types of loading (defined below).

For convenience, the machines have been divided up into the following arbitary classifications and are described and criticised under these headings:-

2.2.1. Pure Hydraulic Loading
2.2.2. Hydraulic/Mechanical Loading
2.2.3. Pure Mechanical-uniaxial machine devices
2.2.4. Pure Mechanical - two separate frames
2.2.5. Pure Mechanical - single frame

First, it is necessary to explain these two terms "hydraulic" and "mdchanical" as used in this context. "Hydraulic" means that the load is applied to the surface of the test specimen by means of hydraulic pressure with or without an intervening membrane; "mechanical" means that the load is applied to the surface of the test specimen through rigid metal plates, clamps, or other semi-rigid devices. Hence the hydraulic/mechanical loading systems described in section 2.2.2. cover those machines in which one stress is applied hydraulically whilst the other is applied mechanically.

Triaxial machines or loading systems have been largely ignored in this review except where biaxial stresses can be obtained (usually by hydraulic pressure) as the research of this project is mainly concerned with biaxial states of stress and especially with biaxial compression. A total of twenty-five district biaxial stress testing machines are reviewed.

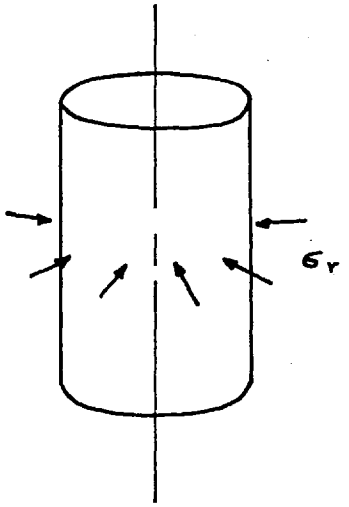
2.2.1. Pure Hydraulic Loading

The most common type of purely hydraulic testing machine for the application of biaxial stress are the triaxial cells used by Rosad Eichinger[24, 32]; Richart, Brandtzaeg and Brown[35]; Balmer[49]; Ackroyd[126] and others. The biaxial compressive stress oondition is attained by the radial hydrostatic pressure on a cylindrical specimen. This arrangement, though free from platen restraint effects, suffers the disadvantage that $\sigma_3 = \sigma_2$ and hence limits experimental work to equal biaxial compression stresses only (fig.2.2.1.1.) Lateral strain measurements are also a problem with this type of apparatus and few cells contain viewing panels.

Recent work has been carried out at Cambridge University in which hydraulic pressure was applied to faces of a cube separately by holding a "hydraulic platen" against each face to be loaded (fig.2.2.1.2.). The cube was selected as intrinsically the best specimen shape for extension to triaxial compression tests at a later date.

The steel platen was recessed around its edge to provide lateral restraint for a rubber seal of circular section. Two such platens together with their seals were clamped to opposite faces of the cube and hydraulic fluid was pumped into the cavity between the cube face and the platen. In this way pressure, up to the limits of containment possible with the rubber seal, could be applied to the cube face, free of platen restraint effects. Figure 2.2.1.3. shows how the basic idea might be extended to triaxial compression.

Unfortunately, the scheme suffers from two major disadvantages. The limitations of the rubber seal have so far kept the workable pressures of the arrangement well below those necessary to cause failure of even a very weak concrete. Secondly, if this first problem could be overcome, there would still remain the problem of the load distribution on the face of the cube shown in fig.2.2.1.4.



$$\sigma_3 = \sigma_2 = \sigma_r$$

FIG. 2.2.1.1. EQUAL BIAXIAL COMPRESSION FROM HYDRAULIC PRESSURE.

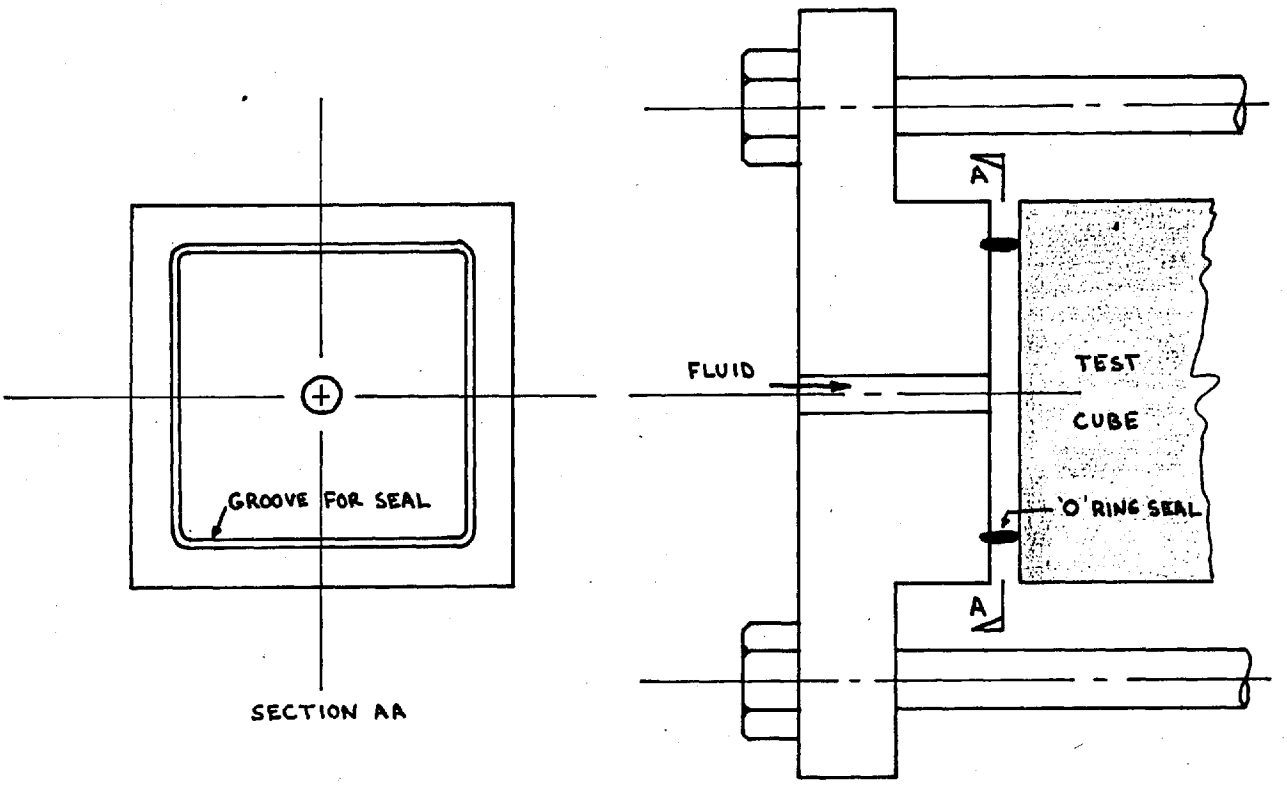


FIG. 2.2.1.2. LINE DIAGRAM OF CAMBRIDGE HYDRAULIC PLATEN.

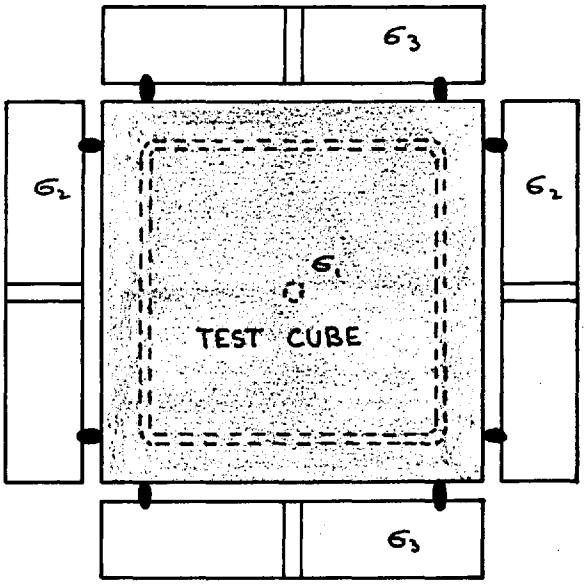


FIG. 2.2.1.3. HYDRAULICPLATEN USED FOR TRIAXIAL COMPRESSION.

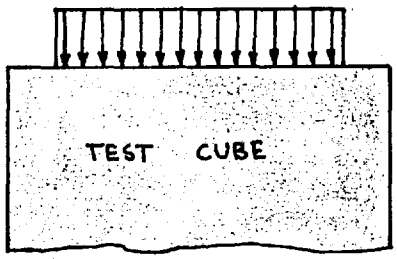


FIG. 2.2.1.4 PRESSURE DISTRIBUTION ON CUBE FACE FROM HYDRAULIC PLATEN.

The resulting test would be some intermediate condition between a direct compression test and an indirect tension splitting test. Certainly no great reliance could be placed on the strength results obtained.

It is understood that the University of Cambridge project has now been temporarily suspended.

An alternative idea for a pure hydraulic testing machine was designed prior to this visit by the author and is described in section 2.4. It is an attempt to overcome the load distribution problem caused by the necessity of incorporating rubber seals to contain the lateral hydraulic pressure.

2.2.2. Hydraulic/Mechanical Loading

Four examples of this type of machine have already been noted in the review in Chapter 1 of previous experimental work.

The three machines used by McHenry and Karni[124], Tsuboi and Swenage[47], and Bellamy[48] and Campbell-Allen[53] utilised hollow cylinders in which internal hydraulic pressure caused a hoop tension whilst external hydraulic pressure caused a hoop compression. The other stress was in all cases compression and was applied mechanically with an initial platen along the axis of the specimen (see section 1.2).

This type of test produces the required critical biaxial stress only on one surface and, as has been pointed out previously, it is very difficult to determine when exactly this type of specimen fails. The state of stress varies across the wall thickness and under biaxial compression and uniaxial-tension platen restraint tends to reinforce the specimen so that unduly high strengths result. In addition, strain measurements are difficult.

Reinius[43], however, combined these two types of loading by clamping hydraulically operated pressure bags to two opposite faces of a prismatic compression specimen (section 1.2.9). This scheme was similar to one suggested by Foppl[22, 57].

The pressure bags were made of rubber and Reinius reported that they tended to bulge out under pressures in excess of 40-50 kg/cm^2. (about 600 p.s.i.). Hence the capacity of the machine was severely limited but the technique did have the big advantage that strain measurements could be taken in all three dimensions.

A point of detail that might cause trouble with this type of machine is where the two types of loading meet at the corners. If the rubber membrane were curtailed too far from the corner, failure of the test specimen might be precipitated by a uniaxial failure at the extreme ends of the specimen.

2.2.3. Pure Mechanical-uniaxial machine devices

Only three such machines have been discovered in the literature and these have already been reviewed in Chapter 1. All of them are intended for biaxial compression only.

Foppl's device[22] (section 1.2.1.) was a most ingenious attempt at the time of development. However, its limitations compared with contemporary standards tend to eliminate its possible usefullness. As has been observed, the device is only capable of applying equal biaxial compressive stresses ($\sigma_3 = \sigma_2$) and exerts both primary shear due to linkage deflections and secondary shear due to platen restraint with a cubical specimen (see section 1.2. for definitions).

Wastlund's original device[40] and the more elaborate modification of it developed by Weigler and Becker[50, 51] are certainly improvements in principal. The ratio of the principal

and intermediate compression stresses can be varied over certain fixed loading ratios and slab rather than cubical specimens can be used. Full access is also provided for strain measurement.

The only drawback with this type of loading device is the stiffness required in the ratio arm (which is thought to be insufficient in the case of Weigler and Becker) and the limitations of clear space available in the average uniaxial compression machine. For example, the clear space needed to test a 10" x 10" x 4" slab as would be necessary if $\frac{3}{4}$" aggregate concretes were to be tested, would be about five feet using a rig similar to this.

2.2.4. Pure Mechanical - two separate frames

Five machines[127, 128, PC, 10, 55] that may be included in this category consist of suspending a secondary test frame horizontally through a primary vertical machine usually with the aid of a counterweight system to enable the secondary frame to follow the movements of the test specimen dictated by the vertical machine (fig.2.2.4.1.). All of these machines were intended for biaxial compression only.

Bridgman[127] developed a machine of this type for testing metals whilst that designed by Hobbs[128] was used for biaxial compression tests on coal and by Hawkes[86] on rocks generally. The other two by Sundara Rajn Iyengar[55] and Robinson[10] were used for tests on mortars and concretes.

An alternative arrangement for biaxial compression was used by Chipman, Lynch and Peterson[129] on mortar models of pressure vessel loading faces in which the two frames were arranged horiz ntally on rollers thus eliminating the specimen alignment problem referred to above.

These arrangements have the advantage that only one jack is required in each testing frame and easy access is available to the

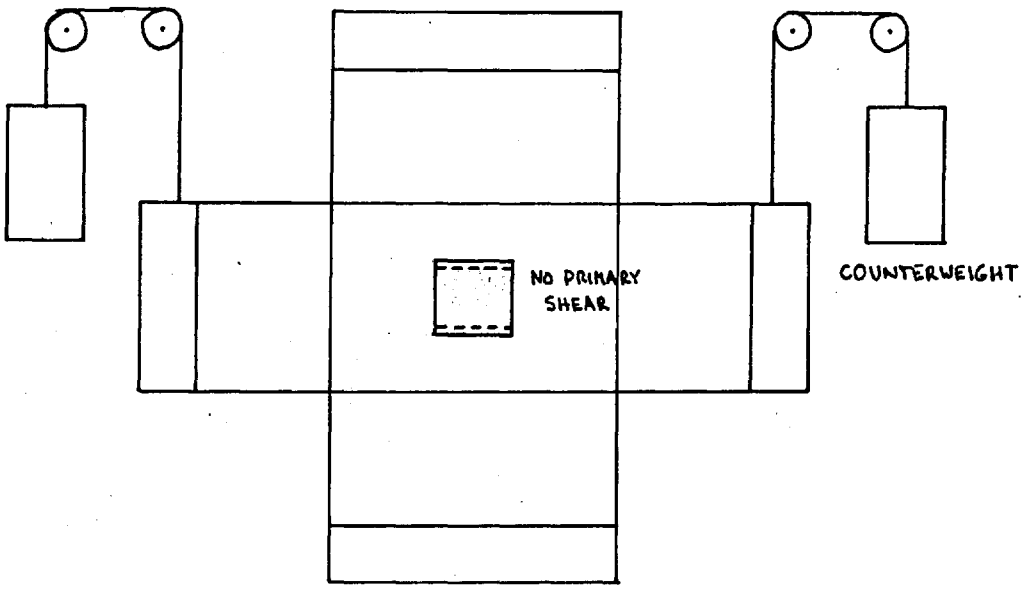


FIG. 2.2.4.1. MECHANICAL MACHINE WITH COUNTERWEIGHTED FRAME.

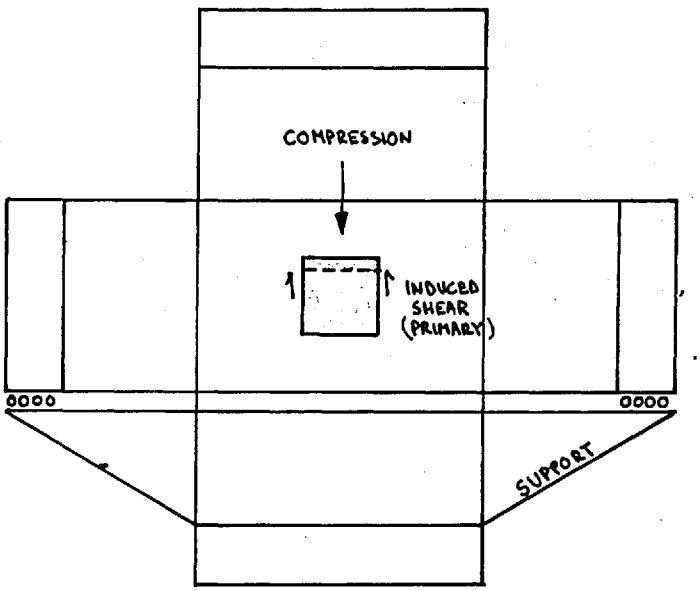


FIG. 2.2.4.2. MECHANICAL MACHINE WITH SUPPORTED FRAME.

specimen for strain measurements and visual observations. The only major problems are alignment on the specimen in one direction, lack of lateral stiffness of the horizontally suspended test frame; and the one common to all mechanical loading devices of platen restraint.

Only Robinson[10] has so far published results on proving trials on the state of stress but he was able to achieve a small zone of uniform strain in the centre of his specimen.

The only mechanical machine for biaxial compression-tension found in the literature was that by Nishingawa[52] already described in section 1.2.15. Again only one ram was used in each of the two testing frames but the test set up is subject to criticism as the necessary conditions of alignment on the specimen are not satisfied; the horizontal tension frame being mounted on the vertical frame and not suspended on counterweights. The misalignment obtained from this set up is illustrated in fig.2.2.4.2.

Lack of published details on the machines used by Hobbs[128] and Sundara Rajn Iyengar[35] make it impossible to comment on their alignment efficiency.

2.2.5. Pure Mechanical - single frame

The machines contained in this classification are similar in action to those described in the last section except that a single reaction frame embraces the whole machine. As a result, to maintain symmetry anf often, alignment on the specimen as well, pairs of opposed jacks are used in each principal stress direction in place of the usual single jack.

Indeed only one two-jack machine was found in this classification and that was designed at Swansea University[PC] for research into the behaviour of rocks under biaxial compression. A line diagram of the machine is shown in fig.2.2.5.1. and the

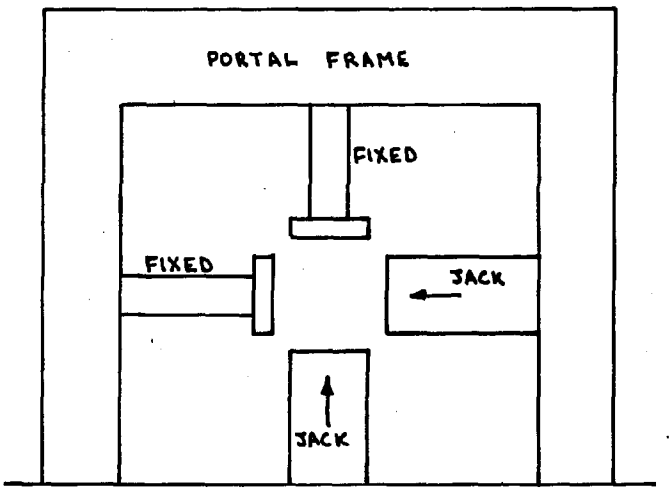


FIG. 2.2.5.1. LINE DIAGRAM OF SWANSEA BIAXIAL COMPRESSION MACHINE.

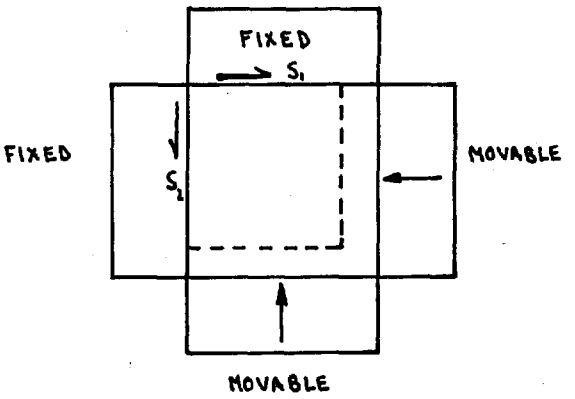


FIG. 2.2.5.2. RESULTING SHEARS (PRIMARY) AND LOADING ECCENTRICITY WITH SWANSEA MACHINE.

deflection problems likely to be encountered with this arrangement are illustrated in fig.2.2.5.2.

As the specimen is compressed shear forces will develop along the length of the fixed platens unless these are mounted on rollers. If they are so mounted, as is believed to be the case, then the centreline of the compressive thrust and possibly also the platen supported by the jack are eccentric to the specimen and will therefore induce secondary stresses.

The other machines by Glomb[46] (section 1.2.11); I.S.M.E.S.[130]; Swain[131]; C.S.I.R. (Council for Scientific and Industrial Research, South Afirca)[PC]; and Hilsdorf[57] are all four ram machines with symmetrical reaction frames. Those by Swain and C.S.I.R. are mainly concerned with rock testing whilst the others have all been specifically developed for concrete work. Gomb's machine has already been discussed.

Hilsdorf's[57] proposed biaxial compression machine is similar to that designed by Glomb[46] in that four jacks in opposed pairs are located within a stiff symmetrical octagonal frame (2.2.5.3.). Alignment on the specimen, as with the other machines, depends on the symmetry of the system. A similar arrangement has been adopted by I.S.M.E.S. for a 200 ton capacity biaxial frame which is often used in a 2,000 ton compression machine for triaxial tests. Specimen size may be up to 30 cm. cube. (fig.2.2.5.4.).

Swain[131], in work on the effects on strength of various slope tunnels through rock, developed the machine shown in fig. 2.2.5.5. The virtue of the design is that the size of specimen that the machine can test is varied by adding more jacks. In addition, each jack has its own platen and, with a number of independent platens along each loaded face of the test specimen, the platen restraint effects will be reduced. However, the machine has two serious drawbacks. Firstly, even with the most careful matching

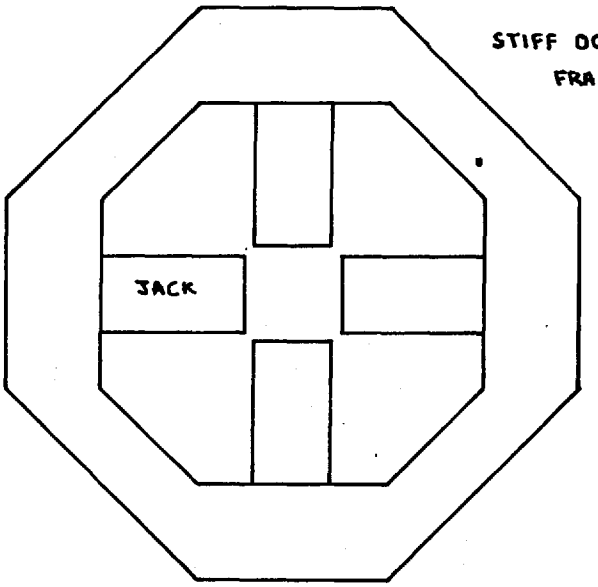


FIG. 2.2.5.3. LINE DIAGRAM OF GLOMB'S BIAXIAL COMPRESSION MACHINE (46).

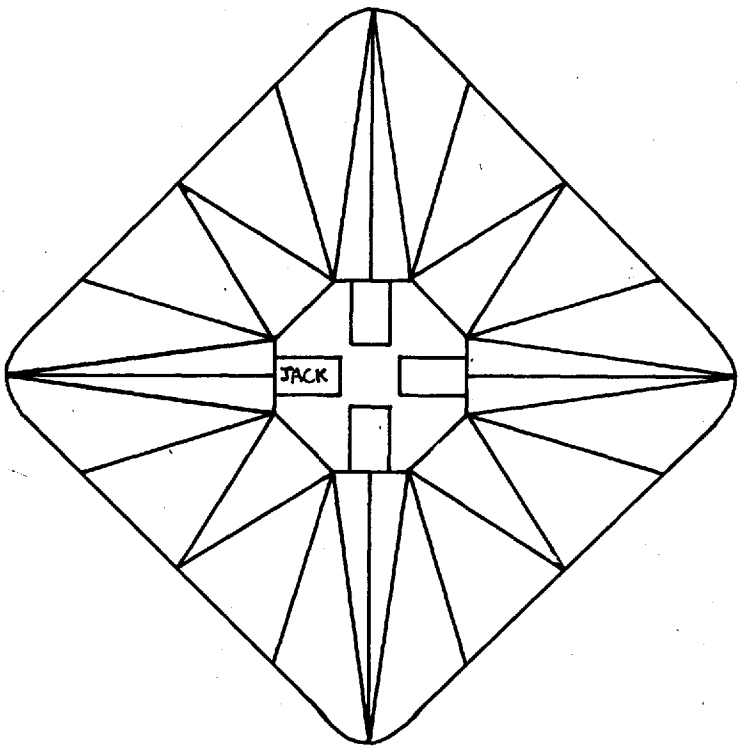


FIG. 2.2.5.4. LINE DIAGRAM OF BIAXIAL COMPRESSION FRAME BY I.S.M.E.S. (130).

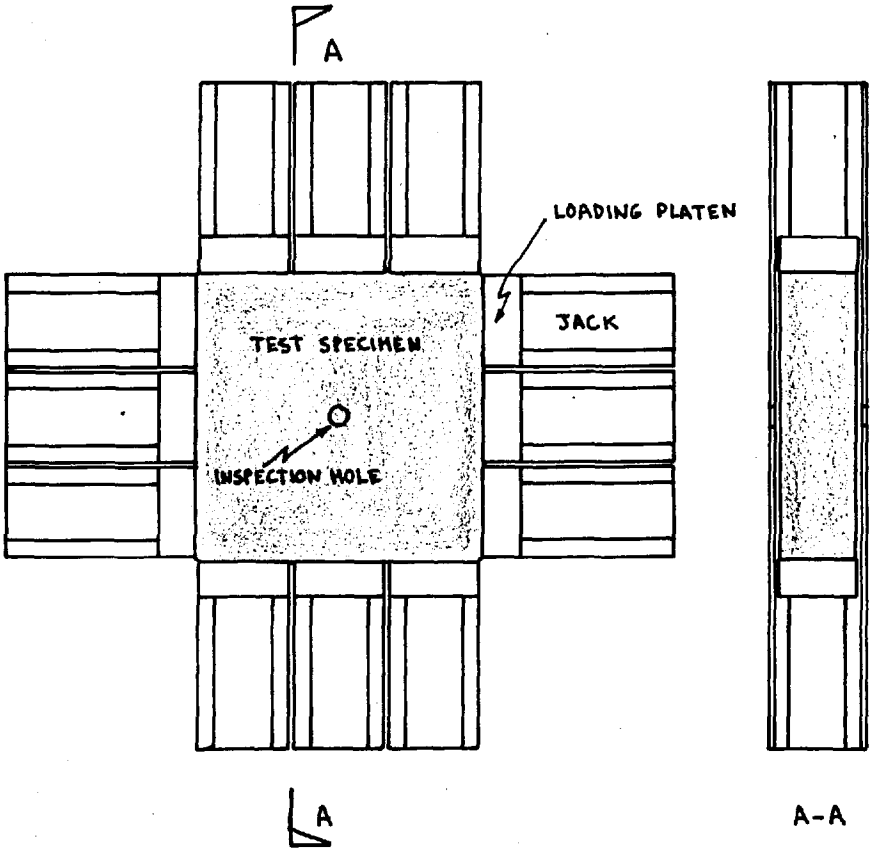


FIG. 2.2.5.5 LINE DIAGRAM OF BIAXIAL COMPRESSION MACHINE BY SWAIN (131). – SHOWN WITH THREE UNIT ARRANGEMENT.

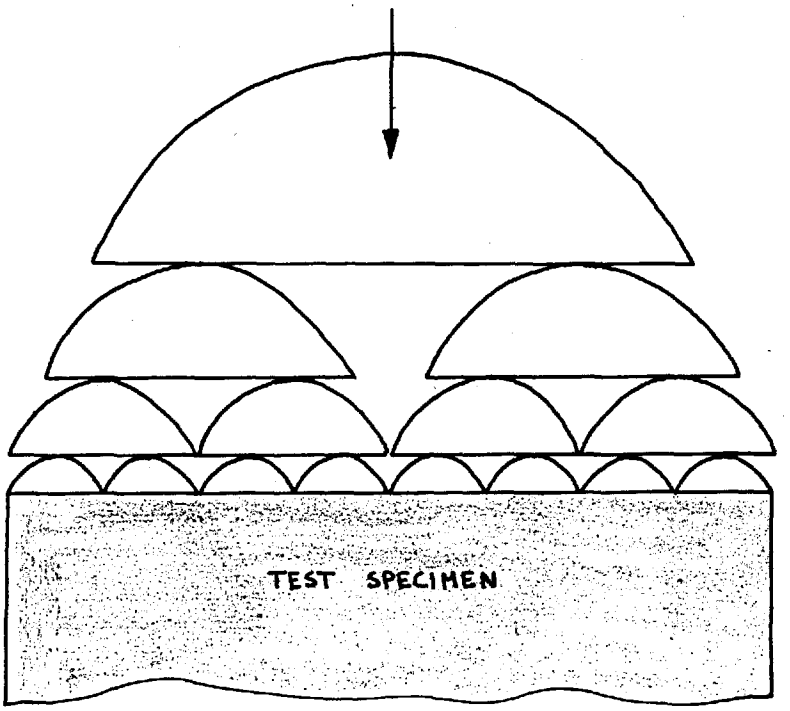


FIG. 2.2.5.6 LINE DIAGRAM OF LOADING PLATEN BY C.S.I.R. (PC).

the jacks will not all have the same identical friction characteristics and hence the pressure along the face of a specimen will not be uniform. Secondly the frame is completely enclosed with the exception of a small viewing hole in the middle which makes it unacceptable for concrete tests were ease of access is essential.

Another attempt to overcome platen restraint effects has been made by C.S.I.R.(PC) in the design of their biaxial compression machine for tests on rock specimens. Semi-circular prisms are placed above each other in pairs of increasing size as shown in fig.2.2.5.6. so that eight such prisms are in contact with the loaded surface of the test specimen yet only one is loaded by the compression jacks. The whole machine frame is shown as a line diagram in fig.2.2.5.7.

Although this arrangement appears to be a good compromise between mechanical loading with its platen restraint effects and completely hydraulic loading, it may suffer from a serious fault. As shown in fig.2.2.5.6. prior to any deformation of the test specimen the system will give a uniform load with much less restraint effects than would be experienced with a single solid loading platen. However, once the specimen deforms, as it must inevitably to carry any load, the system of loading prisms is immediately no longer capable of applying a uniformly distributed load as illustrated in fig.2.2.5.8. (for a case of expansion). Indeed it is probably that the secondary effects on the stress distribution in the test specimen will be greater from the non-uniform loading that this arrangement gives than from the alternative platen restraint effects.

As with Hilsdorf's brush platen discussed in section 1.2.21., the mechanical solutions to the platen restraint problem proposed so far appear to yield worse results than the detrimental effects they are designed to overcome.

A point to notice about all the machines in this section is that they are capable of a biaxial compression facility only.

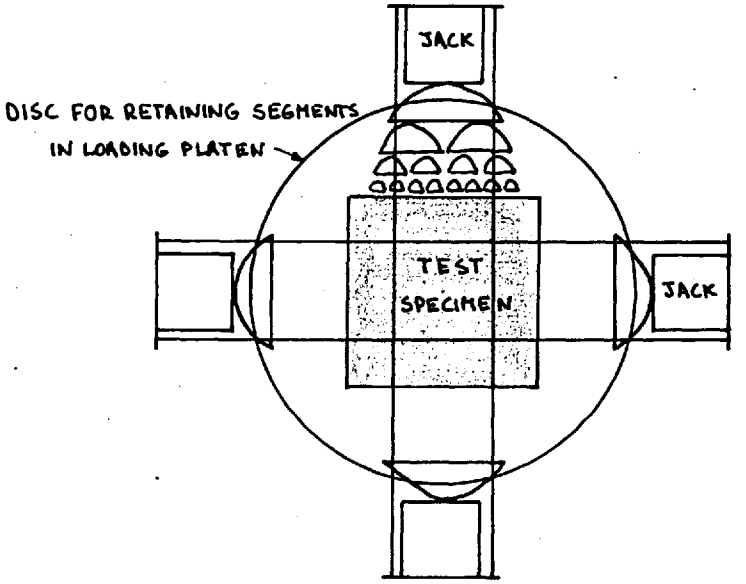


FIG. 2.2.5.7. LINE DIAGRAM OF BIAXIAL COMPRESSION MACHINE BY C.S.I.R.

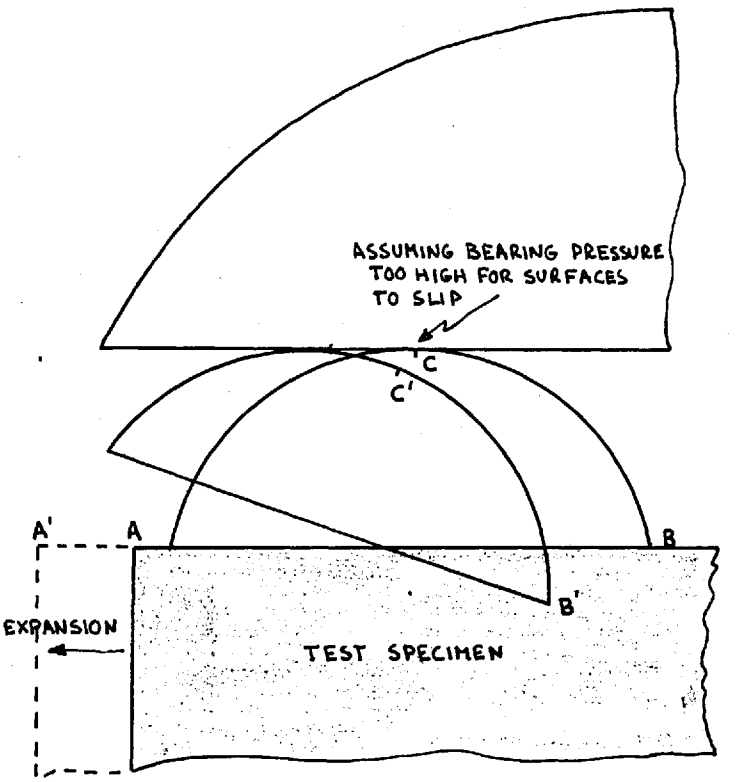


FIG. 2.2.5.8. ROTATION OF PLATEN SEGMENT CONSEQUENT UPON STRAINING OF TEST SPECIMEN IN C.S.I.R. MACHINE ASSUMING NO SLIP.

2.2.6. Reference List

For convenience, a list of all the machines reviewed during this work under their arbitary classification and with the relevant bibliographical references in the margin, is given here.

2.2.1. Pure Hydraulic Loading

1.	Ros and Eichinger	(24, 32)
2.	Richart, Brandtzaeg and Brown	(35)
3.	Balmer	(49)
4.	Ackroyd	(126)
5.	Cambridge hydraulic platen	PC

2.2.2. Hydraulic/Mechanical Loading

6.	McHenry and Karni	(124)
7.	Tsuboi and Suenaga	(47)
8.	Bellamy/Campbell Allen	(48, 53)
9.	Reinius	(43)

2.2.3. Pure Mechanical - uniaxial machine devices

10.	Foppl	(22)
11.	Wastlund	(40)
12.	Weigler and Becker	(50, 51)

2.2.4. Pure Mechanical - two separate frames

13.	Bridgman	(127)
14.	Hobbs	(128)
15.	Hawkes	PC
16.	Robinson	(10)
17.	Sundora Rajn Iyengor	(55)
18.	Chipman, Lynch and Peterson	(129)
19.	Nishigawa	(52)

2.2.5. Pure Mechanical - single frame

20.	Swansea University	PC
21.	Glomb	(46)
22.	Hilsdorf	(57)
23.	I.S.M.E.S.	(130)
24.	Swain	(131)
25.	C.S.I.R.	PC

2.3 General Requirements of a Combined Stress Testing Machine

Before going on to consider the possible types of testing machines or devices that might be adapted for this research project, it is as well to consider the general requirements of a testing machine of this kind.

Any concrete testing machine should be robust in operation because of the large forces involved on a representative sample. (It is suggested that to be representative of concrete, a sample should be not less than a volume of minimum side five times the maximum aggregate size), and also because of its apparent brittle nature observed especially under uniaxial and biaxial stress states when explosive failures are a constant hazard. Robustness is also desirable in the operation of placing specimens in the machine. The number of operations involved in setting up should be kept to a minimum and excessive work on specimen preparation should be avoided. Embedded bars and plates[52] are not desirable in a test specimen nor brush platens[57] used by some workers for applying tension in which the brush has to be stuck to each individual test specimen. A good example of a robust testing machine, which applies a state of stress normally bedevilled by specimen preparation, is the uniaxial tension testing machine designed and developed by M. Ward[7].

A cardinal consideration, which has earned special attention throughout the review of existing machines, is the achievement of uniform and desires states of stress of known magnitude (section 1.2). This is probably the most difficult single problem to be solved by workers in the field of combined stress properties of concrete. Clearly much more thought must be put into this problem before any more testing is done as the lack of assurance on the uniformity of stress state has tended to invalidate much of the work so far.

If a combined stress testing machine is to be developed, then as many facilities should be incorporated as possible. However, it is perhaps significant that all the machines reviewed above are capable of only one, or at the most two, biaxial combinations of stress. The most desirable facilities are those of tension compression, and torsion but, of these, some combinations are very difficult to achieve such as direct biaxial tension or torsion.

Accurate measurement of stress and strain in the three principal directions is essential, and, with concrete, the ability to observe and have access to the surface of the specimen during tests is highly desirable. Ultra-sonic pulse velocity and lateral strain measuring equipment usually needs plenty of room around the test specimen and an adequate view of the specimen also enables photo-elastic measurements to be made.

Adjustment for text specimen size and shape is also a useful asset and automatic self-alignment of the machine on the specimen whilst under load is essential. This latter point was discussed at some length in the preceeding review (section 2.2.).

The independent variation of stress ($\sigma_1 \neq \sigma_2 \neq \sigma_3$) (see section 1.2 for definition) is a basic requirement of this research and the ease of operation to achieve these conditions is an important consideration.

With a mechanical machine as defined in section 2.2., the longitudinal and lateral stiffnesses of the machine frame is important and this will have a bearing on the general arrangements considered. By longitudinal stiffness is means the stiffness in the direction of applied load whilst lateral stiffness is the stiffness at right angles to the applied load. Provision for extension to other stress facilities at a later date should also be seriously considered so that the maximum use can be made of this machine after the termination of this contract.

Lastly, ease of maintenance, a point often forgotten in machine design should be considered throughout especially on a contract of this kind in which all the testing time available is precious and a breakdown could have serious consequences on the progress of the whole project.

2.4 Possible Machine Types

The two dominant considerations here are the achievement of pure states of stress and versatility in stress combinations ($\sigma_1 \neq \sigma_2 \neq \sigma_3$) with emphasis on biaxial compression.

The basic choice of machine type lies between purely hydraulic or mechanical, or a combination of the two. Therefore the advantages and disadvantages of load application by hydraulic or mechanical means should first be considered.

Hydraulic loading ensures good stress transfer with no secondary stresses induced under conditions of plane stress. No alignment problems arise and the hydraulic loading cell is usually very compact such that it will readily fit into a standard compression machine for triaxial tests. However, the tension facility gives problems which are not really solved satisfactorily by applying internal hydraulic pressure to a hollow cylinder as explained in section 1.2.

Torsion facilities are virtually impossible with this type of loading and the problems of access to the test specimen usually mean that there are no strain measurements and no visual aids.

The mechanical loading systems, on the other load, enable a variety of stresses to be applied and strain measurement problems are usually much easier. Stresses can be varied independently and triaxial facilities are relatively simple.

However, the transfer of stress is usually poor with significant secondary stresses induced (section 1.2.) and the

stress condition is close to but not exactly plane strain. Specimen alignment problems are critical and a special frame is required. Finally, machines of this kind usually take up a lot of space.

From the above considerations, two basic ideas developed and were investigated as possibilities for this project. The first was a hydraulic loading system that could be used either purely hydraulically or in combination with mechanical loading along one axis only; the other was an improved scheme for a four jack mechanical machine.

The hydraulic device is shown in fig.2.4.1. and consists of shaped seals held firmly against the corner of a cube or prism test specimen. Four hydraulic cells are produced around the specimen linked hydraulically in opposite pairs enabling the two principal compression stresses σ_3 and σ_2 to be varied independently of each other. The detail of the rubber seal is shown in fig.2.4.2. in which the dimensions are chosen such that $y \geq 2x$ and $x > Z$. The dimension Z needs to be as small as is practical to prevent a differential pressure forcing the seal arm through the gap between the cell case and the test specimen whilst x also needs to be small so that the hydraulic pressure may act right up to the corner giving a uniform pressure distribution on the specimen. Dimension Y should be large to resist the blow-out of the seal mentioned above.

This system could be used around a cube to apply triaxial compression or an axial compression or tension could be applied mechanically to a rectangular prism shaped specimen.

Development time on an idea of this kind could be considerable which is a decided disadvantage in a research contract of limited duration.

The alternative mechanical scheme was developed with a regard for machine stability and stiffness. Hence two similar

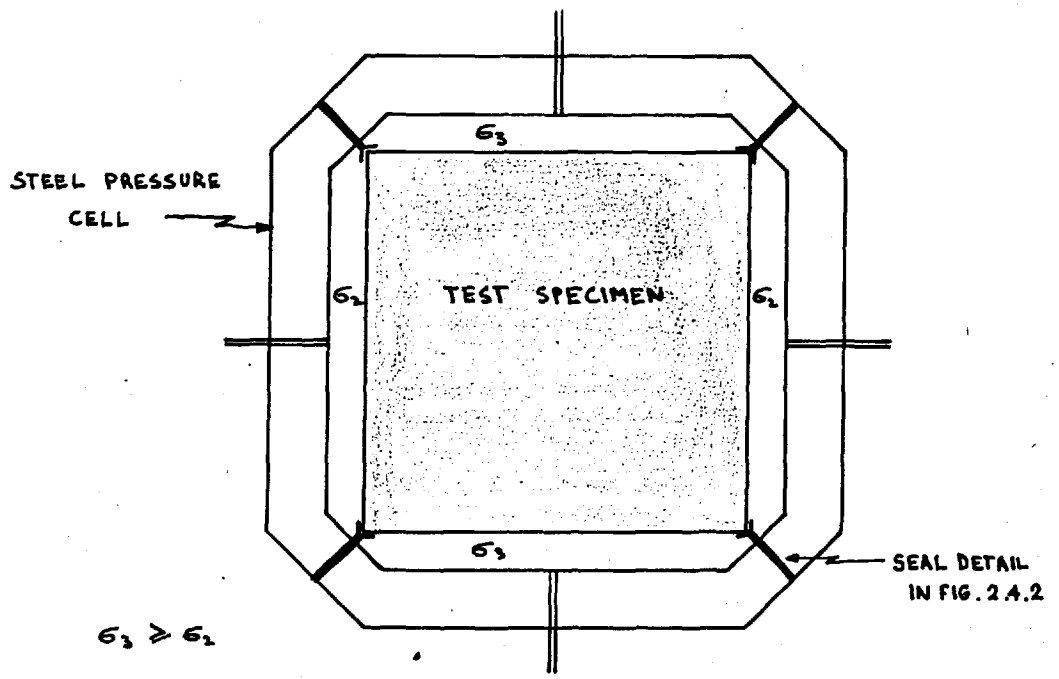


FIG. 2.4.1. LINE DIAGRAM OF HYDRAULIC BIAXIAL (OR TRIAXIAL) COMPRESSION MACHINE BY VILE.

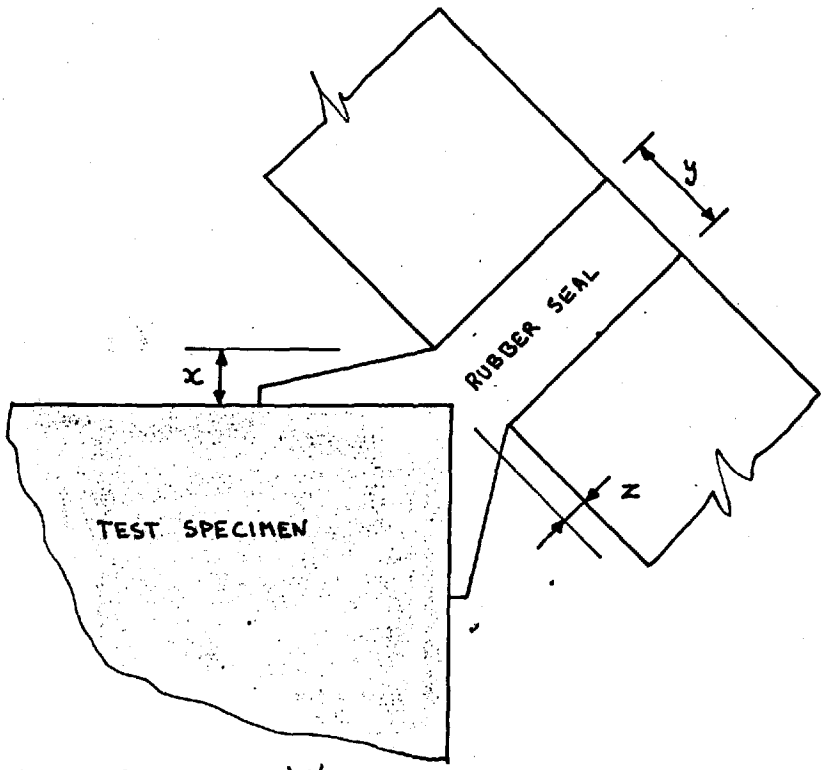


FIG. 2.4.2. DETAIL OF 'Y' SHAPED CORNER SEAL BY VILE.

sub-frames comprising two end plates and four columns each were selected; longitudinal stiffness being the paramount criteria.

These frames were to contain opposed pairs of jacks to give a symmetrical arrangement about the specimen centreline and the loads were to be applied through rigid platens or clamps. Alignment on the specimen was to be maintained by an all embracing frame shown diagrammatically in fig.2.4.3.

Apart from the other advantages of this type of machine already listed above, the machine would be incorporating many well tried techniques and could capitalise on the recent work done by Sigvaldason[9, 12-15] on testing machines. Hence the development time would be less than for the hydraulic machine suggested above.

However, the problem of platen restraint effects has yet to be solved and this will be tackled in the next section.

2.5 Platen Restraint Trials (ref.25)

Although it is doubtful whether the platen restraint effects can ever be eliminated from the whole of a test specimen, any reduction of such effects is helpful. The most that can be expected with even contemporary "frictionless" materials is that platen restraint effects can be reduced a sufficient amount to yield a central volume of the test specimen in a pure state of stress.

Fortunately, at the time of these preliminary design studies, Robinson[10] was also proposing to carry out similar tests in the development of his machine and it was agreed to conduct a joint series of tests. It must, however, be acknowledged that the general concept of the experimental technique used in the tests and much of the analysis of results was largely attributed to Robinson.

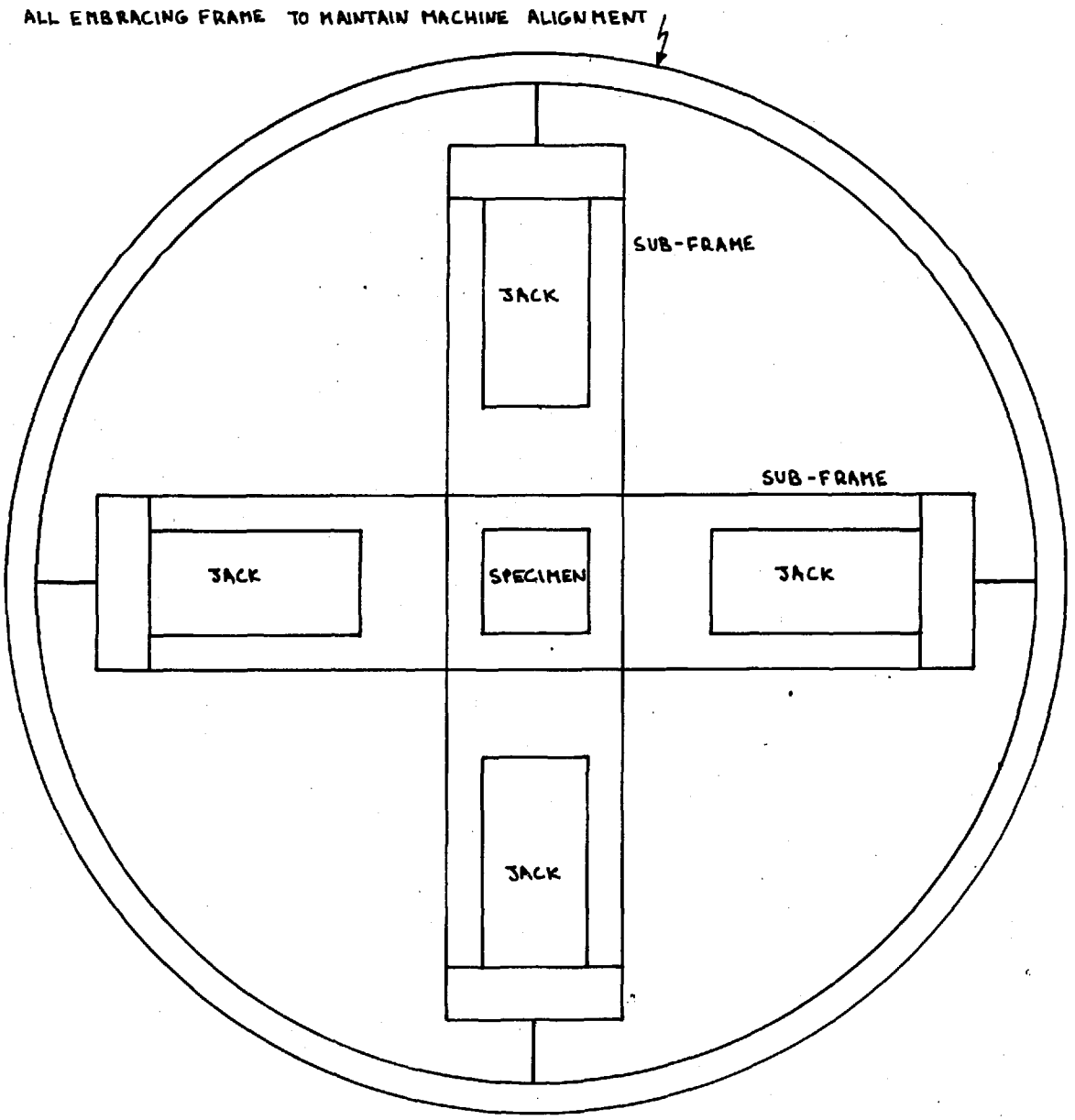


FIG. 2.4.3 DIAGRAMMATIC ARRANGEMENT OF MECHANICAL MACHINE.

Many of the attempts made to reduce platen effects have been noted in the review of previous work. However, apart from A. Foppl and C. Bach[132], and Ros and Eichinger[24], little work was done on this subject until the investigation by Lachance[1-2] referred to in the Introduction (section 1.1). However, this latter work was more concerned with specimen dimensions and much more detailed results using a wider range of packings were required for this investigation. Twelve different packing materials were used and, by varying the coating application in the case of Febcure super, a total of 16 different packing conditions were investigated.

Three additional trials were carried out on an aluminium specimen.

2.5.1. Experimental Procedure

Mortar prisms 4" x 4" x 12" were cast, five from each batch, the mix being (:20:-0.42/-:3/16") and kept in a saturated condition up to the testing time at 28 days old (See section 6.6 for standard casting and curing procedure).

The packing conditions were then taken in groups of five with a plain coating all over in Febcure super being used as the reference condition. Three batches were used for each group of five so that the results of each packing condition was to be seen on three different mortar specimens.

Using a Sogenique linear displacement transducer mounted on a modified Lamb's optical extensometer frame, lateral strain measurements could be taken over the length of the prism and in the steel platen as well. A photograph of the frame is given in fig.2.5.1.2. The transducer had been calibrated against a Sogenique standard and was guaranteed accurate to 1 micro-inch. In service it proved repeatable to $\pm$0.5% or two micro-inches for one set-up and from one set up to the next to $\pm$1.9%. Drift amounted to 3 or 4 mirco-inches and, as all the measurements were taken over a four inch gauge length,

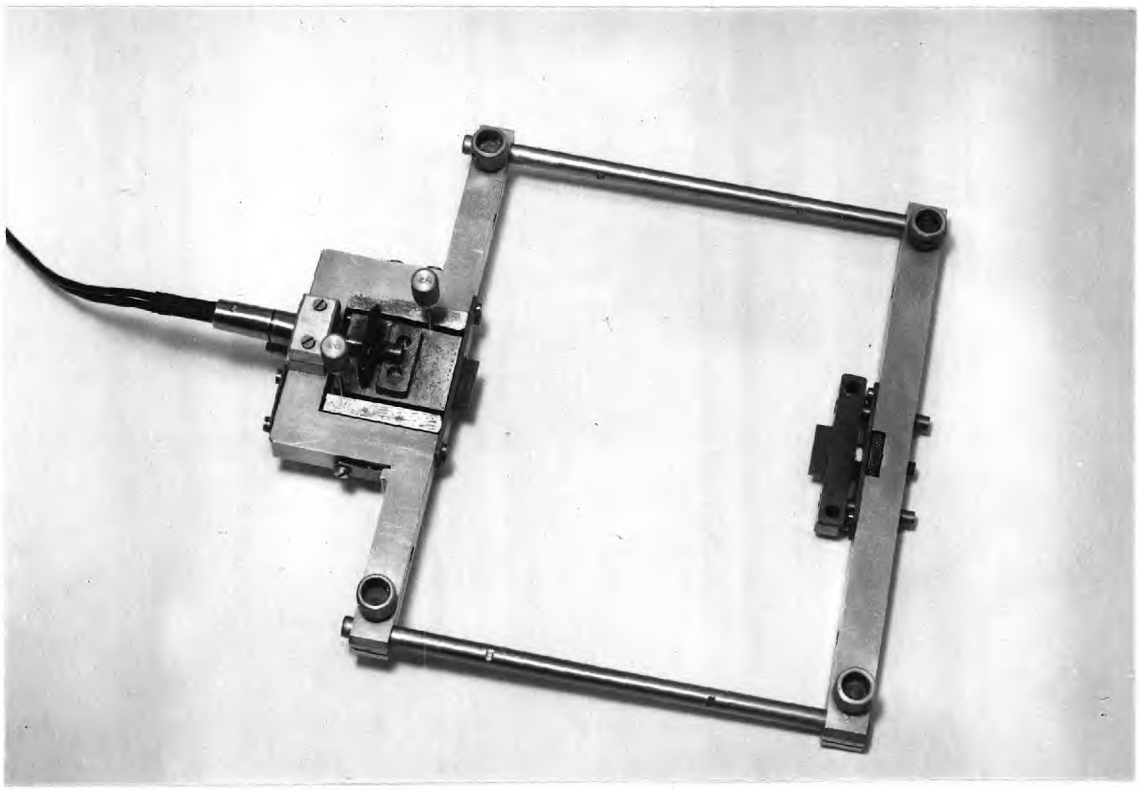


Fig. 2.5.1.2. support frame for Linear Inductance Transducer for measurements on 4" x 4" Cross-sections.

the results are quoted to the nearest microstrain. The positions of the measurements are shown in fig.2.5.1.1.

It was convenient to measure the lateral strain of the prism for a change in compressive load of -1930 p.s.i. as this co-incides with convenient load gauge readings and was only 29% of the ultimate load of the mortar thus guaranteeing elastic behaviour of the test specimen.

Typical results are plotted in fig.2.5.1.3 and a tabular summary of all the results is given in table 2.5.1.1. The full details of the tests have been published by Robinson and Vile[25].

2.5.2. Conclusions

The surprising result of thest tests is that Febcure super (manufactured by Feb. Ltd. of Manchester) applied to the damp concrete is the best packing material despite the fact that it is primarily intended as a curing compound. Although all the tests reported above were only up to 29% of the ultimate load, examination of failed specimens showed that the febcure had retained its surface on the mortar and indeed exhibited a polished surface. It will be shown later in this thesis with tests on biaxial compression specimens that it does in fact retain its friction releasing properties right up to the ultimate load.

A possible explanation of this behaviour is that the curing compound penetrates the concrete or mortar specimen when it is applied wet but subsequently hardens into a visco-elastic layer thick enough to prevent the steel actually coming into contact with the concrete. Indeed a surface tension skin effect seems to develop at the surface of the coating in contact with the steel and this becomes highly polished.

Further tests showed that for the maximum effect, the first coating of curing compound should be applied immediately on surface-drying the concrete with a towel. A second coat should be applied as

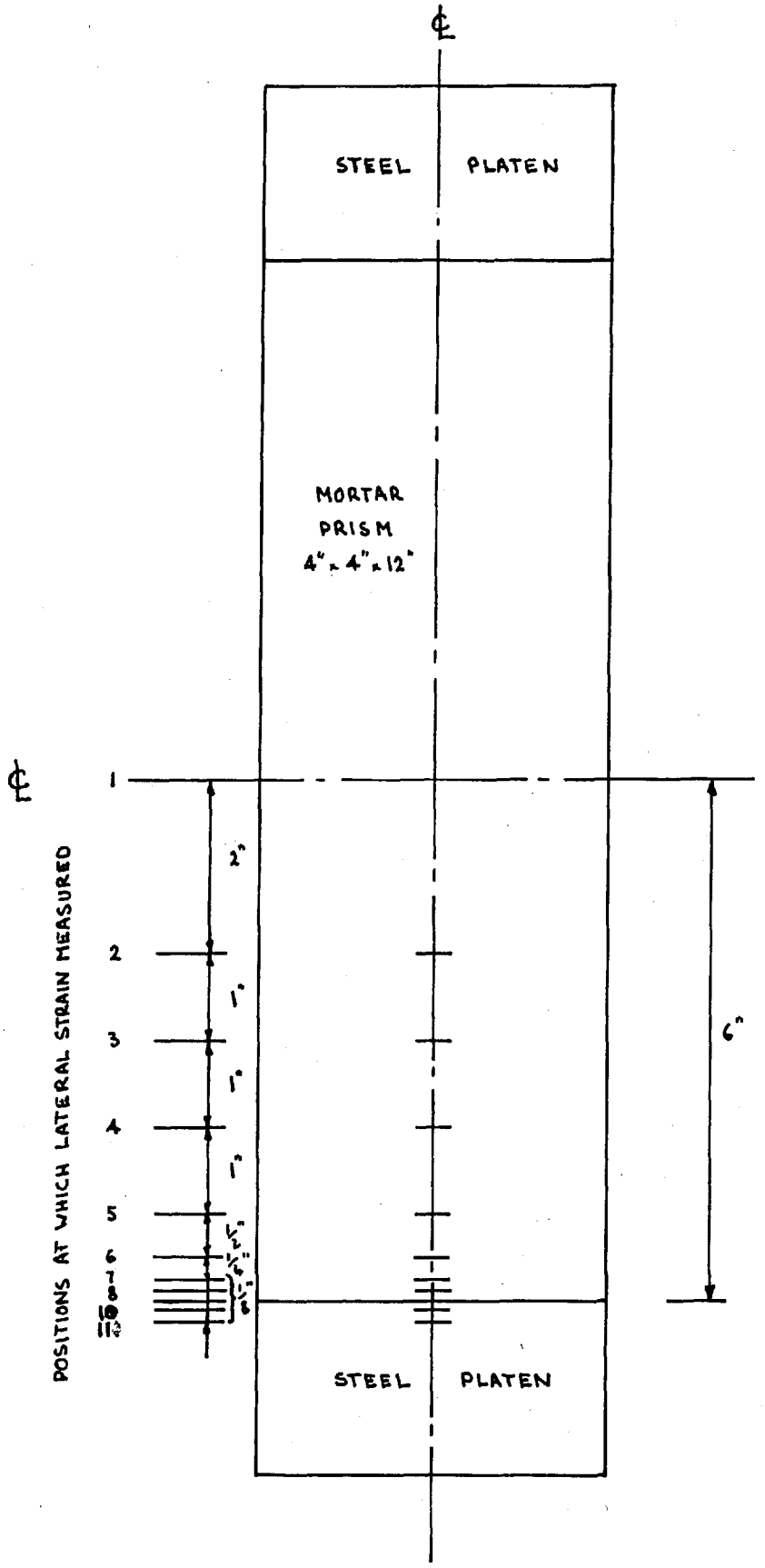


FIG. 2.5.1.1. POSITIONS OF LATERAL STRAIN MEASUREMENTS IN PLATEN RESTRAINT TRIALS.

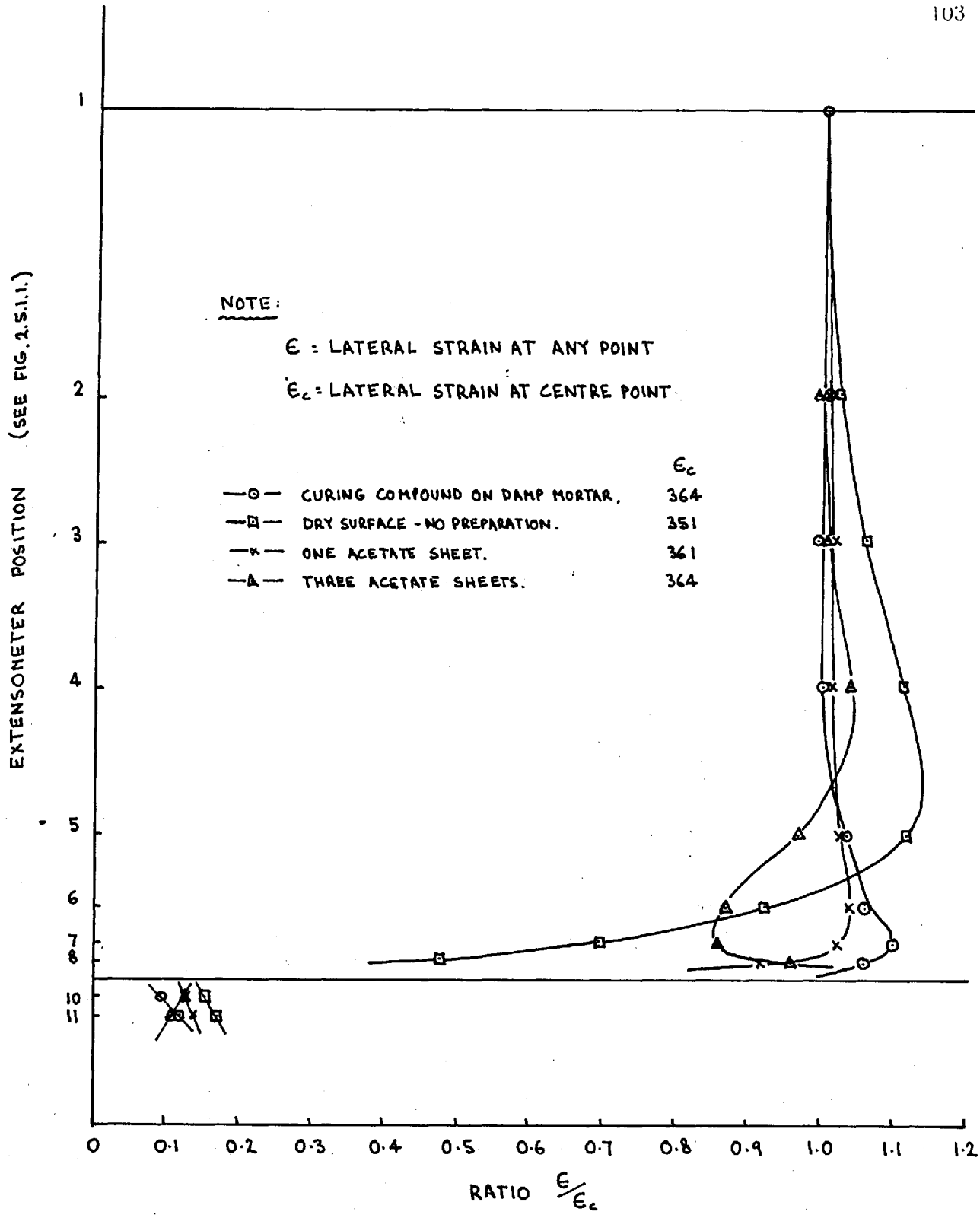


FIG. 2.5.1.3. DISTRIBUTION OF LATERAL STRAIN DUE TO PLATEN RESTRAINT.
(AFTER GSR (10))

soon as the first one has dried (giving a total thickness of about 0.0012") and the specimen tested about 24 hours later.

As the effect on the strain patterns recorded on the 4" x 4" x 12" prism was only 2% at 1.0 to 1.2" from the platen a 10" x 10" x 4" slab on a pro-rata basis would probably contain a fringe of interference equal to or greater than 2% for up to 1.9" from the loading platen face.

2.6. Selection of General Arrangement

On the basis of the conclusions of the platen restraint trials (section 2.5) it was decided to proceed with the design of the mechanical machine outlined in section 2.4 as it appeared that the platen restraint effects could be kept within acceptable bounds. The design proposed was in principle only and it is now necessary to consider the general arrangement in more detail.

2.6.1. Alternative Pre-stressed Concrete Frame

In section 2.4, the plate and column sub-frame was chosen as this provides the greatest longitudinal stiffness for the quantity of steel used whilst still having adequate lateral rigidity. However, one other possibility exists and that is the use of an all-embracing prestressed concrete frame containing four jacks as illustrated in fig.2.6.1.1.

This alternative suggestion, though cheaper than steel, was found to contain a number of disadvantages mainly from the ease of alignment and ease of modification point of view. Although all the facilities to be used on the machine were to be carefully pre-planned (Chapter 3), it was felt that not all the eventualities could be foreseen with sufficient accuracy to allow complete confidence in this design. Moisture movement could cause misalignment of the jacks and creep effects might also be troublesome. It was therefore shelved though use has been made by the Russians[133] of pre-stressed concrete in machine design.

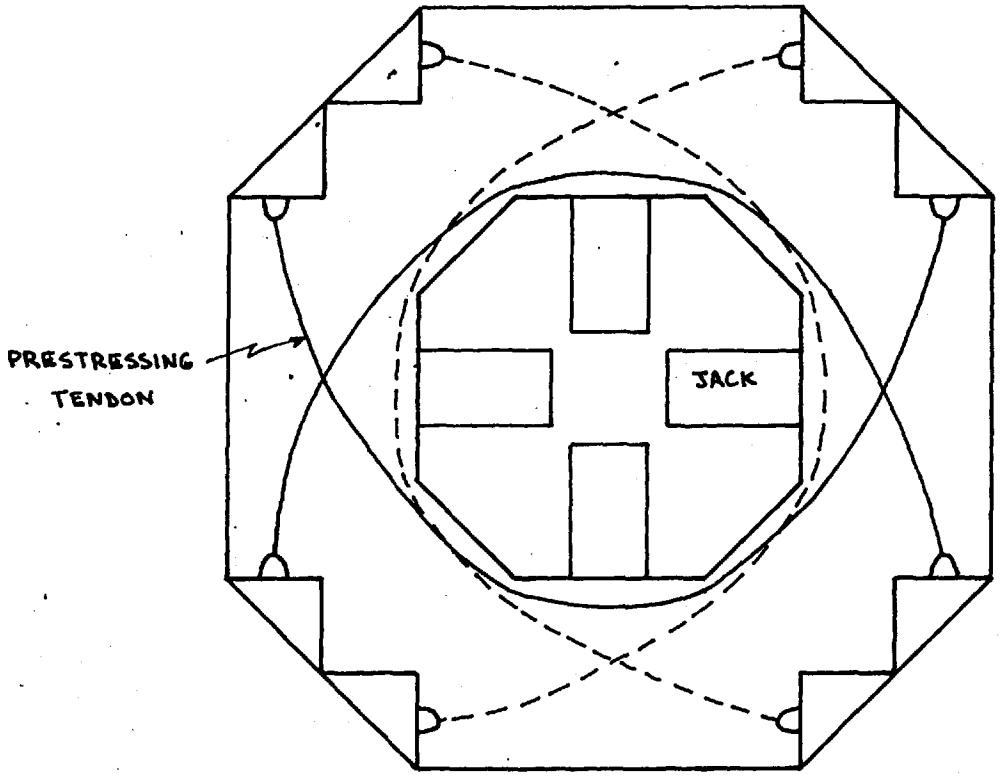


FIG. 2.6.1.1. LINE DIAGRAM OF PRE-STRESSED CONCRETE MACHINE FRAME.

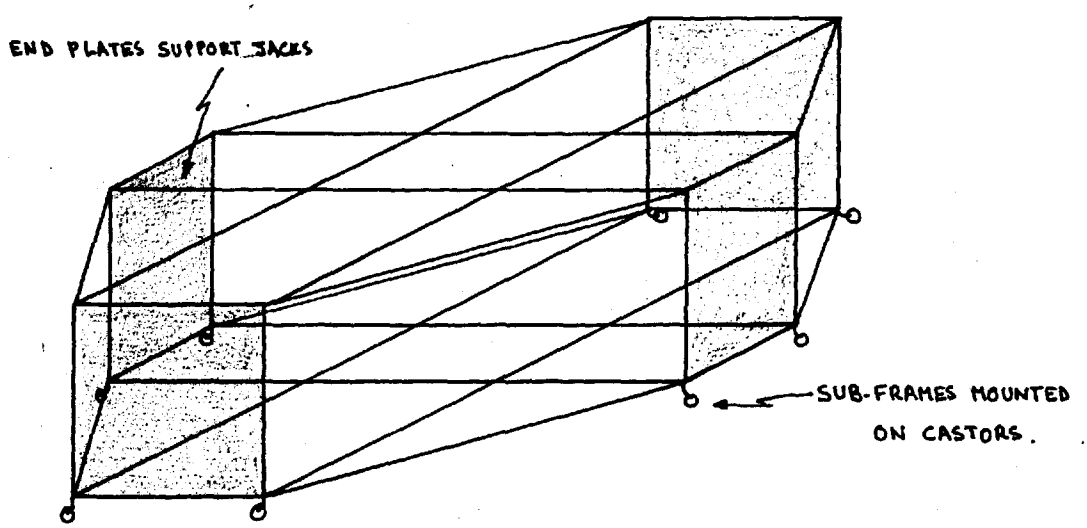


FIG. 2.6.2.1. HORIZONTAL ARRANGEMENT OF MACHINE

2.6.2. Horizontal Arrangement

A horizontal arrangement of the two sub-frames as shown in fig.2.6.2.1. is very good for automatic alignment on the test specimen. This would be simply a more elaborate form of the apparatus used by Chipman, Lynch and Peterson[129] (section 2.2.4.).

However, the arrangement is not readily adaptable to triaxial testing and would tend to occupy a lot of floor space. In addition only one side of the specimen would be visible and there would be the problem of gravity pulling the specimen out from between the platens.

2.6.3 Diagonal Arrangement

From an all-round framing aspect, the diagonal arrangement is good and the test specimen is easily visible from both sides (fig.2.6.3.1.).

Self-alignment on the test specimen would cause difficulties however and the deformation of the frame as a whole during testing would be unsatisfactory (fig.2.6.3.2.). Furthermore the fitting of tension jacks to either side of the bottom plates would be difficult.

2.6.4. Vertical Arrangement

The vertical arrangement shown in fig.2.6.4.1. has the advantage that the specimen is visible from both sides and readily aligned in the machine. However, alignment of the machine on the specimen during testing is a serious problem and requires a special frame to compensate for the deformation of the vertical sub-frame.

Stability of the machine frame as a whole must also be carefully considered particularly in the directions marked x and y in fig.2.6.4.2.

From the above considerations, the vertical general arrangement with steel end plate and column sub-frames and an alignment frame was adopted.

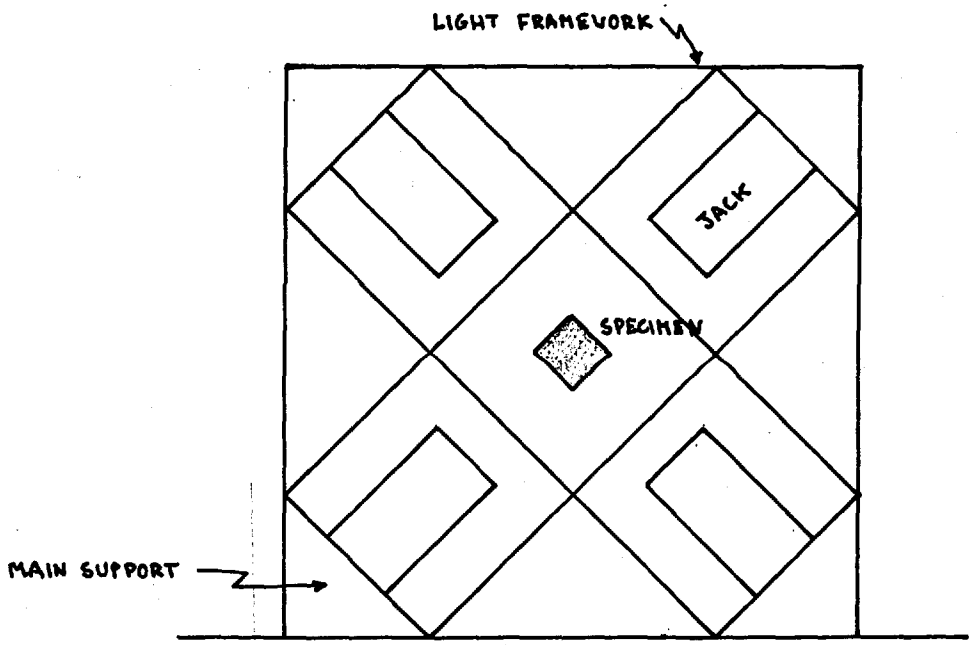


FIG. 2.6.3.1. DIAGONAL ARRANGEMENT OF MACHINE.

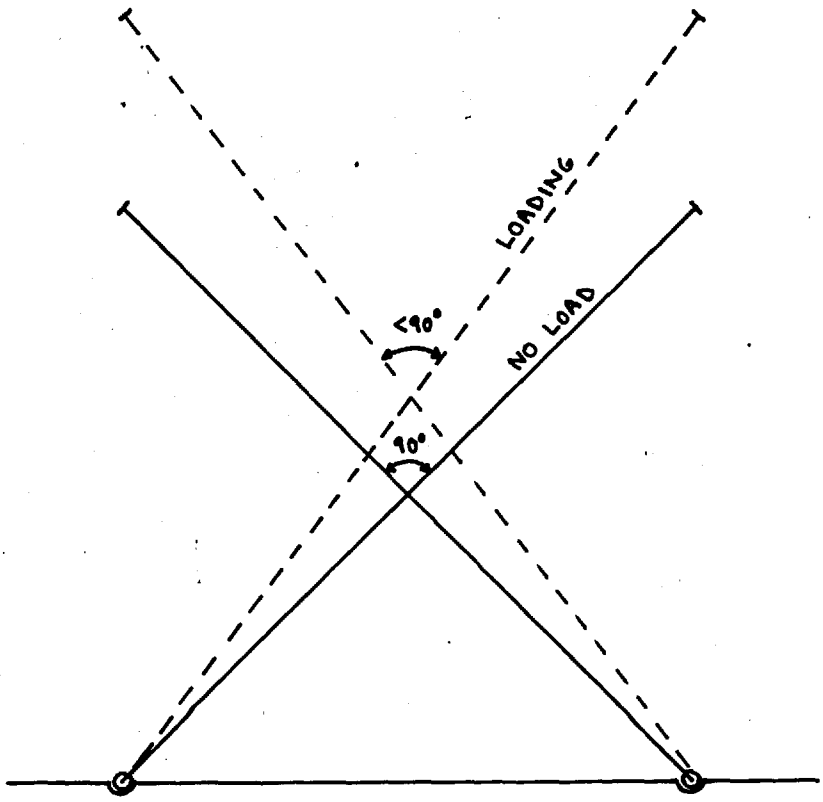


FIG. 2.6.3.2. DEFORMATION OF DIAGONAL MACHINE UNDER LOAD.

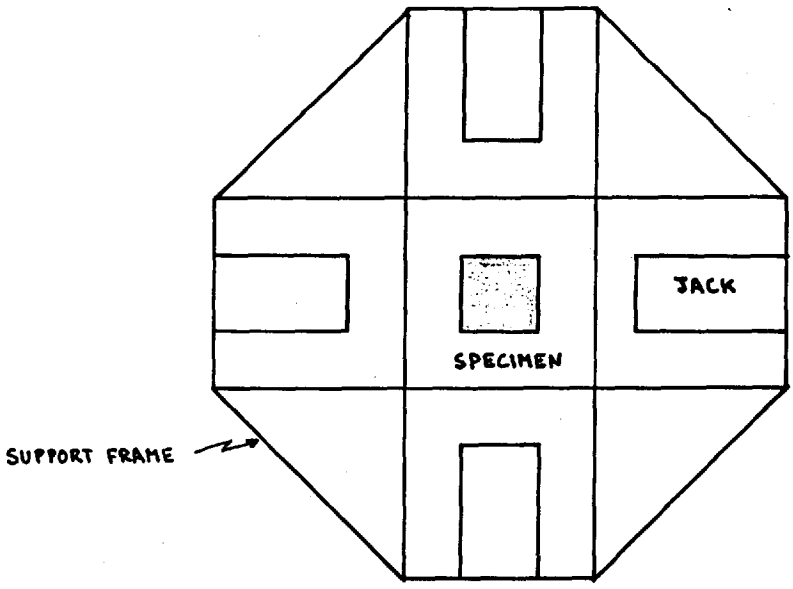


FIG. 2.6.4.1. VERTICAL ARRANGEMENT OF MACHINE.

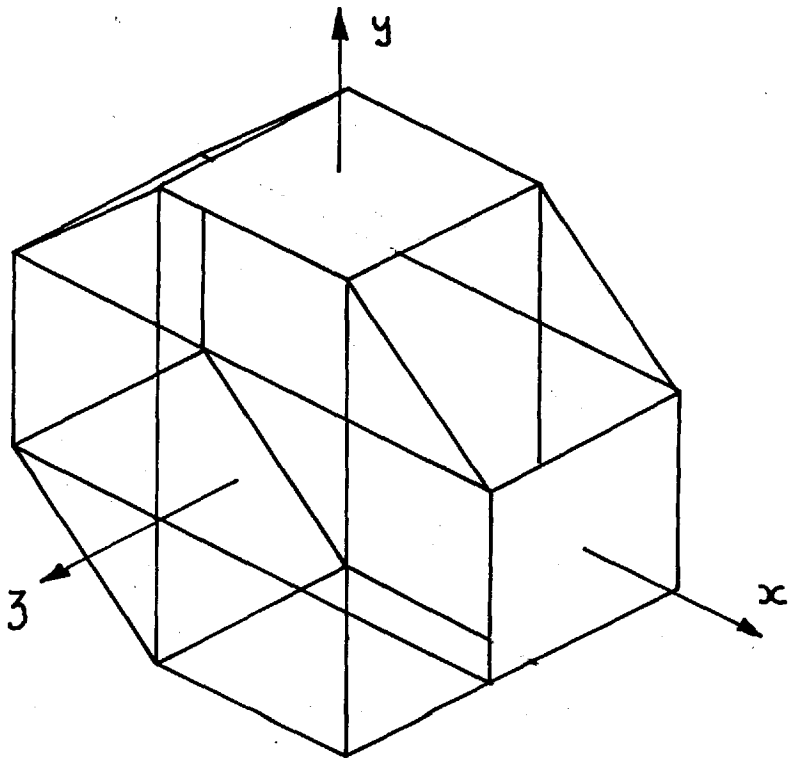


FIG. 2.6.4.2. STABILITY REQUIREMENTS IN DIRECTIONS x AND z.

2.7. Features of the General Arrangement

Certain features of the general arrangement required clarification before the detailed design study could be started and these are described below.

The main problem with the vertical arrangement chosen is, as stated in section 2.6.4, the automatic alignment of the machine on the specimen during testing. The basis of the problem is that the vertical sub-frame will deform relative to its base plate as the test specimen is loaded; for example, if the specimen is a compression test piece, then the vertical frame will extend upwards. The problem then is how to keep the horizontal machine aligned on the centre of the test specimen.

The solution adopted was to use two symetrical portals to support the horizontal frame as shown in fig.2.7.1. The action of these portals would be to raise the horizontal frame half the distance moved by the top plate of the vertical sub-frame. As this, by the symetry of the arrangement, is also the distance moved upwards by the specimen, the alignment of the horizontal frame on the specimen is maintained (fig.2.7.2.).

As a result of the action of these side portals, it is necessary to mount the horizontal frame on rollers to allow the portal free movement and to prevent a reaction transfer between the horizontal frame and the portal (fig.2.7.3.). During testing the horizontal frame does not move as it is acting on the test specimen but stops are provided on the side portals in case of emergency.

To reduce frame distortion, the jacks in the horizontal frame are clamped to the end plates and cantilever from them as shown in fig.2.7.1.

The columns are threaded between each other as shown in fig.2.7.5. with a preference for a symetrical arrangement in the horizontal sub-frame to accommodate torsional equipment. Indeed

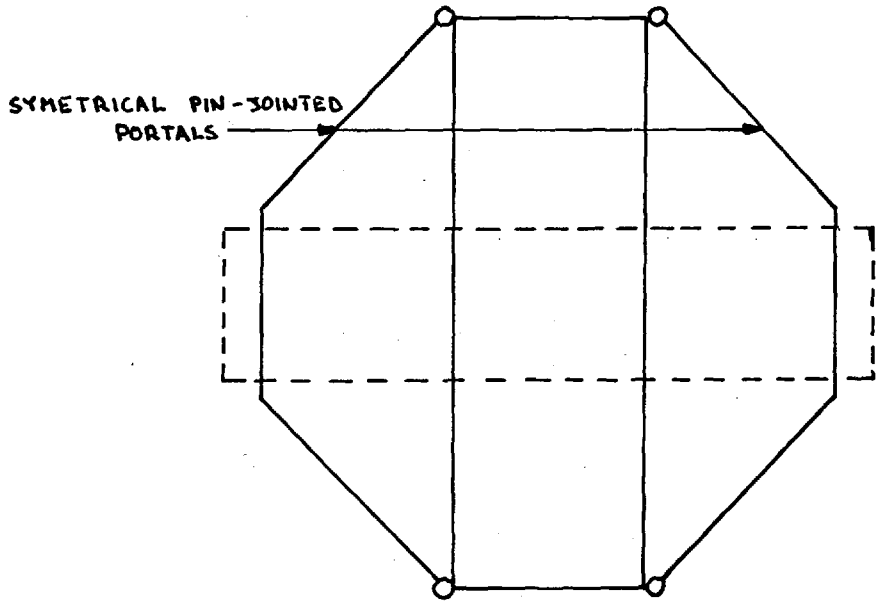


FIG. 2.7.1. PORTAL FRAMES TO SUPPORT HORIZONTAL SUB-FRAME.

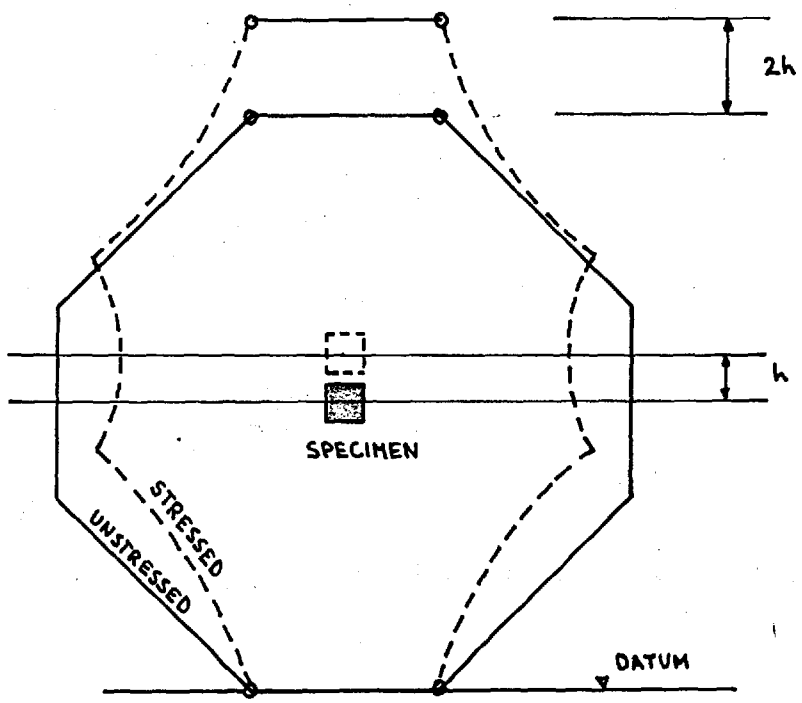


FIG. 2.7.2. SELF-ALIGNMENT MECHANISM OF MACHINE FRAME.

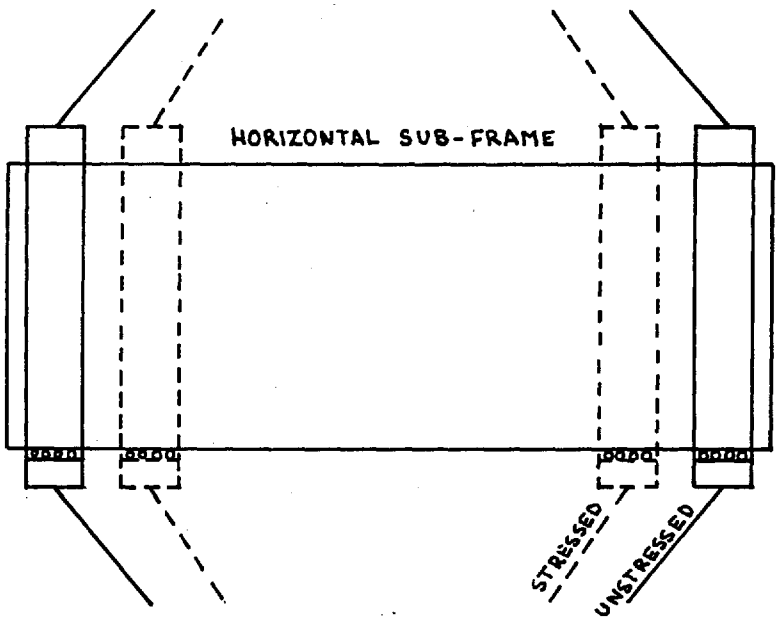


FIG. 2.7.3. FREEDOM OF SIDE PORTAL MOVEMENT.

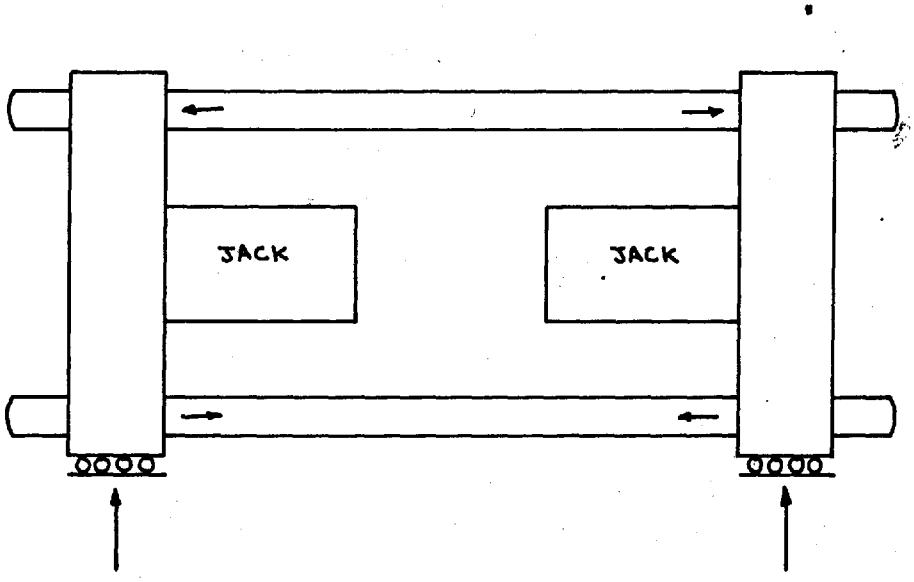


FIG. 2.7.4. CANTILEVER OF JACKS IN HORIZONTAL SUB-FRAME.

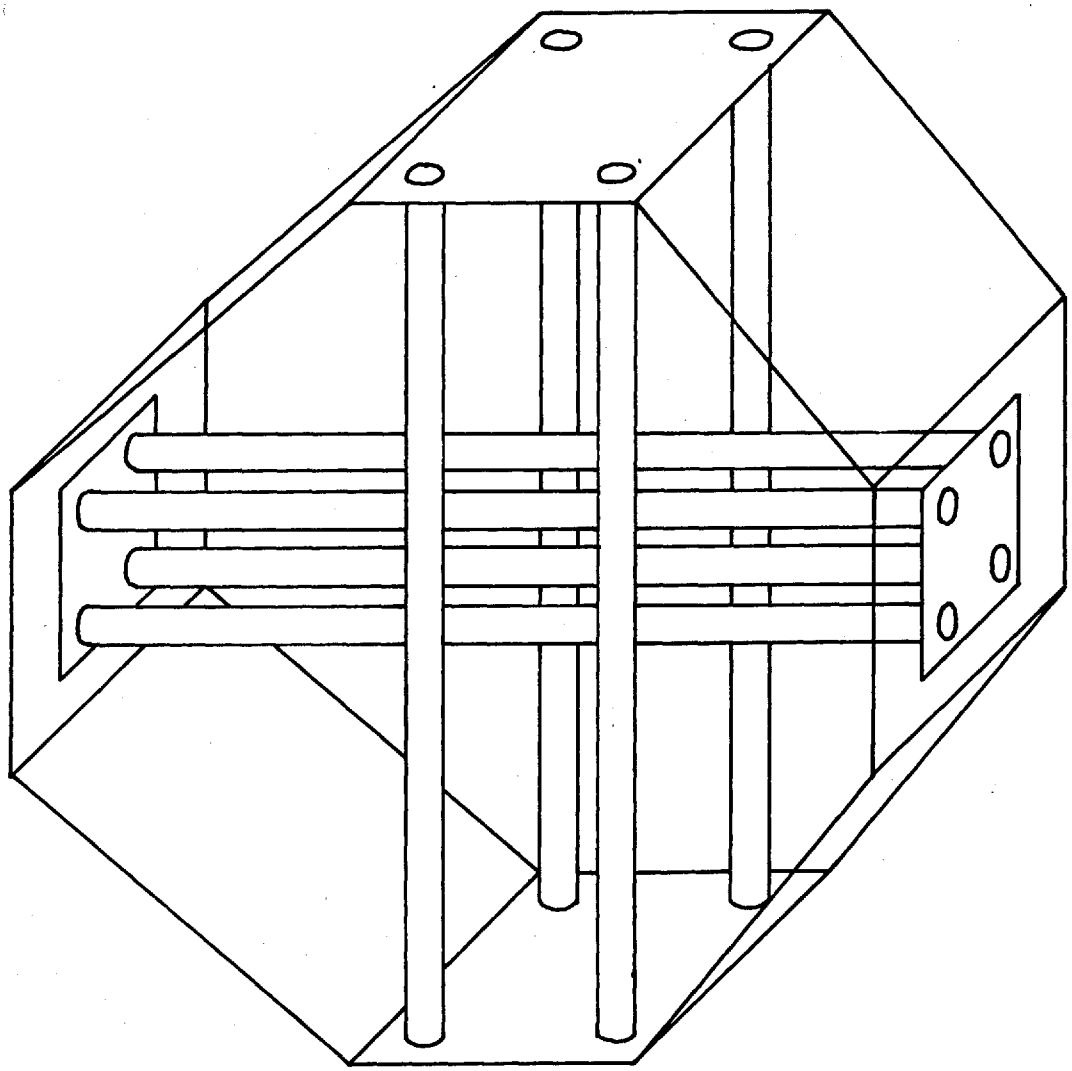


FIG. 2.7.5. ARRANGEMENT OF SUB-FRAME COLUMNS.

the horizontal frame is intended to accommodate tension and torsional facilities as well as compression. A line diagram of the several possible stress combinations available in the machine is given in fig.2.7.6. and a line diagram of the future extension to triaxial testing is shown in fig.2.7.7.

As previously noted, the horizontal sub-frame in the vertical arrangement is best suited to housing the tension jacks and hence the tension facility. This being so, the vertical sub-frame must provide the torsion facility so that the tension-torsion combination can be provided.

The torsion facility is achieved by using two beams set at opposite angles across the cylindrical specimen as shown in fig. 2.7.6. As these two beams are jacked towards each other, the resulting reactions at their ends on the cylinder apply twisting moments of opposite direction at each end thereby subjecting the cylinder to torsion.

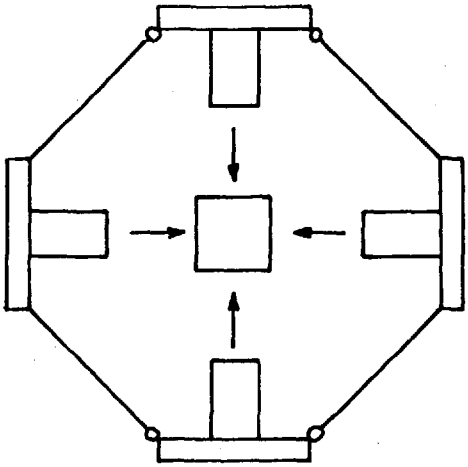
End compression or tension is applied to the torsional specimen via ball races.

The compression is applied to the tension specimen by concrete cubes as shown.

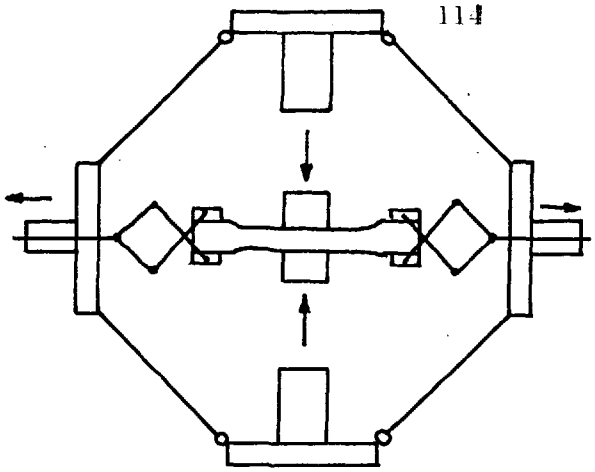
The facilities actually developed, viz. biaxial compression, compression-tension, and compression-torsion are described in detail in Chapters 8, 9 and 10 respectively.

Levelling and alignment screws were provided in the side portals for setting up the horizontal frame during construction (fig.2.7.8.). These ensured that the horizontal frame moved squarely through the side portals.

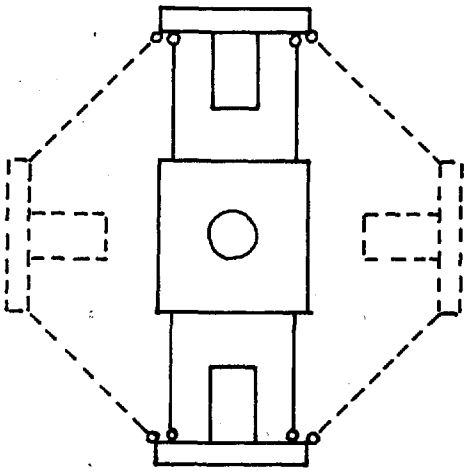
Solid loading platens were selected in preference to such arrangements as the segments used by C.S.I.R.[PC] and the brushes recommended by Hilsdorf[57] as the platen restraint trials had showed that the restraint effects could be restricted without causing the



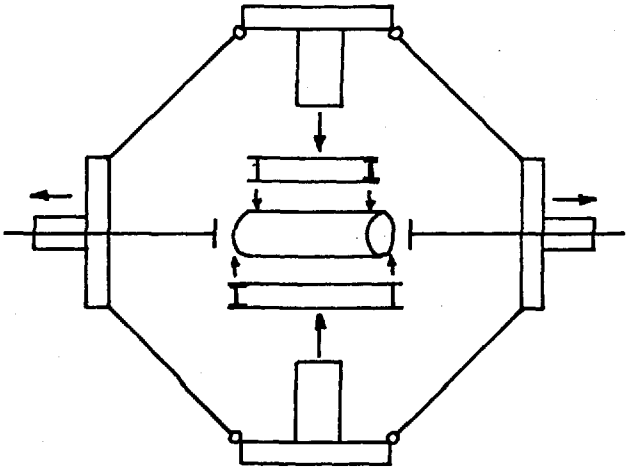
BIAXIAL COMPRESSION



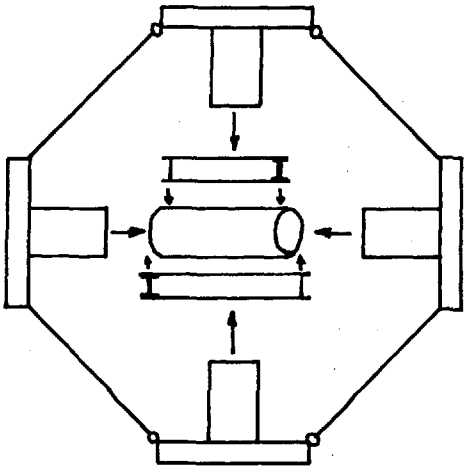
BIAXIAL COMPRESSION - TENSION



TRIAXIAL COMPRESSION



BIAXIAL TENSION-TORSION



BIAXIAL COMPRESSION - TORSION

FIG. 2.7.6. SOME POSSIBLE STRESS COMBINATIONS.

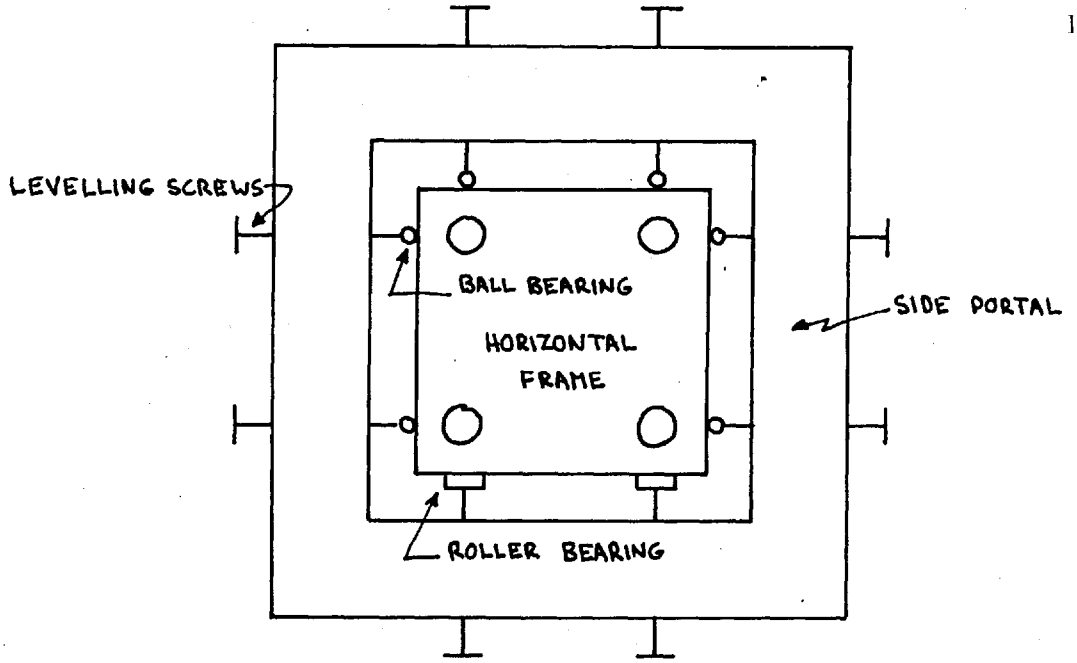


FIG. 2.7.8. ADJUSTMENT FOR HORIZONTAL FRAME IN SIDE PORTALS.

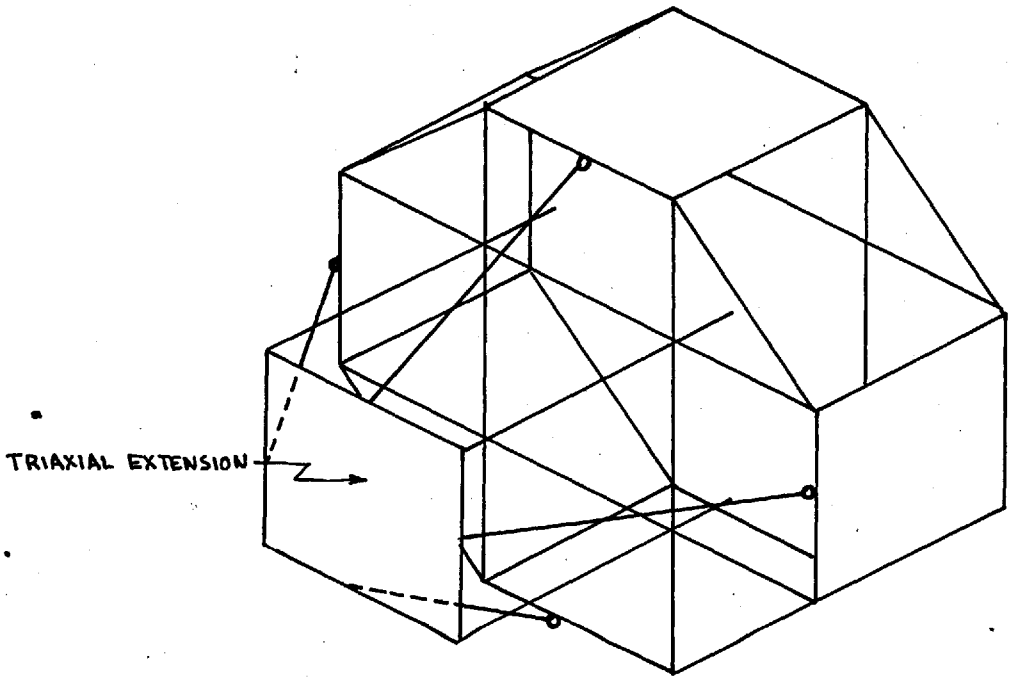


FIG. 2.7.7. EXTENSION OF MACHINE FRAME TO TRIAXIAL TESTING.

secondary effects that it has been shown develop with the more elaborate platens described above. This subject is discussed in more detail in section 4.4.6 in which Hilsdorf's specific objections to the solid platen are dealt with. However, the design of the machine is such that platens of this kind could be used if desired at a later date.

2.8 Summary

The previous machine designs reviewed illustrated a wide range of testing techniques but machines of very limited stress combination facilities.

A consideration of the requirements of a combined stress testing machine to suit this project and the types available led to the problem of platen restraint. A series of tests on this problem showed that it was a manageable difficulty and as a result, a versatile mechanical testing machine was chosen.

The general arrangement of this machine was examined in some detail to ensure that the best solution resulted prior to the detailed design now to be described in Chapter 3.

CHAPTER 3

DESIGN AND CONSTRUCTION OF THE COMBINED STRESS TESTING MACHINE

3.1. Introduction

Having selected the type and general arrangement of the testing machine to be developed, it is now necessary to consider the detailed design requirements in the light of the performance required and the construction problems involved. The problems and limitations of machining the various sections of the machine must also be taken into account.

In this chapter the performance requirements from the research point of view are considered first (section 3.2.) and then the design considerations are listed (section 3.3.). The basis of the detailed design of the machine is then stated (section 3.4.) and a record made of the manufacturer's modifications to the original design (section 3.5.).

The design of the hydraulic control system and the control panel are treated separately (section 3.6.) and finally the construction and alignment of the machine is described (section 3.7.).

3.2. Performance Requirements

The performance requirements were dictated by the needs of the research project and may be summarised as follows:-

3.2.1. Ease of Operation

This requirement was not simply for the convenience of the operator but was a genuine necessity in view of the large quantity of measurements that might have to be taken during a short-term list. The ease with which the machine could be operated will be reflected in the results obtained and is therefore a factor worth careful consideration.

Ease of operation, besides giving the operator more time to fulfil his various functions also has a beneficial psychological effect on all engaged in the testing.

3.2.2. Ease of Facility Exchange

If the versatility of the machine is to be fully exploited, then it must be possible to exchange one testing facility for another without requiring a major reconstruction of the machine.

For this reason, it was convenient that the horizontal sub-frame was selected to house the alternative tension and torsional facilities, as, by careful arrangement of dimensions, fork lift trolleys which are standard laboratory equipment could be used to lift jacks and other ancillary apparatus in and out of position.

This point was also borne in mind in the design of the jack supports and hydraulic pressure lines and fittings.

3.2.3. Independent Control on Jacks

As stated earlier, the proposed method of test involved using pairs of opposed jacks linked hydraulically to maintain a symmetrical loading arrangement centered on the specimen to ensure machine alignment.

However, for the purposes of initial alignment of the specimen in the machine, the hydraulic system must be capable of giving forward or reverse motion to any jack independent of the others.

3.2.4. Machine Capacity

Biaxial compression tests on concrete with up to 7,500 p.s.i. uniaxial compaession strength was the criterion used in determining the machine capacity in addition to the face that a representative volume

of a $\frac{3}{4}$" aggregate concrete should be in a uniform and known state of stress.

At this stage of machine development, Robinson had already reduced the size of his biaxial compression specimen from 12" x 12" x 4" to 10" x 10" x 4" and, although he had not conducted any tests, the results of the platen restraint trials (see section 2.5.1.) were encouraging. These indicated that, within the elastic range, the central 6" x 6" x 4" volume of a 10" x 10" x 4" slab would be in a uniform state of stress.

Hence, on the basis of a 10" x 10" x 4" slab biaxial compression specimen and assuming a 50% increase on the uniaxial strength under biaxial stress in the extreme case of concrete[146], a jack with a capacity of 200 ton. was adopted. The Simplex jacks adopted (see section 3.4.1.) were graded in U.S. tons (i.e., 1 ton = 2,000 lb.) and hence the 200 ton jacks installed exerted a pressure of 10,000 p.s.i. on the 10" x 10" x 4" specimens at working oil pressures (7,500 p.s.i.) and up to a maximum of 13,000 p.s.i. on overload. For tension tests a 30 ton jack was considered adequate for any specimen size up to 12" x 12" x 4" likely to be tested in tension.

3.2.5. Machine Stiffness

Barnard[134-136], Sigvaldason[9] and others[137,138] pointed out the need for a stiff testing machine to avoid explosive failures and to investigate the stress-strain curve beyond the ultimate load.

The problem of attaining adequate stiffness can often be considered in two parts, i.e. stiffness of machine frame and stiffness of hydraulic system: the sum of the two stiffnesses being the overall stiffness of the machine.

The machine designed by Barnard[135] overcame the problems of stiffness and controlled strain rates (to within 7%) but was of only 25 ton capacity and required a piston of 11" diameter to maintain the controlled strain rate facility. For the 200 ton capacity machine envisaged for this research, repetition of Barnard's design but on a pro-rata scale required four pistons of 33" diameter which rendered the design unwieldy and expensive.

These arguments and the need for flexibility in the uses of the machine and in its design generally led to the selection of a conventional hydraulic system of standard jacks and controls. However, the machine frame was designed as a stiff frame which was desirable in itself, regardless of the overall stiffness, as explained by Sigvaldason[9].

As mentioned later in this thesis, the problem of hydraulic stiffness can be solved (see section 4.7.).

3.3. Design Considerations

It was noted in the Introduction to Chapter 1 of this thesis that work had been done at Imperial College by Sigvaldason[9] on testing machine design and later, that it was hoped to capitalise on this work in the design of the combined stress testing machine.

This section considers the principal results of this research and related them to the actual machine under consideration.

3.3.1. Longitudinal Stiffness

Although the longitudinal stiffness of the testing machine has no apparent effect on the ultimate strength results obtained (12, 139), it is desirable to have a "hard" machine to prevent explosive failures, which increase wear on the machine components, and to study the post-ultimate stress-strain curves as stated in section 3.2.5.

Hydraulic stiffness remains a problem but the criteria for the overall machine stiffness is that it should exceed that of the specimen under test.

In this design an elastic modulus of 6×10^6 p.s.i. was assumed for concrete and the machine frame was designed to have a longitudinal stiffness greater than this.

3.3.2. Lateral Stiffness

This was not a great problem because, as stated below, all the specimens were tested with their loaded faces effectively locked. Hence instability problems did not arise.

However, an effort was made in the design detailing to make the subframes as rigid as possible and Sigvaldason's recommended minimum of 10×10^4 lb. in.[12] was exceeded.

3.3.3. Spherical Seatings

The design of the spherical seatings was important because these determine the loading conditions on the specimen, i.e. platens effectively fixed or pinned.

Newman and Sigvaldason[15] have stated that: "A loading method where both ends are pinned is the most suitable for research investigations into the intrinsic weakness of a material for it is under this system only that the failure mode will exhibit the true characteristic weakness of the material".

However, this ideal must be judged against the problems of achieving it in practice and the desirability of its successful achievement in view of the short-comings of the material under test and the application of the results obtained.

The problems of achieving this loading state in practice are particularly difficult with concrete, as slight segregation of the aggregate would result in a complex state of stress in the specimen

with shears and bending moments at the ends of the specimen/super-imposed on the normal loading system. Premature failure would occur in critical sections of the test specimen and the results obtained would be erroneous. Slight misalignment of the test specimen of even the most homogeneous of materials would produce the same effect and in both cases the testing machine operators would be in danger of test specimens being ejected from the machine due to the eccentricity of loading.

Spherical seating rotation under load is possible under pressures in excess of 24,000 p.s.i. on the seating surfaces, as has been shown by Sigvaldason[9], given suitable lubrication.

From the above considerations, however desirable the condition of effectively pinned ends may be, it is clear that this is not the best condition to adopt at first when a new testing technique is being established.

There remains the question of the desirability of achieving this loading condition in view of material short-comings and the application of the test results obtained.

As noted above, this loading condition is quite unacceptable for materials of varying stiffness due to segregation and other causes. Hence it must be ruled out for most concrete mixes on this count alone and therefore for all mortar and concrete testing if a standardised testing technique is to be established.

This conclusion is supported by a consideration of the application of the test results obtained to actual structures. In the majority of structural applications of concrete, the material is subjected to conditions of plane strain and the critical section of the structural member is restrained by the surrounding concrete. With effectively locked loading platens, the same condition of strain is achieved in that the platens apply plane strain and the central uniformly strained zone, which is the critical failure zone, is free to move being restraihed only by the surrounding concrete.

As a result of these consideration, partially lapped spherical seatings were specified and used in the testing machine unlubricated except for a light mineral oil to prevent corrosion. However, should a comparison between fixed and pinned loading conditions be required, it is recommended that to achieve pinned ends the spherical seatings be coated with a 1:2 black lead and tallow lubricant[9].

3.3.4. Construction Material

Prestressed concrete has already been rejected as a suitable material for the machine frame (section 2.6.1.) and the use of aluminium alloys was not justified as weight was not a problem.

Mild steel was chosen for all frame components because of its high elastic modulus, low cost, and ease with which it can be worked. The loading platens were also of mild steel but surface hardened.

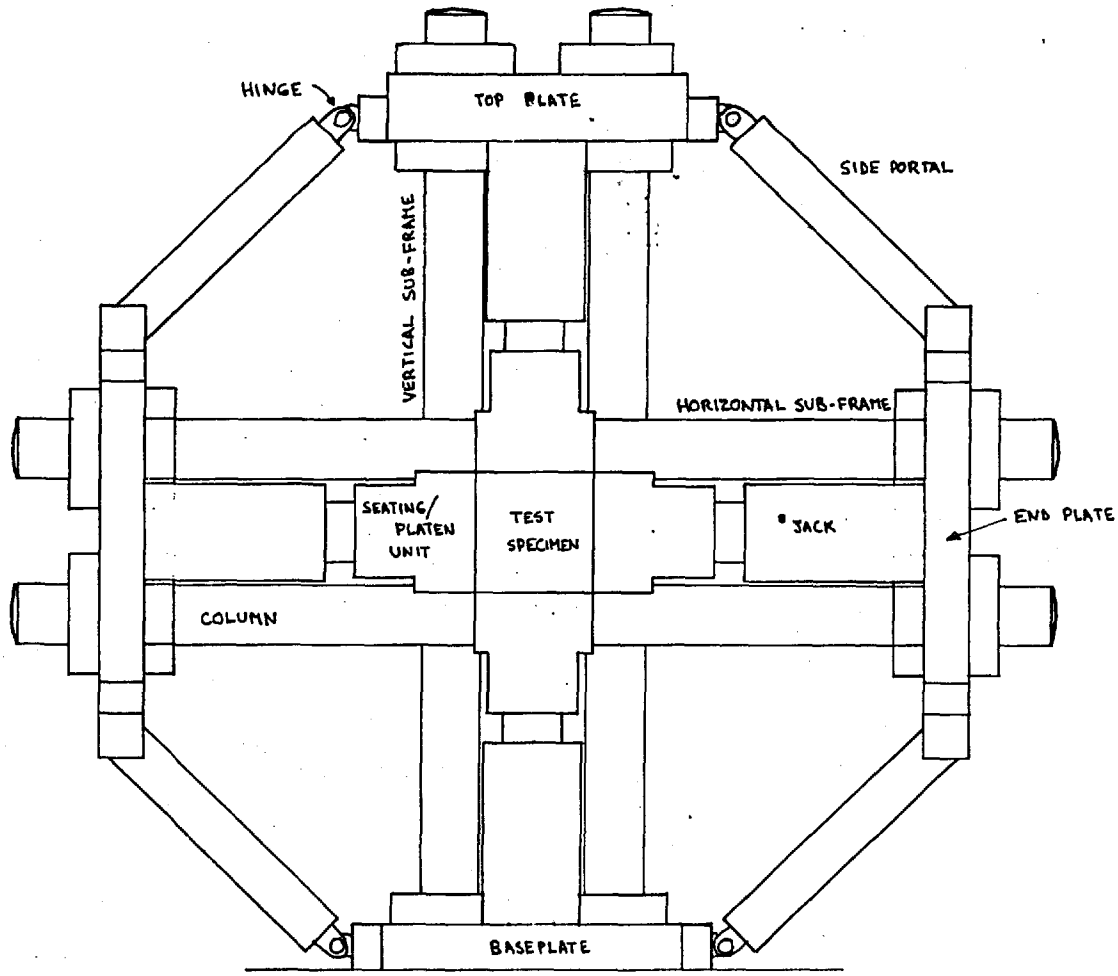
3.4. Detailed Design of Machine

The design calculations for the machine components is included in the appendix and a general arrangement drawing of the machine is given in fig.3.4.1. Detail drawings of the various components are given in fig.3.4.2.-3.4.5. and a photograph of the completed machine in 3.4.6.

This section explains the thought behind the design calculations and the actual interpretation of the considerations and requirements of the previous two sections (3.2. and 3.3.).

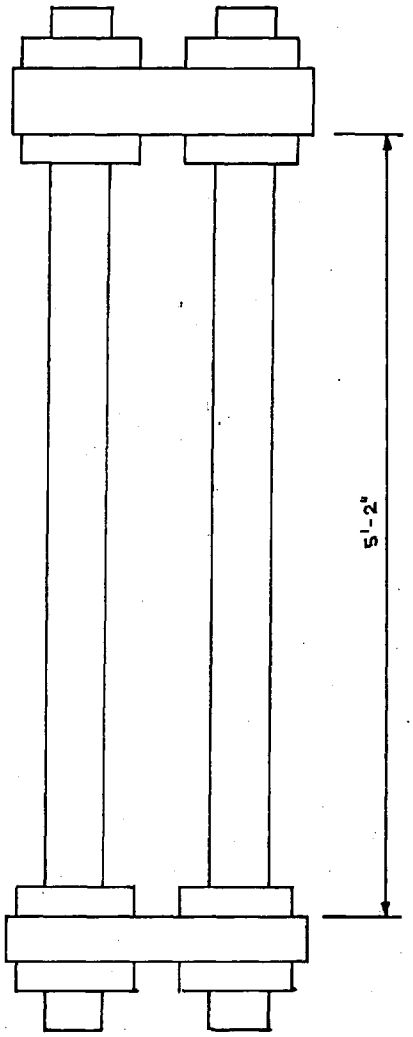
3.4.1. Selection of Jacks

The selection of suitable hydraulic jacks was the first consideration as their size would automatically determine all the

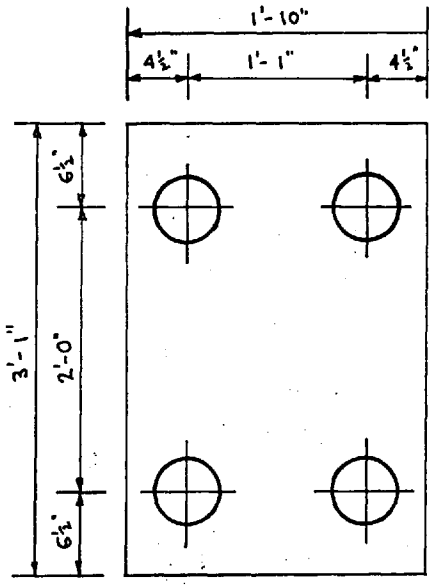


DIMENSIONS GIVEN ON DRAWING AT BACK OF THESIS.

FIG. 3.4.1. GENERAL ARRANGEMENT OF MACHINE SET-UP FOR BIAXIAL COMPRESSION.

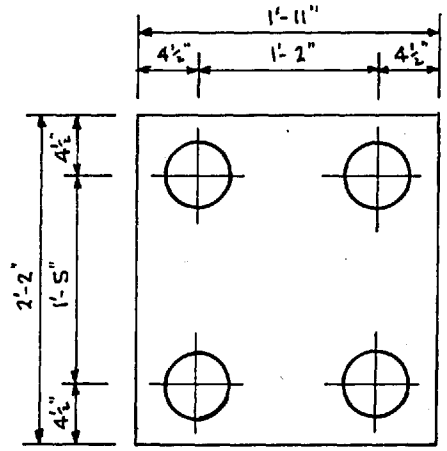


ELEVATION

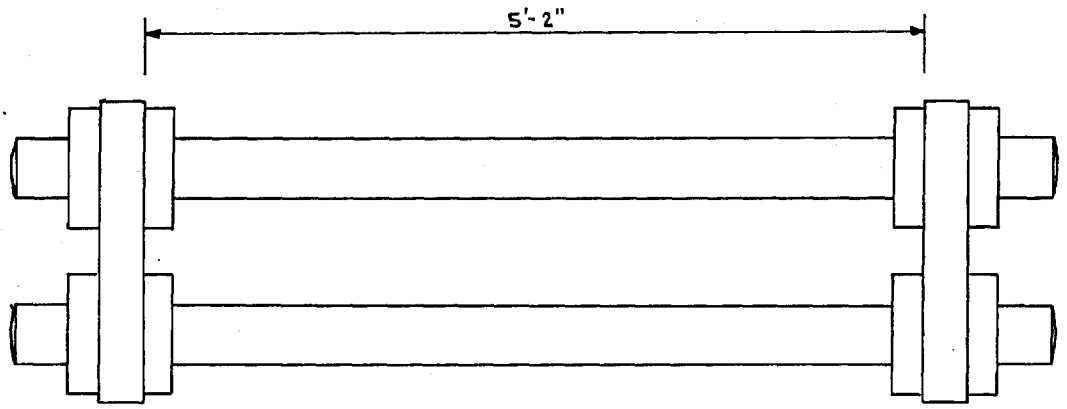


PLAN

FIG. 3.4.2. GENERAL ARRANGEMENT OF VERTICAL SUB-FRAME.



END PLATE ELEVATION



SIDE ELEVATION

FIG. 3.4.3. GENERAL ARRANGEMENT OF HORIZONTAL SUB-FRAME.

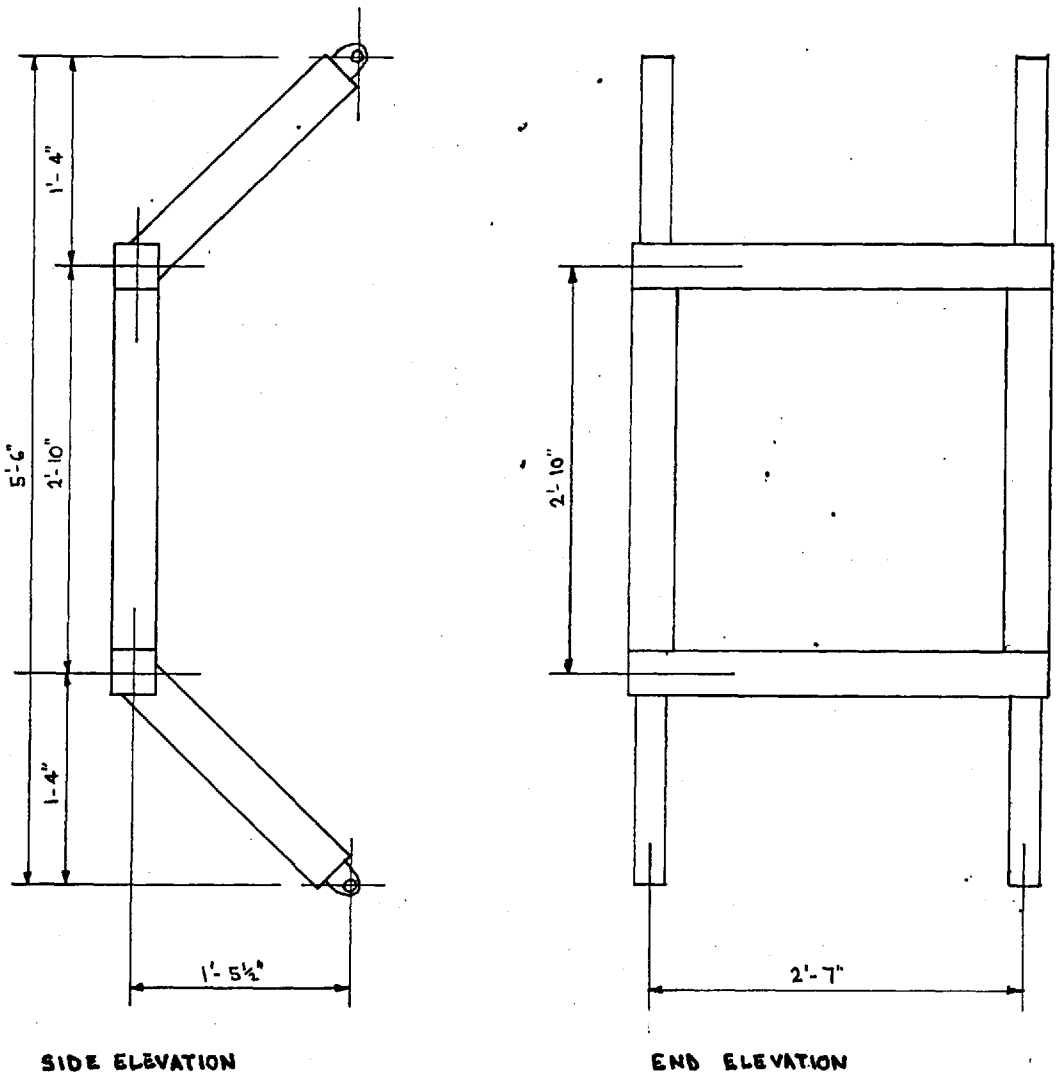


FIG. 3.4.4. GENERAL ARRANGEMENT OF SIDE PORTALS.

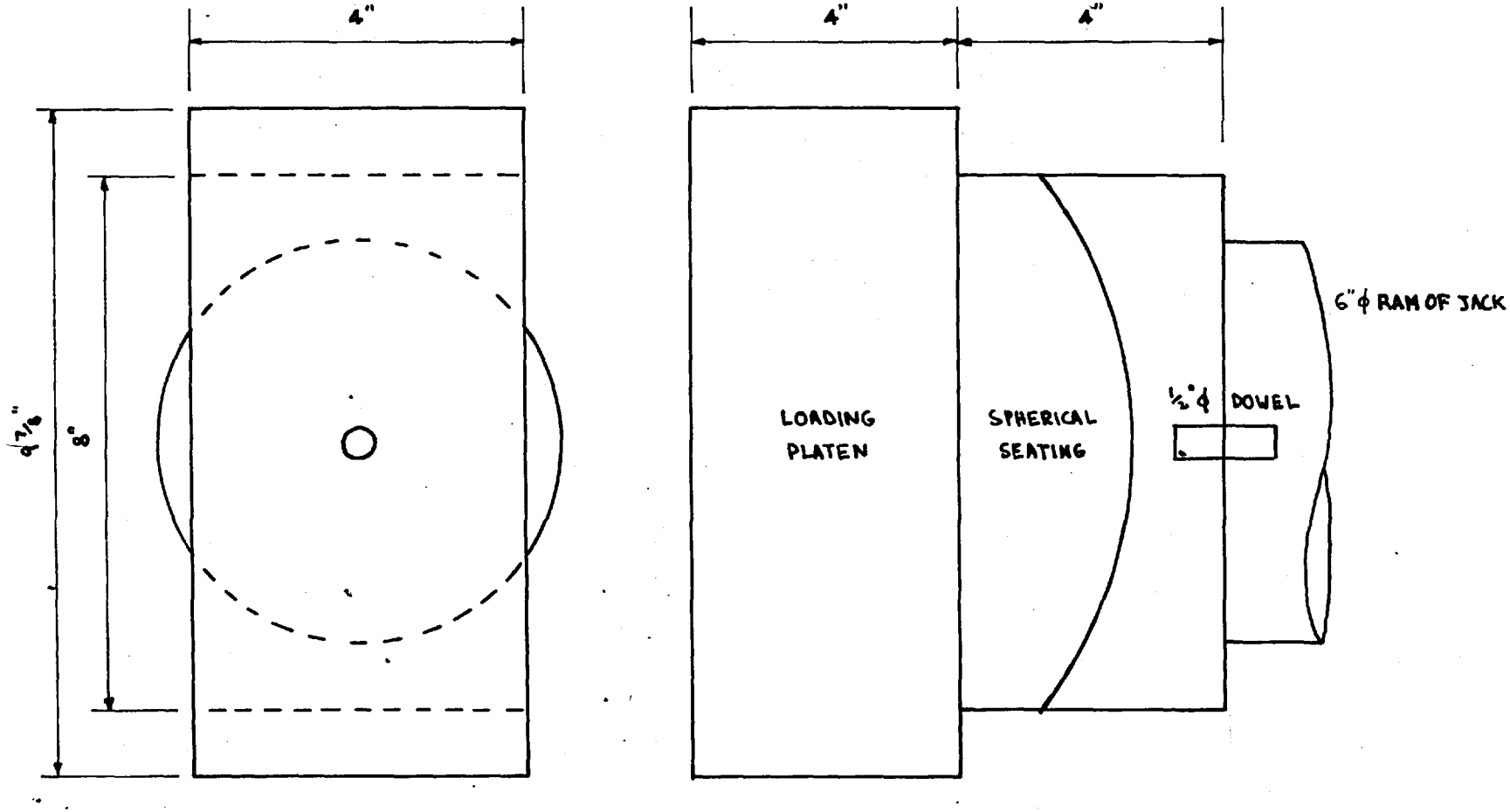


FIG. 3.4.5. GENERAL ARRANGEMENT OF LOADING PLATEN /SPHERICAL SEATING UNIT

(2)

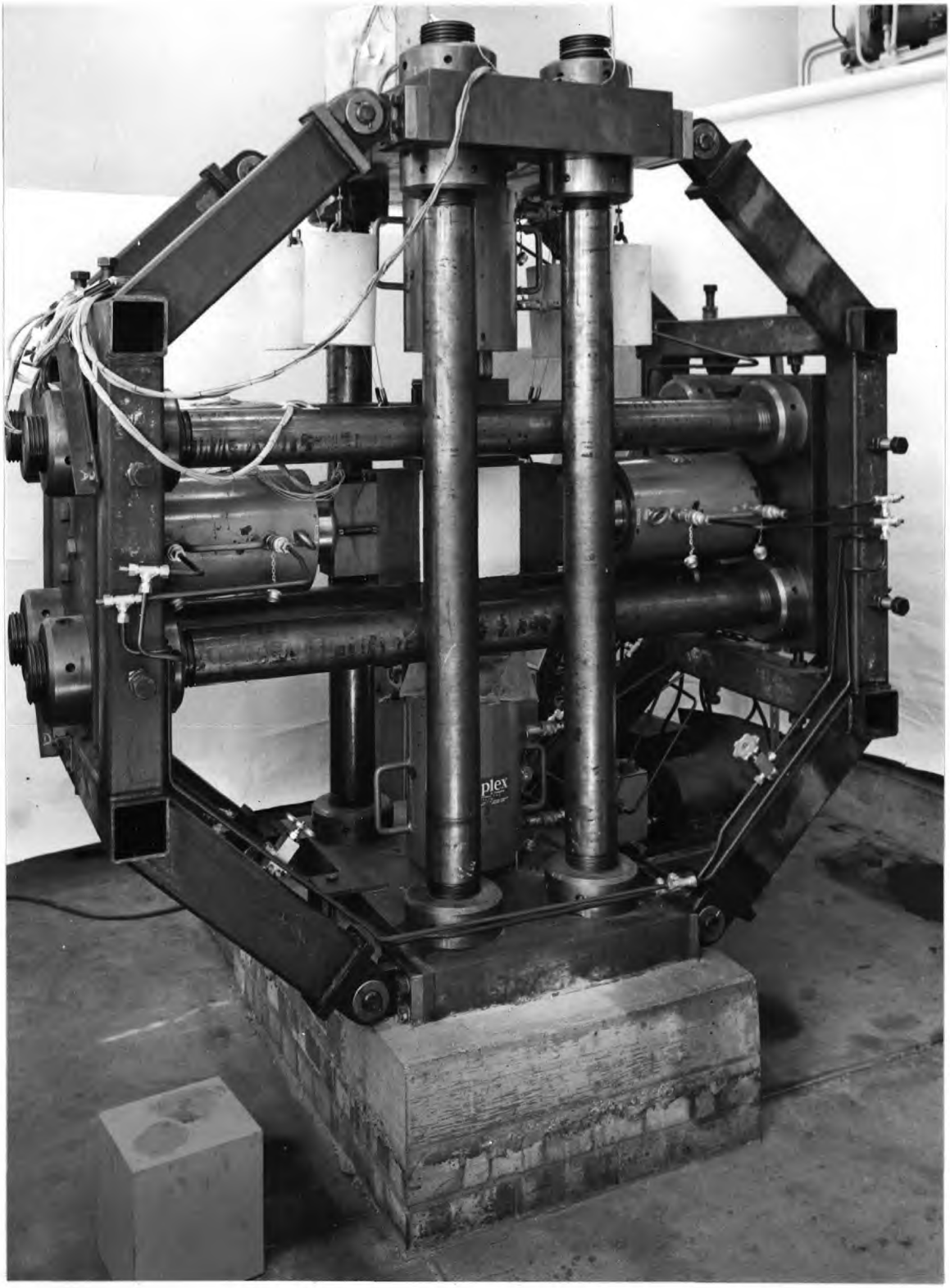


Fig. 3.4.6. Combined Stress Concrete Testing Machine set up for biaxial compression tests.

leading dimensions of the machine frame. A major concern was the operation of jacks on their side as would be necessary in the horizontal sub-frame and the effect that this would have on their friction and ram alignment characteristics.

After an extensive investigation of several designs of commercially available jacks, the design by Simplex was chosen as the most suitable in view of its close manufacturing tolerances and double seals at each end of the ram (fig.3.4.7.). In addition PTFE seals were available at an additional cost and these were specified. The Portland Cement Association Research Laboratories report[141] that these seals gave greatly improved friction characteristics compared with the conventional rubber seals.

Two minor disadvantages had to be tolerated and these were the small ram diameter (an advantage commercially but a disadvantage in research) and the high operating pressures. (7,500 p.s.i. for 200 ton compression jack and 9,200 p.s.i. for 30 ton tension jack).

However these two disadvantages were successfully overcome (sections 3.4.7. and 3.6.) and the subsequent calibrations of the horizontal and vertical sub-frames justified the selection of these jacks (section 4.3.).

3.4.2. Longitudinal Stiffness

The selection of the jacks and the eight inch depth required for the spherical seating and platen in a biaxial compression test (section 3.4.7.) gave an effective column length of 31" allowing for a maximum specimen size of 12".

This column length together with the end plate span between columns (governed by access between columns and jack size) was used to determine suitable member sizes to fulfil the conditions of stiffness laid down in section 3.3.1.

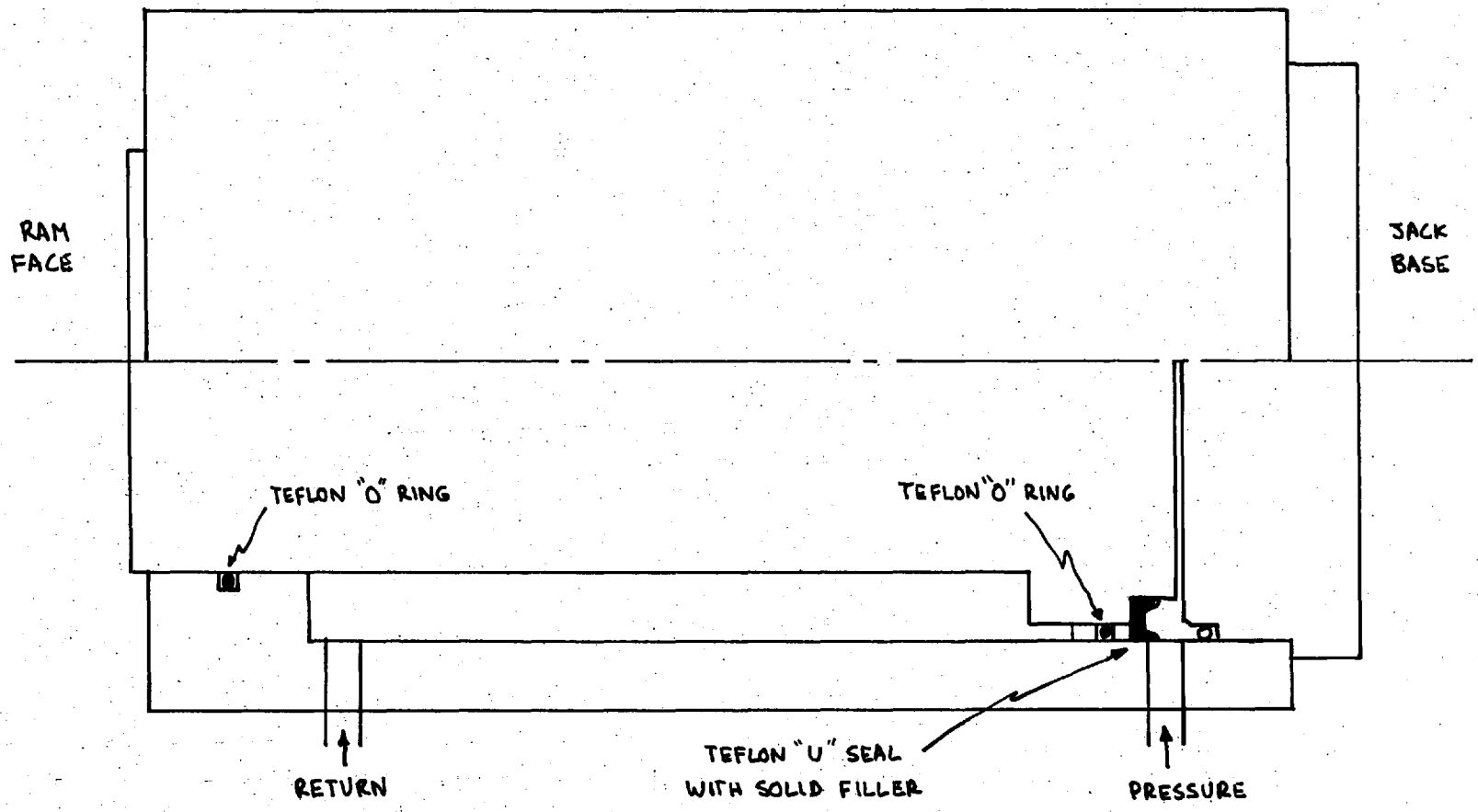


FIG. 3.4.7. LINE DIAGRAM OF "SIMPLEX" MODEL R.200 6/ST. RAM
SHOWING TEFLON (P.T.F.E.) SEALS.

The column length selected was checked to verify that the machine frame could accommodate Ward's tension calipers[7] and also the torsion equipment equipment designed for this machine (refer section 2.7.).

3.4.3. Lateral Stiffness

To give adequate stiffness to the machine frame as a whole, the vertical columns were designed to be a sliding fit down the whole depth of the base plate and stepped nuts (fig.3.4.3.1.) were used in the top plate. Thus the potential lateral rigidity of the columns was utilised by using semi-rigid column/plate connections (fig.3.4.3.2.).

Stiffness of the horizontal frame was achieved by using stepped nuts at both end plates and by the support given by the side portals.

3.4.4. Side Portals

These were required to maintain the alignment of the horizontal frame and at the same time give it adequate lateral support.

Therefore rectangular hollow sections were selected because of their flexural and torsional rigidity and ease of welding into frames. The wall thickness was specified at a minimum of $\frac{3}{8}$" where levelling screws and roller supports were provided. The four legged portal shown in fig.3.4.3.3. also has adequate stability and torsional resistance to avoid distortion during testing.

3.4.5. Portal Pinned Joint

In theory the joint between the portals and the vertical frame can be rigid or pinned but a pinned joint was selected as this eliminated any stresses due to lack of fit during construction.

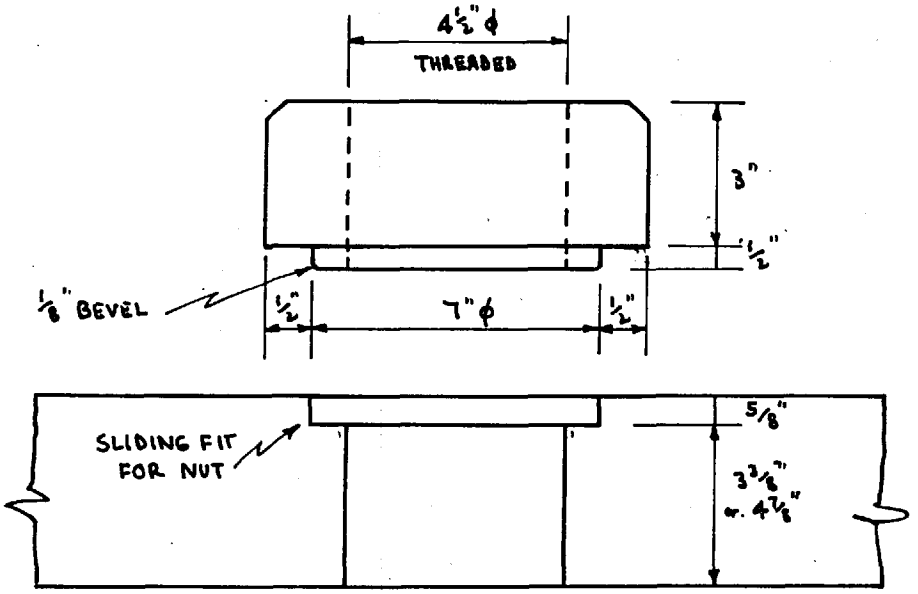


FIG. 3.4.3.1. DETAIL OF STEPPED NUT AND RECESS IN PLATE.

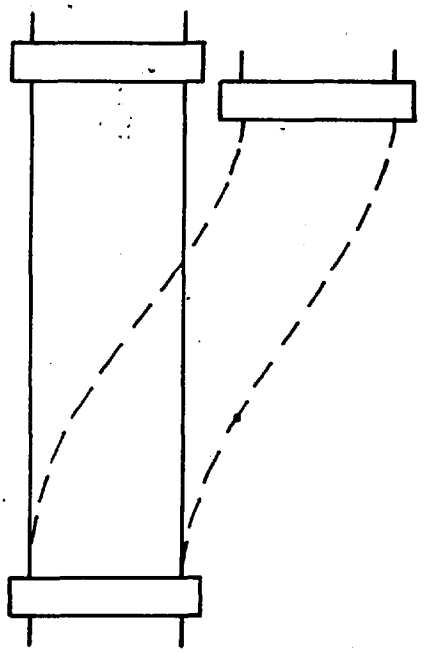


FIG. 3.4.3.2. RESULTING MODE OF LATERAL DEFORMATION OF SUB-FRAME

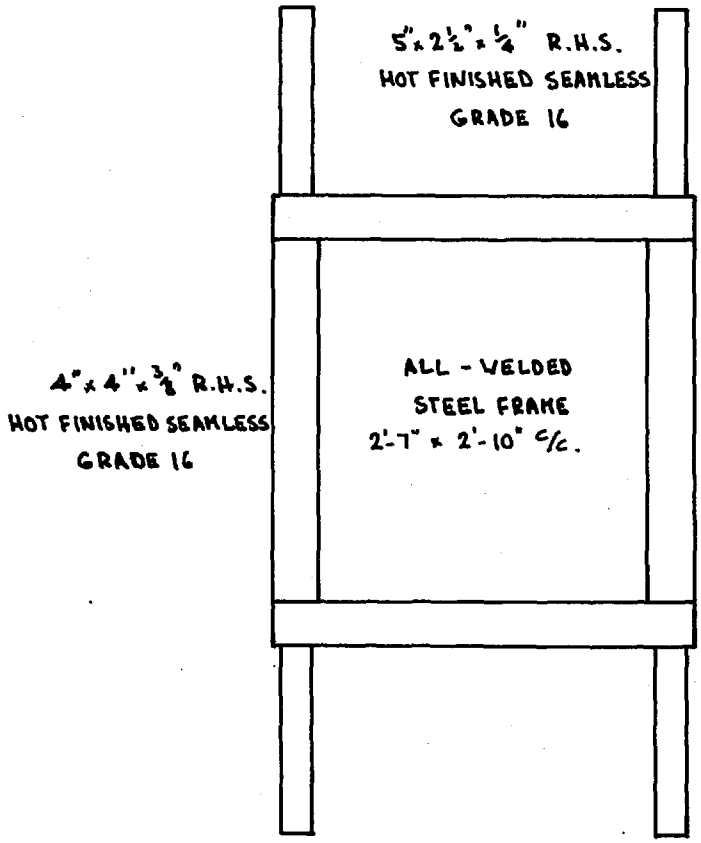


FIG. 3.4.3.3. RIGIDITY OF SIDE PORTALS.

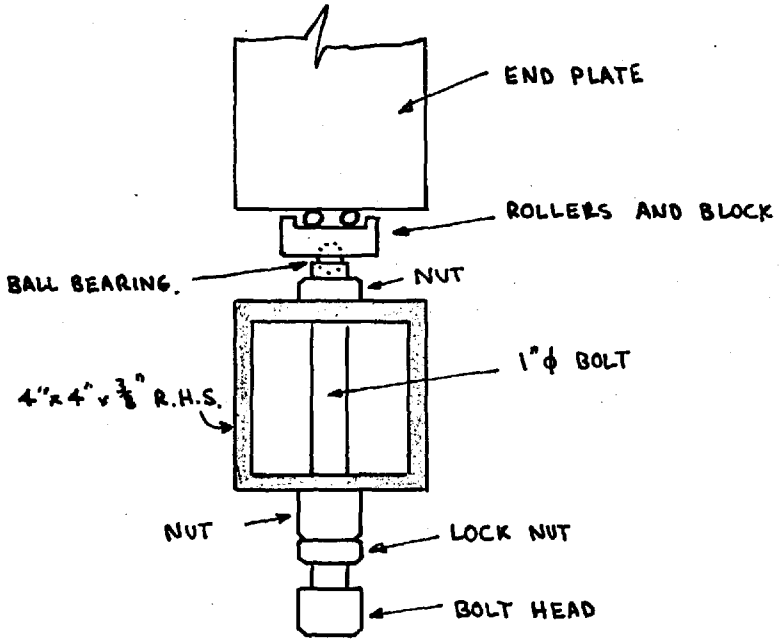


FIG. 3.4.6.1. SUPPORTING ROLLERS AND LEVELLING SCREW.

This, in turn, enabled greater tolerances to be allowed and helped to reduce the cost of the machine structure.

The pins were specified as a forced fit to avoid backlash in the alignment mechanism, and the side portals and pins were delivered as a single unit.

3.4.6. Rollers and Levelling Screws

Nylon encased steel roller pads by Ina Needle Bearing Ltd. were used to support the horizontal frame, two being used at each end.

The screws used for supporting the roller pads and blocks, and for levelling the horizontal frame end plates within the portal were similar. They consisted of a $\frac{3}{4}$" ball bearing placed in the end of a 1" Ø bolt with nuts and lock nuts as shown in fig.3.4.6.1.

The resulting arrangement was very stiff and provided a firm guide for the relative movements of the horizontal frame and the side portals.

3.4.7. Platen and Spherical Seating Design

As the ram diameter of the compression jacks to be used was 6 in. and the standard size selected for biaxial compression specimens was 10 x 10 x 4 in., the combined unit of spherical seating and loading platen was required to cantilever 2 in. from each side of the centre-line.

As the machine was principally a rigid frame, the clearances required between the end of the platen and the test specimen was $^1/16$" compared with $\frac{1}{8}$" used in Robinson's machine[10]. Besides improving the distribution of load on the specimen it also enabled the specimen to be more accurately aligned in the machine. The platen widths were 4 in. which served as a further aid to specimen alignment as shown in fig.3.4.7.1.

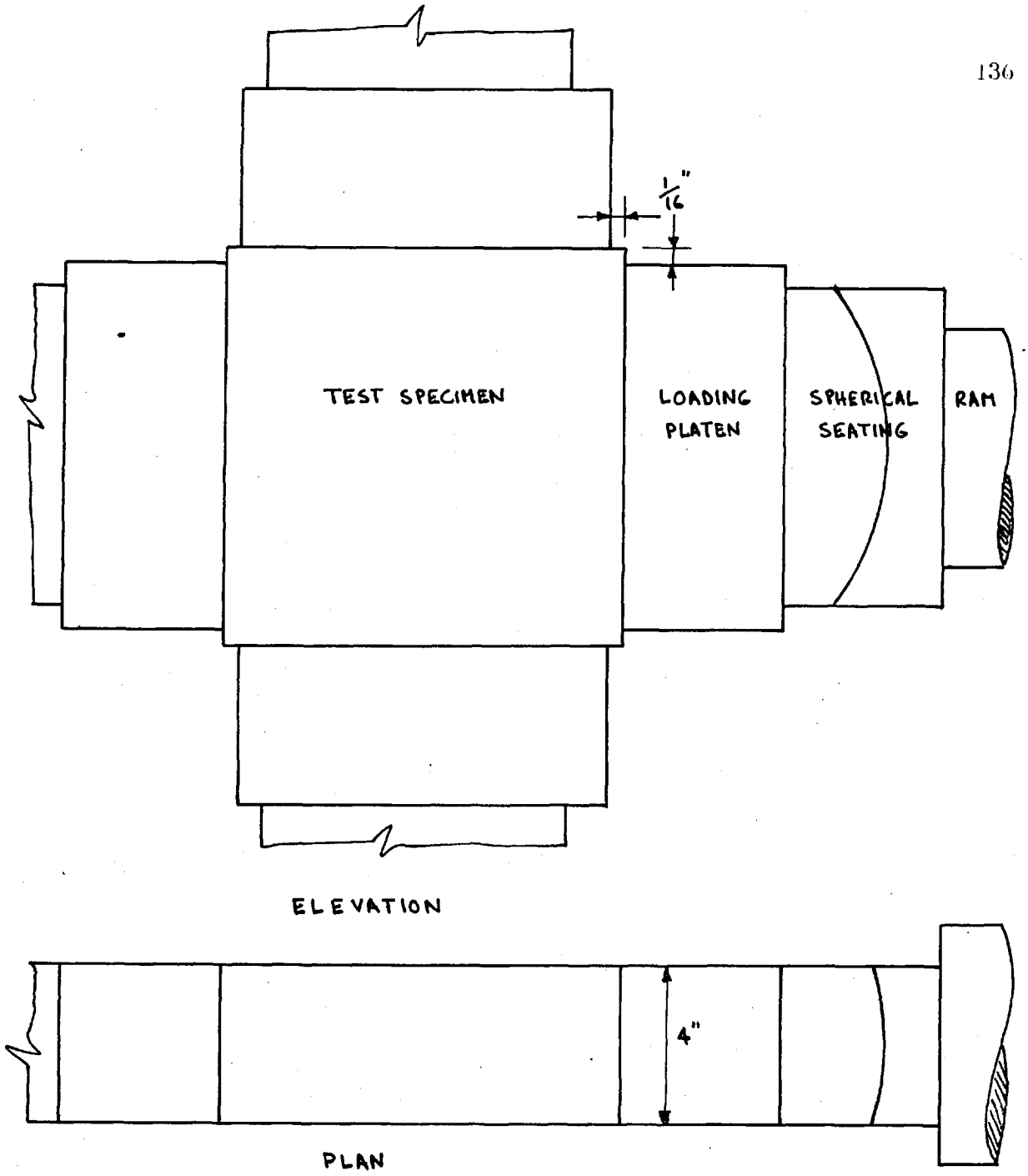


FIG. 3.4.7.1. DIMENSIONS OF LOADING PLATEN / SPHERICAL SEATING UNIT FOR ALIGNMENT ON BIAXIAL COMPRESSION SPECIMEN.

The dimensions shown in fig.3.4.5. were obtained from calculations based on the beam as an elastic foundation concept and were considered to provide adequate stiffness for the loading platen. The results of strain surveys reported in section 4.4. confirm this conclusion.

Photographs of the combined unit illustrating its construction are shown in fig.3.4.7.2. and the disposition of the platen/seating units in the machine is given in fig.3.4.7.3. The counterweights for the top ram and platen are such that they exert an upwards force on the system equivalent to that due to gravity on the lowas jack.

An interesting combined platen and spherical seating has been designed by Carter[142] providing continuous rotation of the spherical seating under load. Essentially it consists of a loading platen suspended in an oil bath.

3.5. Manufacturers Modifications

No major modification to the original detailed design drawings was asked for by the manufacturers of the machine frame. However, the use of a truncated thread on the bottom threaded ends of the vertical sub-frame columns was recommended to assist the sliding fit of these columns in the base plate and this recommendation was adopted.

3.6. Hydraulic System and Control Panel

To allow infinite versatility in operation as required by the research project, two identical systems were developed - one for each sub-frame - but were housed in the same panel for ease of operational control.

The control system was similar to the elementary one of control by bleed-off or bypass valve used in all the research machines at Imperial College (Concrete Department) but with the addition of a spool valve to operate the hydraulic return system and stop valves to isolate individual jacks. A line diagram of one complete system is shown in fig.3.6.1.



a) Dismantled Spherical Seating.

(3)



b) Assembled Spherical Seating.

Fig. 3.4.7.1. a) Construction of Spherical Seating/Loading Platen Unit.



c) Attachment of Seating to Platen

(3B)



d) Complete Seating/Platen Unit showing Dowel to mate with Ram Head.

Fig. 3.4.7.1. b) Construction of Spherical Seating/ Loading Platen Unit.

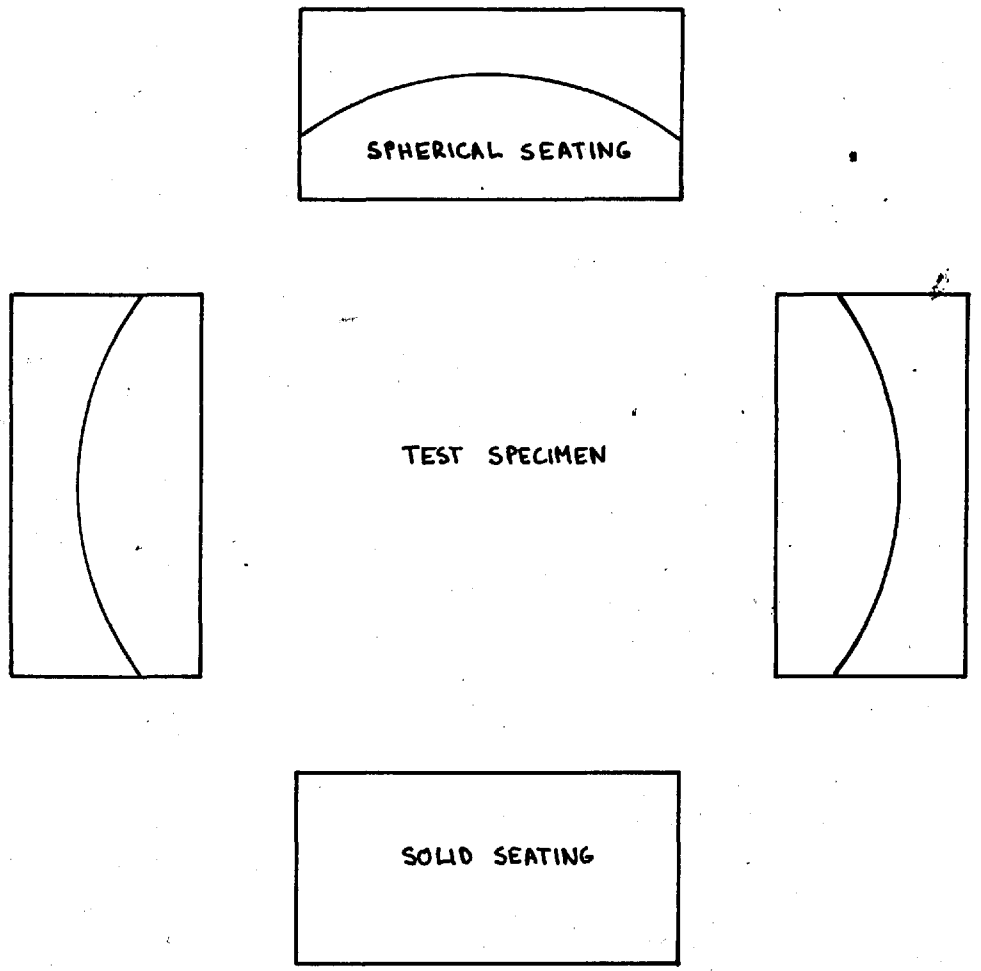


FIG. 3.4.7.3. DIAGRAMMATIC ARRANGEMENT OF SPHERICAL AND SOLID SEATINGS AROUND TEST SPECIMEN.

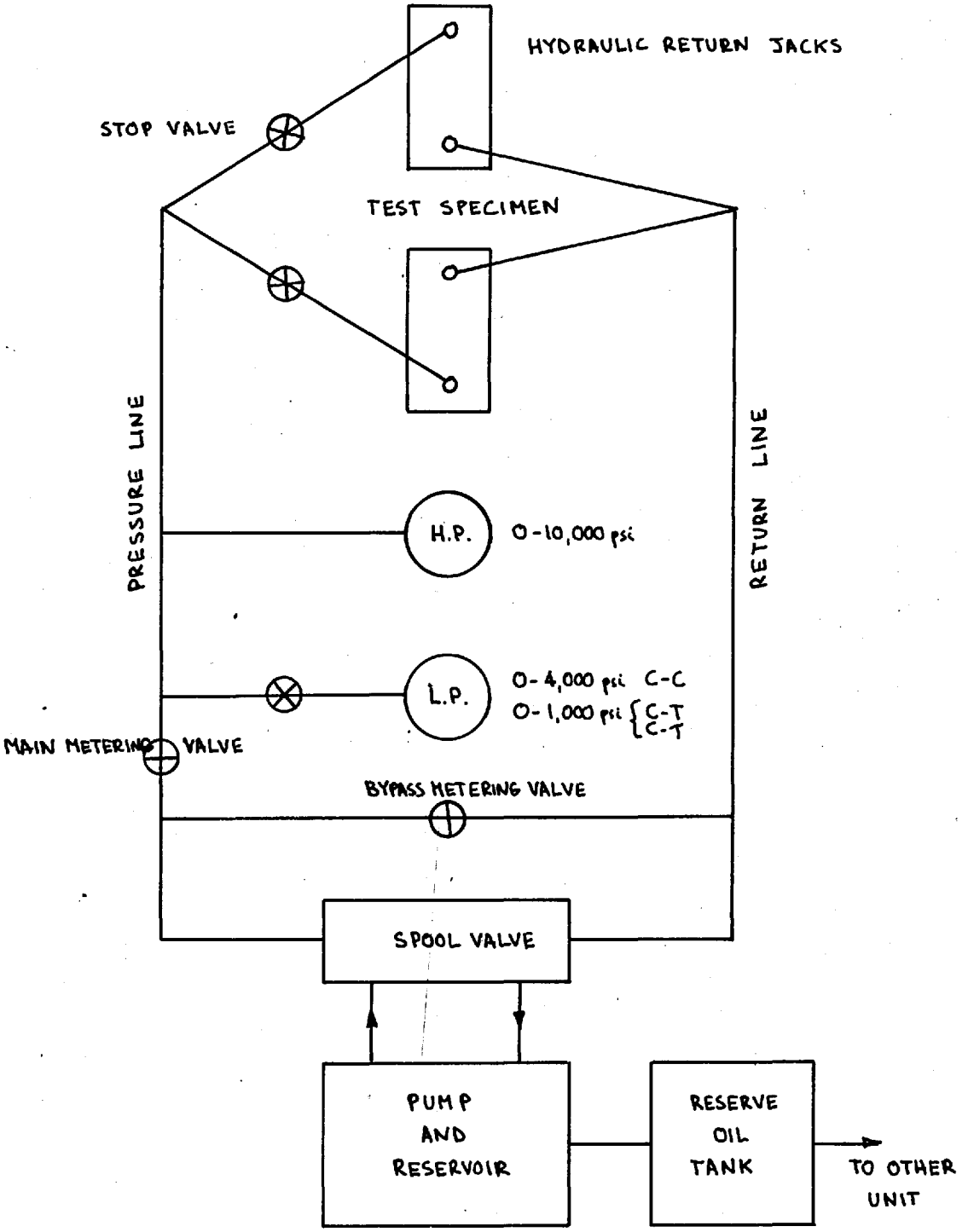


FIG. 3.6.1. DIAGRAMMATIC LAYOUT OF HYDRAULIC SYSTEM FOR ONE PAIR OF JACKS.

The control system on the horizontal sub-frame was required to operate up to 9,200 p.s.i. for the 30 ton tension jacks whilst that on the vertical sub-frame was only required up to 7,500 p.s.i. However, to allow complete interchangeability of equipment, a common working pressure of 10,000 p.s.i. was selected for all pipes, valves, pumps and moters, and fittings.

All the components are listed in the table in fig.3.6.2. and the operational instructions are given in fig.3.6.3. Safety pressure release valves were incorporated in the system as a whole but no relief valves were incorporated with the Budenberg pressure gauges. This would be a useful safety addition for a later date.

A photograph of the control panel with its two identical sets of controls is shown in fig.3.6.4. and it will be observed that the Cawkell USPV. equipment has been housed in the centre of the panel immediately in front of the operator. The exact function of each control is given in fig.3.6.5. and a rear view of the panel is shown in fig.3.6.6.

3.7. Construction and Alignment of Machine

The construction of the machine was complicated by the lack of lifting facilities in the Concrete Material Research Laboratory. No overhead cranes or lifting beam supports were available and hence a scaffold gantry was designed by the author (fig.3.7.1.).

Essentially it consisted of two free-standing towers linked by two braced ties and supporting a single lifting beam centrally. Two chain blocks (20 cwt., and 10 cwt) were used and these, together with the Sherpa fork-lift trolley, were the only lifting appliances used. The brickwork for the base of the machine plinth and the wire hausers used are also visible in the photograph.

The erection sequence is given in figs.3.7.2. and 3.7.3. but is not described in detail here as it is only of interest from the design point of view or to anyone wishing to dismantle the machine.

1. Machine frame components as shown on drg. no. 2 (see Appendix C).
   J.N. Electronic Supplies Ltd.

2. Hydraulic jacks manufactured by Simplex, U.S.A.
   4 No. R200 6S Re-mo-trol Heavy Duty 2-way Ram
   200 ton capacity with "Teflon" seals.
   2 No. R315 2-way Hollow ram
   30 ton capacity with "Teflon" seals.
   2 No. Four-way spool valve (1704) part no. 500-2781-6
   with sub-plate, part no. 500-6120-8.
   The Equipment and Engineering Co. Ltd.

3. Hydraulic pressure gauges.
   2 No. 0 - 10,000 psi
   2 No. 0 - 4,000 psi
   2 No. 0 - 1,000 psi
   Budenberg Gauge Co. Ltd.

4. Needle control/stop valves
   6 No. type no. 104/1 suitable for 10,000 psi hydraulic.
   I.V. Pressure Controllers Ltd.

5. Pump/motor units.
   2 No. Beacham type RTU1/SW00 bedplate unit for a
   pressure of 10,000 psi coupled to 1 H.P. electric
   motor for a 440/3/50 supply.
   2 No. Adjustable pressure relief valves type S.V.1.
   0 - 10,000 psi.
   Oswalds and Ridgway Ltd.

6. Stop valves, pipes, fittings.
   All guaranteed or tested to withstand 10,000 psi.
   British Ermeto Co. Ltd.

fig. 3.6.2. List of major equipment and suppliers

OPERATION INSTRUCTIONS

1. Before starting pump motors:
   check that spool valve control lever is in neutral.

2. Before putting spool valve control into pressure or return:
   check that stop valves to relevant jack are open;
   check that low pressure gauge stop valve is closed
   if pressure range of gauge is likely to be exceeded;
   watch ram movement - if ram stops of its own accord
   put spool valve control lever into neutral immediately
   and investigate;
   check that bypass valve is at least partially open if
   loading platens are in contact with a test specimen.

3. Control of pressure:
   set main control valve to half open position and control
   pressure and pressure rate with bypass valve.

4. On failure of specimen:
   put spool valve control lever to neutral position
   immediately.

5. After using machine:
   retract all rams to end of travel;
   switch off electric motors;
   isolate electric motors from mains supply.

fig. 3.6.3. Operational instructions and checks system.

(3B)



Fig. 3.6.4. Testing Machine Control Panel.

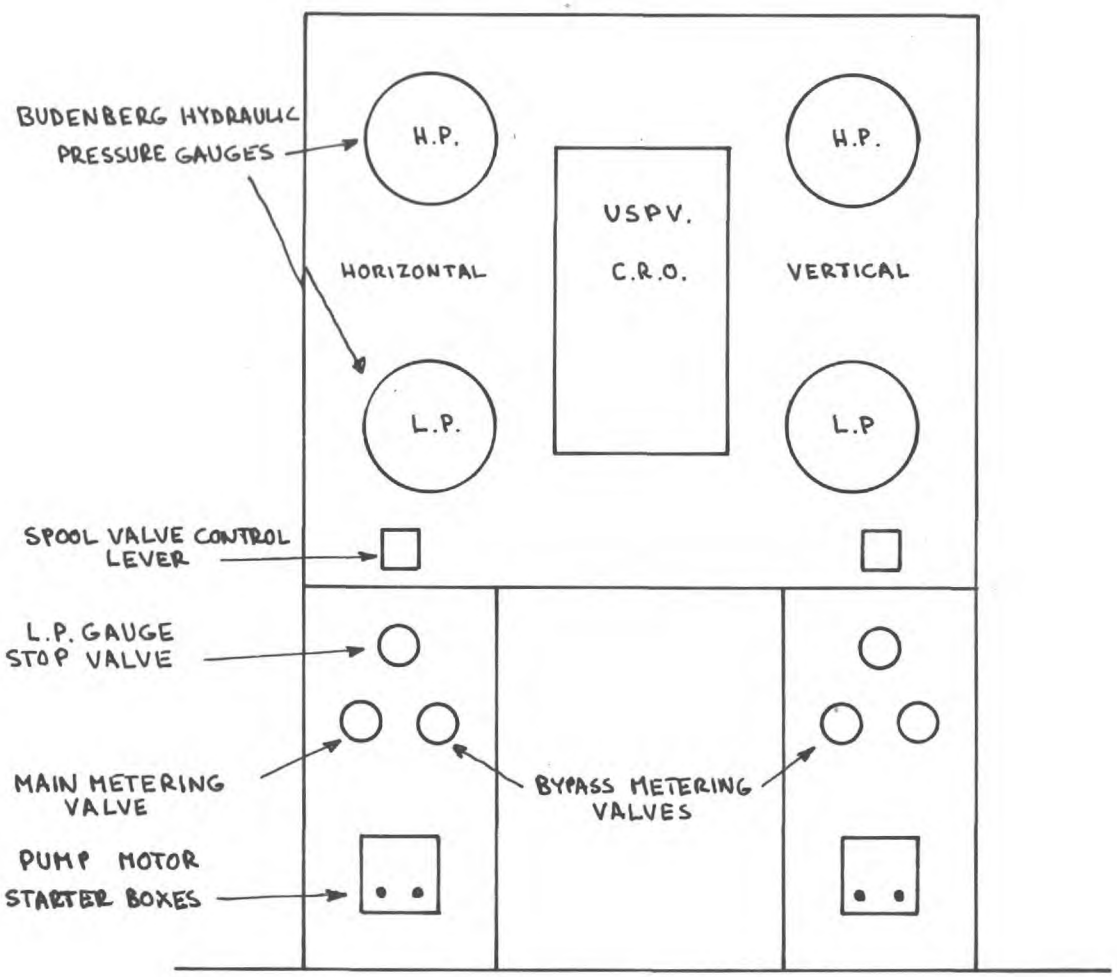


FIG. 3.6.5. LAYOUT OF HYDRAULIC CONTROL PANEL

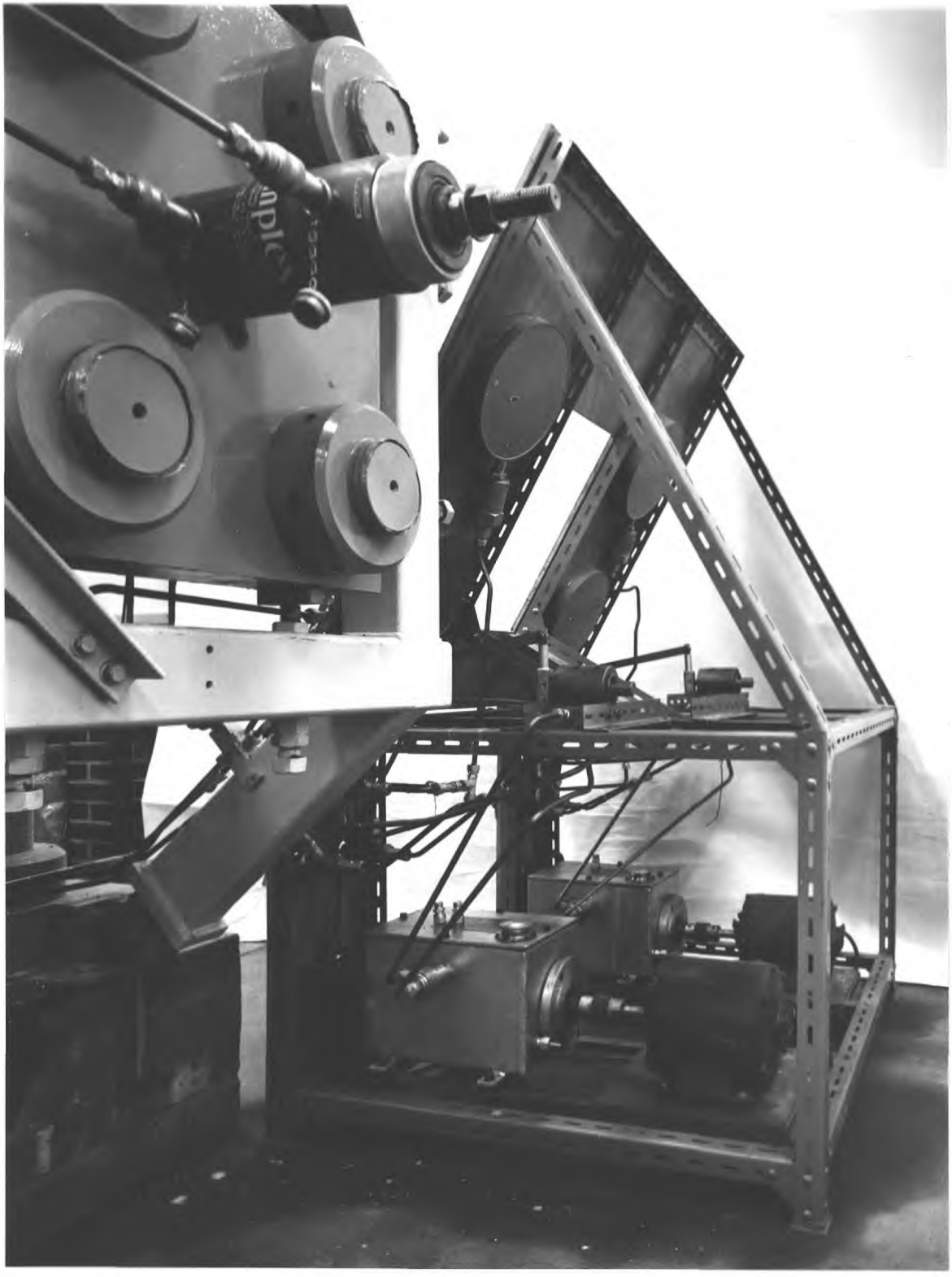


Fig. 3.6.6. Rear view of Control Panel showing pumps, motors, spool valves, etc.

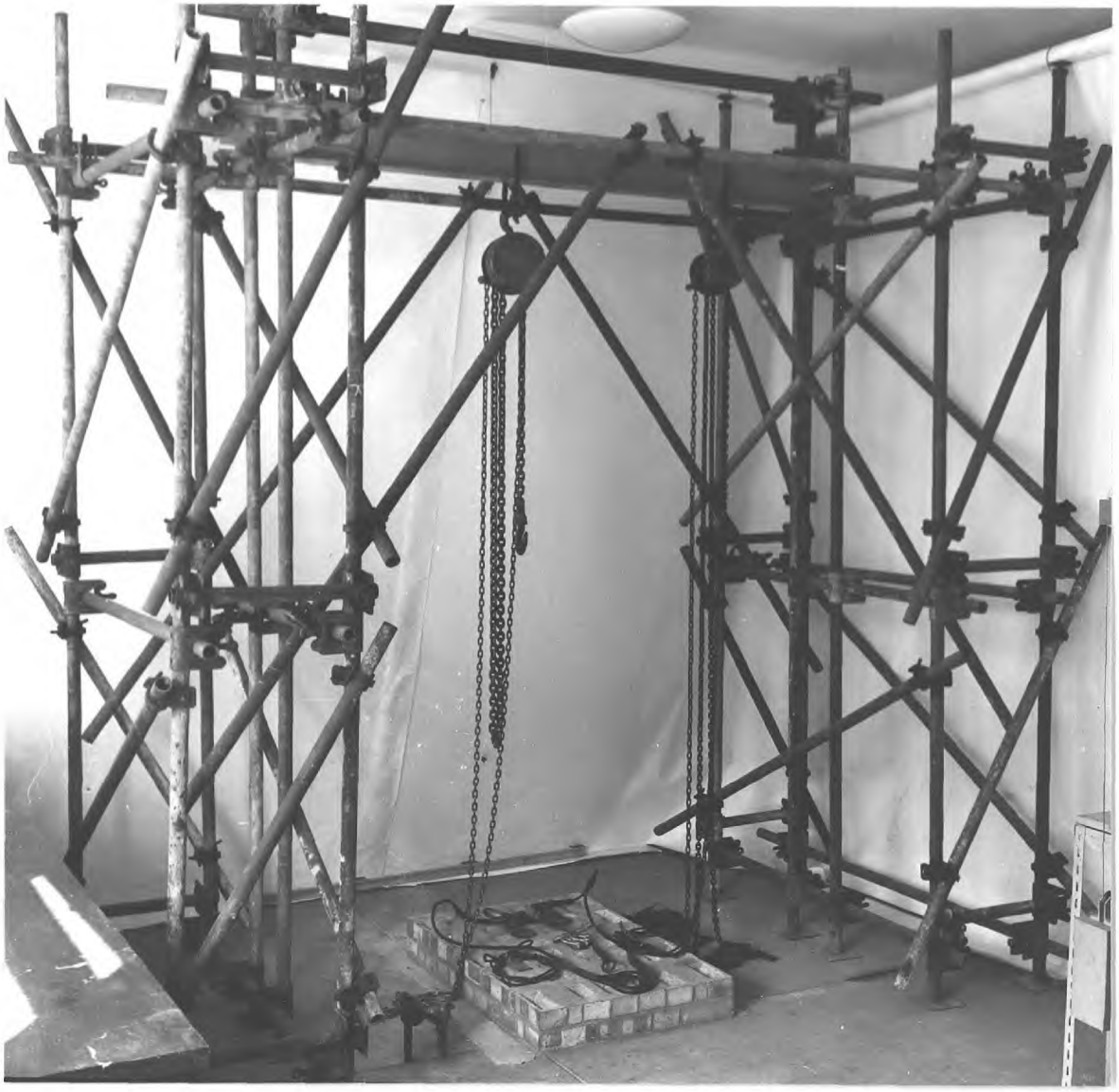
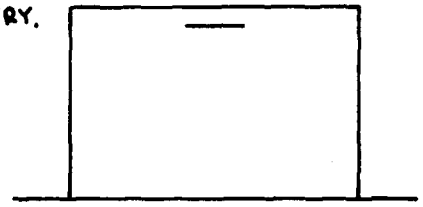
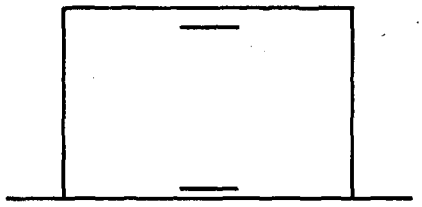


Fig. 3. 7. 1. Temporary Scaffold Gantry and Lifting Tackle.

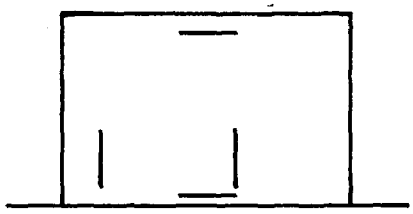
1. TOP PLATE SUSPENDED AT TOP OF GANTRY.



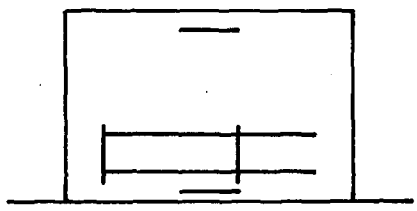
2. BASEPLATE LEVELLED ON PLINTH.



3. ONE END PLATE AND TEMPLATE IN.



4. HORIZONTAL SUB-FRAME COLUMNS IN.



5. OTHER END PLATE IN AND TEMPLATE REMOVED.

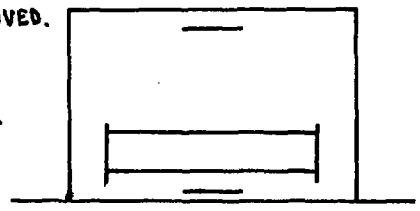
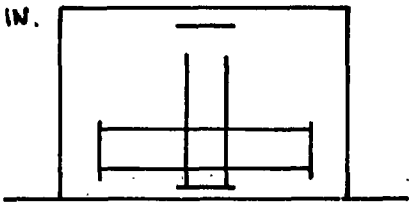
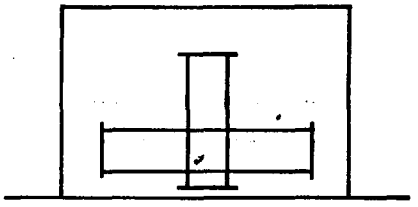


FIG. 3.7.2. MACHINE ERECTION SEQUENCE. STAGES 1-5.

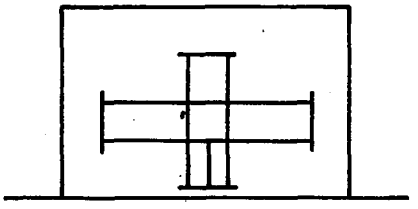
6. COLUMNS OF VERTICAL SUB-FRAME IN.



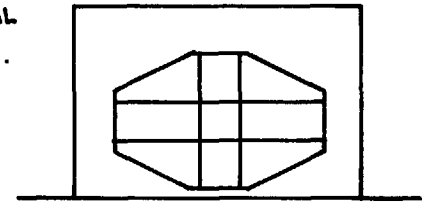
7. TOP PLATE LOWERED ONTO COLUMNS.



8. HORIZONTAL SUB-FRAME LIFTED TO CENTRAL POSITION AND SUPPORTED ON TEMPORARY PACKING.



9. SIDE PORTALS FITTED AND HORIZONTAL SUB-FRAME LOWERED ONTO ROLLERS.



10. JACKS IN.

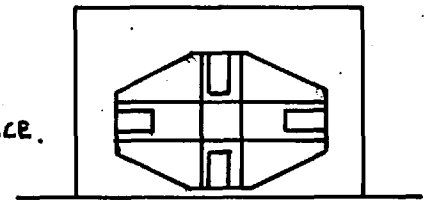
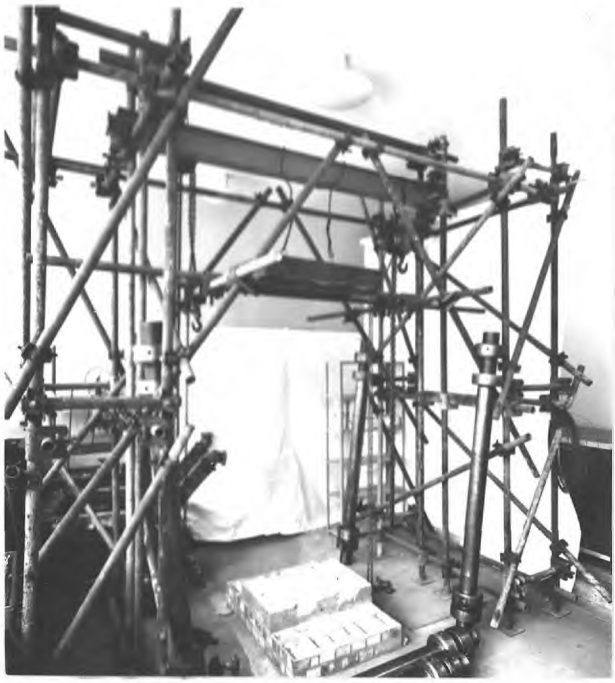
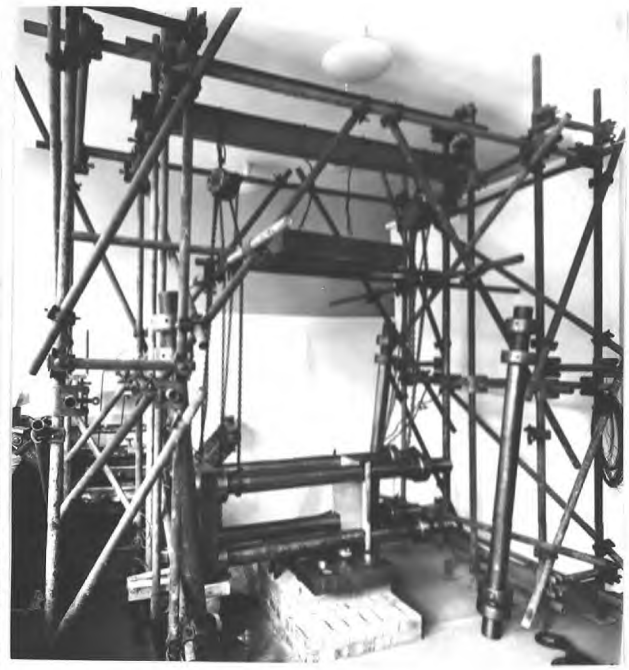


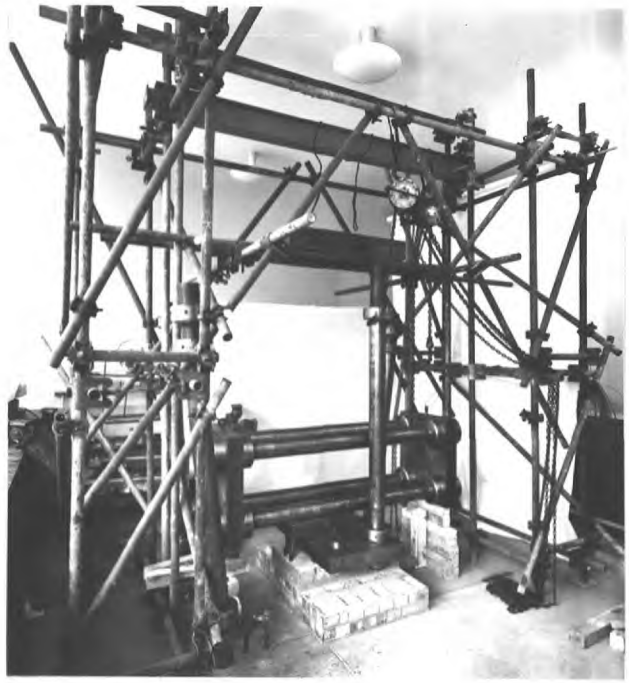
FIG. 37.3. MACHINE ERECTION SEQUENCE. STAGES 6-10.



Stage 1

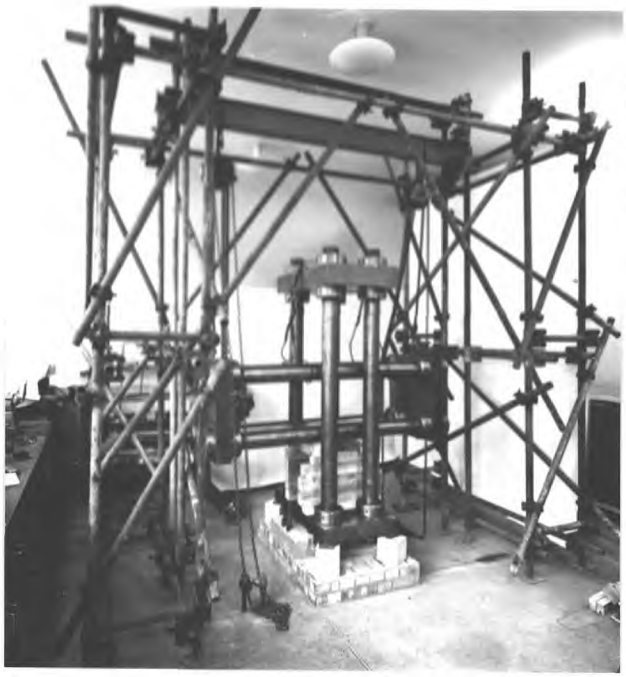


Stage 4

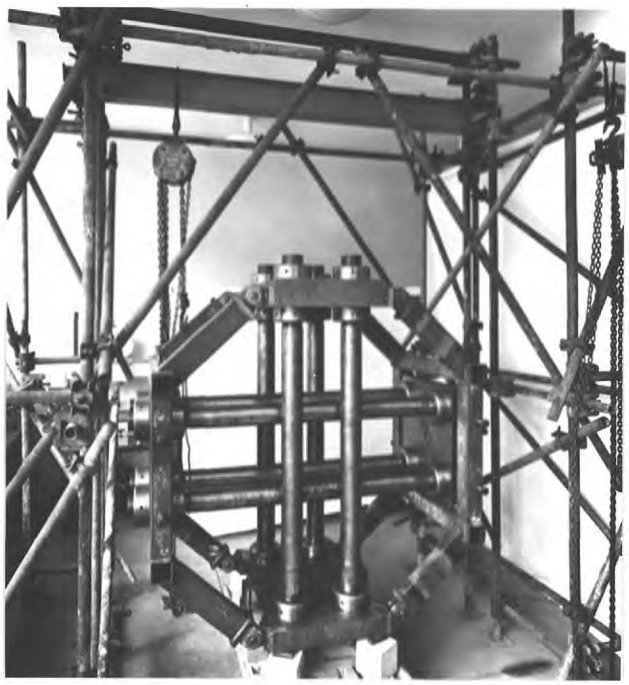


Stage 6

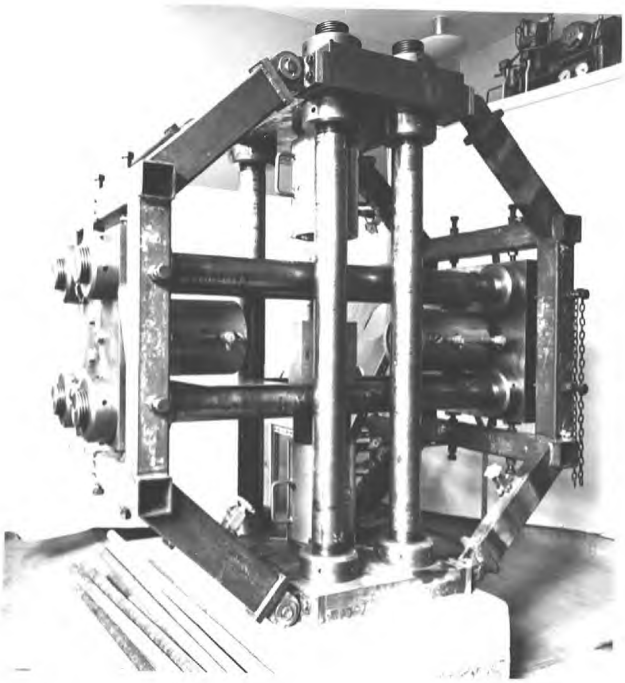
Fig. 3.7.4. Sequence of Machine Frame erection (Part 1)



Stage 8

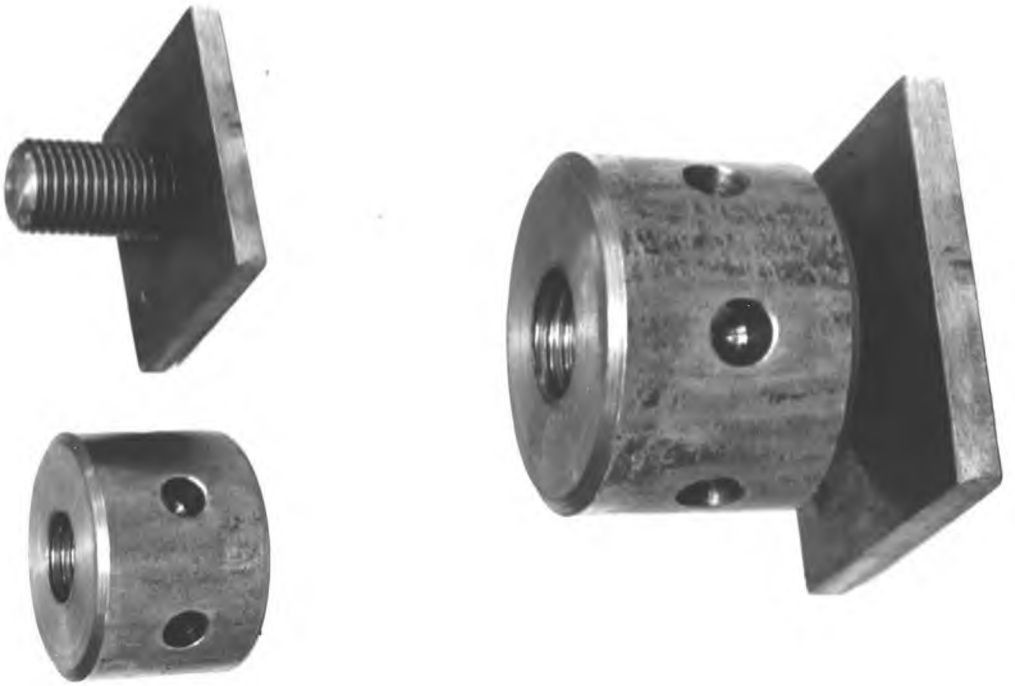


Stage 9



Stage 10

Fig. 3.7.5. Sequence of Machine Frame erection (Part 2)



(7)



Fig. 3.7.6. Base Plate Levelling Screws - details and arrangement on machine plinth.

The progress photographs shown in figs.3.7.4. and 3.7.5. indicate the sequence of assembly.

Purpose made levelling screws were used for the baseplate and these are shown in fig.3.7.6.

Alignment of the jacks was carried out in the following sequence:-

1) Top jack of vertical frame clamped centrally on top platen and used as datum.
2) Bottom jack plumbed into position from top jack.
3) Side jacks clamped to end plates.
4) Aluminium 10 x 10 x 4 reference specimen held centrally between top and bottom jacks in vertical frame.
5) Horizontal frame levelled and aligned with adjusting screws so that with seatings locked, the platens aligned with the aluminium specimen.
6) Proving trials described in section 4.2. were then carried out to check the alignment.

3.8. Summary

This chapter has described the design and assembly of the machine resulting from the general arrangement selected and the performance and machine design requirements regarded as significant.

The selection of all the major components has been detailed as well as the reasons for their selection.

Much information has been included which is mainly of interest to potential operators so that the record in this chapter might not only be complete but also serve as a briefing manual for future workers.

Lastly a brief description of the construction and alignment of the machine was given.

CHAPTER 4

MACHINE AND SPECIMEN PROVING TRIALS

4.1. Introduction

Although these proving trials are referred to as "machine" and "specimen" trials meaning trials on machine frame behaviour and on specimen state of stress, it is clear that both will have a decided effect on the results ultimately achieved. Such trials are essential in order to establish the degree of reliability that can be placed in the subsequent experimental work. It should be remembered that a major criticism of most previous work was that such trials had not been reported (section 1.3.).

Proving trials act as a check on the design assumptions of the various compromises inevitable in any new development work. They are the test to which all such designs must be submitted and indicate the degree of success achieved. Furthermore the information gathered, good or bad, is of considerable use in subsequent research projects of a similar nature.

The ultimate intention is to establish beyond all reasonable doubt, the stress and strain patterns actually achieved in the test specimen and their territorial limits.

It is necessary to define clearly the aims of each trial. For convenience the trials have been grouped as follows:-

4.2. Machine Trials
   1. Alignment of sub-frames
   2. Self-alignment mechanism of whole machine frame.

4.3. Machine Calibration
   1. Compression facilities
   2. Tension facility
   3. Torsion facility

4.4. Biaxial Compression Specimen
   1. Surface strain by strain gauge
   2. Surface strain by photo-elastic technique
   3. Lateral strain
   4. Coefficient of variation of Test Results
   5. Order of loading

4.5. Biaxial Compression-Tension Specimen
   1. Direct Tension Facility
   2. Combined Compression-Tension Facility

4.6. Biaxial Compression-Torsion Specimen
   1. Surface strain

4.7. Hydraulic Control System.

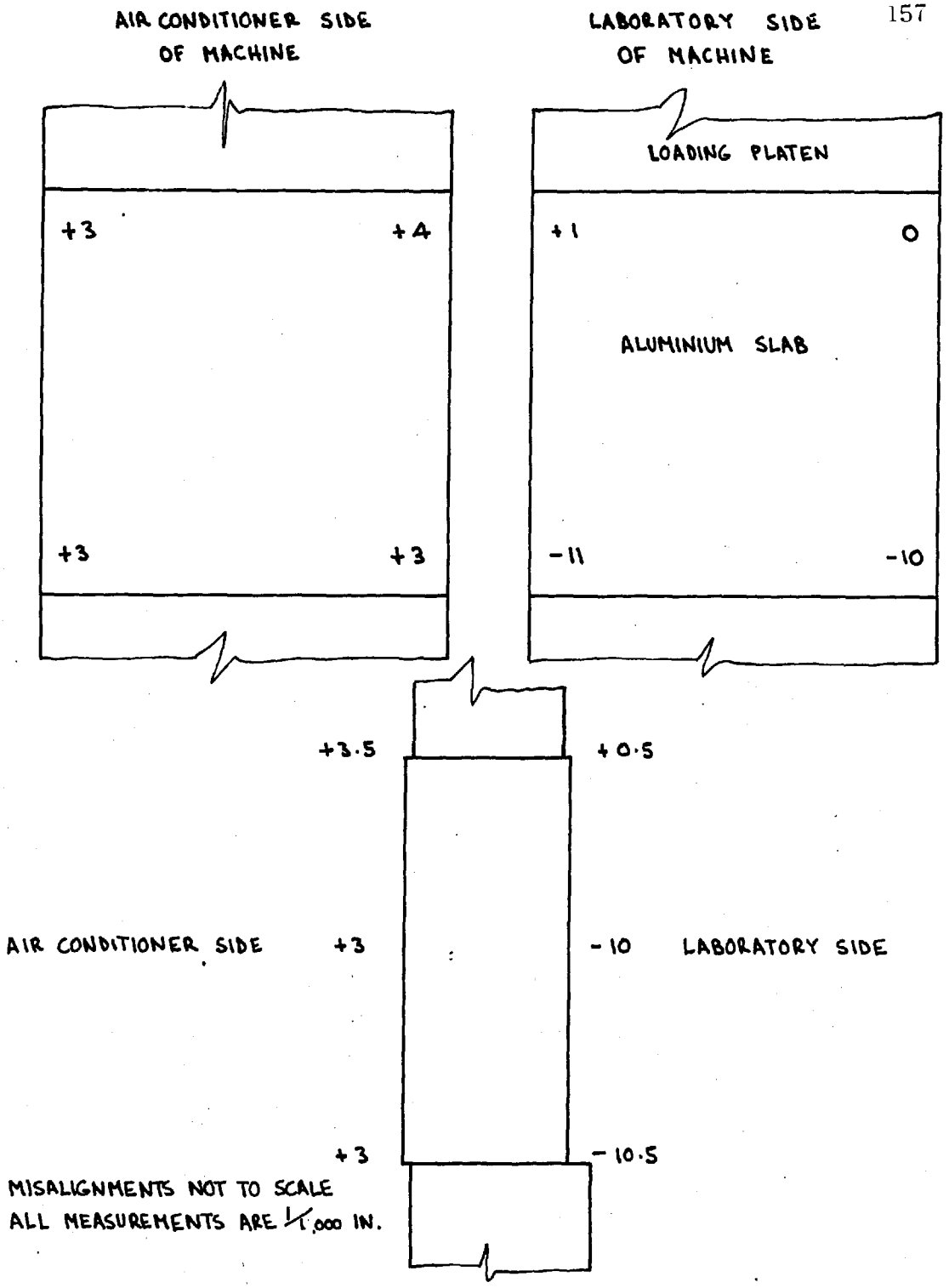
4.2. Machine Trials

These trials were concerned exclusively with the mutual geometrical alignment of the sub-frames and jacks, and with the subsequent self-operating alignment-maintaining action of the side portals.

4.2.1. Alignment of Sub-frames

The actual alignment procedure has been described in section 3.7. In this trial the physical alignment of the biaxial compression loading platens on a precisely made aluminium 10" x 10" x 4" slab was checked and then the strain variation between faces recorded. In this latter trial any significant eccentricity of loading would be reflected in the strain readings obtained.

By using a depth micrometer and measuring the alignment of the aluminium slab and the platen sides on both faces of the slab and in all four corners on each face it was possible to adjust the jacks until the worst reading was within 0.002 in. The results obtained with the vertical frame alignment are given in fig.4.2.1.1. in which positive signifies that the member in question is proud of its adjacent member.



NETT MISALIGNMENT OF LOADING PLATENS = 0.003 in.

FIG. 4.2.1.1. ALIGNMENT OF VERTICAL JACKS.

In the strain trial, the aluminium slab was again used and electrical resistance strain gauges were positioned as shown in fig.4.2.1.2. on both faces of the slab.

With the gauges aligned vertically as in fig.4.2.1.2., the vertical machine compressed the slab between 133 p.s.i. and 1330 p.s.i. which values happened to be convenient gauge readings. The exercising was done twice to check repeatability of strain reading and then repeated for two subsequent set-ups. Hence both the strain repeatability and comparison between faces was observed both for one set-up and between set-ups. The same procedure was repeated for another three set-ups with the horizontal compression at 533 p.s.i.; again a convenient pressure gauge reading.

The whole procedure was then repeated in its entirety with the aluminium specimen turned through 90° and the pressure from the vertical machine maintained constant whilst the pressure from the horizontal machine was varied using the same pressure increments as before. Thus a check was made on strain distributions in both directions as well as the comparison between faces and between different set-ups. The results are condensed in fig.4.2.1.2.

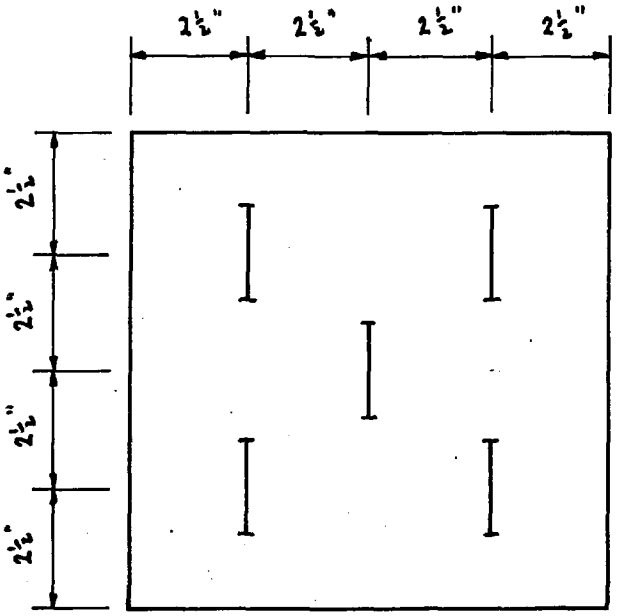
These results show that no significant variations in strain distribution were obtained due to misalignment effects.

4.2.2. Self-alignment of Machine Frame

The achievement of a self aligning mechanism in the machine frame was a major consideration in the machine design. These trials were carried out to discover whether the design objective had in fact been achieved.

An independent scaffold datum frame was cantilevered off the machine baseplate (fig.4.2.2.2.) and spanned across the whole machine frame so that dial gauges (sensitive to 0.0001) might be placed at the strategic points marked in fig.4.2.2.1.

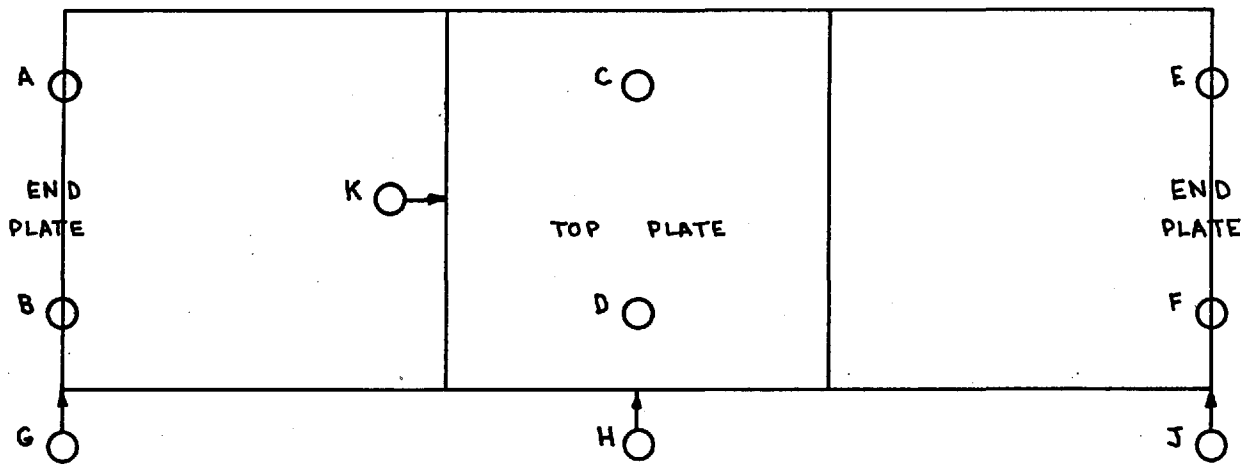
Gauges C and D measured the total vertical extension of the vertical sub-frame whilst H and K measures its sideway. The vertical movement of the horizontal sub-frame was recorded by gauges A, B, E



STRAIN GAUGE LAYOUT

STRAIN VARIATIONS TABLE				
1) ANY ONE GAUGE TAKEN BY ITSELF.				
	σ_1 RANGE	σ_2	WITHIN ONE SET-UP.	BETWEEN SET-UPS.
a) VERTICAL STRAINS				
UNIAXIAL	133 - 1330	0	± 1 %	± 2 %
BIAXIAL	133 - 1330	533	± 2 %	± 3 %
b) HORIZONTAL STRAINS				
UNIAXIAL	133 - 1330	0	± 2 %	± 3 %
BIAXIAL	133 - 1330	533	± 3 %	± 3 %
2) BETWEEN OPPOSITE FACES.				
	σ_1 RANGE	σ_2	AVERAGE STRAIN.	VARIATION (MAX.)
a) VERTICAL STRAINS				
UNIAXIAL	133 - 1330	0	119	± 2
BIAXIAL	133 - 1330	533	118	± 7
b) HORIZONTAL STRAINS				
UNIAXIAL	133 - 1330	0	108	± 8
BIAXIAL	133 - 1330	533	107	± 9

FIG. 4.2.1.2. VARIATIONS AND REPEATABILITY OF STRAIN MEASUREMENTS.



PLAN OF MACHINE SHOWING LAYOUT OF DIAL GAUGES.

NOTE: GAUGES A B C D E F MEASURE VERTICAL MOVEMENT.
GAUGES G H J K MEASURE HORIZONTAL MOVEMENT.

ABSTRACT FROM TRIAL No. 2.

RANGE σ_3	σ_2	GAUGE	ACTUAL READING $\times 10^{-4}$ IN.	AVERAGE	CORRECTION	CORRECTED DISPLACEMENT
267	267	A	19.9	20.3	+5.1	25.4
-1333		B	20.5			
psi	psi	C	44.5	45.2	+3.0	48.2
		D	46.0			
		E	34.5	30.9	-5.1	25.8
		F	27.3			
		G	0.1			0.1
		H	0.3			0.3
		J	0.1			0.1
		K	10.2		-10.2	0

TEST SPECIMEN STRAIN = 328×10^{-6}

EQUIVALENT MACHINE STRAIN = $48.2 \times \frac{1}{71.5} \times 10^{-4} = 67 \times 10^{-6}$

FIG. 4.2.2.1. SELF- ALIGNMENT OF MACHINE FRAME.

(8)

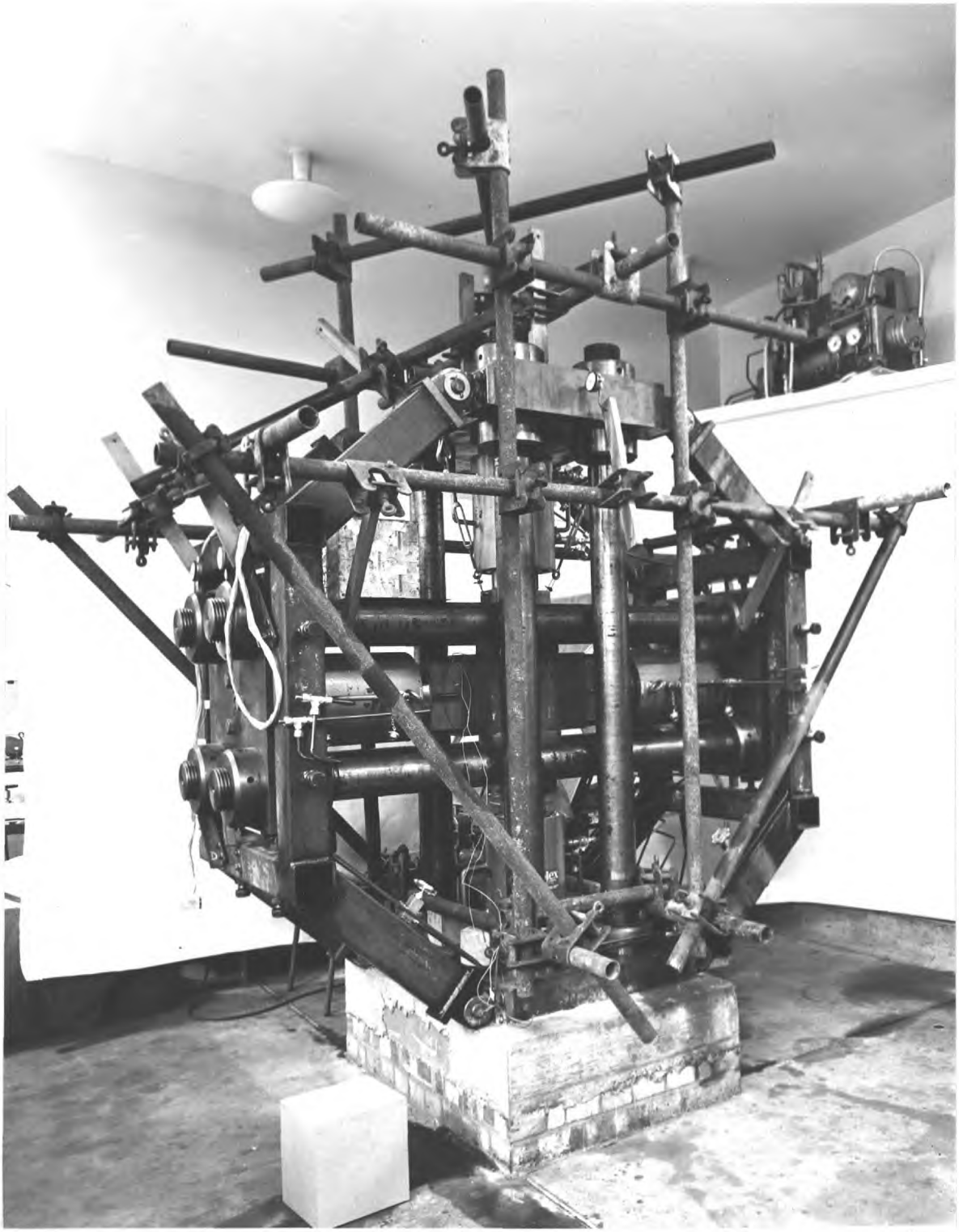


Fig. 4.2.2.2. Scaffold Datum Frame and Dial Gauges for measuring machine deflection.

and F and the lateral movement of the side portals by G and J.

A fault developed in the scaffold frame in that it swayed relative to the machine in the direction of K; the fact that the machine did not sway was confirmed by an independent frame constructed later and mounted on the laboratory floor. The fault could not be specifically located, though the very small base on which the scaffold frame was based lent itself to such movements. Therefore all the readings have been corrected to allow for this movement; fortunately it does not conceal the general relative movements taking place in the whole frame.

The results in fig.4.2.2.1. show that, after correction, the machine sidesway is less than 0.0005 in. and that the self-alignment mechanism works satisfactorily.

Note that the accuracy of the dial gauges is of the order of 0.ØØ02 in. and therefore it can be concluded that the machine frame behaved as designed and the self-alignment mechanism works correctly.

The surface strain trials described in section 4.4.1. failed to detect any overall shear forces acting on the specimen thus confirming the satisfactory machine alignment predicted by the results of this trial.

4.3. Machine Calibration

The calibration of the various jacks and loading devices has been included under the general title of proving trials as the results of such calibrations are also useful indications of the behaviour of the machine over its entire range.

Of especial interest is the repeatability of the pressure gauge readings and, in the case of this particular machine, the comparison between the horizontal and vertical compression facilities. It will be recalled that the efficiency of the horizontal jacks was a point of special concern during the design stage.

The calibration trials fall conveniently under the three headings of compression, tension and torsion facilities as the reference equipment used and the calibration procedures differed in each case.

4.3.1. Compression Facilities

The compression facilities were calibfated with a National Physical Laboratory designed load cell fitted with a mechanical dial gauge and measuring about $10\frac{1}{2}$" overall thus corresponding roughly to the size of the biaxial oompression specimen. The load cell was owned by the Cement and Concrete Association who kindly undertook to calibrate the machine on two occasions during the period of this research contract (14th November 1964 and 25th June 1965). Mr. D. Cole, who also supervised the calibration, deserves special thanks for his interest in this problem. The load cell had been calibrated previously against dead-weight at the National Physical Laboratory at Teddington.

The calibration procedure which was identical in the case of both sub-frames was to exercise the load cell initially six times and then, after removing the load, for five minutes, reapply it up to the maximum capacity of the cell (in this case 175 ton) and then unload again. This loading was repeated twice giving three readings in all at all load stages chosen, the load being completely removed between each unloading and subsequent loading.

The results of the two calibrations are tabulated in fig.4.3.1.1. and plotted in fig.4.3.1.2.

The close agreement between the calibration of the vertical and horizontal sub-frames is quite remarkable and speaks well of the jacks used as does the agreement between the two separate calibration trials.

CALIBRATION OF BIAXIAL COMPRESSION FACILITY.				
PRESSURE GAUGE READING psi.	VERTICAL MACHINE		HORIZONTAL MACHINE	
	14.10.64. TON.	25.6.65 TON.	14.10.64 TON.	25.6.65 TON.
1000	22.4	23.2	22.3	23.5
1500	34.2	—	34.1	—
2000	46.1	47.7	46 2	47.5
3000	70.4	71.7	70.3	71.7
4000	95.0	95.9	94.7	95.4
5000	119.0	119.7	119.1	119.5
6000	142.4	142.7	142.7	142.7
7000	166.3	166.6	166.3	—

ALL RESULTS ARE MEAN OF THREE READINGS
MAXIMUM VARIATION 0·1% BETWEEN READINGS IN ONE CALBRATION.

FIG. 4.3.1.1. RESULTS OF CALIBRATION OF BIAXIAL COMPRESSION FACILITY.

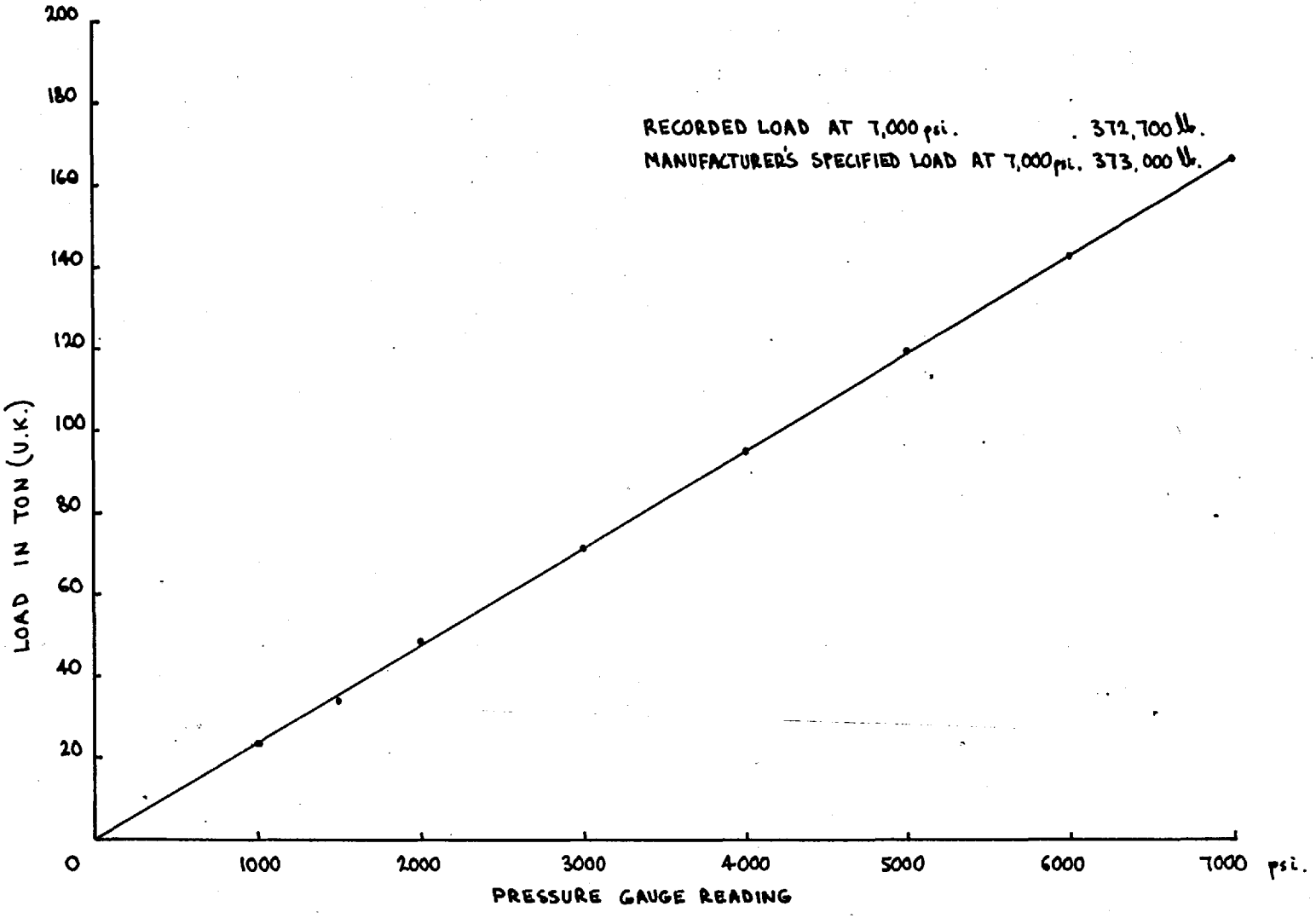


FIG. 4.3.1.2. CALIBRATION CURVE FOR COMPRESSION FACILITY.

4.3.2. Tension Facility

The only available proving device for the tension facility in the horizontal machine was a 10 ton proving ring No.197; the property of the Engineering Structures Department at Imperial College.

This required a 19" gap between pin centrelines and hence one of the tension calipers had to be removed from the machine for the calibration and replaced by the 5" link shown in fig.4.3.2.1.

The arrangement worked satisfactorily and after exercising six times, the load was ruh up to 10 tons and down again taking convenient intermediate readings. This procedure was repeated twice to give three sets of readings and the results are recorded in table 4.3.2.2. and plotted in fig.4.3.2.3.

Again a very linear load/pressure characteristic was obtained due to a large extent to the jacks selected for the task.

4.3.3. Torsion Facility

No basic jack calibration was required in this case as the existing compression jacks were to be used in their usual position for biaxial compression but with special equipment fitted (see fig. 10.2.1.2.).

However, the actual torque applied to the cylindrical specimen would be the equivalent effect from the vertical sub-frame jacks less the ball-race friction (see fig.10.2.1.1.).

Hence a series of tests were conducted in which for various longitudinal compression stresses on the cylinder, the force and hence torque needed to just turn the specimen was recorded using 1 cwt. spring balance.

The results are given in fig.4.3.3.1.

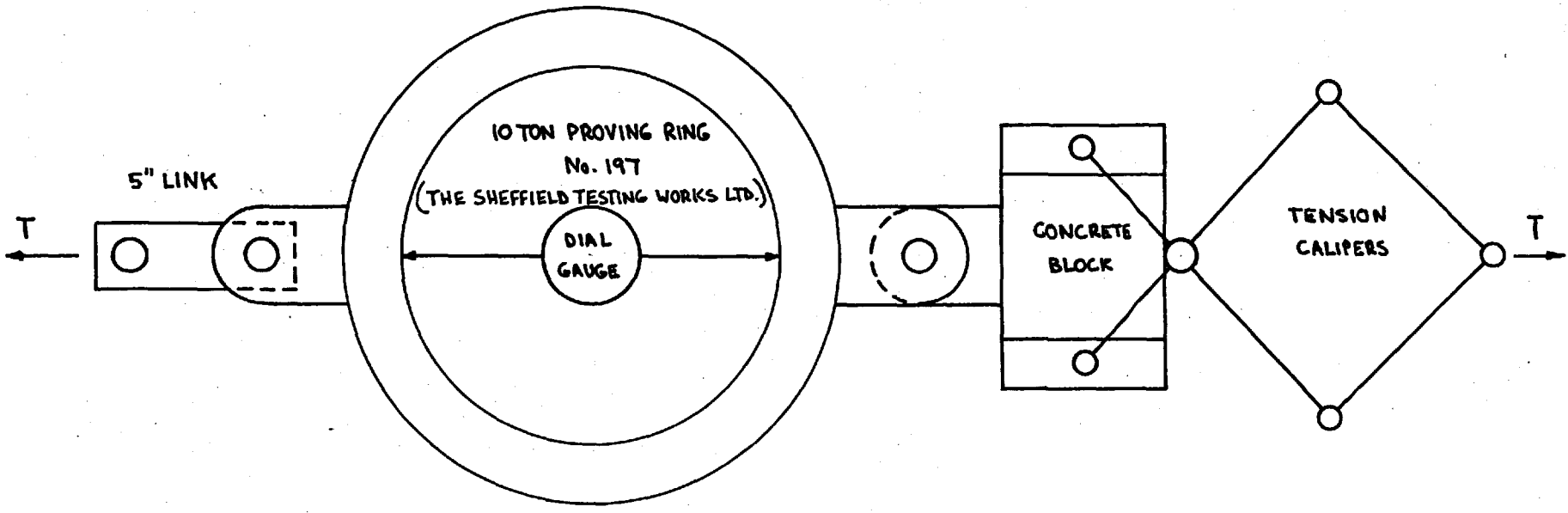


FIG. 4.3.2.1. DIAGRAMMATIC ARRANGEMENT OF TENSION FACILITY CALIBRATION IN HORIZONTAL MACHINE.

CALIBRATION OF TENSION FACILITY				
GAUGE PRESSURE psi	DIAL GAUGE READINGS			ACTUAL LOAD LB. $\times 10^3$
	1ST LOADING	2ND LOADING	3RD LOADING	
0	13000	13003	13003	0
50	009.5	009.5	009	0.160
100	019	020	020.5	0.430
150	031	033	033	0.790
200	044	045	045	1.090
300	068	068.5	068.5	1.720
400	092	092	092	2.360
500	116	116.5	116.5	3.000
600	140	140	141	3.700
800	190	190	189	5.040
1,000	236	236.5	237	6340
1,200	284	284.5	284.5	7.660
1,400	330.5	332	332	8.940
0	13003	13003	13003	

FIG. 4.3.2.2. RESULTS OF CALIBRATION OF TENSION FACILITY.

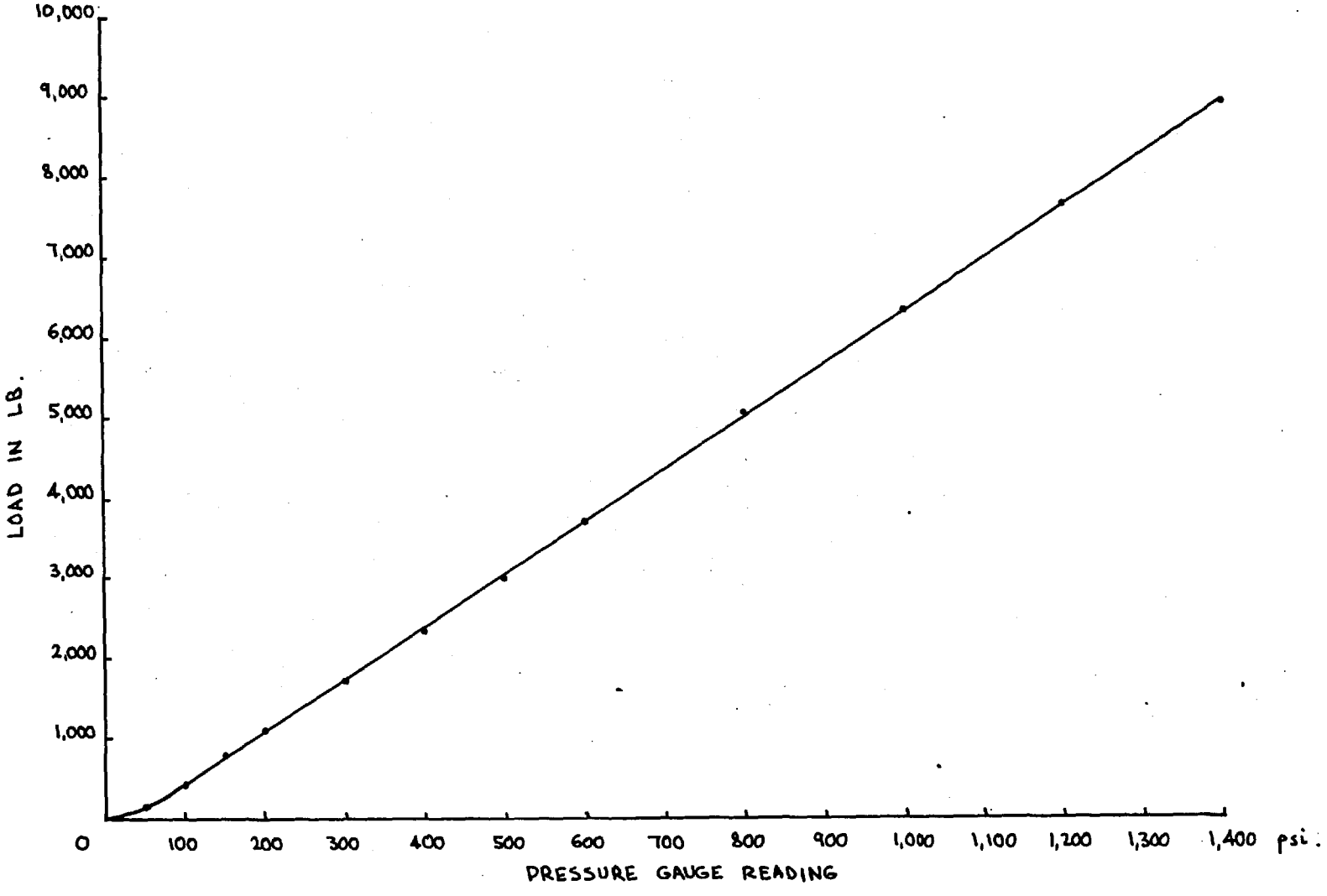
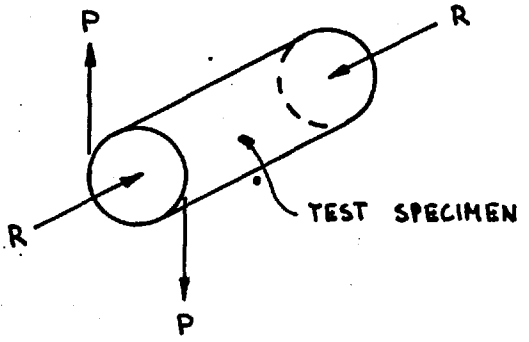


FIG. 4.3.2.3. CALIBRATION CURVE FOR TENSION FACILITY.

GAUGE READING R psi	TORSIONAL LOAD P LB.
100	29
200	52
300	84



TORSIONAL LOAD P HAS A LEVER ARM OF 7 in.

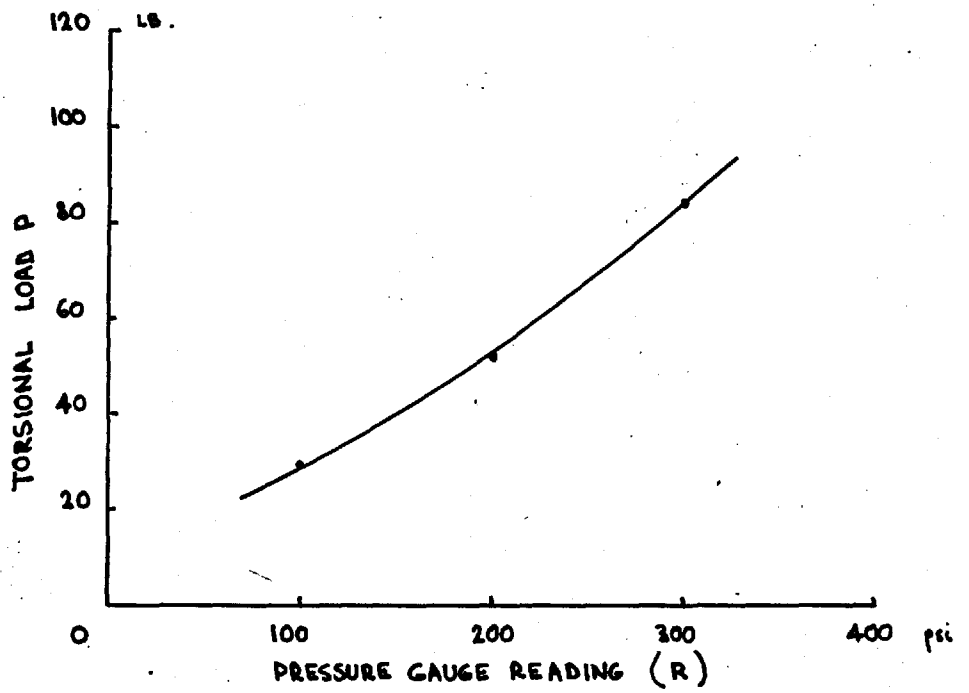


FIG. 4.3.3.1. TORQUE REQUIRED TO OVERCOME BALL-RACE FRICTION.

4.4. Biaxial Compression Specimen

These trials on the state of stress achieved in the biaxial compression specimen were particularly detailed as the biaxial compression tests were to be the main series in this research project. Furthermore the feasibility of obtaining a uniform state of stress i such a test had been questioned by some investigators[57] who considered that the influence of the platen restraint would spread right into the centre of the specimen.

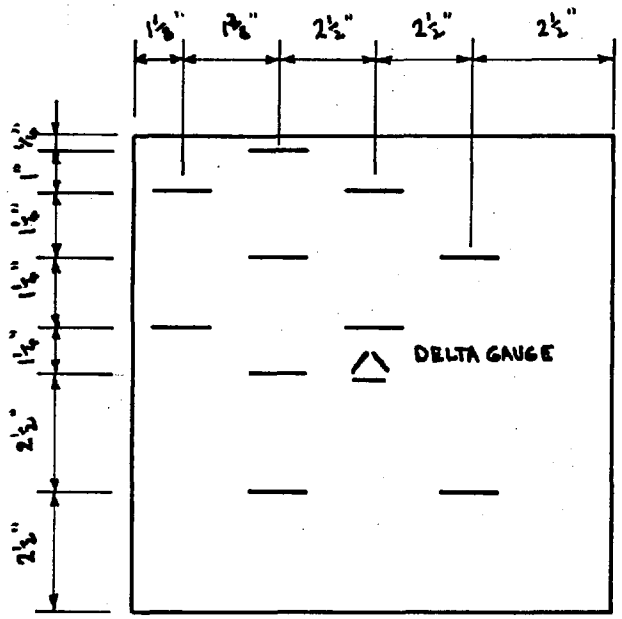
In these trials, mortar specimens were used for investigations into strain distribution because of their homogeneous nature whereas the more heterogeneous concrete mixes were used to determine the maximum coefficient of variation and the effect of order of loading. However two concrete specimens were used in a study of surface strain distribution to give a comparison with the mortar and the results are described in section 4.4.1.

4.4.1. Surface Strain by Strain Gauge

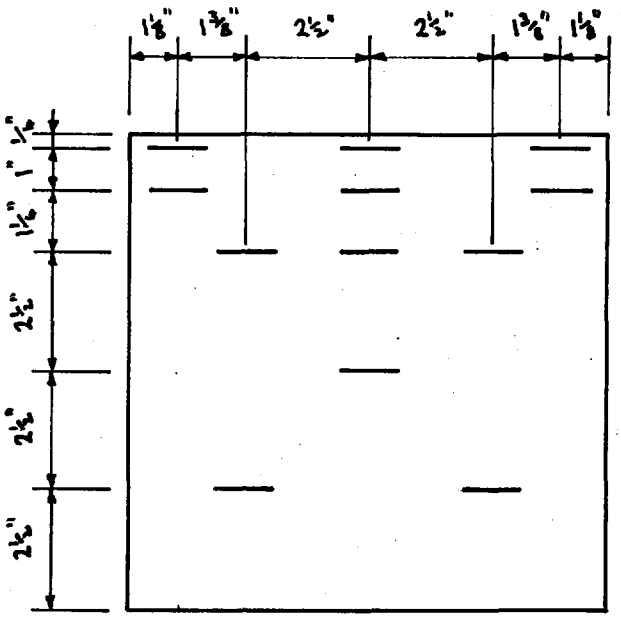
Surface strain measurements, though not the complete solution, are a very useful indication of the uniformity of a stress field. Any variations of stress within a continuous or even semi-continuous body will be reflected on the surface and these variations can therefore be picked up by surface strain measurements.

However, to ensure that all other eventualities have been dealt with, lateral strain measurements are used later to confirm the findings of this section.

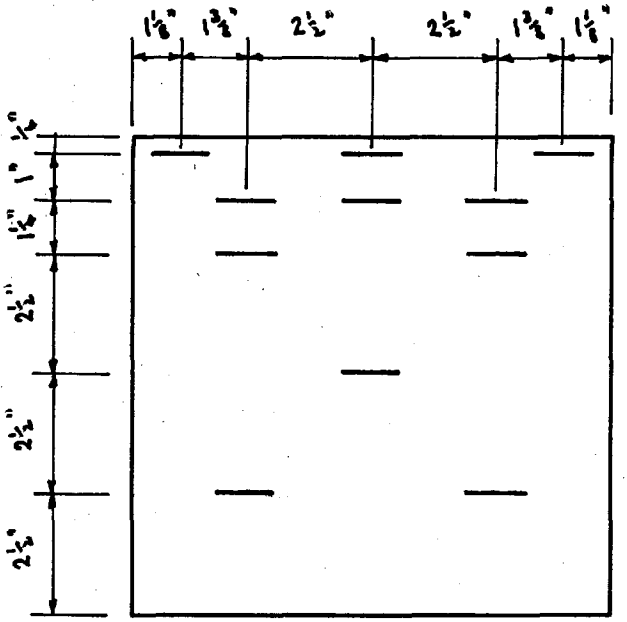
Three mortar specimens, illustrated in fig.4.4.1.1., were used within their "elastic" range (defined by the discontinuity stress) to investigate the surface strain distribution obtained for various combinations of load. The layout of the electrical resistance strain gauges was different in the case of all three specimens and a tabular summary of the positions used and the stress combinations investigated is given in table 4.4.1.2.



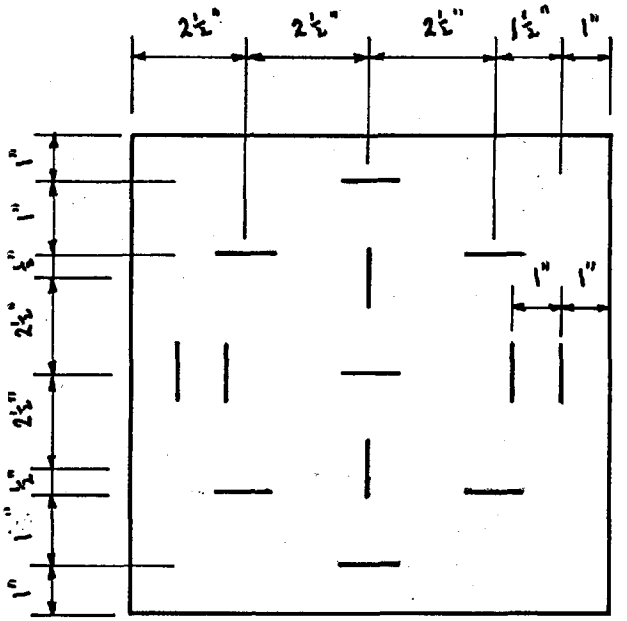
MORTAR SPECIMEN 'A'



MORTAR SPECIMEN 'B'



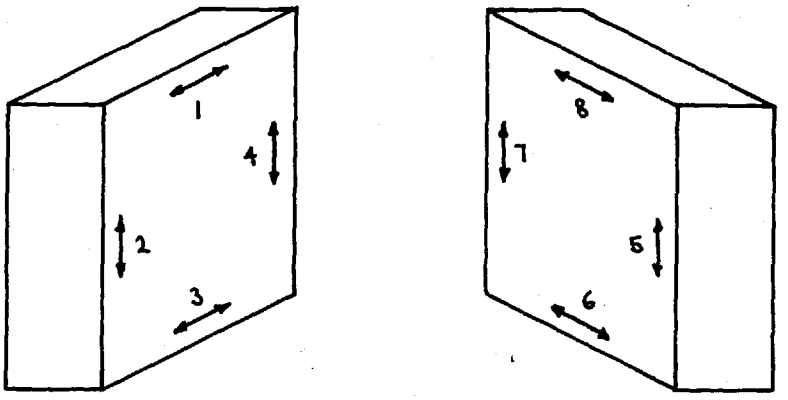
MORTAR SPECIMEN 'C'



CONCRETE SPECIMENS

FIG. 4.4.1.1. LAYOUT OF STRAIN GAUGES ON PROVING TRIAL SPECIMENS.

PROVING TRIAL SPECIMEN POSITIONS.



LABORATORY FACE

AIR CONDITIONER FACE

⟷ INDICATES DIRECTION OF MAJORITORY OF STAIN GAUGES AND THEIR BIAS.

SPECIMEN	σ_3 RANGE psi.	σ_2 psi	POSITIONS
A	267-1,333	0 267 667 1333	1 + 8
B	267-1,333	0 267 1333	1,2,3 + 8.
C	267-1,333	0 267 667 1333 2000	4 + 5
D	SEE FIG. 8.4.6.1.		ALL

FIG. 4.4.1.2. POSITIONS AND STRESS COMBINATIONS OF PROVING TRIAL SPECIMENS.

Although three different specimens from the same mix "MBC" were used the results were remarkably similar indicating the same minimum zones of uniform strain for each stress combination. Hence two typical results are given in fig.4.4.1.3. to illustrate how the strain dropped off outside these zones.

The zones of uniform strain were defined on those areas within which the whole length of a strain gauge yielded a strain reading within 5% of the strain recorded at the centre of the specimen. The strain readings from gauges extending outside this zone were rejected and hence the zonal limits tend to be conservative.

The results of all three specimens are summarised in the minimum zonal limits given in fig.4.4.1.4.

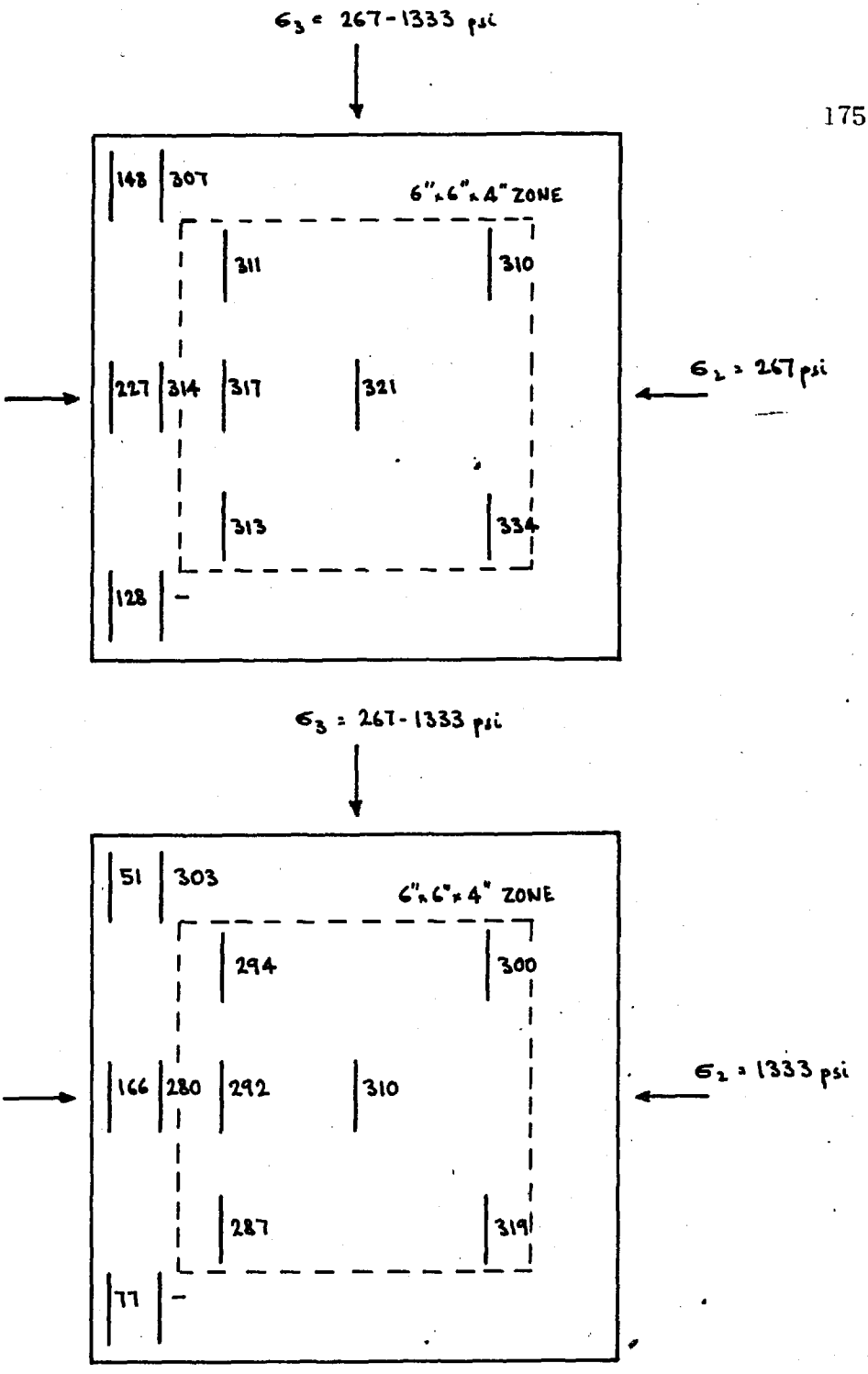
The two concrete specimens with strain gauge layouts given in fig.4.4.1.1. were used initially for measurements of the variation of elastic modulus with mean normal stress (Chapter 8) but the strain gauge positions were chosen such that the variation of strain up to failure on a $\frac{3}{4}$" aggregate concrete might be investigated.

The zonal limits of fig.4.4.1.4. were confirmed for concrete below the discontinuity point as defined by the ultrasonic pulse velocity and figs.4.4.1.5. and 4.4.1.6. show some typical strain distributions obtained at varying percentages of ultimate load.

4.4.2. Surface Strain by Photo-elastic Technique

The short-combings of finite length strain gauges to define zones of uniform strain were apparent in the preceeding section where the device of minimum zones had to be used.

Although it is doubtful whether reliable quantitative measurements could be obtained with the equipment used in this test because of the small strains to be measured, it was hoped that the qualitative pictures obtained would give a reliable definition of uniform strain zone limits - probably slightly larger than the minimum zones of fig.4.4.1.4.



ALL STRAIN VALUES ARE MICROSTRAIN (ie, STRAIN x 10^6)

FIG. 4.4.1.3. TYPICAL RESULTS FROM SPECIMEN B IN POSITION 2.

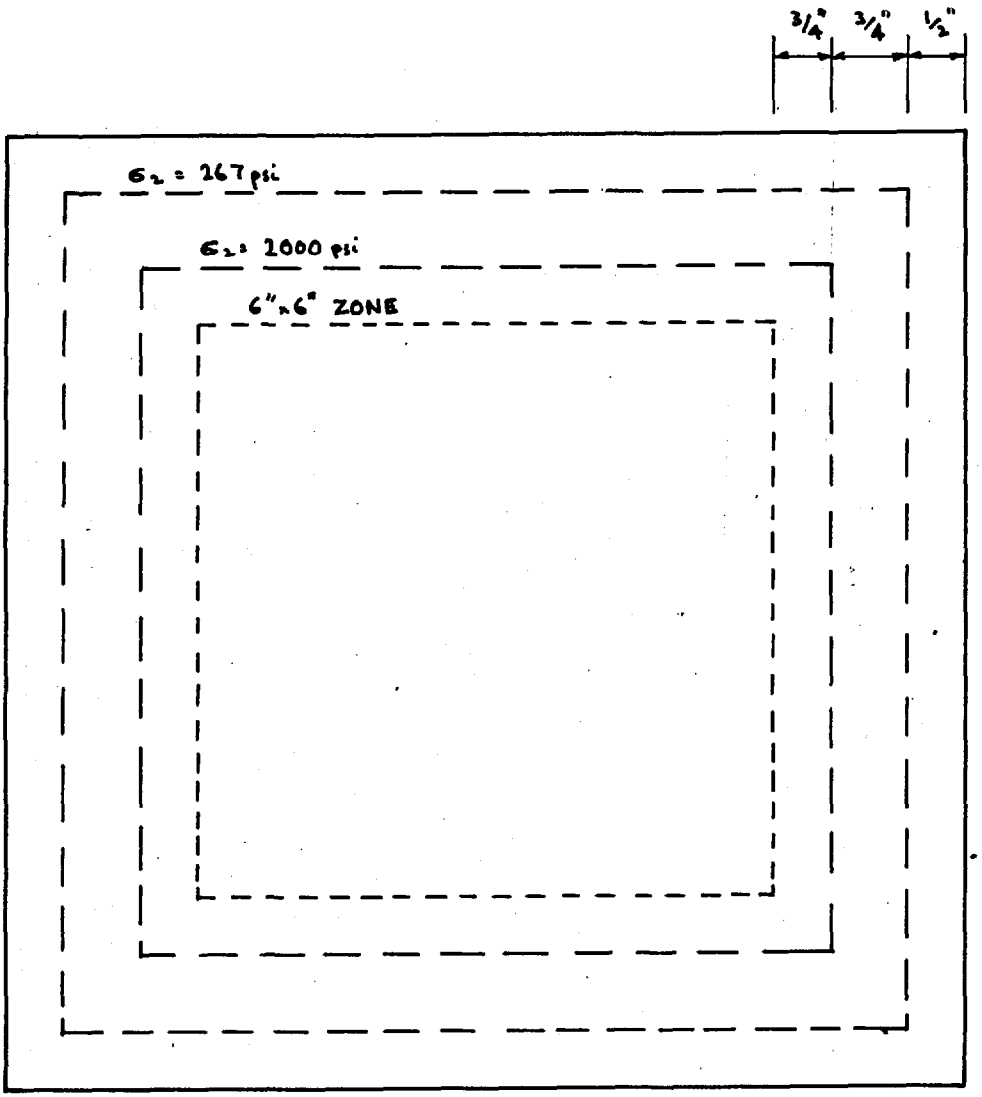
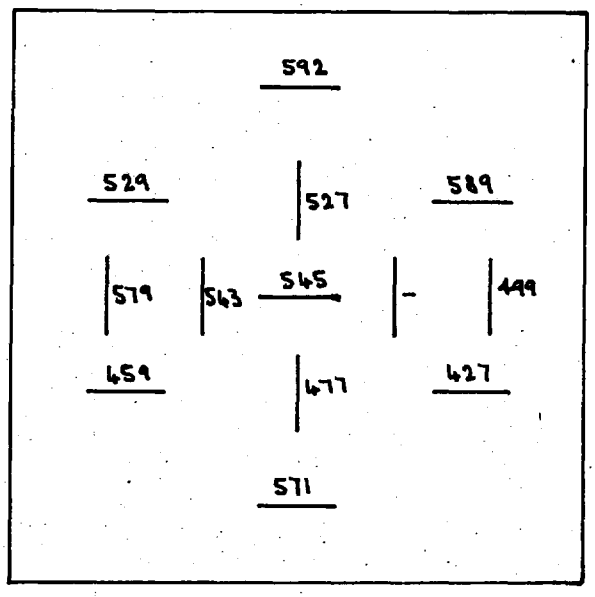
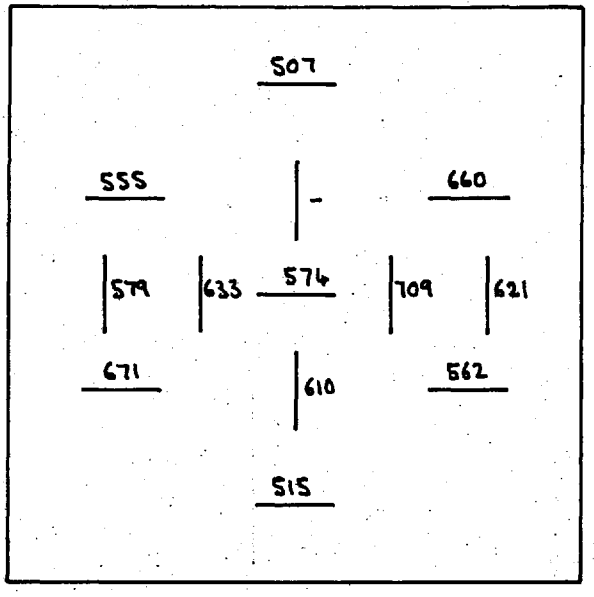


FIG. 4.4.1.4. ESTIMATED MINIMUM ZONES OF UNIFORM STRAIN FROM SURFACE STRAIN MEASUREMENTS.



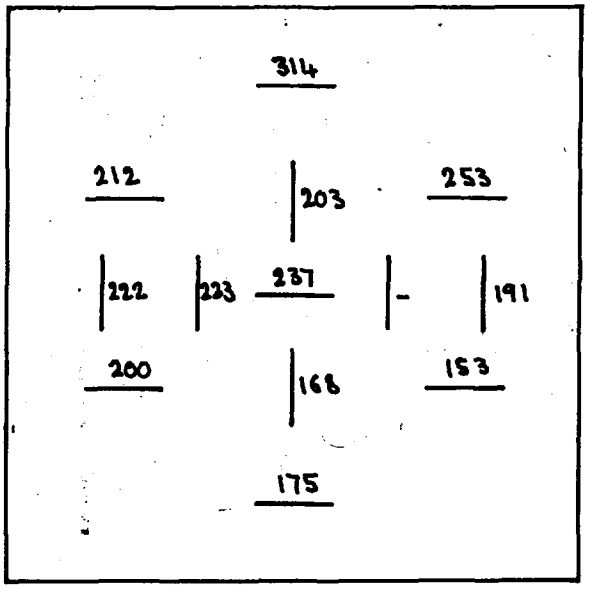
1) CAST FACE



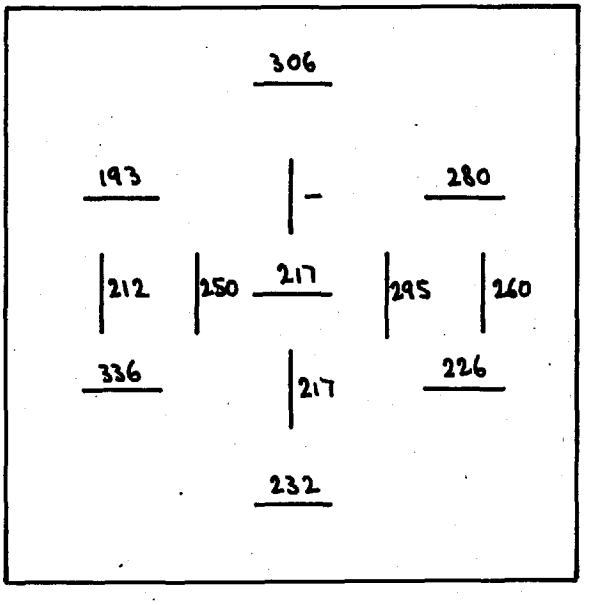
2) MOULDED FACE

NB FOR EXACT GAUGE POSITIONS SEE FIG. 4.4.1.1.

FIG. 4.4.1.5. TYPICAL STRAIN GAUGE READINGS ON CONCRETE SPECIMEN.
(MIX 61B - 3/4" MAX. AGGREGATE SIZE)
[PART 1 & 2 : $\sigma_1 = \sigma_2 = 2940$ psi]



1) CAST FACE



2) MOULDED FACE

FIG. 4.4.1.6. TYPICAL STRAIN GAUGE READINGS ON CONCRETE SPECIMEN.
(MIX 61B – 3/4" MAX. AGGREGATE SIZE)
[PART 2 of 2 : $\sigma_3 = \sigma_2 = 1333$ psi]

The apparatus used was the trial kit supplied by Messrs. Horstman of Bath, Somerset and consisted of a 10" x 10" sheet of birefringent material stuck onto a mortar specimen with a special reflecting cement. Light was beamed onto the sheet through the polariser of the precision hand viewer and the isochromatics were observed through the analyser. (140).

A range of different biaxial compression stresses (0-2667 p.s.i. in 667 p.s.i. increments) were investigated which were below the discontinuity stress and the zones of uniform isochromatic measured. Unfortunately the special photographic technique which had to be developed to take pictures of the actual patterns obtained was not developed in time for photographs to be included in this thesis, However, the measurements referred to above of uniform isochromatic zone are shown in fig.4.4.2.1.

From both the electrical resistance strain gauges and the photo-elastic measurements it would appear that a central volume of the test specimen measuring 6" x 6" x 4" is always under a uniform state of strain. Lateral strain measurements were included as an additional check as surface strain measurements are sometimes insensitive to variations in lateral strain especially if the zone indicated as uniform by surface strain measurements is small.

4.4.3. Lateral Strain

The linear displacement transducers and their supporting frame described in section 5.3. of the thesis were used in this trial.

The frame had been especially designed so that the transducers could scan the major portion of the test specimen across a diagonal and a mortar specimen was used with glass pads for the displacement transducers in the position shown in fig. 4.4.3.1.

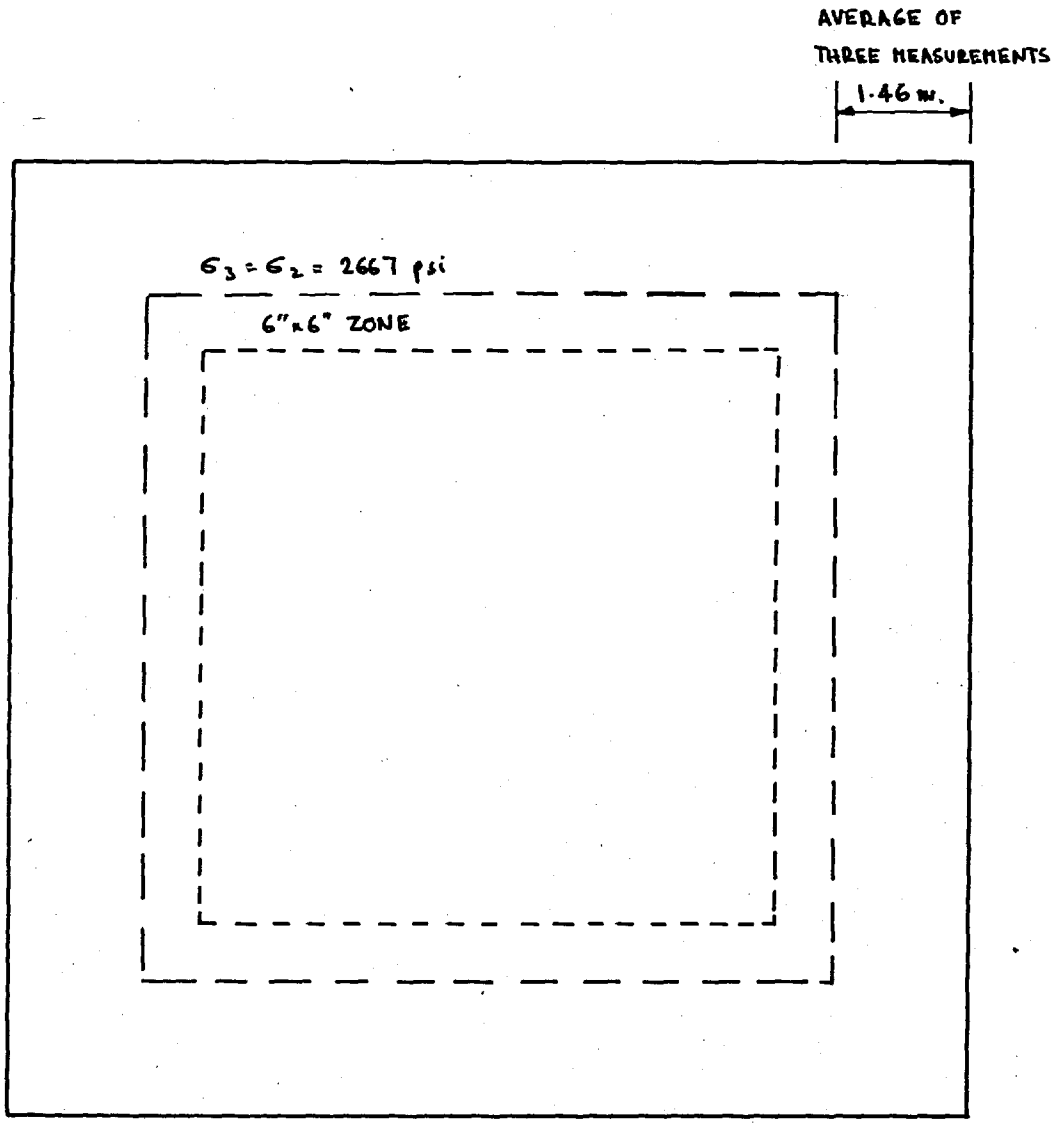


FIG. 4.4.2.1. ESTIMATED MINIMUM ZONE OF UNIFORM STRAIN FROM PHOTO-ELASTIC MEASUREMENTS.

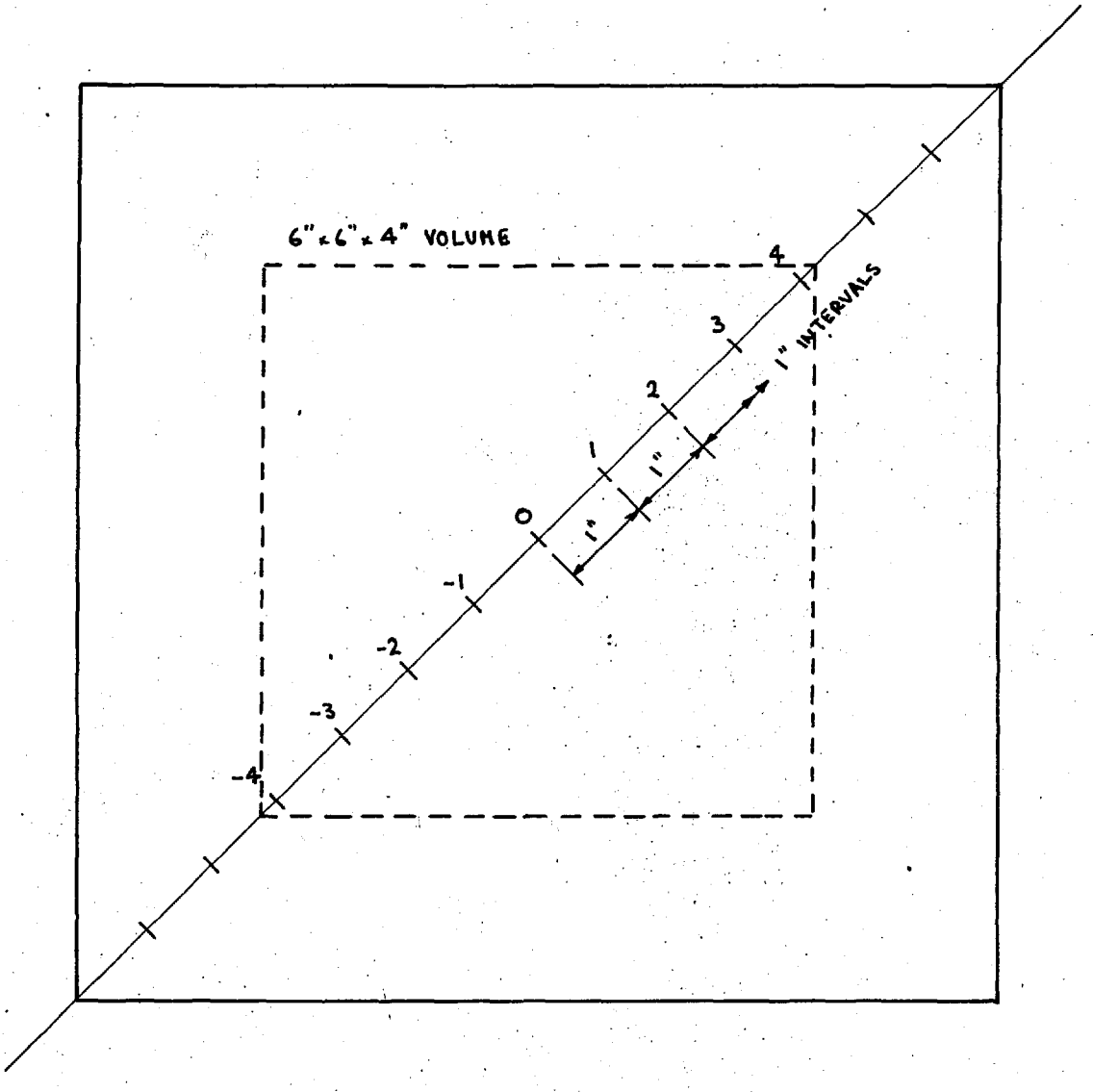


FIG. 4.4.3.1. POSITIONS OF LATERAL STRAIN MEASURING POINTS USED IN LATERAL STRAIN SURVEY.

At each point on the traverse, the specimen was first exercised over a range of 533 p.s.i. with different constant lateral pressures as shown in fig.4.4.3.2. The stresses chosen were convenient readings on the pressure gauge.

The results of a local exercise have been plotted in fig. 4.4.3.3. and the cumulative strains are plotted in fig.4.4.3.4.

From the results, it will be seen that an effectively uniform lateral expansion was achieved over the whole of the 6" x 6" x 4" uniform strain zone thus confirming the results of the surface strain trials.

4.4.4. Coefficient of Variation of Test Results

A problem with the biaxial compression series of tests is that, as the specimen size is large and many parameters require investigation to make the results meaningful, only a limited number of specimens can be cut from each mix to establish stress and strain relationships. Although a series of points as a curve have a statistical significance a single point by itself has no significance at all unless the variability of the results is known.

Hence six concrete specimens (mix 61B) were cut and tested to destruction to establish a coefficient of variation for both discontinuity and ultimate stresses and for surface strain measurements.

The experimental technique used was identical to that described in detail in Chapter 8.

A tabular summary of the stress results obtained and the coefficients of variation for both stress and strain measurements is given in fig.4.4.4.1. and the stress results are plotted in fig. 4.4.4.2.

POSITION	STRESS RANGE σ_3 psi	LATERAL STRAIN INCREMENT FOR VALUES OF σ_3 psi				TOTAL $\times 10^6$
		267	667	1333	2000	
4	533 psi FOR σ_3 = 667, 1333 AND 2000 267 psi FOR σ_3 = 267 BUT STRAIN INCREMENT = 2. MEAN STRESS OF RANGE = σ_3	32	31	34	34	131
3		44	37	33	30	144
2		48	41	37	33	159
1		46	39	38	33	156
0		48	41	36	33	158
-1						
-2		54	47	41	35	177
-3						
-4		50	46	41	36	173

FIG. 4.4.3.2. SUMMARY OF RESULTS OF LATERAL STRAIN PROVING TRIAL.

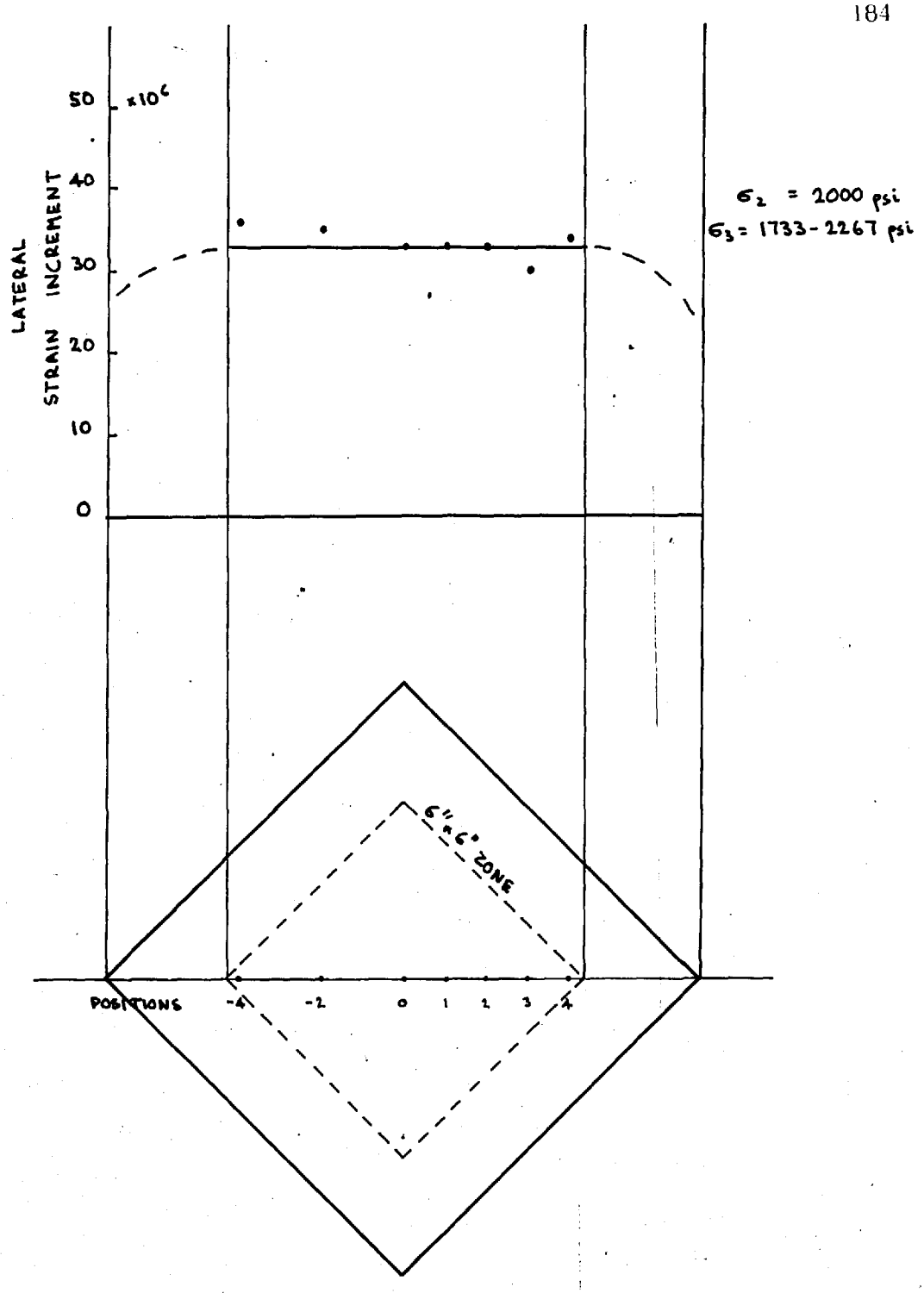


FIG. 4.4.3.3. VARIATION OF LATERAL STRAIN ACROSS TEST SPECIMEN.

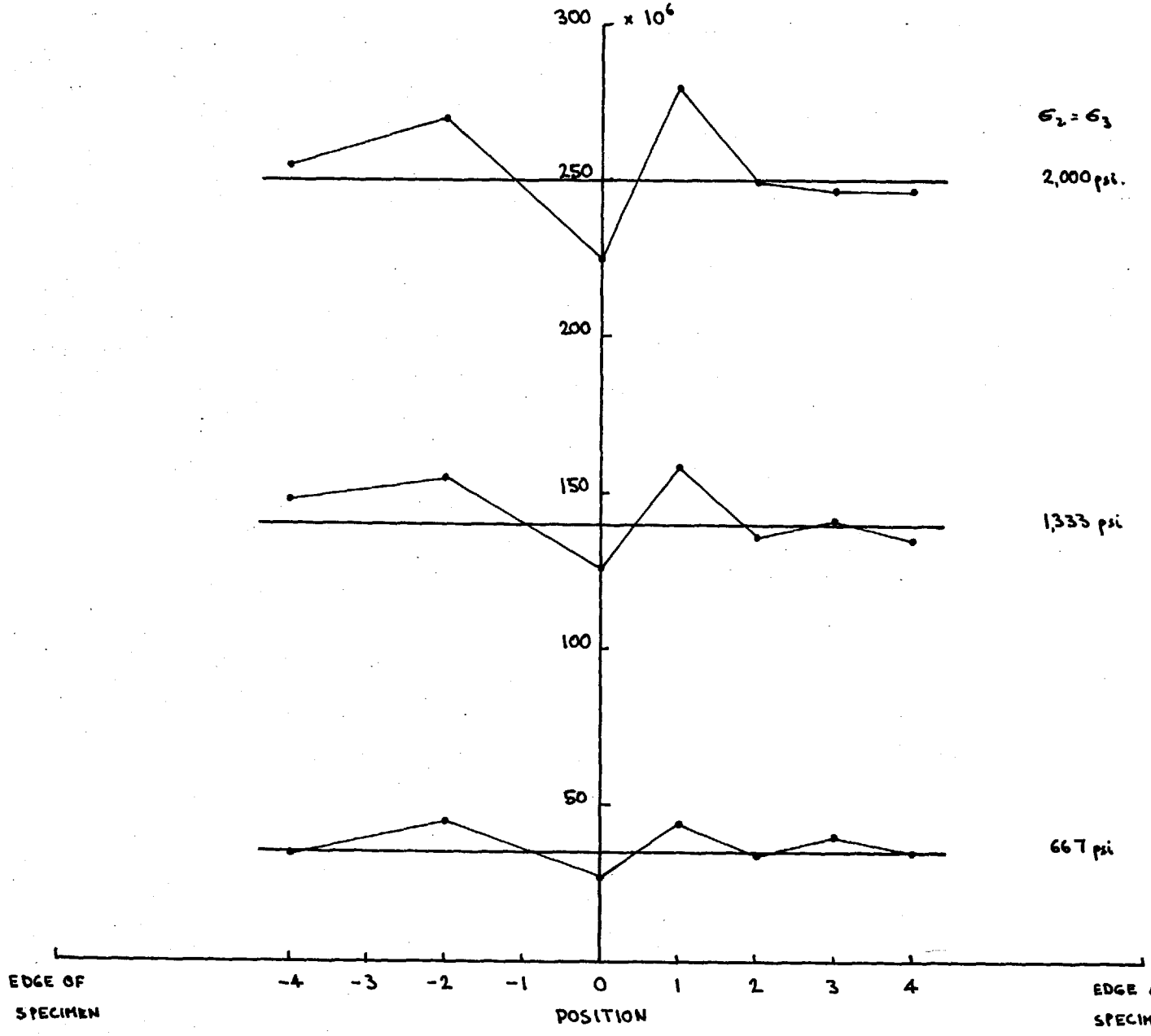


FIG. 4.4.3.4. CUMULATIVE LATERAL STRAIN.

SPECIMEN	LOAD STAGES	DISCONTINUITY STRESS σ_{pv} psi.	ULTIMATE STRESS σ_u psi.	SURFACE STRAIN @ $\sigma_2 = \sigma_3 = 267$ psi.	
				ϵ_3	ϵ_2
CV 1	CV1 - CV3 $\sigma_2 = \sigma_3$ IN 267 psi INCREMENTS	2540	5280	452	452
CV 2		3070	5230	484	484
CV3		3070	5280	484	484
CV4	CV4 - CV6 AS ABOVE UNTIL $\sigma_3 = 2667$ THEN σ_2 = CONST. σ_3 IN 267 psi INCREMENTS	3070	5540	564	486
CV5		2940	5540	682	467
CV6		2410	5670	543	-

COEFFICIENT OF VARIATION :-

1) DISCONTINUITY STRESS σ_{pv} . 8·2%.
2) ULTIMATE STRESS σ_u . 1·2%. BUT ASSUME 5%.
3) SURFACE STRAIN ϵ . 11·4%.

FIG. 4.4.4.1. RESULTS OF COEFFICIENT OF VARIATION TRIALS.

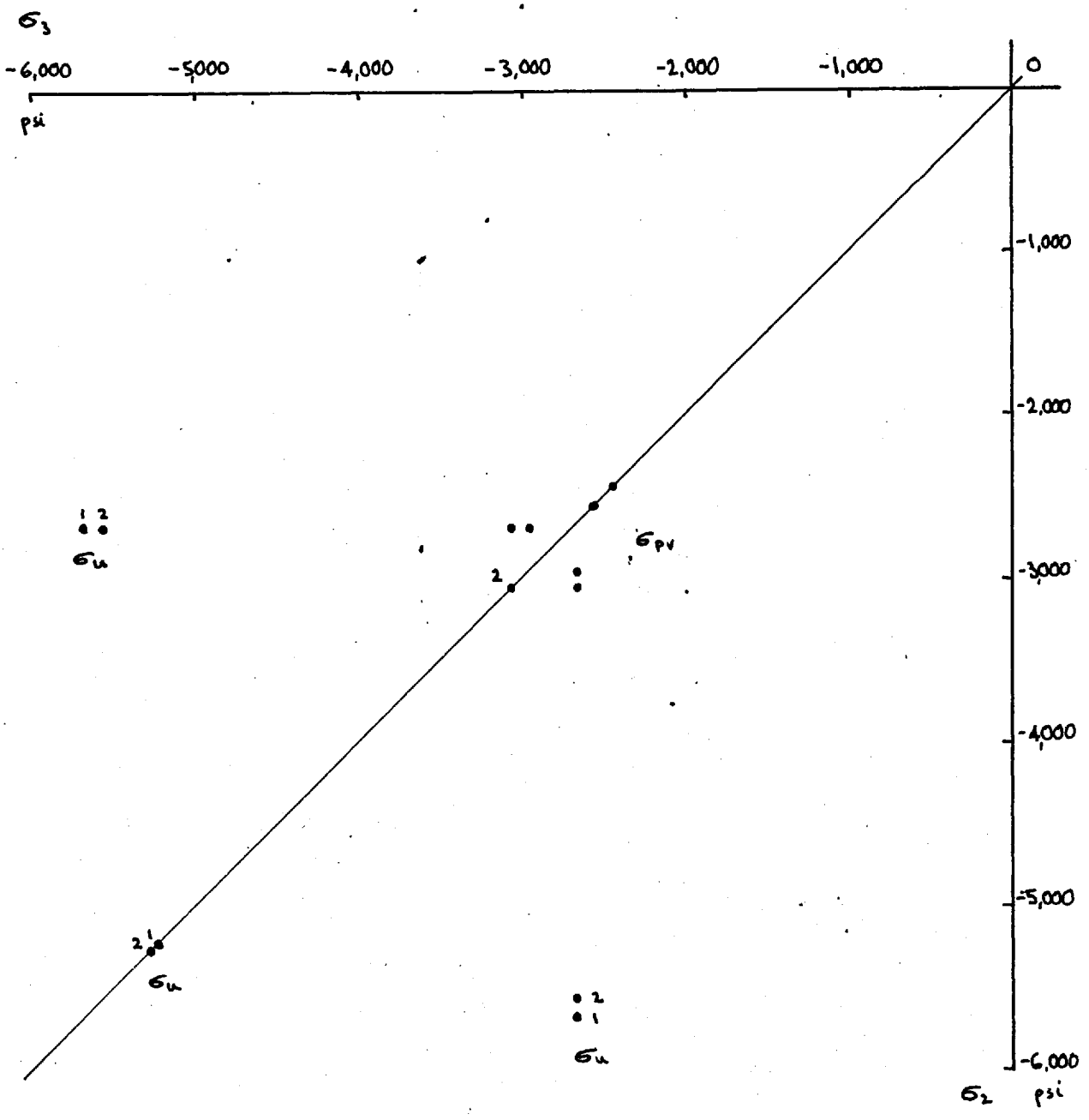


FIG. 4.4.4.2. STRESS RESULTS OF COEFFICIENT OF VARIATION TRIALS.

4.4.5. Order of Loading

The purpose of this trial was to discover whether the discontinuity or ultimate stress values obtained were dependent on the order in which the specimens were loaded.

Four concrete specimens (61B mix) were cast and the four 'routes' or loading sequences A B C and D were arranged such that the phenomenon was investigated on the mid-range of the stress characteristic ($\sigma_3 = 0.5\,\sigma_2$) and three results were obtained at each of one point on the discontinuity and ultimate load curves (fig.4.4.5.1.). The slope in the D route was to avoid passing outside the discontinuity envelope under uniaxial stress at point x.

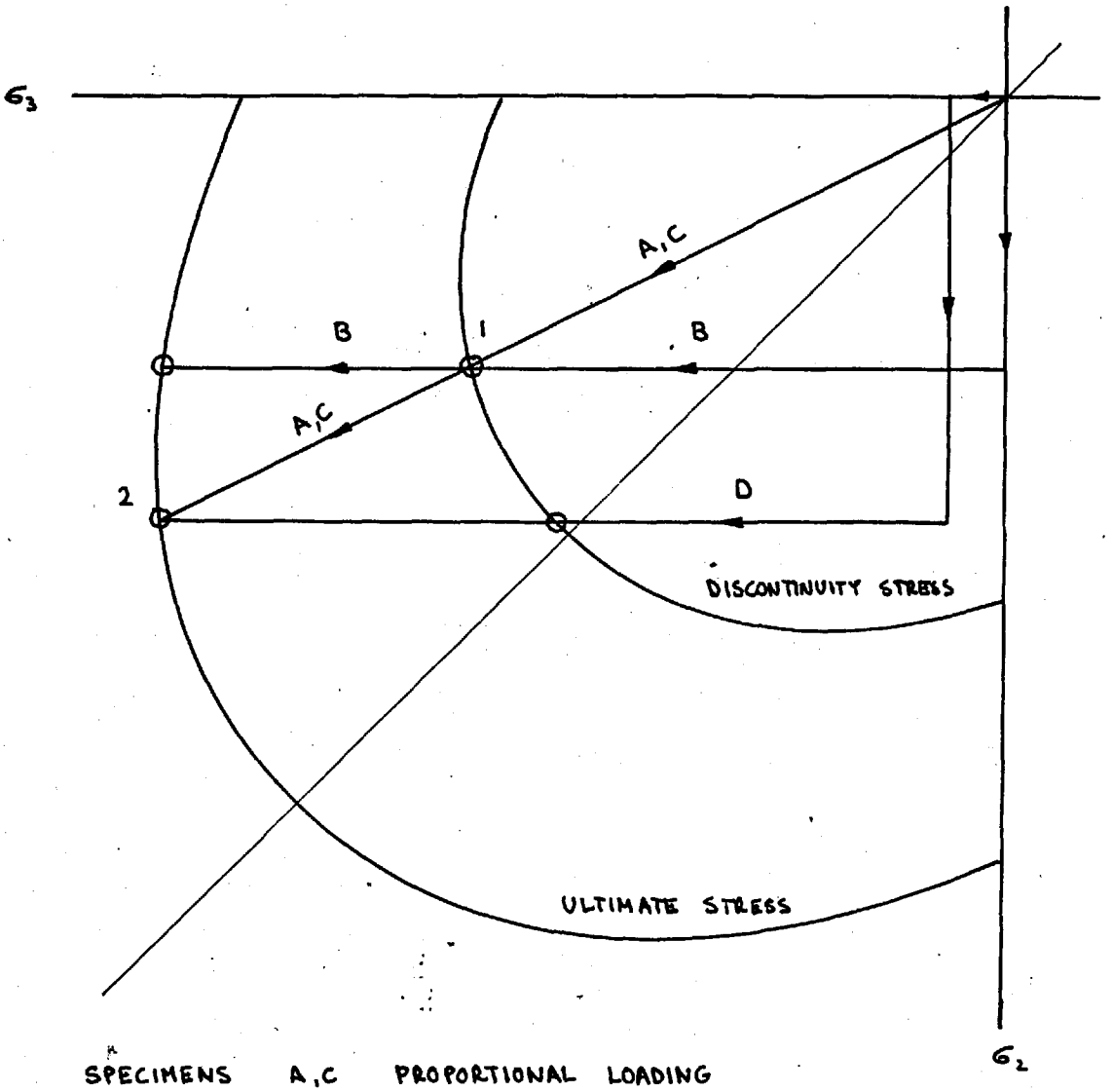
Strain readings were also taken in accordance with the standard testung procedure described in Chapter 8.

The stress and strain values obtained from these tests are given in fig.4.4.5.2. and the stress results are plotted in fig.4.4.5.3.

The results confirm the claims of Sundara Rajn Iyengar et al[55] that the ultimate stress values obtained in biaxial compression tests are independent of the order of loading and it is interesting to note that a similar claim was made by McHenry and Karni[124], and Bresler and Pister[45] for states of biaxial compression-tension.

Although the resulting co-incidence of results was to be expected at the discontinuity stress as the material behaves 'elastically' up to this point, the ultimate load result is surprising and, as other workers have obtained the same result, suggests that crack history is not as critical as has been supposed; at least under these states of stress.

Other results obtained later in the main series of biaxial compression tests only confirm this result and the smoothness of the stress curves obtained with a mixture of sequence and proportional



SPECIMENS A,C PROPORTIONAL LOADING
SPECIMENS B,D SEQUENCE LOADING.

STRESSES AT POINTS 1 AND 2 DETERMINED BY A and C
AND THEN CHECKED BY B and D.

FIG. 4.4.5.1. ORDER OF LOADING TRIALS.

SPECIMEN	DISCONTINUITY STRESS psi		ULTIMATE STRESS psi		DISCONTINUITY STRAIN $\times 10^6$		
	σ_3	σ_2	σ_3	σ_2	ϵ_3	ϵ_2	ϵ_1
A	-4810	-2410	-7550	-3880	-1170	-321	+300
B	-4810	-2410	-7480	-2410	-999	-319	+197
C	-4540	-2270	-7680	-3880	-968	-230	+265
D	-5350	-3880	-7750	-3880	-1242	-718	+512

FIG. 4.4.5.2. SUMMARY OF RESULTS OF ORDER OF LOADING TRIALS.

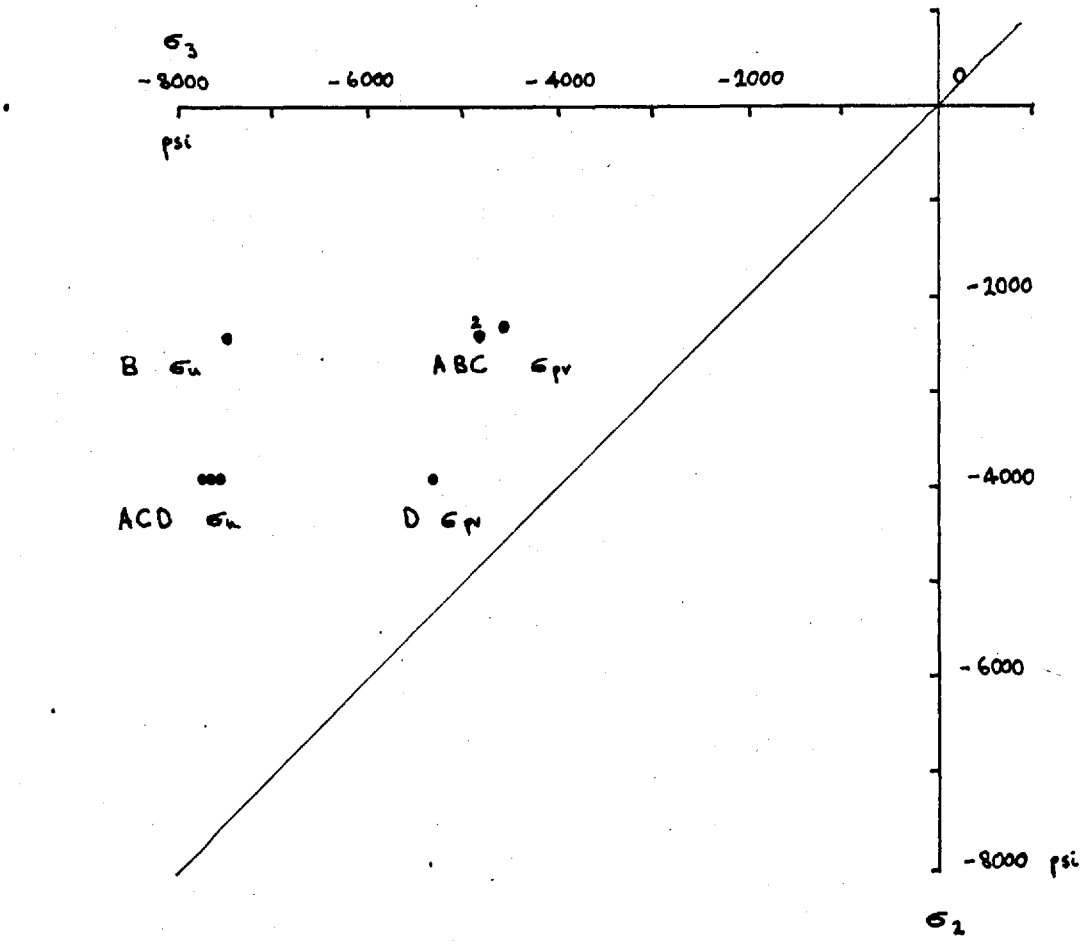


FIG. 4.4.5.3. STRESS RESULTS OF ORDER OF LOADING TRIALS.

loading is additional evidence - both on the independence of the results in order of loading and the freedom from machine effects achieved with this machine.

4.4.6. State of Stress on Central Volume

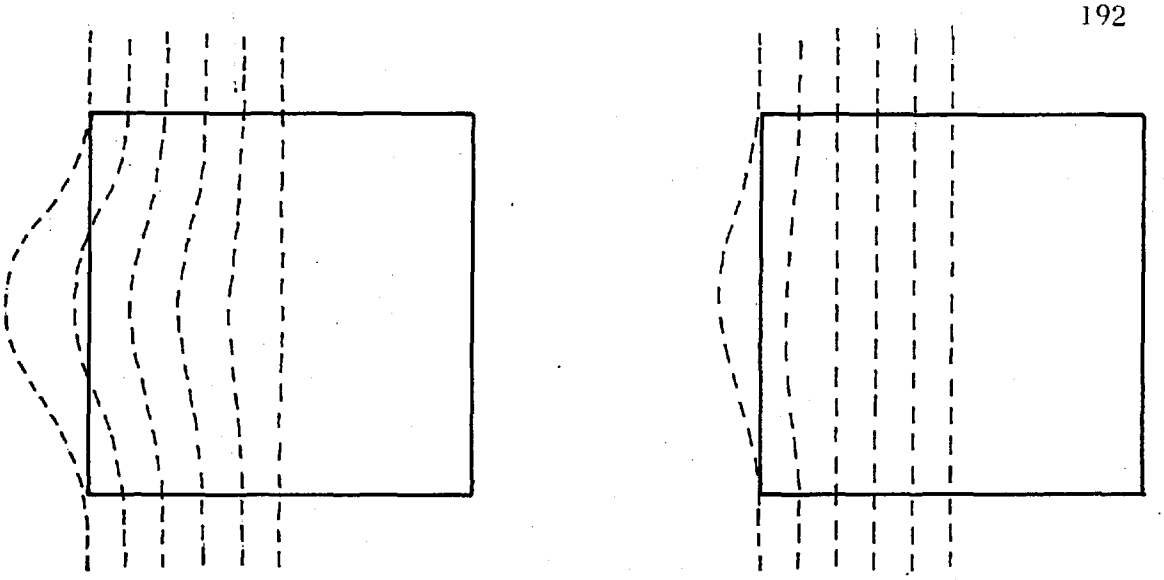
In the review of previous research in section 1.3., the work of Hilsdorf[57] was mentioned (section 1.3.21) in which it was claimed that the use of solid loading platens such as have been used in this machine meant that the stress on the uniformly strained central volume was always unknown and in any case less than the average applied stress.

The results of the proving trials described above dispute this claim. The fact that such a large proportion of the total volume is subjected to a uniform pattern of surface and lateral strain means that the central set of stress trajectories could not be deflected by the effect of the side platens otherwise a strain difference would exist right across the central zone. This argument is illustrated in fig.4.4.6.1. in which in one instance the trajectories are deflected and in the other they are not: in both cases the influences needed to cause this state to arise and the resulting surface strain patterns are shown diagrammatically.

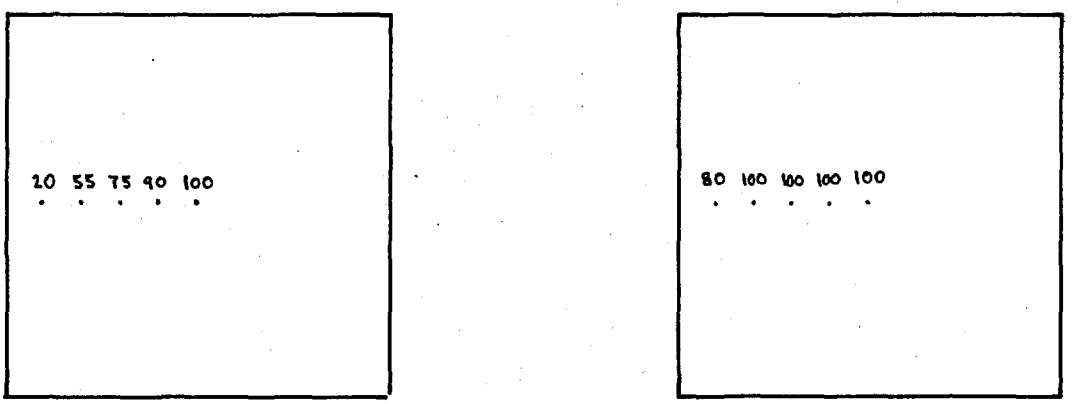
It is concluded that the stress on the central uniformly strained volume must be equal to the average applied strain if the strain patterns on the specimen are as demonstrated in the proving trials described above.

4.5. Biaxial Compression-Tension Specimen

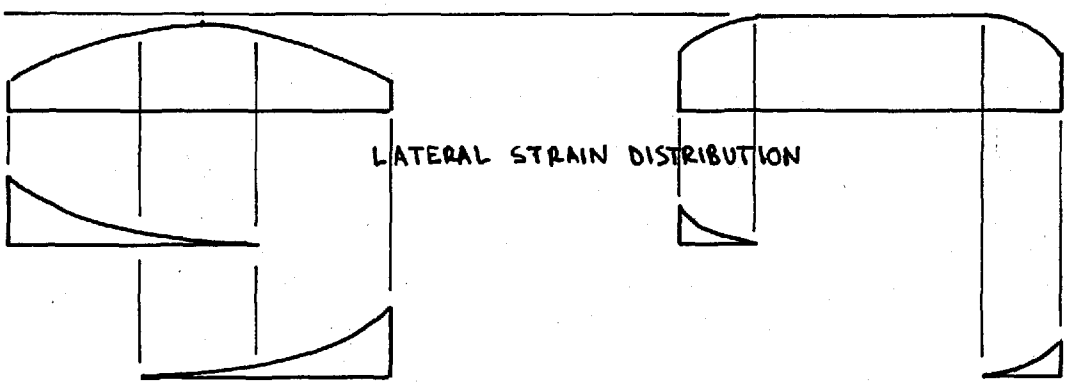
Direct tension tests for concrete have always been very difficult to achieve and hence difficulties in developing a satisfactory direct-stress biaxial compression-tension test are to be expected.



STRESS TRAJECTORIES



SURFACE STRAIN DISTRIBUTION



LATERAL STRAIN DISTRIBUTION

PLATEN INFLUENCES

a) HILSDORF SITUATION

b) UNIFORM CENTRAL VOLUME

FIG. 4.4.6.1. COMPARISON OF LOADING PLATEN EFFECTS ON STRESS AND STRAIN PATTERNS.

The use of tension grips developed by Ward[7] and the attendant modifications required meant that a first priority was to check the state of stress under uniaxial tension. This was a particularly necessary measure as the tension grips were used horizontally in the combined stress testing machine whereas they were used vertically in Ward's tension machine. The cube effects in the compression facility were to be investigated next and lastly, the combined strain effects of the two facilities working together were to be measured.

4.5.1. Direct Tension Facility

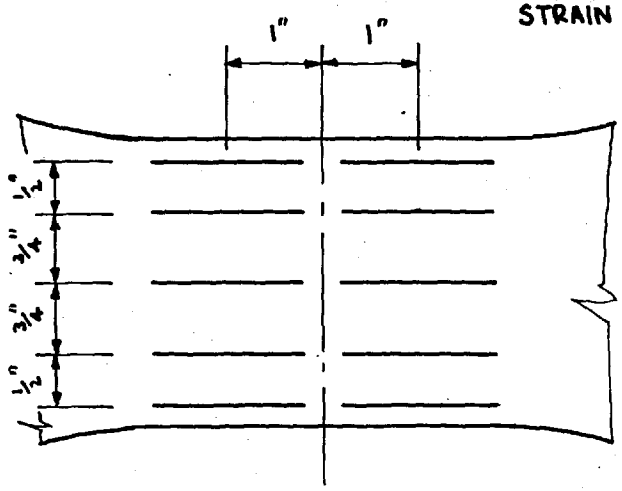
Three standard tension specimens were cast in mortar (MBC) and had strain gauges fixed to them in the positions shown in fig.4.5.1.1.

Then tensile stresses were applied to the specimens up to values just below the discontinuity stress and the strain distributions recorded.

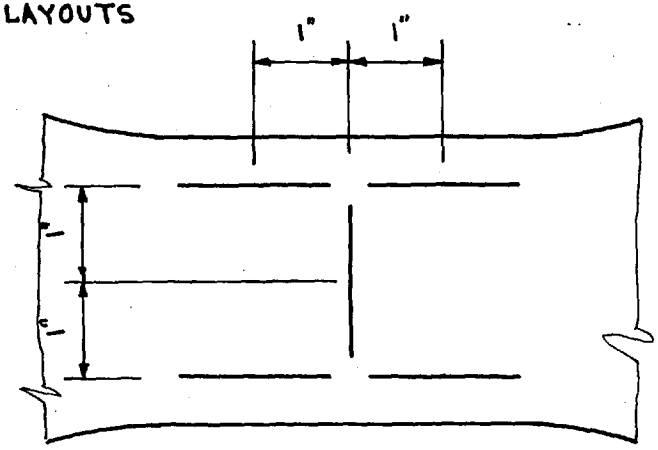
Some typical results are given in fig.4.5.1.2. and a summary of the maximum strain variations are given in fig.4.5.1.3.

It will be observed from the results that the strain distribution is slightly affected by the subsequent set-ups and that it is not as good as that obtained by Ward in his proving trials on the same grips used in a vertical machine. However, the strain distribution is very close to pure tension ($\pm$ 3%) which should be satisfactory. The author was able to obtain uniaxial tension ultimate stress results with an average coefficient of variation of 6.8% with this arrangement whereas the average coefficient of variation that he obtained on Ward's machine was 7.7%. This may have been due to operator tecnique differences between the two machines and the fact that the combined stress testing machine was much stiffer and had better hydraulic pressure controls. However, these considerations indicate that the uniaxial tension facility in the testing machine operated satisfactorily in the horizontal position.

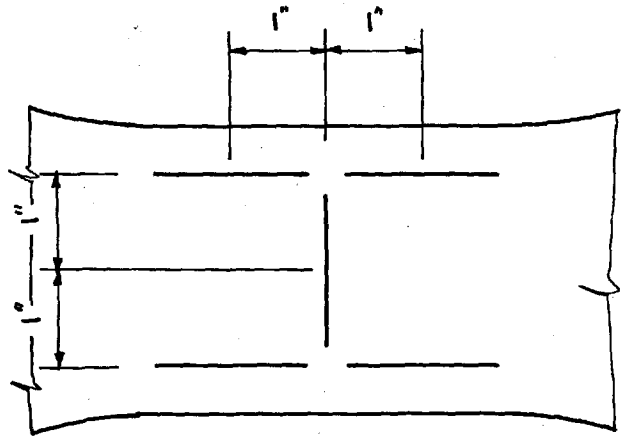
STRAIN GAUGE LAYOUTS



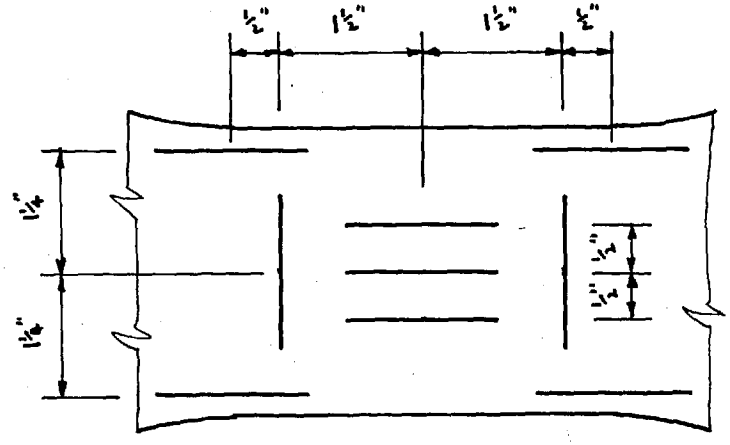
SPECIMEN A
(DIRECT TENSION)



SPECIMEN B
(DIRECT TENSION)

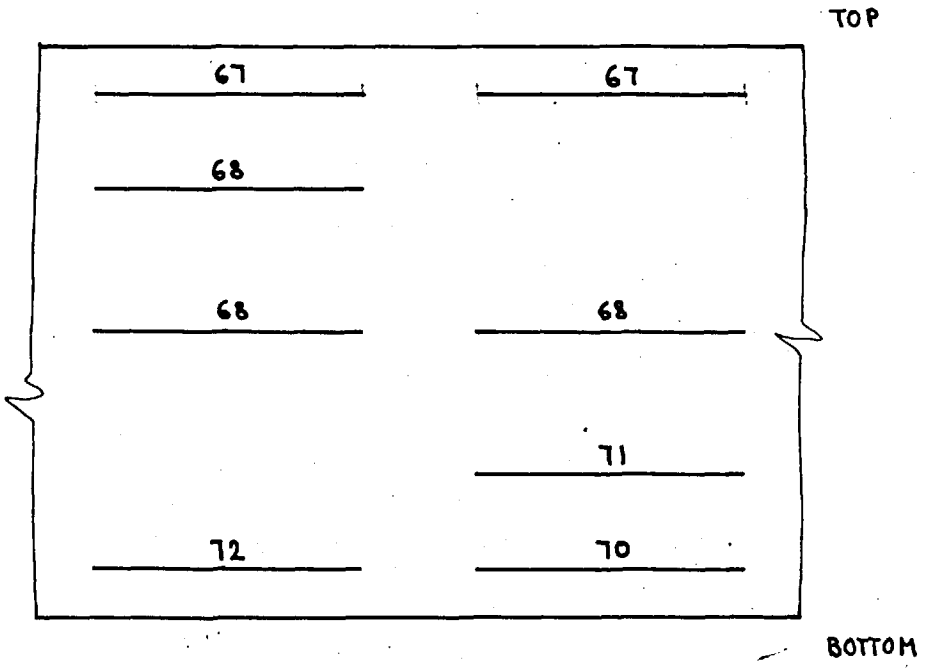


SPECIMEN C
(DIRECT TENSION AND COMPRESSION)



SPECIMEN D
(BIAXIAL COMPRESSION-TENSION)

FIG. 4.5.1.1. COMPRESSION-TENSION FACILITY PROVING TRIAL SPECIMENS



SPECIMEN A TEST No. 3.

(ALL FIGURES ARE IN MICROSTRAIN)

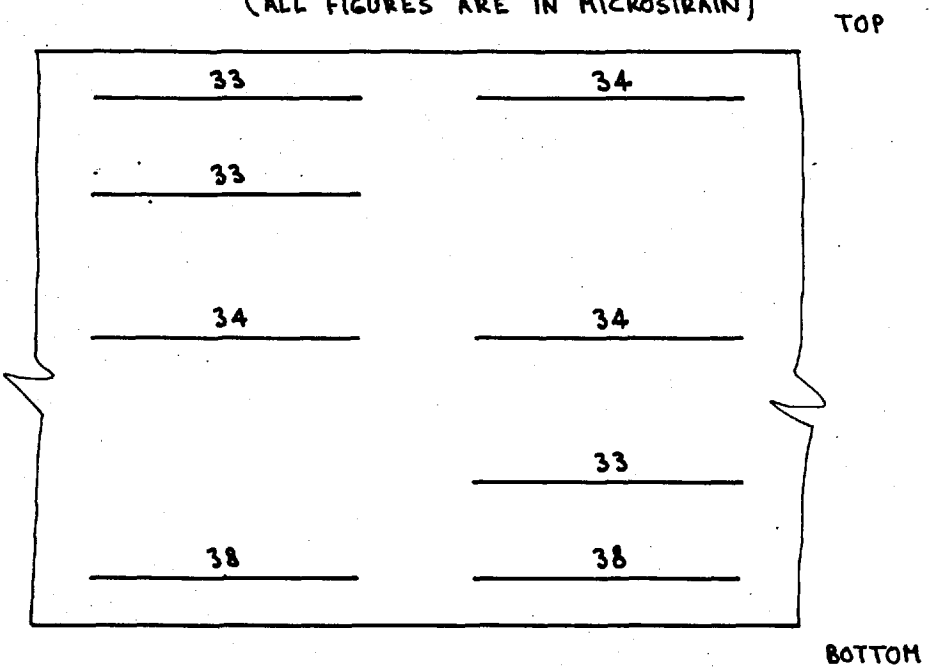


FIG. 4.5.1.2. TYPICAL STRAIN DISTRIBUTIONS OBTAINED IN DIRECT TENSION ON CENTRAL 4"x3" ZONE.

SPECIMEN	VARIATION OF STRAIN ON 4"×3" FACE	
	ONE SET-UP	BETWEEN SET-UPS.
A	5%	1%
B	6%	1%
C	4%	1%.

FIG. 4.5.1.3. VARIATION OF TENSILE STRAIN DISTRIBUTION UNDER DIRECT TENSILE STRESS.

4.5.2. Combined Compression-Tension Facility

In these tests only one specimen was used with the gauge layouts shown in fig.4.5.1.1.

Tensile stresses were varied for given compressive stresses resulting in the distributions shown in fig.4.5.2.1. Note that both the best and the worst distributions have been recorded to show the results that can be achieved with this technique and at the same time, the undesirable concentrations that can arise.

The net result of these tests is that the facility can give very good results but that very careful setting up of the two compression cubes is essential and that the same degree of reliance cannot be placed in the results obtained as can be with results of biaxial compression tests. The results obtained were more consistent if the compression stress was applied before the tension stress. Furthermore considerable care is required in the initial alignment of the tension jacks so that the tension rods are able to move freely through the brass bushes in the central hole in the ram.

4.6. Biaxial Compression-Torsion Specimen

The biaxial compression-torsion specimen and the test set-up in the testing machine are described in detail in Chapter 10.

The purpose of this test was to investigate the distribution of surface strain on the outside of the test specimen as a measure of the effect of the thickened ends of the specimen and to see whether the central third of the specimen was in a uniform state of strain.

A simple mortar (MBC mix) specimen was used for these tests in which for a series of constant axial compressive stresses, the magnitude of the torsion stress was varied to study the resultant incremental change in surface strain.

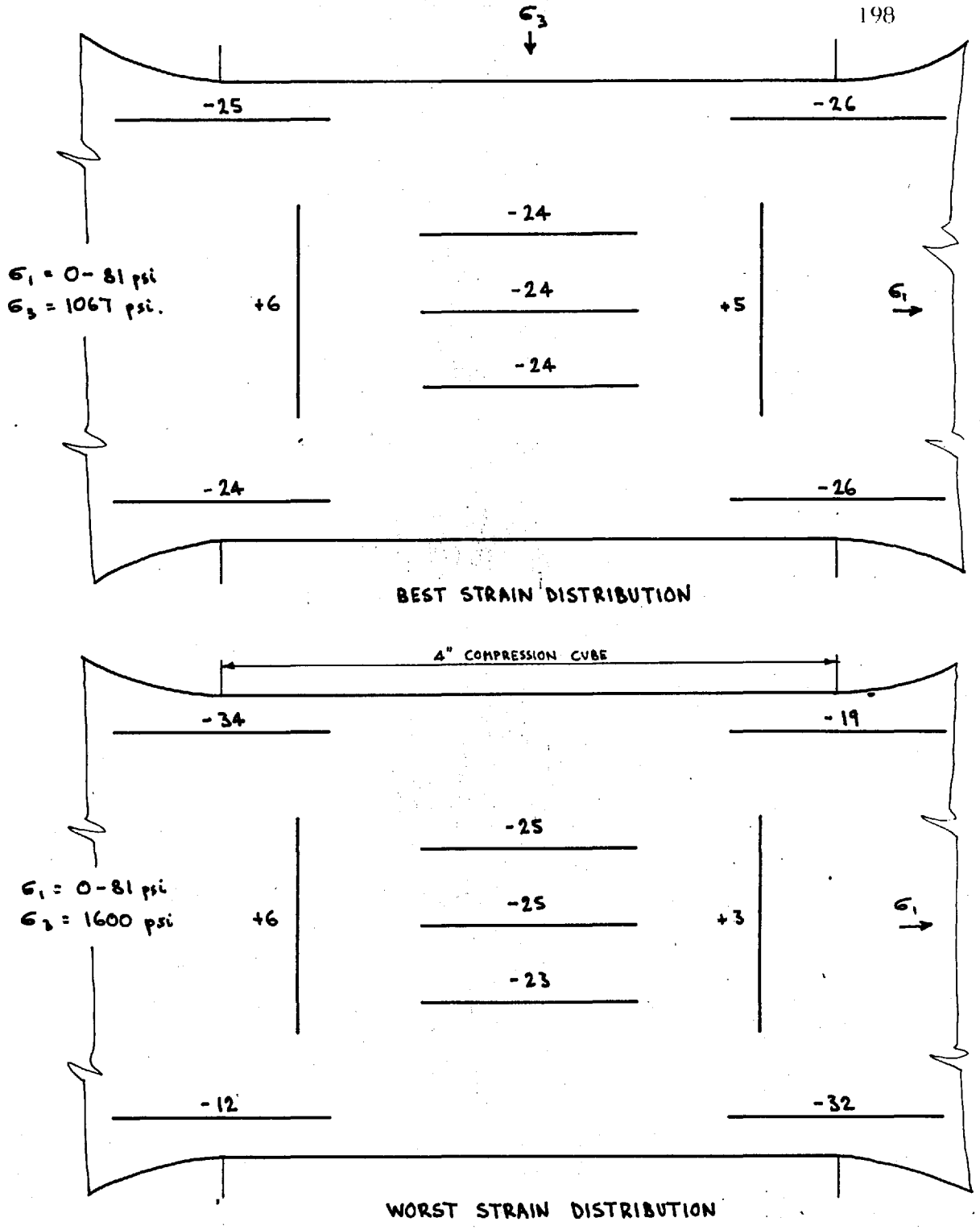


FIG. 4.5.2.1. COMPARISON OF THE BEST AND WORST STRAIN DISTRIBUTIONS OBTAINED FROM SPECIMEN D IN BIAXIAL COMPRESSION-TENSION.

The location of the electrical resistance strain gauges and the details of the loading sequence are given in fig.4.6.1.1. whilst the resulting strain distributions are shown in fig.4.6.1.2.

This, and several other, test specimens were loaded to failure so that the failure crack-patterns might be studied as a further item of evidence on the stress-strain distributions achieved using this testing technique. A change of failure mode from clearage under pure torsion to splitting with increasing compressive stress was noted. The failure in all cases appeared to initiate at the centre section and seemed quite unaffected by the clamped ends of the test specimen (fig.4.6.1.3.).

These results indicate that this testing facility operated satisfactorily but some thorough trials are recommended prior to any test series in which measures should be made of the surface strain distribution in more detail and the lateral (or radial) strain distribution if possible.

4.7. Hydraulic Central System

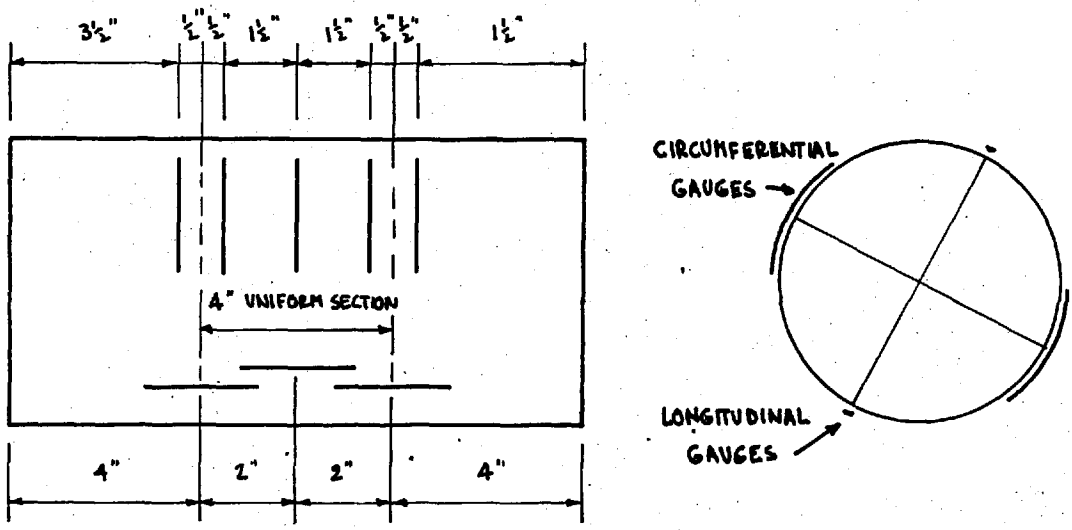
This faculty of the machine does not lend itself to the formal type of proving trial described in the section above but is more in the nature of an operators report of service over a period of time - in this case nine months.

The serviceability of all components was good except for the Ermeto stop valves which either leak or, when the gland is tightened down, are very stiff in operation. It is recommended that these be replaced by stop valves manufactured by I.V. Pressure Ltd.

A lesser criticism is the operation of the needle valves at high oil pressures, say above 4,000 p.s.i. where it is difficult to control the pressure to within 20 p.s.i. though the action is smooth and steady. The indications are that the needle is adequate for the job but some gearing down on the spindle, say about 5:1 would be advantageous. Alternatively, as only two bypass valves are involved, a more elaborate control valve could provide a better answer.

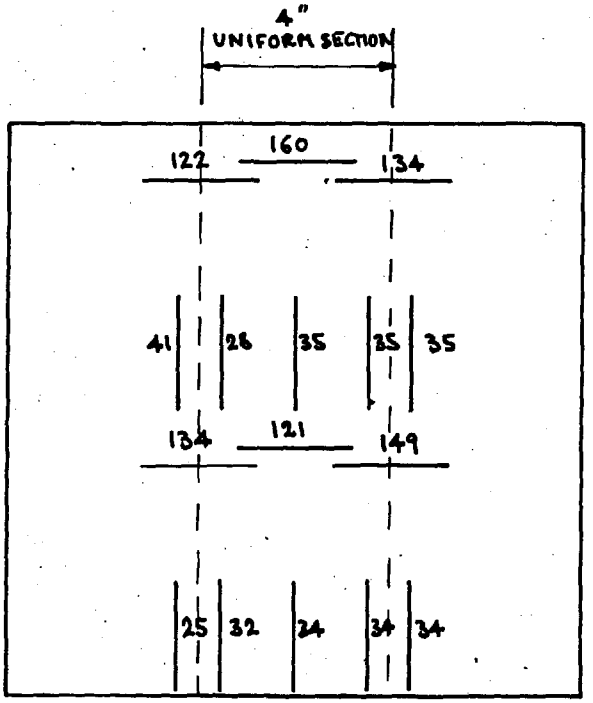
TEST No.	COMPRESSIVE STRESS psi	TORSIONAL LOAD STAGES. lb-in.			
1	155	0	667	1333	0
2	310	0	667	1333	0
3	465	0	667	1333	0
4 (TO FAILURE)	465	FROM ZERO IN 267 psi LOAD STAGES.			

LOADING SEQUENCE OF PROVING TRIAL SPECIMEN.

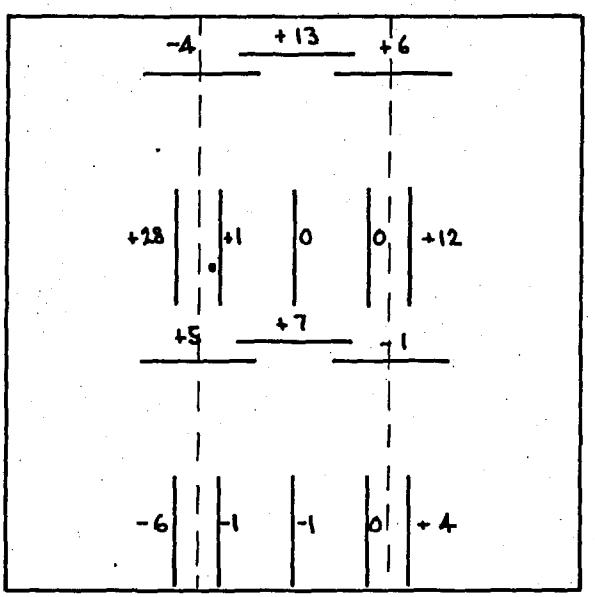


LAYOUT OF STRAIN GAUGES ON PROVING TRIAL SPECIMEN

FIG. 4.6.1.1. LAYOUT OF STRAIN GAUGES AND LOADING SEQUENCE FOR BIAXIAL COMPRESSION-TORSION PROVING TRIAL.

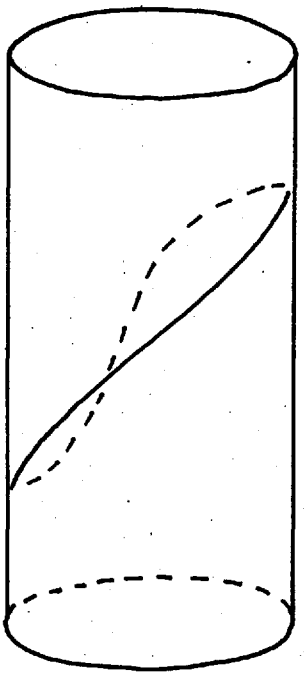


TEST No. 2.
(COMPRESSIVE STRESS = 310 psi TORSIONAL LOAD = 667 LB.FT.)

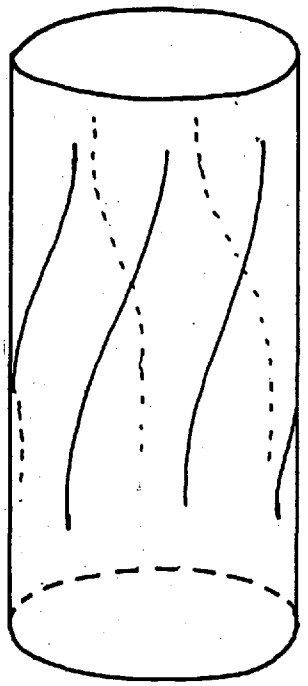


TEST No.1.
(COMPRESSIVE STRESS = 155 psi TORSIONAL LOAD = 1333 LB.FT.)

FIG. 4.6.1.2. TYPICAL STRAIN DISTRIBUTIONS IN BIAXIAL COMPRESSION-TORSION PROVING TRIAL.



CLEAVAGE TYPE FAILURE
UNDER LARGE TORSIONAL
LOAD



SPLITTING TYPE FAILURE
UNDER LARGE COMPRESSIVE
LOAD

FIG. 4.6.1.3. FAILURE MODES IN BIAXIAL COMPRESSION TORSION.

The storage of elastic strain energy in the hydraulic system above acceptable loads resulting in explosive test specimen failures was recognised as a problem in the design study in section 3.2.5. and the incorporation of a hydraulic compensator (138) would be a great asset.

However, within the limits of the project requirements, all controls were satisfactory and very reliable. No tests were ever delayed because of machine faults.

The procedure for bleeding air from the hydraulic system needs some attention as with an elaborate hydraulic system there are many chances for air to be drawn in with serious results. Unfortunately the jacks were not fitted with bleed valves and therefore this can only be achieved by jacking the ram to either extreme of its travel. One item, apart from all other high points in the pipework, not to be overlooked are the pressure gauges which can generally be bled from their joint with the pipework.

4.8. Summary

The operation of the machine frame and its subsequent calibration verified the design assumptions and confirmed the choice of the jacks used.

Trials on the strain distribution in the biaxial compression specimen showed that a uniform strain pattern was achieved over a substantial value of the specimen which could be defined.

The results from trials on the biaxial compression-tension facility were not quite so satisfactory and indicated a lower degree of confidence in the results subsequently obtained.

Similarly the single trial on the biaxial compression-torsion specimen gave a good general picture but several more detailed tests would be necessary before a major series of tests using this facility could be planned with confidence.

Lastly the hydraulic control system was reported to be highly reliable and satisfactory for the tests conducted but requiring several modifications for other types of testing as listed in Chapter 13.

It is recommended that trials after the fashion of those described for the machine and biaxial compression specimen in sections 4.2 and 4.4. should be the standard procedure for any future work on combined stress state testing.

CHAPTER 5

ANCILLARY EQUIPMENT

5.1 Introduction

The strain and ultrasonic pulse velocity measuring equipment used in the experimental work is discussed briefly. Although standard equipment which has been described in detail elsewhere (7, 9, 10) was used, the test methods and the modifications made in some cases were peculiar to this project and deserve special mention.

The zones or surface areas of test specimens within which representative measurements of strain and ultrasonic pulse velocity can be made have been defined by the trials described in Chapter 4. However, the problem of accommodating all the necessary devices within these zones had to be solved and is also described.

Equipment reliability and maintenance will be included in the discussion where they are of consequence to the experimental work.

The description of the equipment used has been sub-divided into the following groups for convenience:-

5.2 Measurement of Surface Strain
5.3 Measurement of Lateral Strain
5.4 Ultrasonic Pulse Velocity Equipment.

5.2 Measurement of Surface Strain

The problem of recording representative average strains of a uniformly stressed volume of concrete is a major consideration. Mechanical gauges are often difficult to attach to a specimen (this would be especially true of a biaxial compression specimen) and are of limited sensitivity. On the other hand electrical resistance strain gauges have to be a suitable length with respect to aggregate size[143,144] which usually makes for an expensive solution and the recorded differences between on-surface and in-surface measurements[145] are a further complicating factor.

The limitations of existing equipment are also apparent. Whilst the overall average strains are of interest to the engineer as these give sufficient data for design purposes, the measurement of local strains at points inside the concrete are at present impossible to any degree of reliable accuracy and yet it is these very strains that determine the failure of the material.

This project was confined to the measurement of on-surface average strains over a suitable gauge length using electrical resistance strain gauges. Although more expensive than mechanical gauges, the electrical resistance strain gauges were considered more accurate (apart from certain optical methods) and were more versatile in that they could be placed with ease in any position or direction. Furthermore, a large number of these gauges could be read in a very short time thus causing the minimum delay at load stages and hence reducing creep effects and resulting in more definable stress values.

The gauge selected as a standard for the tests was a Tokyo Sokki Kenkyojo 30 mm. effective length gauge designated by the manufacturers as PL30 with a gauge factor in the range 2.10 to 2.12, depending on the batch, and a resistance of 120 $\pm$0.3 ohm. Cron-sensitivity was of the order of 0.2%.

A length of only 30 mm. might be considered rather short for a $\frac{3}{4}$" aggregate concrete but the results of the proving trial reported in section 4.4.1. and illustrated in figs.4.4.1.5 and 4.4.1.6. show that this gauge length was representative of the average surface strain of a $\frac{3}{4}$" aggregate concrete. The maximum variation within the central uniformly strained zone being only $\pm$.6% which compared favourably with a variation of $\pm$ 3$\frac{1}{2}$% reported in the same section for a 3/16" aggregate mortar.

The reliability of the readings is increased by the placing of a gauge on each side of the specimen to give an effective gauge length of 60 mm. ($\simeq 2\frac{1}{2}$ in.).

Furthermore, it is useful to compare the accuracies claimed by other workers for the accuracy of the electrical resistance strain gauge readings related to the ratio of the effective gauge length to the maximum aggregate size. In this case the gauge length selected of effectively 60 mm. gives a gauge length to aggregate size ratio of about 3.15 for $\frac{3}{4}$" aggregate which is the largest size used. Peattie[183] claimed a 5% error from a ratio of 2.0 using $\frac{1}{4}$" aggregate whilst Grassam and Fisher[182] claimed a 4% error from 2.65 using $\frac{3}{8}$" aggregate. On the other hand Binns and Mygind[143] suggest an error of about 5% with a ratio of 3.15 whilst Cooke and Sneddon[144] suggest a 7% error for the same ratio.

Taking all these results together, a 5% error may be assumed with some confidence for the strain gauge length chosen.

To stick strain gauges onto the surfaces of saturated specimens, centre-line positions of the gauges were marked by pencil on the sides of the specimens. After painting the specimens with curing compound as described in section 6.9, the surface laitance was scraped from the areas which were to take gauges and the surface allowed to dry out. As soon as the surface was dry the gauge positions were re-marked and a thin layer of "PS adhesive" supplied by the strain gauge manufacturers applied to the surface. The gauge was then pressed into position on the adhesive, care being taken to ensure that no air bubbles were trapped under the gauge. A thin sheet of acetate supplied with the gaug was then placed carefully over the gauge, a metal weight being used to hold the whole firmly in position until the adhesive had set.

It was found by experience that a better bond was achieved by using 50% more hardener than recommended by the manufacturers. Although the adhesive set more rapidly, the results obtained during the preliminary proving trials showed that the gauges rarely come unstuck and much more consistent results were obtained. An additional improvement

in the technique was to place a rubber pad between the acetate sheet and the metal weight.

Vaseline grease was smeared around the edges of the adhesive on those areas not covered by the curing compound to prevent further evaporation of water. It was noticed that about 30 minutes after the grease was applied, the specimen moisture appeared to have returned to the surface.

For the uniaxial tests where only two gauges had to be read, a Peekel transistorised strain indicator (type T200) was used. However, for the specimen proving trials and the biaxial tests, the large number of gauges to be read necessitated the use of the automatic data logger manufactured by Solartron and illustrated in fig.5.2.1. The data logger read two gauges per second and was designed to drift less than 1 microstrain in 72 hours. The total time required to read a set of gauges was two seconds plus half the total number of gauges in seconds. The readings obtained were printed out on a line printer and punched tape; both can be seen in the photograph.

With both the Peekel transistorised strain indicators and the Solartron data logging system complete sets of dummy gauges were used to ensure temperature compensation. Hence the accuracy of the strain readings using this equipment was to within 1 microstrain or 1% of the reading whichever was the greater. This should not be confused with the overall accuracy of measuring the average strain which is dependent upon such factors as the quality of the adhesion of the gauge to the specimen, the electrical lead characteristics, the gauge length, the accuracy of manufacture of the gauges and operator efficiency.

(9)

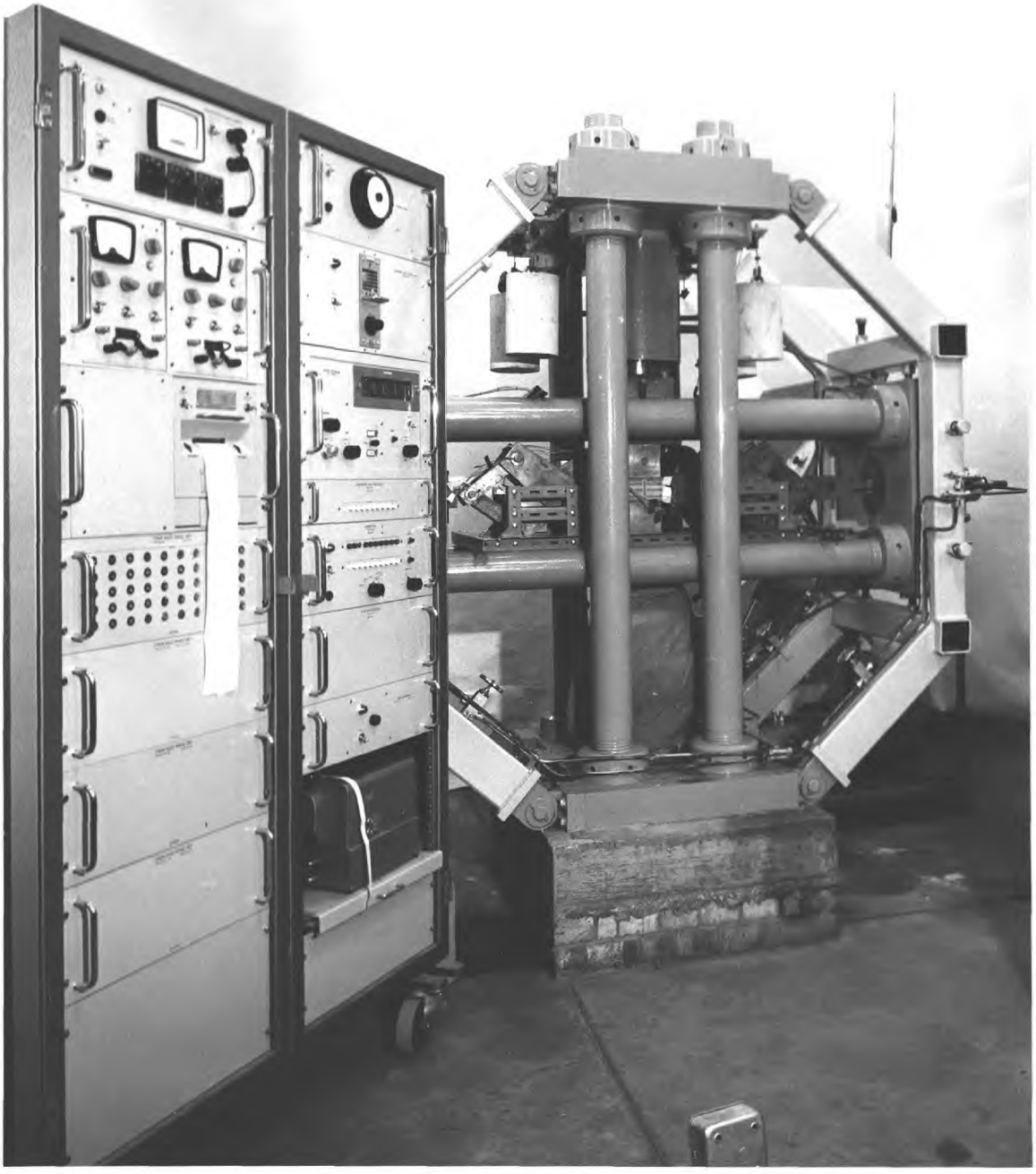


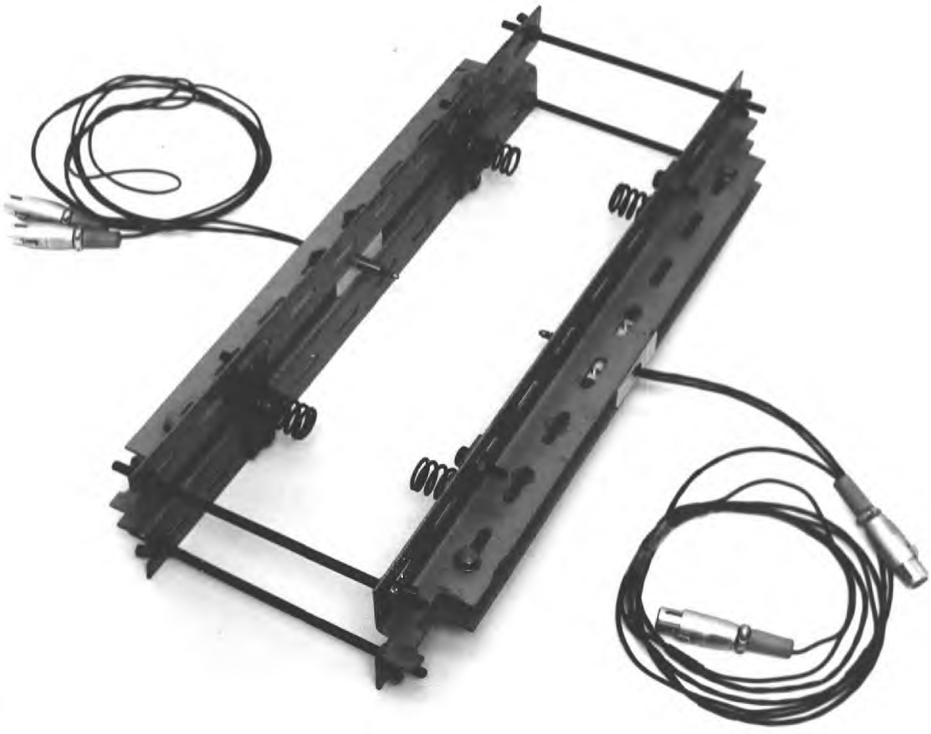
Fig. 5.2.1. Solartron Automatic Data Logger shown during proving trial on biaxial compression-tension facility.

5.3. Measurement of Lateral Strains

As explained in section 2.5. which described the platen restraint investigation, the most convenient method for measuring lateral strains available in the Concrete Technology Department was by the use of the linear displacement transducers manufactured by Sogenique Ltd. of Newport Pagnell. This method of strain measurement has advantages over the electrical resistance strain gauges as a method of strain measurement as it measures the total overall deformation of the test specimen. However, similar to the problem of representative gauge lengths, is the problem of the uniformity of the lateral deformation and the question of how representative is a measurement taken at one cross-section only. The results of the platen restraint trials (section 2.5.) in which lateral strain measurements were taken at several cross-sections showed that a single section reading was representative to $\pm 2\%$ and this reliability was subsequently confirmed on concrete specimens. The variations between specimens of the same batch also confirmed this.

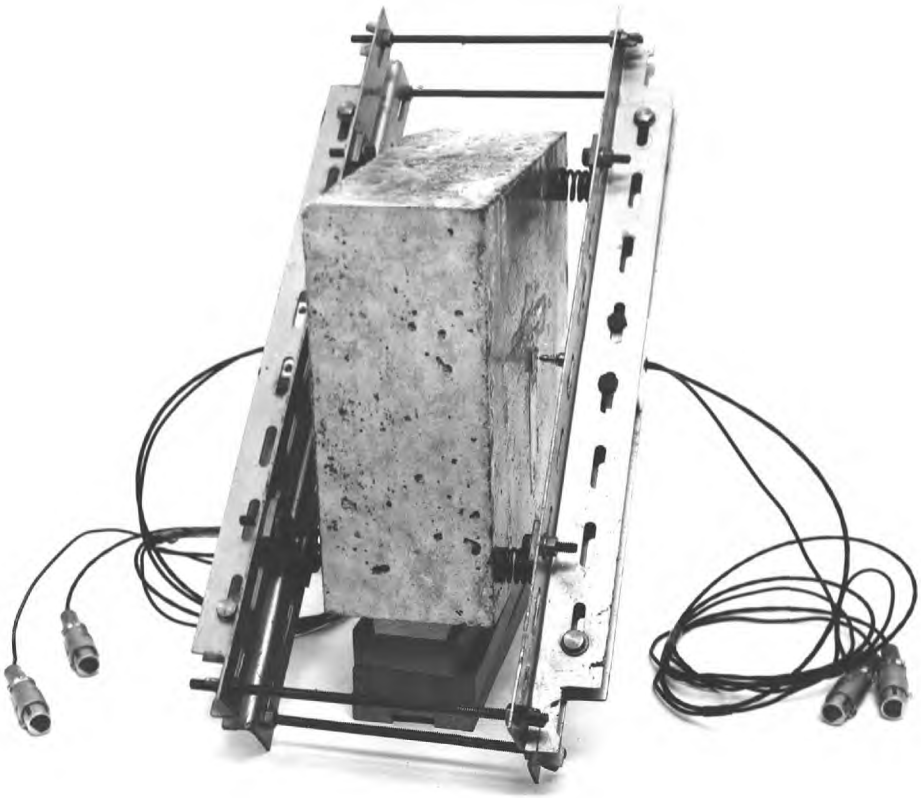
The support frame for the transducers developed by Robinson[10] for 4" cross-sections has already been described and is illustrated in fig.2.5.1.2. However, for biaxial compression tests where the specimen was much larger, a new support frame was called for.

Photographs of the frame designed are shown in figs.5.3.1. and 5.3.2. The basic principal of the arrangement is that the stiff double angle members and the end tie bars form a rigid datum frame whilst the springs clamp the frame lightly to the test specimen. As the specimen expands laterally, the springs are compressed but their reaction on the datum frame does not change significantly. Hence the linear displacement transducers which bear on glass pads stuck to the concrete surface are held firmly in position relative to each other and thus give a measure of the lateral expansion. One further advantage of the springs is that they can accommodate changes in specimen shape in the plane of the specimen surface.



(10)

Fig. 5.3.1. Support Frame for Linear Inductance Transducers for biaxial compression tests showing Transducers in position.



(11)

Fig. 5.3.2. Support Frame for Linear Inductance Transducers as clamped to biaxial compression test specimen.

With equipment as sensitive as this, the relatively easy solutions of mounting the transducers on frames supported from the laboratory floor or on the machine itself are not advisable as the readings would be affected by even minute local vibrations. Also the test specimen would move past the transducers as the machine deflected thus measuring the angle of inclination and roughness of the glass pads as well as the specimen expansion.

It is important when setting up the frame on a specimen to ensure that the wing nuts on the end ties are firmly clamped against their lock nut and that the springs bear on the specimen at symetrical points on the diagonsl so that, as the specimen deforms, the frame maintains the transducers centrally on the glass pad.

An additional feature of this frame were the blocks in which the transducers are placed; these can be moved to a variety of positions along the double angled members and thus enable the survey of lateral strain along a diagonal to be carried out (section 4.4.3.).

Note that although the transducer holding blocks are a push fit between the two angles of the frame, two bolts are used to clamp them firmly into place.

This frame was calibrated using the reference aluminium specimen against the electrical resistance strain gauges used in the research project and described above in section 5.2.

Two gauges were placed on the aluminium slab in the position shown in fig.5.3.3. and the linear inductance transducers were placed centrally on the specimen. The specimen was then compressed uniaxially and the two strain readings were taken and compared. The results showed that after an initial lag of about 12 micro-strain (± 2), probably due to a difference between the expansion of the aluminium specimen in the centre and at the ends, the resulting strains were identical. This was the average of six trial runs and the calibration chart is given in fig.5.3.3.

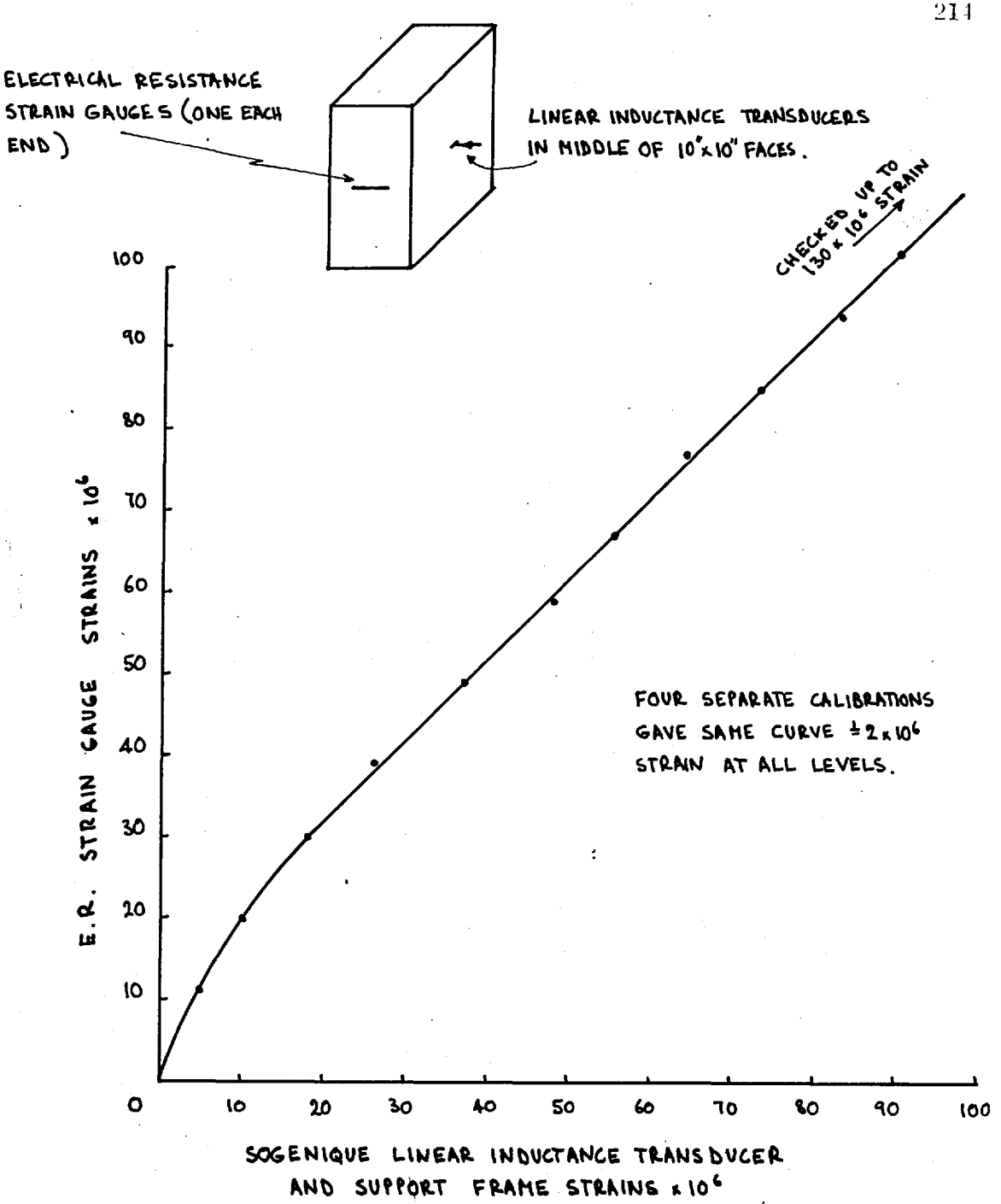


FIG. 5.3.3. CALIBRATION CURVE FOR LINEAR INDUCTANCE TRANSDUCER AND SUPPORT FRAME.

The Sogenique equipment has already been referred to in section 2.5.1. but in that case only one transducer was used. In this application two transducers are used and the recording box illustrated in fig.5.3.4. automatically records the average reading $\frac{A+B}{2}$ in micro-inches to the same accuracy as for a single transducer, i.e. 1 micro-inch. Hence over a 4 inch. thickness of specimen the reading is twice the value of the strain.

For all sets of readings on the recording box, the transducer frames were adjusted until the reading was at about the centre of the range which is the most linear position on the Sogenique calibration curve.

This equipment was later replaced by a more elaborate switch box and recorder capable of housing six transducers.

5.4 Ultrasonic Pulse Velocity Equipment

The discussion on the philosophy of strength contained in section 1.5 brought out the significance of the point of discontinuity and it was noted that Robinson[10] had shown the relationship between the various methods of determining this point. Of these several methods, the measurement of variation in ultrasonic pulse velocity appeared to be the most clearly defined method and hence was used throughout this research project to determine the discontinuity point.

A standard Cawkell ultrasonic materials tester, type UCT2 with varium titinate crystals was used in all the tests and has been described by Ward[7]. In addition, the reading techniques developed by Robinson[10] were used to determine the changes in ultrasonic pulse velocity in all tests.

The significant improvements in technique developed in this project were in the positioning of the transducers to obtain more representative results by scanning across critical volumes (figs.5.4.1. and 5.4.2.).



(12)

Fig. 5.3.4. Sogenique Strain Measuring Box with Linear Inductance Transducers in their housing blocks.

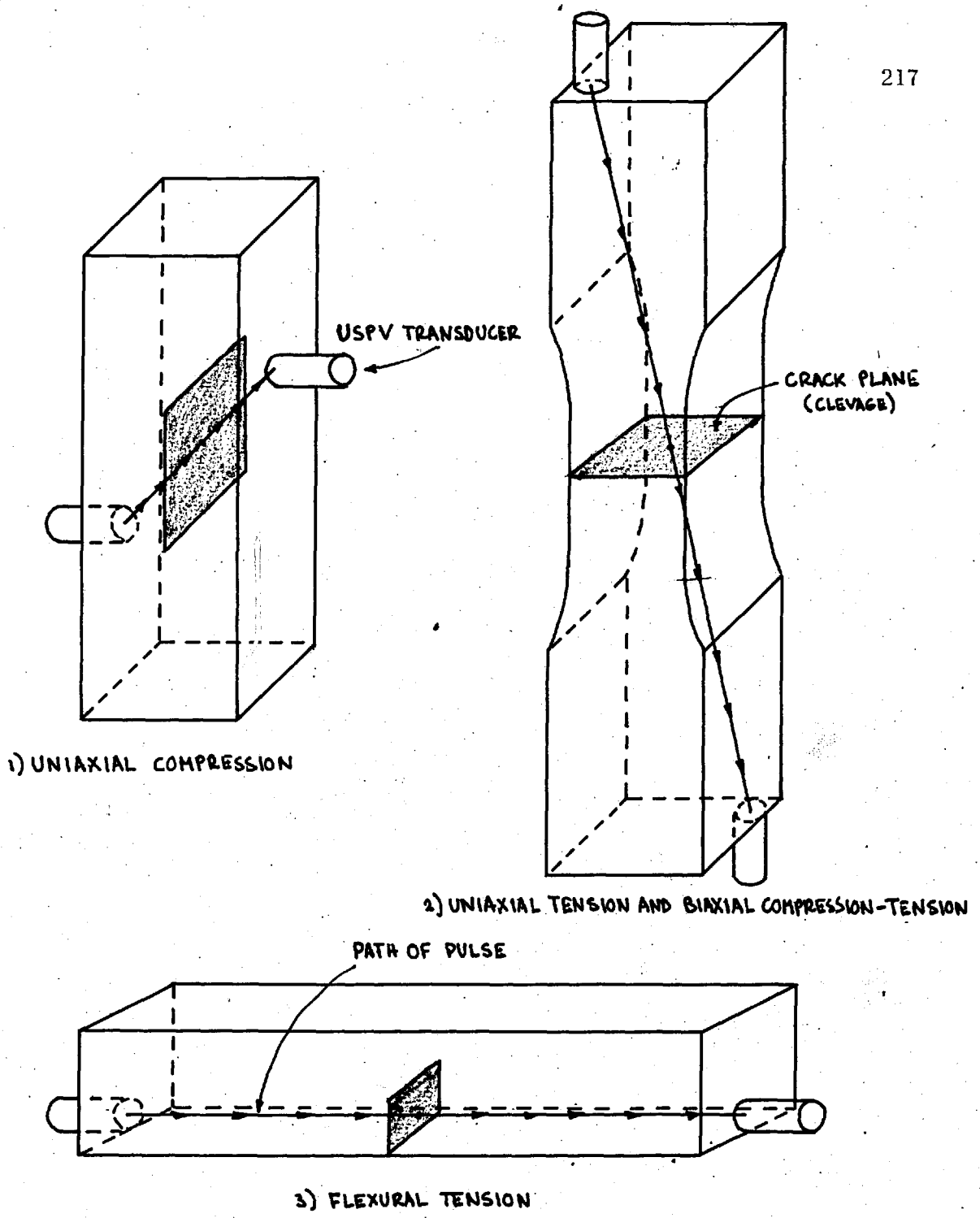
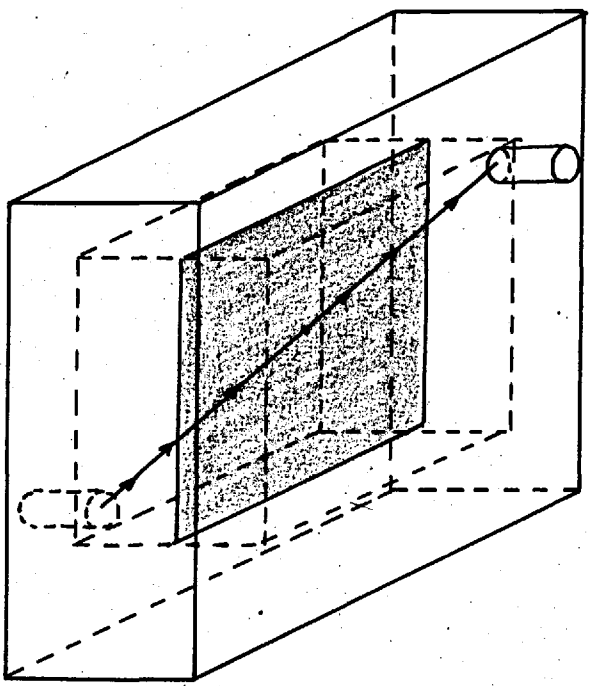
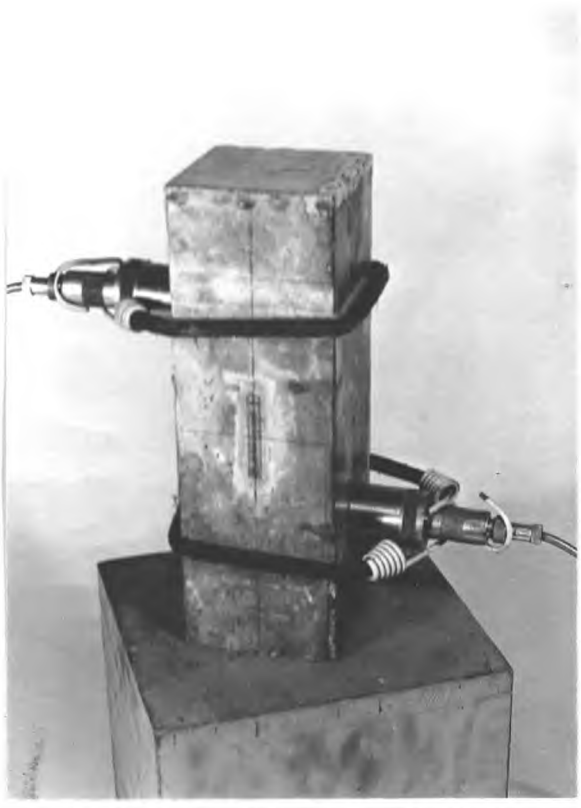


FIG. 5.4.1.a. USE AND POSITIONING OF ULTRASONIC PULSE VELOCITY TRANSDUCERS RELATIVE TO PROBABLE CLEAVAGE PLANES.

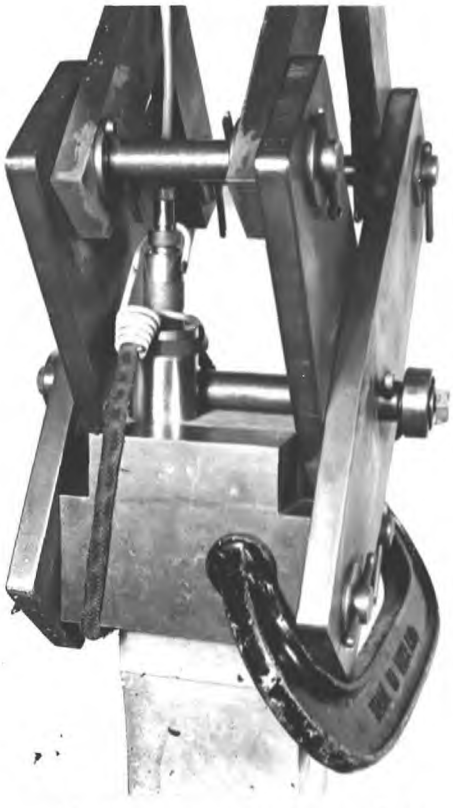


4) UNIAXIAL AND BIAXIAL COMPRESSION

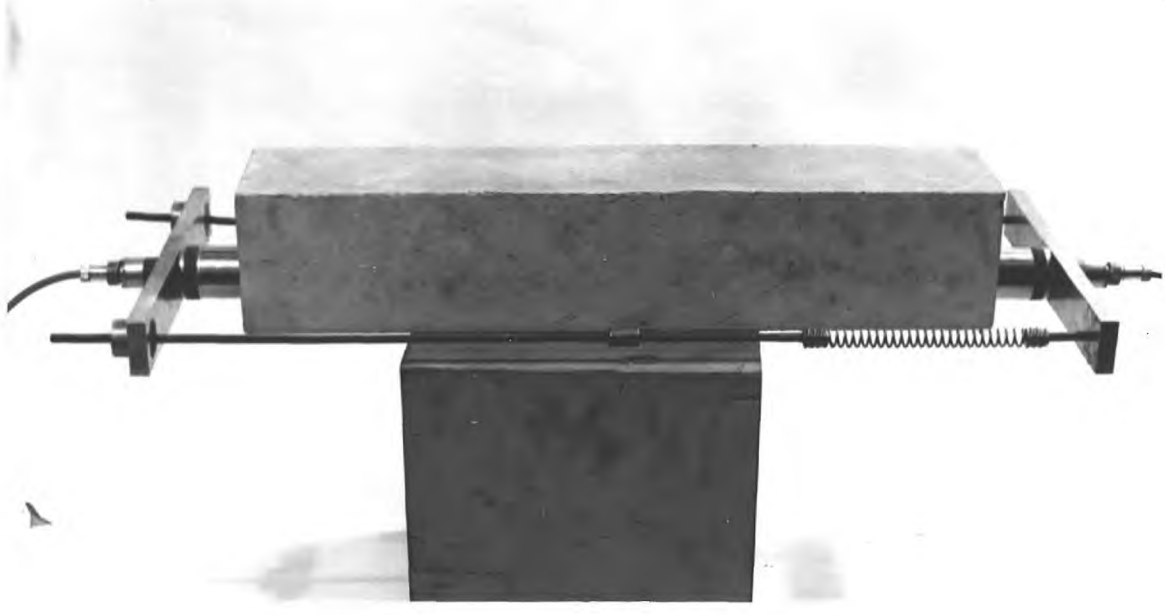
FIG. 5.4.1.b. USE AND POSITIONING OF ULTRASONIC PULSE VELOCITY TRANSDUCERS RELATIVE TO PROBABLE CLEAVAGE PLANES.



(13) a) Uniaxial Compression Test



b) Uniaxial Tension Test



c) Flexural Beam Test

Fig. 5.4.2. Arrangement of Ultrasonic Pulse Velocity Transducers for crack detection in uniaxial tests.

For uniaxial compression tests, the transducers were placed at diametrically opposite third points to scan the centre third of the prism which Ros[24], Lachance[1] and others have shown to be in a uniform state of stress.

In uniaxial tension the transducers were placed in diagonally opposite corners and hence scanned the whole specimen. As in the case of uniaxial compression tests, the pulse travels normal to the planes of crack propagation and hence should detect the discontinuity point. The fact that there was adequate access to the ends of the tension specimen in the testing technique developed by Ward[7] for the ultrasonic pulse velocity transducers was a new and hitherto unacclaimed advantage of this technique over other tension tests.

For the flexural beam test, the transducers were aligned with the centre of the beam soffit to detect flexural tension cracking as shown in figs.5.4.1 and 5.4.2.

The scanning principle was repeated in the biaxial compression tests in which the centre 6" x 6" x 4" volume of uniform stress state was scanned from diagonally opposite corners thus picking up most of the initial cracking within that volume.

In all these tests, either the critical volume alone is scanned or, if less critical volumes are included such as in the flexural and direct tension tests, they are free from any possible crack formation by virtue of the reduced stress level in these volumes.

Prior to positioning the transducer on the specimen, the surface laitance and curing compound were removed from the concrete, and the face of the transduced coated with Vaseline to help provide a uniform contact.

Difficulties were discovered later in applying this technique to biaxial compression-torsion and triaxial compression tests and these are described in Chapter 10.

Besides its application as a method of detecting crack formation[104, 109, 146] the ultrasonic pulse velocity equipment has been developed by Jones and others[147-151] as a general non-destructive method of testing concrete. Together with the determination of natural frequency of vibration (described by Ward[7]) it can be used to calculate the dynamic elastic modulus and Poissons ratio of a given mix. Furthermore the results of these two tests have been related to the damping coefficient by Jones[152] and to the compression strength and Poissons ratio by Kaplan[153-155].

5.5 Summary

The strain measuring and ultra-sonic pulse velocity equipment described above was deemed to be adequate for this research project and subsequent results confirmed its efficiency and reliability.

CHAPTER 6

OUTLINE OF EXPERIMENTAL WORK

6.1. Introduction

After the design and proving of a suitable testing machine and ancillary equipment as described above, the second stage of the research project as outlined in section 1.7 was the planning of a comprehensive series of biaxial compression and compression-tension tests and the development of several other facilities.

This chapter outlines the planning of these tests including the selection of suitable mixes and describes the materials and standard laboratory procedures used throughout.

Many of the techniques used in testing the materials, manufacture and preparation of specimens, and subsequent non-destructive testing have been deliberately kept the same as those developed by Ward[7] and Robinson[10] in order to compare the results obtained. Where this has been done reference is made to the relevant thesis.

6.2. Plan of Test Series

The general theme of the research project was to work towards a failure criterion for concrete under universal and combined states of stress with particular emphasis on the behaviour under biaxial compression. To this end the following test series were designed.

6.2.1. Uniaxial Test Series

Whilst much work has already been carried out under uniaxial states of stress there is still much to be learnt from these basic tests. The intention in these tests was to attempt to obtain Poissons ratio valves and discontinuity stress loads in

uniaxial compression and tension in addition to measurements of elastic modulus (both static and dynamic), ultimate stresses, and the overall stress-strain characteristics. The three recent advances in experimental technique to be applied were Ward's uniaxial tension device, the ultrasonic pulse velocity as used by Robinson, and the measurement of lateral strain by linear displacement inductance transducer. It was considered that, by extending the range of measurements in uniaixial tests along these lines, more information could be obtained on concrete behaviour under these simple states of stress than hitherto. Furthermore, the tests would match those carried out under biaxial compression and act as control tests for the same.

The uniaxial tests selected were uniaxial compressive, using 4" x 4" x 12" prisms, uniaxial tension on a modified 4" x 4" x 20" beam as specified by Ward[7], and a standard flexural beam test on a 4" x 4" x 20" beam[156]. Three specimens were considered statistically significant for each test[157, 158] and so a total of nine uniaxial specimens were cast from each mix making a total of 180 specimens for the whole short-term testing series.

Work by Kaplan[21] had shown that the clearage strain at the onset of rapid crack propagation (or discontinuity, see section 1.5) was dependent upon the volume fraction of coarse aggregate in the mix. Although good correlation was achieved between these two parameters, only two coarse aggregate types were used. The proposed series of tests were planned, therefore, to cover a wider range of aggregate types and see whether the same degree of correlation could be obtained. At the same time, these tests would act as a check on the results of this and other recent investigations under uniaxial stresses.

As may be seen from the programme of the research project contained in section 1.8, many of these tests were carried out whilst the machine was under construction and hence enabled

experience to be gained in the use and limitations of the ancillary equipment prior to the more rigorous testing procedures adopted during the biaxial compression tests.

The mixes selected for testing during this period were chosen so that they complimented those mixes selected for biaxial compression tests to avoid duplication (see section 6.4.).

An additional uniaxial test series was concerned with the ultimate strength of concrete under long-term loading and how this would relate to the short-term loading results. Accordingly, a number of 4" x 4" x 12" prisms were loaded to various stress levels in uniaxial compression and the load maintained for up to two hours. The rate of strain increase was used as a measure of creep effects plus structural deterioration.

The uniaxial tests and the results obtained are described in Chapter 7.

6.2.2. Biaxial Compression Test Series

This main series of tests followed immediately on the completion of the machine and biaxial compression specimen proving trials.

From the results of the uniaxial test series, it became clear that certain parameters not normally considered to be important had a large effect on the failure of concrete such as the volume concentrations of fine and coarse aggregate. Hence, in the selection of mixes for this test series, as many parameters as possible were taken into account.

Due to the limited number of 10" x 10" x 4" specimen moulds available only six biaxial compression specimens could be cast from each mix along with the nine uniaxial control specimens specified above. However, a low coefficient of variation was obtained in the proving trials and by plotting limits of two standard

deviations either side of each envelope using the results of the proving trials, an indication of the statistical significnnce of the curve would be obtained. Furthermore, the envelope as drawn through the six results would be a 'best fit' and this would tend to enhance the reliability of the envelope.

In these and other combined stress tests the type of failure was of special interest and records were kept of the observed modes.

6.2.3. Biaxial Compression-Tension Test Series

These tests were subsidiary to the main biaxial compression test series. The plan was to develop the testing facility as far as project time allowed and then to take a number of the mixes chosen for biaxial compression tests and extend the results into the compression-tension sector of the general biaxial stress envelope.

Eight direct tension specimens and three prisms were cast; the prisms acted as the control specimens for the mix and the eight tension specimens being spread over the biaxial compression-tension sector to give a representative picture of behaviour.

Again a special effort was to be made to obtain a complete record of failure modes.

6.2.4. Torsional and Triaxial Test Series

These tests were included at the end of the testing programme and were intended more as a logical extension of the development of the testing machine facilities rather than an investigation into the properties of concrete under other states of stress. However, some interesting qualitative results were obtained.

The tests were also designed to indicate possible future venues of research that might prove profitable and could be pursued using equipment already developed and available as a result of this research project.

6.3 Selection of Variable Parameters

Before listing the parameters whose variation was considered to be of prime significance in the tests, the more important fixed parameters were selected and are specified below:-

1) Plain concrete or mortar specimens, no reinforcement.
2) All specimens tested fully saturated to avoid problems of shrinkage and varying moisture content.
3) Short-term loadings (with one exception) to avoid creep problems and to enable other paramters to be investigated in the time available.
4) For each state of stress only one condition of degrees of freedom of movement for the specimen between the loading devices, e.g. all four faces effectively fixed, etc.
5) Only one type of cement used.
6) All tests carried out at 28 days on specimens cured under identical conditions (159-161).
7) Size of specimen for each state of stress was fixed.

The variable parameters selected for these tests were as follows:-

1) Size of maximum particles of aggregate. Three sizes chosen, viz. 3/16"; $\frac{3}{8}$"; $\frac{3}{4}$".
2) Volume fraction of coarse aggregate. Two extremes slected (30% and 50%) within range at which full compaction can be achieved.
3) Type of coarse aggregate. Four types selected on a basis of aggregate texture and elastic modulus.
4) Water-cement ratio. Three ratios selected (0.45; 0.55; 0.65) within the range of normal mixes.

It will be noted that great emphasis was placed on the aggregate content of the mix which was influenced by previous work on aggregate effects (162-166). However, the number of possible combinations (3 x 2 x 4 x 3 = 72) were too many and the following section describes the compromises made to reduce the combinations to a practical number.

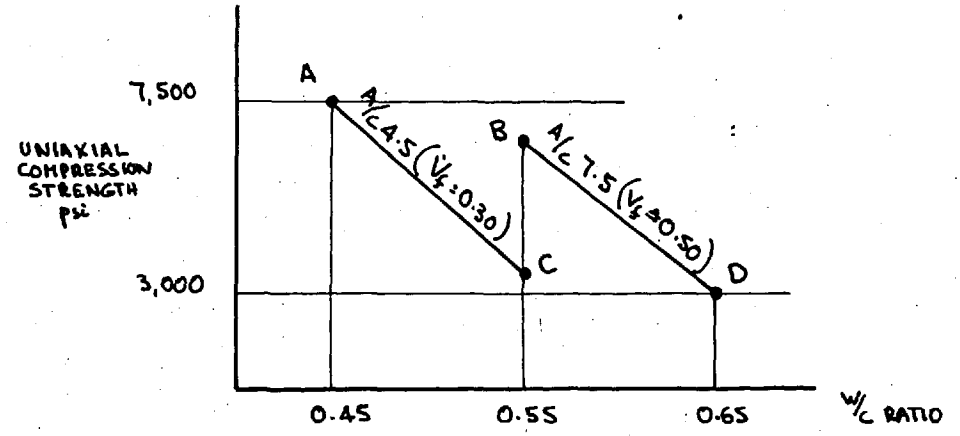
6.4. Selection of Mixes

The selection of suitable mixes was made having regard to the variable parameters selected in section 6.3. and the philosophy of the research project defined in section 1.6.

In addition the criteria for the concrete mixes laid down in the outline of the research project in section 1.8. called for maximum aggregate sizes up to $\frac{3}{4}$" and uniaxial compressive strengths over the range 3,000-7,500 p.s.i. These latter include the range of design mixes used in both British Standard Codes of practice CP.114: 1957 and CP.115: 1959.

As stated at the end of section 6.3., not all the possible combinations of parameters could be investigated and therefore a certain amount of compromise and curtailment was necessary in the mix selection. The main test series was biaxial compression and therefore the selection of mixes for this series took priority. The major compromise was achieved by examining only sand and one coarse aggregate ($\frac{3}{4}$" irregular) under all three water:cement ratios and then, for one fixed water:cement ratio, examine the remaining aggregates.

The scheme of mix selection is given in fig.6.4.1. in which the mixes selected for all the test series are shown. In a second diagram (fig.6.4.2.) the mixes used for comparative studies of the effect of one parameter are listed for the mixes used in biaxial compression only. For example, the effect of varying aggregate size whilst keeping the water/cement ratio,



QUANTITY OR DETAILS OF MIX.	MIX PROPORTIONS			
	A	B	C	D
WATER : CEMENT RATIO	0.45	0.55	0.55	0.65
SAND : CEMENT RATIO	2.4	2.4	2.4	2.4
COARSE AGG. : CEMENT RATIO	2.1	2.1	5.1	5.1
TOTAL AGG. : CEMENT RATIO	4.5	4.5	7.5	7.5
NOMINAL VOLUME FRACTION OF COARSE AGGREGATE ASSUMING S.G. = 2.65	0.32	0.32	0.51	0.51
MORTAR	MA	MBC	MBC	MD
3/8" IRREGULAR AGG.	(3IA)	3IB	3IC	(3ID)
3/4" IRREGULAR AGG.	6IA	6IB	6IC	6ID
3/4" ROUNDED AGG.	(6RA)	6RB	6RC	(6RD)
3/4" ANGULAR AGG.		6AB		
3/4" LECA AGG. (LIGHTWEIGHT)	(6LA)	6LB		

NOTATION : EXAMPLE 6RC MEANS 6/8" (i.e. 3/4") R ROUNDED AGGREGATE CONCRETE WITH MIX PROPORTIONS C SPECIFIED ABOVE.
MIXES TESTED IN UNIAXIAL STRESSES ONLY MARKED THUS (6RD)
MIXES TESTED IN BIAXIAL COMPRESSION-TENSION MARKED THUS 6IB
ALL OTHER MIXES TESTED IN UNIAXIAL STRESSES AND BIAXIAL COMPRESSION. (13 No.) INCLUDING ALL THOSE TESTED IN BIAXIAL COMPRESSION-TENSION.

FIG. 6.4.1. MIX PROPORTIONS AND THEIR SELECTION.

INVESTIGATION OF EFFECT OF SINGLE PARAMETERS IN BIAXIAL COMPRESSION.	
PARAMETER TO INVESTIGATE	MIXES TO COMPARE
1/ WATER CEMENT RATIO	MA — MBC — MD 61A — 61B 61C — 61D
2/ VOLUME FRACTION OF COARSE AGGREGATE	31B — 31C 61B — 61C 6RB — 6RC
3/ AGGREGATE SIZE	MBC — 31B — 61B MBC — 31C — 61C
4/ AGGREGATE TYPE	6RB — 61B — 6AB — 6LB 6RC — 61C

FIG. 6.4.2. EFFECT OF MIX SELECTION ON THE INVESTIGATION OF THE EFFECT OF SINGLE PARAMETERS IN BIAXIAL COMPRESSION.

volume fraction of aggregate, and aggregate type constant was illustrated by the results from mixes MBC; 31B; and 61B and from MBC; 31C; 61C.

For other tests and proving trials, the two standard mixes were MBC (typical mortar) and 61B (typical $\frac{3}{4}$" aggregate concrete). The only exception was 31B which was used in a biaxial compression-torsion test where $\frac{3}{4}$" aggregate was considered too large for a 1 in. thick cylinder wall.

Pastes were excluded from all test series as it was considered that the techniques involved in the preparation of the specimens in this research project would have a profound effect on the results likely to be obtained from testing pastes (167). The techniques used involved partial drying of the specimen which would have invalidated the results obtained.

6.5. Materials

This section describes all the materials used in the manufacture of the test specimens and the various tests carried out on them.

6.5.1. Cement

Ordinary portland cement, supplied by the Cement Marketing Co. Ltd. was blended in the laboratory plant and supplies were drawn from this source for all tests. Tests carried out on the cement in accordance with B.S.12: 1958 showed no significant variation in quality over the period of the research project. These tests showed that the cement was of average quality; the results are given in fig.6.5.1.1.

6.5.2. Fine Aggregate

An irregular Thames Valley river sand, supplied by Messrs. T. W. Howard from their Thorney Mill Road pit[168] was used throughout the test series. Its natural grading with the limits derived from three sieve tests conducted in accordance with BS.410: 1962[169] and BS.812: 1960[173] is given in fig.6.5.2.1. and it conformed with the quality requirements of BS.882; 1954[170].

After delivery the sand was heated to 100°C in a rotary drier for 5 minutes before being placed in the storage hoppers. To avoid segregation in the final storage hoppers the dried sand was sieved into two sizes (3/16" to 25; 25 down) and stored separately. It was then recombined at the time of mixing in the ratio 30:70, fine to coarse, and sieve analyses on the two halves gave the recommended gradings shown in fig.6.5.2.2.

Saturated surface dry specific gravity and absorption tests were carried out on the sand and, in the case of the absorption test, the values obtained at 4 hours and 24 hours were compared[171-173]. The results are given in fig.6.5.2.3.

CEMENT QUALITY TESTS.

3 DAYS.	7 DAYS.
1) 4800	1) 6450
2) 4400	2) 6000
3) 4100	3) 6300
AVERAGE 4450 psi.	6250 psi.

FIG. 6.5.1.1. CEMENT QUALITY.

SIEVE	% PASSING
3/8"	100
3/16"	100
No. 7.	79
No. 14.	55
No. 25	27
No. 52.	2
No. 100.	0
REMAINDER	0

FIG. 6.5.2.1.

NATURAL GRADING OF SAND.

SIEVE	% PASSING
3/8"	100
3/16"	100
No. 7	82
No. 14	59
No. 25	31
No. 52	5
No. 100	1
REMAINDER	0

FIG. 6.5.2.2.

RECOMBINED GRADING OF SAND.

ALL RESULTS ARE THE AVERAGE OF THREE ANALYSES.

1) SPECIFIC GRAVITY		2.52
2) APPARENT SPECIFIC GRAVITY		2.67
3) WATER ABSORBTION	4 HOURS	—
	24 HOURS	2.30

FIG. 6.5.2.3. SPECIFIC GRAVITY AND ABSORBTION TESTS ON SAND.

SIEVE	% PASSING			
	SAND	3/8" IRREGULAR	3/4" IRREGULAR	3/4" ROUNDED	3/4" ANGULAR
1½"		100	100	100	100
3/4"		100	100	99	100
3/8"	100	100	15	7	1
3/16"	100	2	0	1	0
No.7	92	0	0	0	0
No.14	79				
No.25	31				
No.52	11				
No.100	1				
REMAINDER	0				

ALL RESULTS ARE THE AVERAGE OF THREE ANALYSES. VARIATION NEVER EXCEEDED ±1.5%. SAND GRADING IS RECOMBINED GRADING GIVEN IN FIG. 6.5.2.2.

FIG. 6.5.3.1. SAND AND AGGREGATE GRADINGS.

6.5.3. Coarse Aggregate

All coarse aggregates, with the exception of the light-weight Leca aggregate, were firstly dried by the process described above for the fine aggregate and sieved to provide the required range of aggregate size. They were then stored in separate hoppers in the laboratory batching plant and carefully labelled.

The resulting gradings taken as a spot check after sieving are given in fig.6.5.3.1. The aggregates used and their suppliers were as follows:-

1) $\frac{3}{8}$" Thames Valley Natural River Gravel - irregular shape. Supplied by Messrs. T. W. Howard and Son Ltd., Larbourne Farm, Thorney Mill Road, Iver Bucks. (April 1964).
2) $\frac{3}{4}$" - as above.
3) $\frac{3}{4}$" Bridport Natural River Gravel - rounded shape. Supplied by Norman Good Ltd. of Bridport, Dorset. (April 1964).
4) $\frac{3}{4}$" Mountsorrel Crushed Granite - angular shape. Supplied by Mountsorrel Granite Co. Ltd. from the Mountsorrel pit, (April 1964).
5) $\frac{3}{4}$" Leca lightweight aggregate. Supplied by W. C. French Ltd., London. (April 1964).

Saturated surface dry specific gravity and absorption tests were carried out on these aggregates in the same manner as on the sand. The results given in fig.6.5.3.2. together with the results from the sand in fig.6.5.2.3. show that there is no significant difference in the absorption values recorded over 4 and 24 hours. Hence for the convenience of this research project, the pre-soaking time for all aggregates prior to mixing was reduced from the Concrete Material Research Group standard of 24 hours[7,10] to 4 hours. This results tends to support Newman's suggestion[172] that the bulk of the absorption of water by normal aggregates takes place in the first hour.

COARSE AGGREGATE	SPECIFIC GRAVITY	APPARENT SPECIFIC GRAVITY	WATER AT 4 HOURS	ABSORBTION AT 24 HOURS
3/8" IRREGULAR	2.37	2.61	—	3.90
3/4" IRREGULAR	2.47	2.61	2.10	2.10
3/4" ROUNDED	2.51	2.57	0.95	0.95
3/4" ANGULAR	2.64	2.68	0.50	0.55

WATER ABSORBTION IS PERCENTAGE OF WATER ABSORBED BY WEIGHT OF DRY AGGREGATE.

FIG. 6.5.3.2. SPECIFIC GRAVITY AND ABSORBTION TESTS ON COARSE AGGREGATE.

Results of petrological anqlysis on the aggregates are given in fig.6.5.3.3. and measurements of particle texture[174] and shape[175-177] are summarised in fig.6.5.3.4.

6.5.4. Water

Mixing water was drawn from the laboratory water supply systems and was within 5°F of the laboratory ambient temperature at the time of mixing.

6.6. Manufacture of Specimens

The mixing, casting and curing procedures for 28 day tests on saturated specimens used in this project were those now standard in the Concrete Materials Research Group and have been described in detail by Ward[7] and Robinson[10]. However there were features of these operations peculiar to this research project and these are described below.

All aggregates, both fine and coarse, were pre-soaked in water for a minimum of four hours prior to mixing (see section 6.5.3.) and it was convenient to batch all the materials at this time. This procedure had the advantage that all the materials used in the mix were at the ambient temperature of the laboratory (68°F ±2°F) at the time of mixing.

Despite the careful draining of the pre-soaked aggregate, it was noiced that some small quantity of fines was lost from the fine aggregate. Therefore a comparison was made between the sieve analysis of the sand taken before and after pre-soaking. The results given in fig.6.6.1. show that the loss of fines due to draining off the excess pre-soaki g water was negligible.

The quantity of batch water was adjusted assuming a 1% by weight absorption of water by the aggregate. This was later corrected by the results of the absorption tests and the actual resulting mix propertions are given in fig.6.7.1.

Sample No.519-¾" Angular crushed granite

This is a coarse-textured, crushed, crystalline igneous rock, pinkish-white in colour with black 'spots'. The essential constituents are quartz alkaline felspar (pink), biotite (blackish-brown), muscovite, iron ores and apatite. The structure is holocrystalline, the minerals reasonably fresh. All constituents of this sample are derived from the same mother-rock which is an alkaline BIOTITE GRANITE, and as such is classified in the Granite group in B.S.812:1960, clause 10d., similarly in revised draft of same D65/7547, July 1965.

Sample No.520-⅜" Irregular aggregate

This aggregate is composed essentially of angular to sub-angular flint pebbles, some angular and flaky; well rounded vein-quartz pebbles (10%); basalt etc.(4%). The petrological designation is FLINT GRAVEL. (Note: B.S.812 recognises Flint group, into which this aggregate enters.)

Sample No.521-¾" Irregular aggregate

This aggregate is composed of angular, subangular and rounded flints, vein-quartz and few quartzite pebbles, well rounded, (6%), rounded dark grey basalt pebbles etc.,(12%). The petrological classification is FLINT GRAVEL as above.

Sample No.522-¾" Rounded aggregate

This sample represents a natural, well water-worn gravel deposit. The component pebbles are for the most part rounded in shape and are composed of flint, vein-quartz, quartzite, and basalt (the latter about 6%). Probably the best designation petrologically for this aggregate is QUARTZOSE GRAVEL. This type of 'mixed rock' deposit is not envisaged in B.S.812.

Sample No.523- Irregular Sand

This sample of sand effervesces moderately with dilute HCl, but the reaction is not long sustained. The composition is essentially silica sand (detrital quartz) in rounded to subangular grains, angular flint particles, chert and miscellaneous igneous rock particles (the latter relatively scarce) and clay. Acid treatment leaches a certain amount of brown limonite from the sample. Petrologically this is a SILICA SAND, containing a little lime derived from shell fragments.

FIG. 6.5.3.3. PETROLOGICAL ANALYSIS OF AGGREGATES.

	Surface Texture	Flakiness Index %.	Shape	Elongation Index %.
3/8" Irregular	Smooth.	31.6		17.7
3/4" Irregular	Smooth	24.6		17.2
3/4" Rounded	Smooth	13.8		24.7
3/4" Angular	Crystaline	28.9		7.7

FIG. 6.5.3.4 Coarse aggregate particle texture and shape.

SIEVE	% PASSING.	
	BEFORE SOAKING	AFTER SOAKING.
3/8"	100	100
3/16"	100	100
No. 7	82	83
No. 14	57	58
No. 25	27	28
No. 52	5	5
No. 100	0.5	0
REMAINDER	0	0

FIG. 6.6.1. EFFECT OF SOAKING AND DRAINING OF SAND ON GRADING.

MIX	NOMINAL PROPORTIONS				ACTUAL WEIGHTS LB.				THEORETICAL WET DENSITY	ACTUAL DENSITY	ACTUAL V_f	SLUMP	COMPACTING FACTOR
	CEMENT	SAND	C.A.	WATER	CEMENT	SAND	C.A.	WATER					
MA	1	2.4	0	0.45	112.0	268.6	0	53.1	144.5	139.5	0	½" CO	.844
MBC	1	2.4	0	0.55	105.6	253.5	0	60.6	139.9	135.7	0	7" C	.933
MD	1	2.4	0	0.65	100.0	240.0	0	67.4	138.2	136.1	0	—	—
31A	1	2.4	2.1	0.45	35.4	84.9	74.3	17.5	153.2	—	.326	⅜"	—
31B	1	2.4	2.1	0.55	72.8	174.9	153.2	43.5	148.2	142.9	.313	½" CO	.858
31C	1	2.4	5.1	0.55	50.6	121.4	258.0	31.6	154.0	143.1	.526	¼" C	.680
31D	1	2.4	5.1	0.65	23.0	55.2	117.2	16.7	150.4	—	.512	3"	—
61A	1	2.4	2.1	0.45	75.9	186.6	159.5	37.6	153.3	144.8	.326	0	.777
61B	1	2.4	2.1	0.55	72.8	175.0	153.2	43.5	148.2	143.9	.313	1¾"	.905
61C	1	2.4	5.1	0.55	50.6	121.4	258.0	31.6	153.9	146.9	.526	0 C	.744
61D	1	2.4	5.1	0.65	49.3	118.3	251.3	32.2	150.4	143.8	.512	0	.778
6RA	1	2.4	2.1	0.45	75.9	186.6	159.5	37.6	152.8	—	.329	—	—
6RB	1	2.4	2.1	0.55	36.4	87.5	76.6	21.7	141.6	142.0	.316	6" C	.976
6RC	1	2.4	5.1	0.55	25.3	60.6	129.1	15.7	153.9	146.1	.528	½" CO	.742
6RD	1	2.4	5.1	0.65	23.0	55.2	117.2	16.7	150.3	—	.514	—	—
6AB	1	2.4	2.1	0.55	72.8	175.0	153.2	43.5	148.2	147.9	.319	2" CO	.917
6LB	1	2.4	2.1	0.55	72.8	175.0	36.8	42.3	109.0	108.8	.320	½" C	.825

NB. SLUMP C = COLLAPSE. DENSITIES IN LB/FT^3.
CO = COHESIVE.

FIG. 6.7.1. PROPORTIONS AND WORKABILITY OF MIXES.

The materials were mixed in a 1 cu.ft. rotating drum mixer for the uniaxial test series and in a 2 cu.ft. rotating pan mixer for the remainder of the experimental work.

Mixing procedure was to mix the aggregate and water together for three minutes, and then add the cement and mix for another three minutes. The mix was then tipped out onto damp steel sheets and mixed a little by hand where necessary to overcome segregation. In the biaxial compression test series, two batches of 1.5 cu.yd. each were required to cast the fifteen test specimens and in this case the two batches were mixed together on the steel sheets.

Slump and compacting factor tests as described in section 6.7. were then performed on the mix.

Steel moulds were used for all specimens. The beam mould 4" x 4" x 20" was a standard BS.1881: 1952[156] design whilst the prism would was simply a beam mould with a stop-end at 12 in. made to the same tolerance as the cube and beam moulds of BS.1881. The design of the uniaxial tension mould has been described by Ward[7] but modified for this research project (see section 7.2.) and the biaxial compression slab mould by Robinson[10]. These moulds, lightly greased, were placed on the vibrating table and compacted in two layers to obtain a good degree of compaction (1% voids) without excessive segregation.

As soon as the top surface of the specimen had set the moulds were covered with damp hessian and then a polythene sheet to reduce evaporation. The moulds were stripped after 18-22 hours and the specimens immersed in water in the temperature controlled ($68^{o}F \pm 2^{o}F$) curing tanks until the day before testing (see section 6.9.).

6.7. Tests on Fresh Concrete

British standard slumps and compacting factor tests were conducted on all biaxial compression mixes in accordance with BS.1881: 1952[156]. The weight of the fully compacted concrete required in the compacting factor calculation was assessed from the mix proportions.

The results of these tests together with the actual mix proportions used and the corrections applied to allow for water absorption by the different aggregates are given in fig.6.7.1.

As Ward[7] has pointed out, neither of these tests are suitable for lightweight concrete mixes and so these tests were not performed.

6.8. Tests on Hardened Concrete

Under this loading are included non-destructive tests of hardened concrete only. Destructive tests are described in Chapters 7-10.

The non-destructive tests were of three types:-

1) Density measurement
2) Ultrasonic pulse velocity
3) Resonant frequency.

These three tests are used to determine the density of the concrete and its dynamic elastic modulus and Poissons ratio. Three 4 x 4 x 20 standard beam specimens were used from each mix and the test procedures adopted were those developed by Ward[7].

The results of these tests are given in Chapter 8 along with the results of the biaxial compression test series.

6.9. Preparation of Specimens for Testing

The procedure established by the Concrete Materials Research Group[10] for preparing a specimen for testing has been followed throughout.

The test specimens were removed from the curing tank about 24 hours before testing, washed in tap water to force them from all loose laitance, and surface dried with a towel.

A single coat of Febcure super curing compound (about .0006 in. thick) was immediately painted over the whole specimen except on any pre-marked areas where electrical resistance strain gauges or glass pads were to be stuck.

This coat was allowed to dry for about three hours and then a second coat was added giving a total thickness of about .0012 in. (see section 2.5.).

The areas free of curing compound were scraped to remove laitance and the electrical resistance strain gauges and glass pads stuck onto the surface as described in section 5.2.

The curing compound and the strain gauge adhesive then had about 16 hours to dry before testing.

CHAPTER 7

UNIAXIAL TEST SERIES

7.1. Introduction

The need to look more closely at the mechanical behaviour of mortars and concretes under uniaxial states of stress has already been referred to in section 1.8. as an adjunct to the work under combined states of stress and as an opportunity to take advantage of the recent developments in measuring techniques described in section 6.2.

As mentioned in section 6.2.1., the uniaxial tests described in this chapter were carried out as control tests on all the biaxial compression test mixes as well as being carried out on those mixes selected in section 6.4. for uniaxial tests only. Therefore, in this chapter, all the uniaxial tests are taken together (both from the uniaxial test series and the biaxial test series) so that a complete picture of all the results may be obtained.

7.1.1. States of Stress

From lateral deformation profiles on specimens of various shapes, Lachance[1-3] verified that prisms or cylinders with a height to width ratio of at least 2.5 provide a reliable uniaxial compression test in which the middle third of the specimen is in a uniform state of compression. It is noteworthy that this same conclusion was reached on the basis of ultimate strength values by Ros and Eichinger in 1928[24] and tentatively by Foppl and Bach[132].

In addition a reliable uniaxial tension test has been developed and proved by Ward[7] in which coefficients of variation of the same order as those usual in well controlled uniaxial compression tests, i.e. 3-4%, can be obtained. This test has the additional advantages of having all the necessary equipment available

in the Concrete Materials Research Laboratory (see section 1.7.) and of being a remarkably robust testing method requiring no more specimen preparation than the more usual test methods.

By adopting this test method for uniaxial tension, it is also possible to obtain a direct comparison between the uniaxial compressive and tensile states of stress over similar values, as the uniformly stressed volume in the 12" x 4" x 4" compression prism measures 4" x 4" x 4" and the equivalent volume in the Ward tension specimen is 4" x 3" x 4". (see fig.5.4.1.).

The British Standard flexural test[156] was included in the series as it was the one standard reference test in which the state of stress had been carefully investigated[178] and found to be of an almost fundamental character. The beam tests provided a link between the direct and flexural states of stress and the beam specimens were also required for density, dynamic elastic modulus and dynamic Poisson's ratio measurements as explained in Chapter 6.

However, the cylinder splitting tensile test was omitted from the test series as the actual state of stress achieved is rather complex and a relationship between the results of this test and the direct uniaxial tension test using Ward's apparatus had already been established by Ward[7]. Furthermore, at this stage, the laboratory casting capacity was strained to its limit and so only the three fundamental tests described above were carried out.

7.1.2. Strain and Cracking Measurements

For these tests, the Sogenique linear displacement transducer equipment described in section 5.3. was available. This had proved reliable to within $\pm\frac{1}{2}\%$ or 2 microinches for measuring the lateral strain of mortar prisms in the Platen Restraint trials described in section 2.5. Preliminary tests on tension specimens showed that the same order of sensitivity could be achieved in the measurement of lateral strain in this test as well.

The results of Robinson's work[10] on crack detection and especially his verification of the use of the ultrasonic pulse velocity equipment in detecting the onset of major cracking or discontinuity point were known and available. This technique offered a more positive definition of the discontinuity point than could be obtained from the stress-strain curve. Hence the ultrasonic pulse velocity technique was used in all three tests to measure the discontinuity point. Occasional checks using the stress-strain curves simply verified its reliability. The method of recording the pulse velocity, the change in velocity, and the location of the transducers in the three tests have all been detailed in section 5.4.

7.1.3. Concrete Mixes

Apart from acting as a useful adjunct to the main series of biaxial compression tests in the search for a comprehensive failure criterion, this experimental work was intended to extend the recent contributions of Kaplan[21] and Ward[7] in the uniaxial testing field.

As was mentioned in planning the test series (section 6.2.), Kaplan had shown a significant trend relating the cleavage strain at discontinuity with the volume fraction of coarse aggregate in the mix. However, his range of aggregates was restricted to a rounded gravel and a crushed limestone which, although having difficult aggregate textures, may have been of similar elastic modulus. In addition, some of the mixes with a small proportion of coarse aggregate (i.e. <30%) would have had a very high slump and it is difficult to see how these could have been properly compacted without serious segregation.

It was decided, therefore, to investigate this trend in more detail using a wider range of aggregate types but restricting the coarse aggregate volume fractions to the two nominal figures of 0.30 and 0.50. These coarse aggregate volume fractions represent the extreme ends of the range of workable concretes since mixes with

volume fractions less than 0.30 are normally subject to considerable segregation whilst those greater than 0.50 are too coarse and unworkable for proper compaction. Lightweight aggregate was also included in the selection of aggregate types as it was thought that the ratio of the elastic modulus of the aggregate to that of the surrounding mortar matrix was important in the failure of concrete.

For statistically significant results to be obtained, the number of specimens of each type cast from each mix was fixed at three for all uniaxial tests.

7.1.4. Cleavage Strains at Discontinuity

A point not made in Kaplan's report[21] is the much larger cleavage strain at discontinuity measured on uniaxial compression specimens than on uniaxial tension specimens. As no direct measurements of the lateral strain under uniaxial compression were made in his tests, the value of Poisson's ratio determined by the ASTM method (ASTM C215-60) had to be used. A comparison of his results from uniaxial compression and tension tests is given in fig.7.1.4. from which this phenomenon is clearly apparent. In the test series to be described, the cleavage strain in compression was obtained by direct measurement using the linear displacement transducer and frame to give a check on the results of Kaplan.

Ward[7], in developing his tension testing machine, carried out a series of tests in which he compared his uniaxial tension results with those from several other simple loading tests including uniaxial compression. However, in his tension tests, Ward relied upon the stage at which the stress-strain curve became non-linear to obtain the discontinuity point because the strain gauges used to measure the lateral strains in the uniaxial tension specimen were accurate to ± 5 microstrain. In uniaxial tension, the lateral strains at failure are only of the order of 15 microstrain depending on the mix.

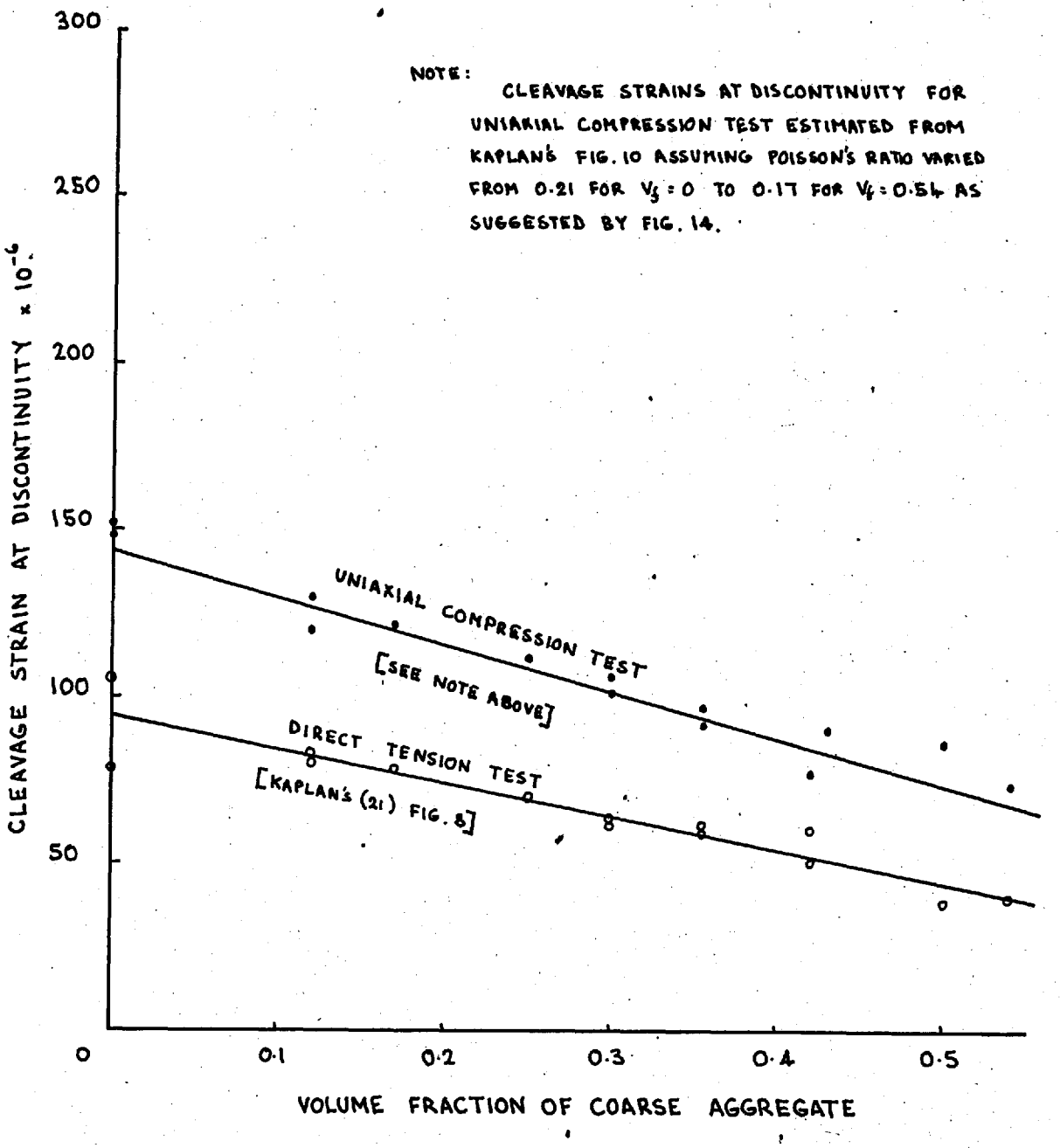


FIG. 7.1.4.1. VARIATION OF CLEAVAGE STRAIN AT DISCONTINUITY WITH VOLUME FRACTION OF COARSE AGGREGATE (FROM KAPLAN(21)).

Therefore, a further purpose of these tests was to check Ward's tension results and provide more definite information on the discontinuity stress and the lateral strain behaviour. Several of the aggregates used by Ward were incorporated in this test series so that a direct comparison could be made.

7.1.5. Rates of Loading

As these are all short-term tests, the rates of loading are especially critical.

The British Standard 1881:1952[156] recommends a comprehensive rate of loading of 2,000 p.s.i./min. which is considered to be disproportionately high when compared with the 100 p.s.i./min. recommended for the flexural beam test. Furthermore the strain and pulse velocity measurements would become impossible to take in the biaxial compression tests at this high rate of loading. Therefore, as recommended by Robinson[10], the loading rate in uniaxial compression was reduced to 700 p.s.i./min. Assuming equal elastic modulus under uniaxial tension and compression (7) and a Poisson's ratio of about 0.15, this gives a loading rate of 110 p.s.i./min. for the tensile tests if the ultimate strength is to be reached after the same time in compression and tension.

The rates of loading were fixed therefore as follows:-

Uniaxial Compression	700 p.s.i./min.
Uniaxial Tension	110 p.s.i./min.
Flexure (Extreme Fibre)	110 p.s.i./min.

These are identical to those used by Robinson. Ward's rate of loading for tension tests was 100 p.s.i./min. but this is unlikely to lead to significantly different results. Kaplan's rates of loading were also of a similar order being 1060 p.s.i./min. for uniaxial compression and 150 p.s.i./min. for tension and flexural tests which is also the ASTM standard rate.

A supplementary series of uniaxial compression tests was also carried out to investigate the suggested connection between the discontinuity stress at which major cracking occurs and the long-term ultimate strength of concrete (21, 101, 102). A number of prisms were loaded at the short-term rate of loading to load stages above and below the discontinuity stress and then the load was maintained at a constant value. After this stage, the rate of strain increase with time at constant load was recorded and taken to be a measure of structural deterioration. These tests are also described in this chapter.

7.1.6. Age and Condition of Specimens at Test

All uniaxial test specimens from a particular mix were always tested on the same day and this was at 27-29 days after casting.

The specimens were all tested in a saturated condition which was maintained throughout the test by the use of a curing compound as described in section 6.9.

7.2. Testing Machines and Equipment Used

Equipment availability and laboratory reorganisation meant that two different machines had to be used at different times for both the uniaxial compression and the flexural tests.

All uniaxial compression tests on mixes tested under uniaxial states of stress only were tested in the 200 ton. compression machine of Robinson described in section 1.7. For uniaxial compression tests on the biaxial compression test series mixes, the combined stress testing machine was used.

With the flexural tests, Sigvaldason's flexural testing machine (described in section 1.7.) was used throughout the uniaxial test series and also on some of the biaxial compression test series specimens. The remaining mixes were tested in an earlier standard

beam testing machine described by Ward and listed in section 1.7. Whereas the Sigvaldason machine had adequate stability and good hydraulic control, the older machine tended to rack and had poor hydraulic control with the result that it was difficult to maintain a steady loading rate. Therefore the flexural beam test results given in tables 7.5.1.1. and 7.5.1.2. have been marked to indicate which testing machine was used.

All uniaxial tension tests were conducted using Ward's uniaxial tension testing machine listed in section 1.7.

One important modification was made to the uniaxial tension moulds used by Ward and Robinson after it was discovered that the actual dimensions of the reduced section did not agree with those specified by Ward. It was decided to make new inserts with a flat section to accommodate a 4" compression cube for the biaxial compression-tension series of tests and to make the thickness of the reduced section up to the round figure of 3" - there being no advantage in the fractional figure of 2.86 in. used previously.

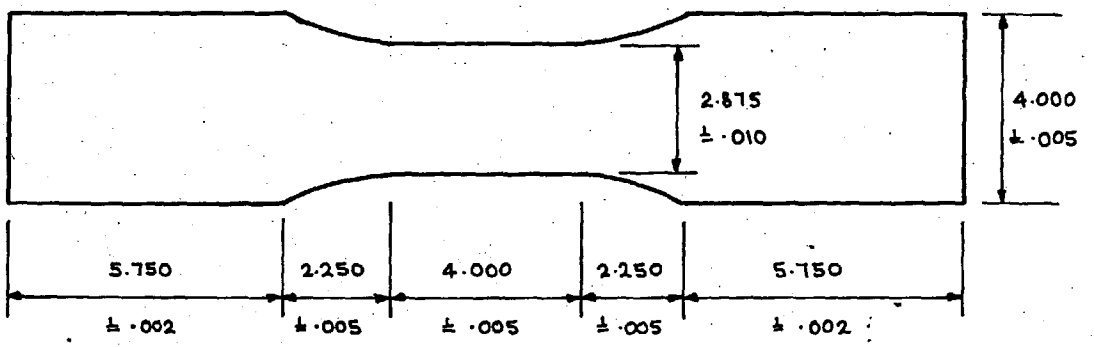
The actual and specified dimensions of the Ward specimen together with the modified specimen dimensions are given in fig. 7.2.1.

Ultrasonic pulse velocity and Sogenique linear inductance transducer equipment used has been described in Chapter 5. For reading the electrical resistance strain gauges used to measure the longitudinal strains on all three types of specimen, a small Peckel transistorised strain measuring box type T200 was used.

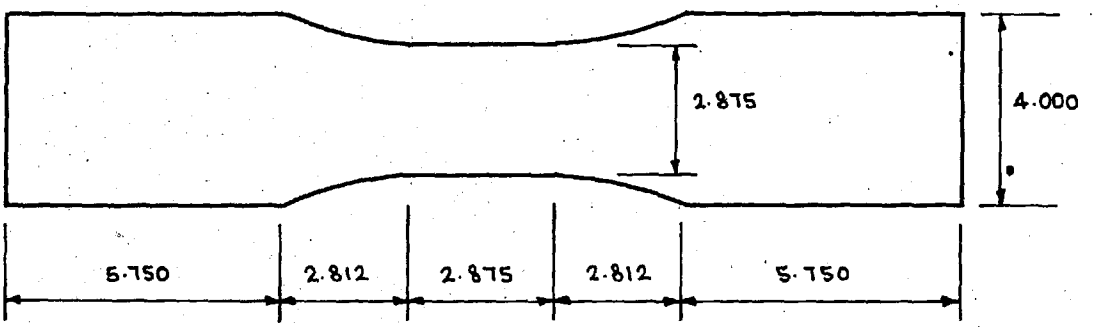
For the long-term strength tests on uniaxial compression specimens, the combined stress testing machine was used throughout.

The loading rates and constant load timing were maintained in all the short-term and long-term tests using a stop watch graduated in tenths of a second and able to record the total time up to 1 hour.

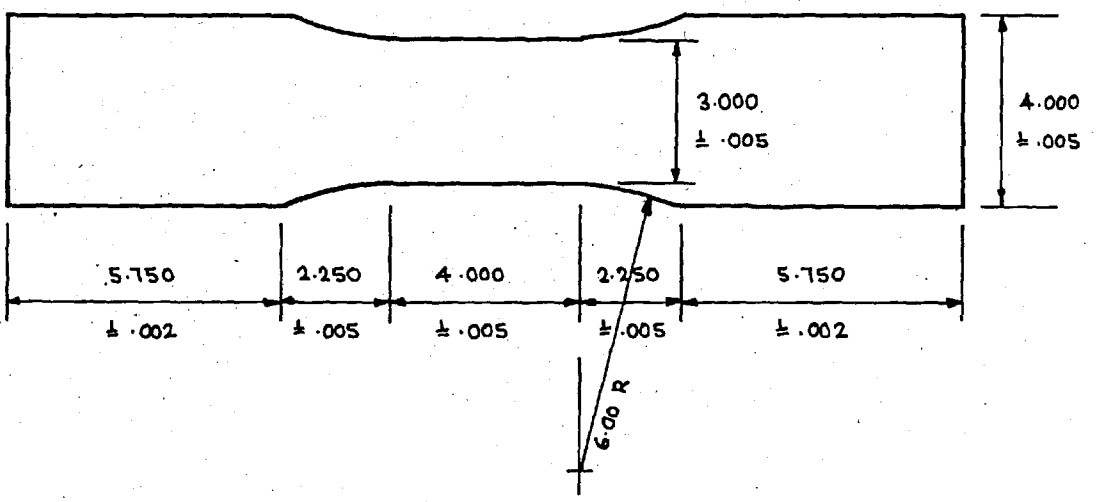
NB: ALL DIMENSIONS ARE IN INCHES.



1) DIMENSIONS SPECIFIED BY WARD (7).



2) ACTUAL DIMENSIONS OF WARD'S MOULD.



3) MODIFIED MOULD BY VILE.

FIG. 7.2.1. DIMENSIONS OF UNIAXIAL TENSION SPECIMEN.

7.3. Testing Arrangement

The arrangement of the ultrasonic pulse velocity transducers with particular reference to their scanning action across the critical section and normal to the anticipated cleavage planes has previously been specified in section 5.4. and was strictly adhered to in these tests.

The positions of the electrical resistance strain gauges, the pulse velocity transducers, and the location of the linear displacement transducer frame for all these tests are illustrated in fig.7.3.1. The two active electrical resistance strain gauges were coupled with two identical dummy gauges to provide a correction for temperature and connected in series so that the strains on the two opposite faces of the test specimen were automatically averaged. In the case of the flexural test only one active and dummy gauge were used.

A summary of the strain and pulse velocity measurements taken in each of the three tests is given below:-

1) Uniaxial Compression (short and long-term tests)

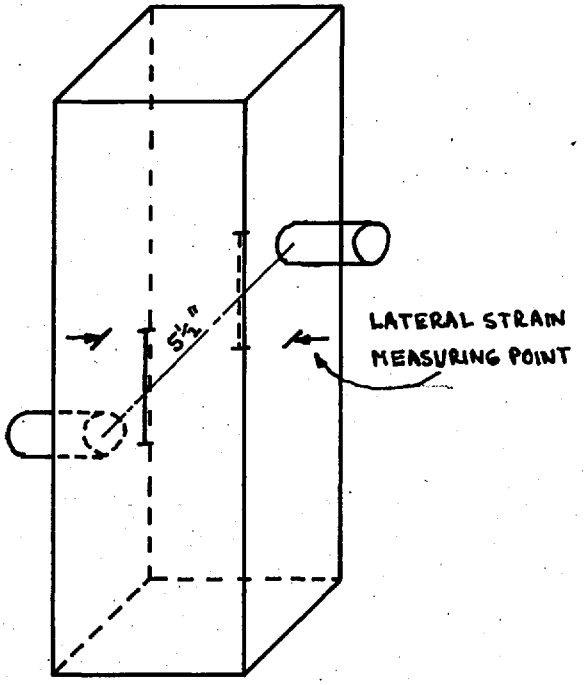
Longitudinal strain	2 x 30 mm. E.R.S.G.
Lateral strain	4 in. guage length - Sogenique
Ultrasonic pulse velocity	$5\frac{1}{2}$ in. path length

2) Uniaxial Tension

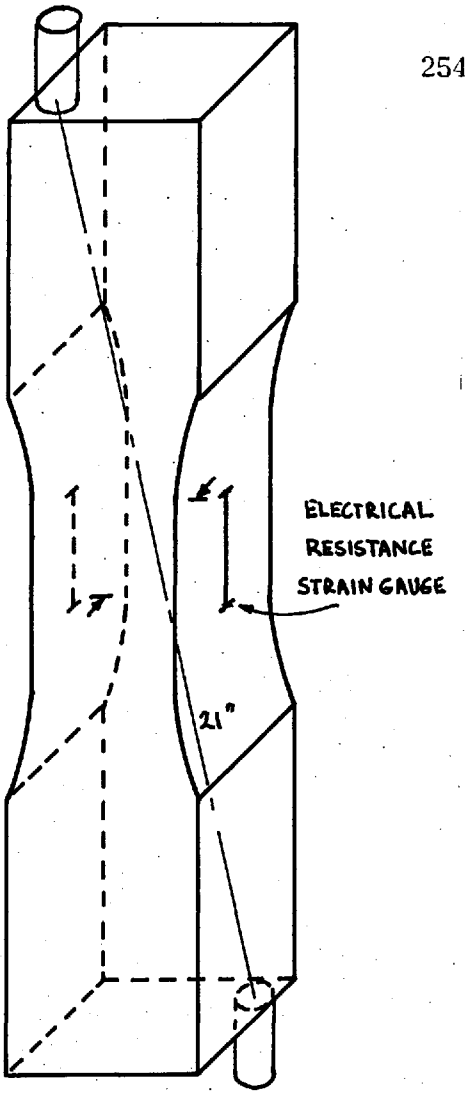
Longitudinal strain	2 x 30 mm. E.R.S.G.
Lateral strain	4 in. gauge length - Sogenique
Ultrasonic pulse velocity	21 in. path length

3) Flexure

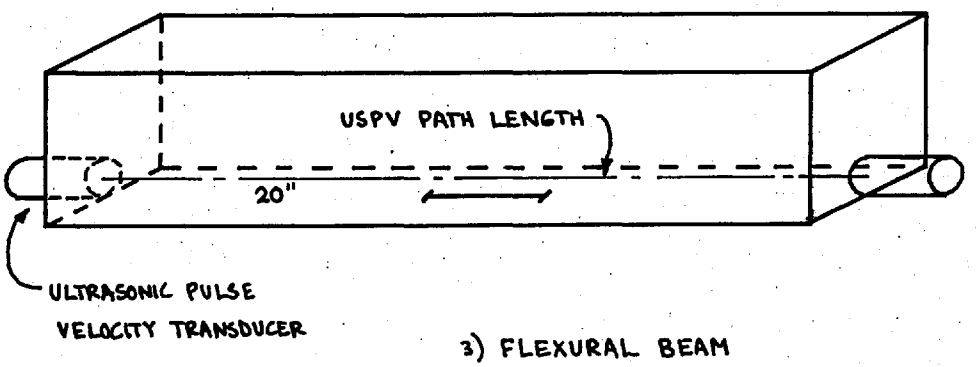
Longitudinal strain	1 x 30 mm. E.R.S.G.
Lateral strain	-
Ultrasonic pulse velocity	20 in. path length



1) UNIAXIAL PRISM



2) UNIAXIAL TENSION SPECIMEN



3) FLEXURAL BEAM

FIG. 7.3.1 UNIAXIAL SPECIMEN TEST SET-UPS.

7.4. Test Procedures

7.4.1. Uniaxial Compression Test

The test specimen was firstly set up in the testing machine and then the measuring devices listed in section 7.3. were attached to it. The specimen was then loaded and unloaded at the rate of 700 p.s.i./min. to a maximum load of about 30% of the estimated ultimate load three times.

This procedure enabled a check to be made on the attachment of the measuring devices to the specimen and on the functioning of the ancillary equipment. A drift in the readings of the ancillary equipment served as an indication of something being wrong.

The loading and unloading procedure may also have had the effect of extending local bond failures around the aggregate particles and hence making the stress-strain curve up to this load stage more linear. This effect is discussed in detail in Chapter 12. However, this initial loading was kept below a level where it might have exceeded the discontinuity stress to avoid any artificial elevation of the discontinuity point.

Providing all the equipment was functioning properly, the specimen was loaded and unloaded twice at a rate of 700 p.s.i./min. to a maximum load of about 25% of the estimated ultimate load. During this second set of loading cycles, longitudinal and lateral strains were measured at zero load and maximum load to obtain representative values of elastic modulus and Poisson's ratio. If any drift in the strain readings was noted during this test, the loading and unloading cycles were repeated until consecutive readings agreed to within 2% or 1 microstrain whichever was the greatest. On rare occasions the test had to be stopped and the faulty item of equipment re-attached to the specimen.

Once these measurements of the elastic constants were complete, the specimen was loaded to failure at the specified loading rate of 700 p.s.i./min. taking measurements of longitudinal and lateral strain, and ultrasonic pulse velocity at convenient pressure gauge readings. For a concrete or mortar with an ultimate load of about 6,000 p.s.i., the number of load stages at which these measurements were taken was twelve. Hence an estimate of the number of load stages used for any other specimen can be obtained on a pro-rata basis.

Once the discontinuity stress had been exceeded as indicated by a sharp decrease in the ultrasonic pulse velocity and a marked increase in lateral strain, the linear displacement transducer frame was removed from the specimen to avoid possible damage to the transducer when the specimen failed. Moreover, subsequent readings of lateral strain have little or no significance since major cracking has occurred and the material is no longer continuous.

7.4.2. Uniaxial Tension Test

The procedure for the uniaxial tension test was in general similar to that for the uniaxial compression test in that, after setting up the specimen in the testing machine in accordance with the procedure laid down by Ward[7], the measuring devices were all attached. The specimen was then loaded and unloaded three times at the specified loading rate of 110 p.s.i./min. (see section 7.1.5.) to a maximum load of about 30% of the estimated ultimate load. Extensive use was made of the tension strengths obtained by Ward in estimating the ultimate load for this purpose.

Then measurements of the elastic modulus and Poisson's ratio were made by loading and unloading the specimen twice at the rate of 110 p.s.i./min. to a maximum load of about 25% of the estimated ultimate load.

Upon satisfactory completion of this test, the specimen was loaded to failure at a rate of 110 p.s.i./min and measurements of longitudinal and lateral strain, and ultrasonic pulse velocity taken at convenient load stages. For a concrete or mortar with an ultimate tensile strength of 500 p.s.i., the number of such load stages would be sixteen.

7.4.3. Flexural Test

The procedure for setting-up the flexural beam test specimen has already been described in detail by Ward and was in accordance with British Standard B.S.1881.

This test was treated as a standard beam test and no preliminary loading and unloading cycles or elastic coefficient measurements were made.

The beam was loaded directly to failure at the specified rate of stress increase at the extreme fibre of 110 p.s.i./min. (see section 7.1.5.). The tensile strain along the axis of the beam was measured at the mid-span using an electrical resistance strain gauge and, for those tests conducted in Sigvaldason's flexural machine, ultrasonic pulse velocity measurements were taken as described in section 5.4. These measurements were taken at convenient load stages; a beam with a flexural tensile strength of 700 p.s.i. having fourteen such load stages.

7.4.4. Long-term Compression Test

In these tests, the testing arrangement was identical to that employed in the short-term compression tests but the loading procedure was in many respects different.

The initial loading and unloading at a rate of 700 p.s.i./min. was carried out three times on each specimen to a maximum load of 30% of the known short-term ultimate load (the mix used was 61B and had already been tested in short-term tests).

Each specimen was then loaded at the specified rate of 700 p.s.i./min. to a series of different load stages and at each load stage the load was maintained constant for five minutes whilst the rate of strain increase in the longitudinal direction was recorded. A final load stage would then be reached either above or below the discontinuity point at which the load would be maintained for a much longer period of time, usually one hour, or until failure occurred.

In this way the rate of increase of longitudinal strain and the time taken to reach ultimate load at stress levels below the short-term ultimate strength but above the discontinuity stress were obtained.

7.5. Test Results

7.5.1. Tabulated Results

The summarised results of the short-term tests are given in tables 7.5.1.1. and 7.5.1.2. and of the long-term tests in tables 7.5.1.3. and 7.5.1.4.

The methods of analysing the results are illustrated in Appendix D where typical concrete and mortar mixes are analysed from the original experimental measurements to show how the summarised results above were obtained.

The complete set of experimental record sheets together with the calculations of the analysis are contained within a box file deposited with the Concrete Materials Research Group at Imperial College and access to these sheets can be obtained through Mr. K. Newman.

7.5.2. Stress-Strain Characteristics

A selection of representative stress-strain characteristics for mortar, normal aggregate concrete, and lightweight concrete under

MIX	ULTIMATE STRESS psi				DISCONTINUITY STRESS psi				ELASTIC MODULUS $\times 10^6$ psi				POISSONS RATIO				DISCONTINUITY CLEAVAGE STRAIN $\times 10^6$			
	C_1	C_2	C_3	AV.	C_1	C_2	C_3	AV.	C_1	C_2	C_3	AV.	C_1	C_2	C_3	AV.	C_1	C_2	C_3	AV.
MA	7250	7125	7100	7158	5400	5400	5100	5300	—	—	—	—	—	—	—	—	270	274	356*	272
MBC	4650	4680	4350	4550	2400	2780	2780	2653	—	3.80	3.68	3.74	.168	—	.158	.163	—	184	174	179
MD	4000	2800*	3630	3815	2800	—	2450	2625	3.18	2.91	—	3.05	.188	—	.210	.199	194	—	196	195
31A	4950	4780	5025	4918	3500	3550	3190	3413	5.08	5.47	5.09	5.21	.16	.18	.18	.17	137	130	113	127
31B	5350	5000	4300	4883	3450	3450	3180	3360	5.04	4.82	4.74	4.87	.137	.122	.087	.115	144	142	123	136
31C	3830	3680	3380	3630	1800	1500	1500	1600	5.22	5.24	4.88	5.11	.104	.100	.095	.100	38	36	31	35
31D	3790	3820	3820	3810	2950*	2780	2880	2830	4.40	4.97	5.42	4.93	.17	.14	.18	.16	113*	97	97	97
61A	5100	4820	4500	4807	2800	2130	2400	2443	5.43	5.36	5.56	5.45	.129	.136	.105	.123	87	68	81	79
61B	4120	4130	4130	4127	2130	2130	2130	2130	5.00	4.87	5.13	5.00	.141	.154	.155	.150	69	77	76	74
61C	3850	4010	4250	4028	2150	2430	2150	2243	6.04	5.75	6.01	5.93	.11	.14	.10	.12	50	68	31	50
61D	4500	4800	4500	4600	3130	3450	3130	3237	5.53	6.11	5.86	5.83	.100	.107	.109	.105	92	98	93	94
6RA	6720	6090	6200	6337	4130	5010	4300	4483	5.73	6.63	5.92	6.09	.15	.17	.18	.17	130	135	130	132
6RB	4400	3900	4420	4240	2780	2780	2780	2780	4.65	—	4.93	4.79	.143	—	.136	.140	119	—	108	114
6RC	4600	4850	—	4725	2800	3400	—	3100	6.39	6.38	7.00	6.59	.080	.104	—	.092	61	62	—	62
6RD	2950	2980	2730	2887	1960	2050	1660	1890	4.76	5.58	6.04	5.46	.20	.15	.16	.18	73	69	58	67
6AB	6350	6500	5400	6083	3760	4100	4100	3987	5.33	5.03	4.60	4.99	.192	.167	.126	.162	211	190	223	208
6LB	2450	2100	2700	2417	1750	1750	1500	1667	2.57	2.50	2.38	2.48	.212	.256	.191	.220	176	217	133	175
6LA'	1900*	2503	2200	2352	—	1720	1600	1660	2.19	2.18	1.93	2.10	.31*	.17	.19	.19	—	150	156	153

X DENOTES SUSPECT RESULT

— DENOTES NO READING TAKEN.

TABLE. 7.5.1.1. UNIAXIAL TEST RESULTS. (PART 1 of 3 — UNIAXIAL COMPRESSION)

MIX	ULTIMATE STRESS psi				DISCONTINUITY STRESS psi				ELASTIC MODULUS $\times 10^6$ psi				POISSONS RATIO				DISCONTINUITY CLEAVAGE STRAIN $\times 10^6$			
	T_1	T_2	T_3	AV.	T_1	T_2	T_3	AV.	T_1	T_2	T_3	AV.	T_1	T_2	T_3	AV.	T_1	T_2	T_3	AV.
MA	475	432	543	483	371	342	400	371	5·41	5·08	–	5·25	·245	·300[X]	–	·272	95	83	96	91
MBC	386	412	371	390	251	223	282	252	4·07	4·36	4·06	4·16	–	·103	·114	·109	73	62	90	75
MD	400	391	400	397	283	283	310	292	5·38[X]	3·45	3·67	3·56	–	·212	·189	·201	91	95	107	98
31A	360	342	344	349	310	283	283	292	5·40	6·35	5·80	5·85	·09	·13	·05	·09	72	68	64	68
31B	–	372	343	357	223	223	223	223	5·29	5·35	4·51	5·05	·137	·126	·224	·162	50	50	50	50
31C	302	325	372	333	194	223	223	213	4·35	4·35	5·80	4·83	–	–	–	–	42	51	45	46
31D	329	235[X]	307	318	235	173	266	225	6·05	5·58	5·56	5·72	·07	·07	–	·07	42	38[X]	66	54
61A	372	379	352	368	283	223	251	252	6·10	5·29	4·52	5·30	·192	–	·185	·188	54	48	52	51
61B	329	400	281	337	235	193	177	202	7·17	5·74	5·11	6·01	·197	·104	·213	·171	47	40	39	42
61C	402	338	372	371	235	283	194	237	6·11	5·80	7·02	6·31	·13	·14	·18	·15	44	53	29	42
61D	364	352	367	361	222	222	207	217	6·82	–	5·48	6·15	·121	–	·143	·132	43	–	38	40
62A	–	420	343	381	–	333	285	309	6·85	6·28	5·40	6·18	–	·14	–	·14	–	64	55	60
62B	282	371	341	331	–	–	–	–	4·35	5·41	5·17	4·98	–	–	–	–	–	–	–	–
62C	459	430	430	440	341	341	341	341	8·13	6·41	7·17	7·24	–	–	–	–	57	61	56	58
62D	392	426	372	397	329	343	313	328	6·03	5·74	5·67	5·81	·17	·09	·11	·12	59	61	60	60
6AB	400	417	460	426	283	283	311	292	5·53	4·36	4·52	4·80	·084	·150	–	·150	61	61	70	64
6LB	311	283	283	292	194	194	194	194	2·49	2·44	2·49	2·47	·172	·168	·193	·178	86	93	90	90
6LA'	235	233	232	233	180	180	180	180	2·26	1·94	1·89	2·03	·16	·10	·18	·15	100	120	120	113

X DENOTES SUSPECT RESULT
– DENOTES NO READING TAKEN

TABLE. T.S.1.2. UNIAXIAL TEST RESULTS (PART 2 of 3 – UNIAXIAL TENSION)

MIX	ULTIMATE STRESS psi.				DISCONTINUITY STRESS psi.				DISCONTINUITY CLEAVAGE STRAIN x 10^6				DYNAMIC ELASTIC MODULUS x 10^6 psi				DYNAMIC POISSONS RATIO			
	B_1	B_2	B_3	AV.	B_1	B_2	B_3	AV.	B_1	B_2	B_3	AV.	B_1	B_2	B_3	AV.	B_1	B_2	B_3	AV.
MA	787	798	834	806	620	620	620	620	174	160	173	169	5.32	5.29	5.22	5.28	.272	.279	.279	.277
MBC	535	608	612	585	291	291	350	311	73	75	91	80	4.35	4.42	4.38	4.38	.236	.236	.246	.239
MD	676	608	620	635	–	–	–	–	–	–	–	–	4.12	4.02	4.06	4.07	.236	.230	.240	.235
31A	700	675	–	688	291	346	–	319	63	70	–	67	–	–	–	–	–	–	–	–
31B	676	646	643	655	–	–	–	–	–	–	–	–	5.55	5.56	5.58	5.56	.209	.209	.209	.209
31C	632	606	593	610	–	–	–	–	–	–	–	–	5.55	5.80	5.90	5.75	.192	.212	.225	.210
31D	526	–	–	526	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
61A	595	675	675	648	–	403	403	403	–	68	78	73	6.20	6.08	6.01	6.10	.228	.204	.225	.219
61B	561	532	567	550	292	347	320	320	67	67	73	69	–	–	–	–	–	–	–	–
61C	739	677	655	690	512	457	457	475	77	65	73	72	6.74	6.71	6.52	6.66	.225	.225	.210	.220
61D	634	704	–	669	–	–	–	–	–	–	–	–	6.47	6.28	6.43	6.39	.230	.213	.204	.216
6RA	695	645	–	670	567	573	–	570	93	130	–	112	–	–	–	–	–	–	–	–
6RB	625	618	687	643	–	–	–	–	–	–	–	–	5.56	5.45	5.59	5.53	.210	.219	.200	.210
6RC	687	676	717	693	–	–	–	–	–	–	–	–	6.65	6.65	6.87	6.72	.192	.184	.175	.184
6RD	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
6AB	748	734	703	728	–	–	–	–	–	–	–	–	5.92	5.99	6.15	6.02	.253	.248	.242	.248
61B	559	559	489	536	–	–	–	–	–	–	–	–	2.96	2.92	3.01	2.96	.273	.270	.270	.271
CLA'	410	368	–	389	238	236	–	237	174	174	–	174	–	–	–	–	–	–	–	–

X DENOTES SUSPECT READING.
– DENOTES NO READING TAKEN

TABLE. 7.5.1.3. UNIAXIAL TEST RESULTS (PART 3 of 3 FLEXURAL TENSION AND DYNAMIC CONSTANTS).

LOAD STAGE psi.	RATE OF LONGITUDINAL STRAIN INCREASE ×10⁶ (δ_T = 5 MIN.) P_1	P_2	P_3
333	0·8	–	–
667	1·2	1·2	–
1000	1·0	–	–
1333	0·5	2·5	2·0
1667	1·0	–	–
2000	1·0	1·5	2·2
2333	1·5	–	–
2667	2·7	4·2	5·4
3000	2·5	–	–
3333	3·7	5·7	9·4
3667	5·2	–	–
4000	6·2	11·5	16·4
4333	8·2	–	–
4667	10·5	22·5	–
5000	20·2	–	FAILED
5333	45·5	26·7	
5667	FAILED	FAILED	
6000			

SPECIMEN	CONSTANT STRESS psi	σ_{pv} psi.	σ_u psi.	TIME TO FAILURE MIN.	TOTAL LONGITUDINAL STRAIN ×10⁶
P_4	NONE	2500	5333	6	1600
P_5	4000	2667	4000	132	≐ 1900
P_6	4333	3000	4333	71	2007

TABLE 7.5.1.4. UNIAXIAL TEST RESULTS (LONG TERM TESTS)

uniaxial compressive, tensile, and flexural states of stress are shown in figs.7.5.2.1. to 7.5.2.9. The discontinuity stress as determined from ultrasonic pulse velocity measurements has been plotted on these curves so that a comparison with the shape of the stress-strain curve can be made. From these curves it can be seen that the determination of the discontinuity point from the shape of the stress-strain curve could be misleading unless coupled with the measurements of lateral (tensile) strain.

In many of the stress-strain characteristics, the curve is remarkably linear over the section below the discontinuity stress. This may be due in part to the initial loading and unloading cycles of stress.

Under uniaxial tension stress, however, a number of mixes showed a marked curvature in their stress-strain characteristics below the discontinuity stress. This was especially noticed in the case of mortars and has been previously noted by Ward[7]. This emphasises once again the comparison between the methods of defining the discontinuity point. It would be very difficult in these cases of curvature to determine the discontinuity stress from the stress-strain curve whereas in all these cases the ultrasonic pulse velocity measurements gave a well defined point.

7.5.3. Discontinuity and Ultimate Stress

The variation of discontinuity and ultimate stresses with the water-cement and aggregate-cement ratios of mortars, the $\frac{3}{8}$" irregular aggregate concretes, and the $\frac{3}{4}$" irregular aggregate concretes is shown in figs.7.5.3.1. to 7.5.3.6. under uniaxial compression and tension. Some of Ward's ultimate stress results have been superimposed for comparison. The variations follow the well established trends.

A direct comparison between the tensile and compressive ultimate and discontinuity stresses is made in fig.7.5.3.7. Again some of Ward's ultimate stress values have been plotted as well. An

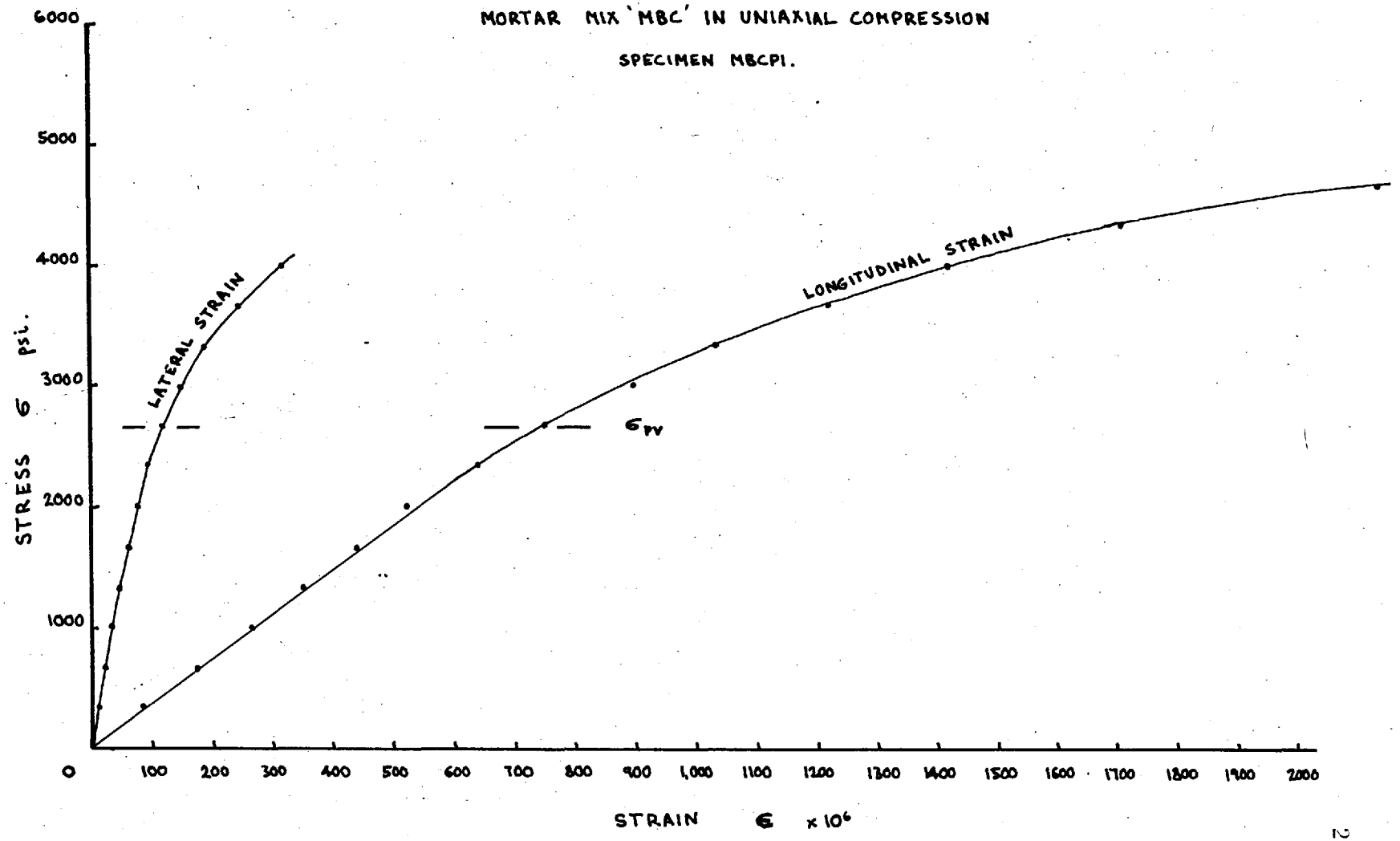


FIG. 7.5.2.1. STRESS-STRAIN CHARACTERISTICS IN UNIAXIAL COMPRESSION OF MBC.

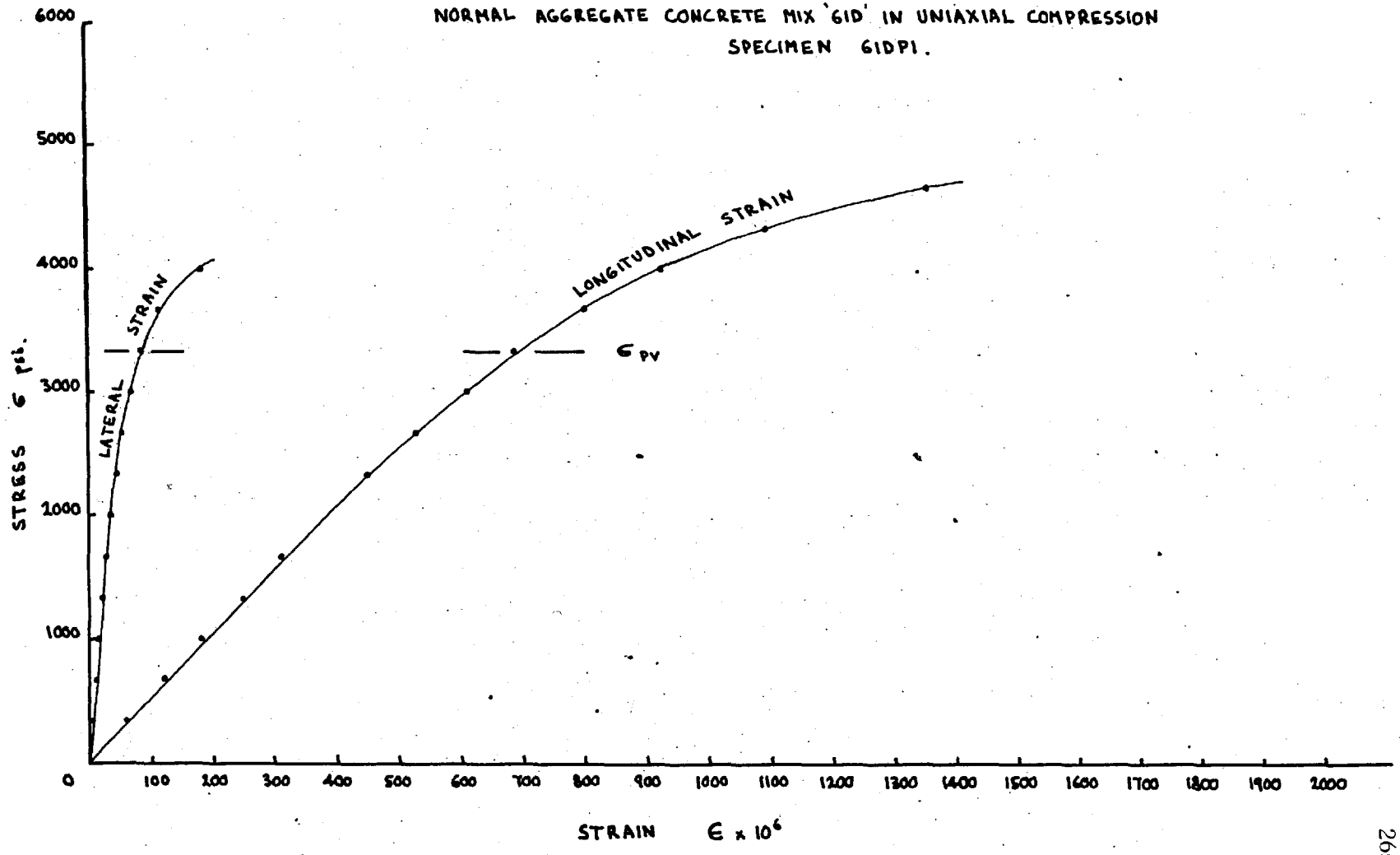


FIG. T.5.2.2. STRESS-STRAIN CHARACTERISTICS IN UNIAXIAL COMPRESSION OF 6ID.

FIG. 7.5.2.3. STRESS-STRAIN CHARACTERISTICS IN UNIAXIAL COMPRESSION OF 6LB.

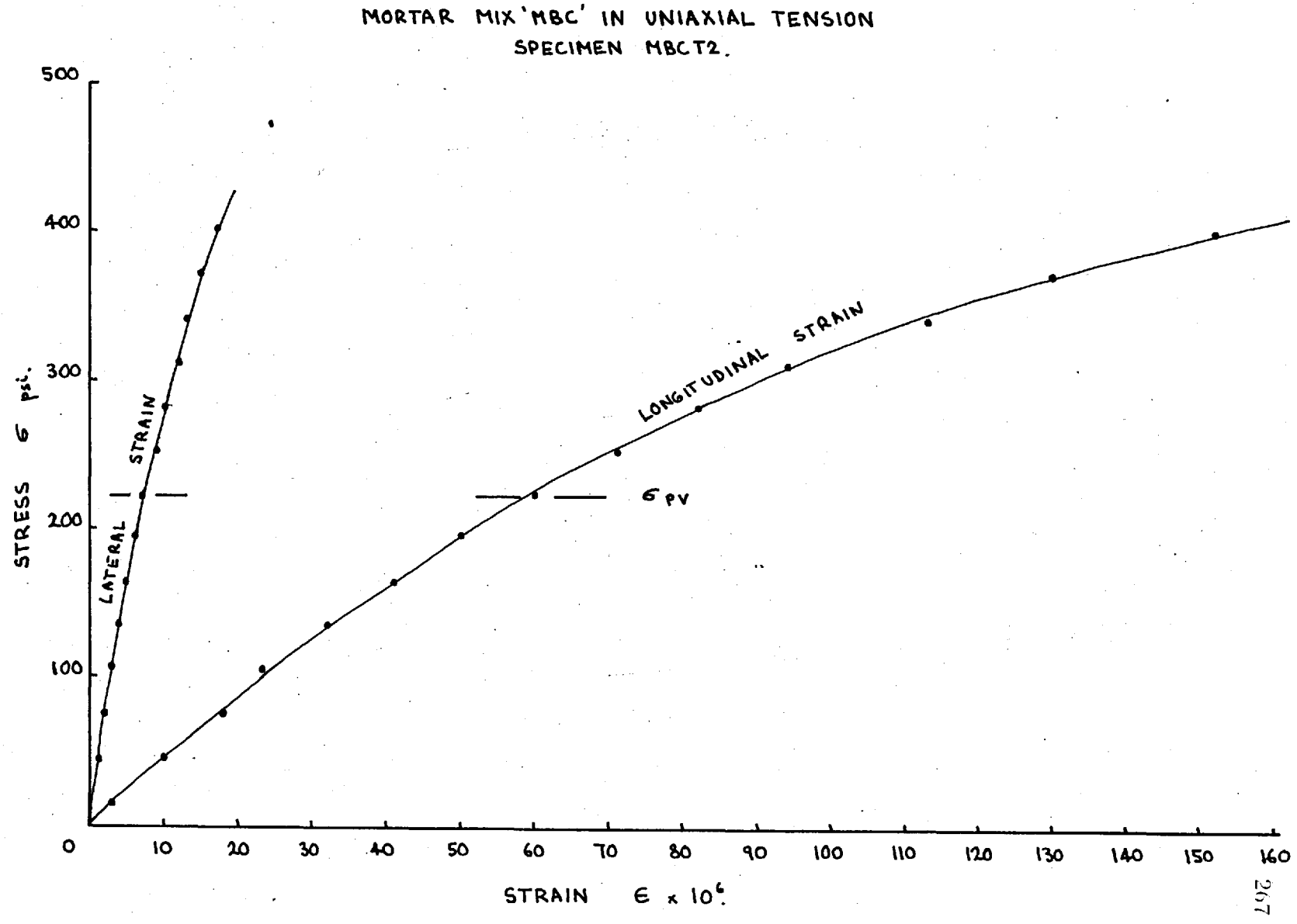


FIG. 7.5.2.4. STRESS-STRAIN CHARACTERISTICS IN UNIAXIAL TENSION OF MBC.

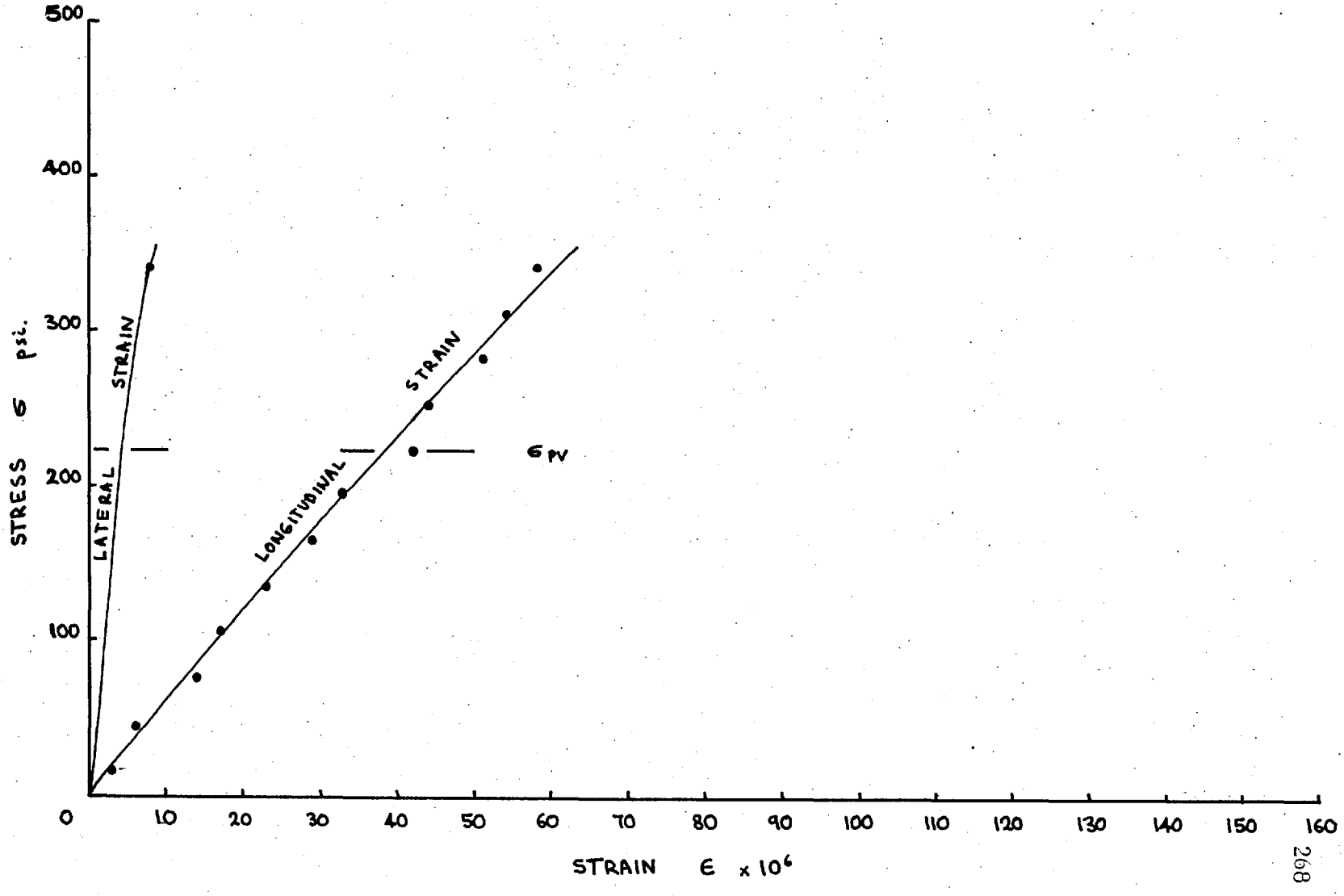


FIG. T.5.2.5. STRESS-STRAIN CHARACTERISTICS IN UNIAXIAL TENSION OF 61D.

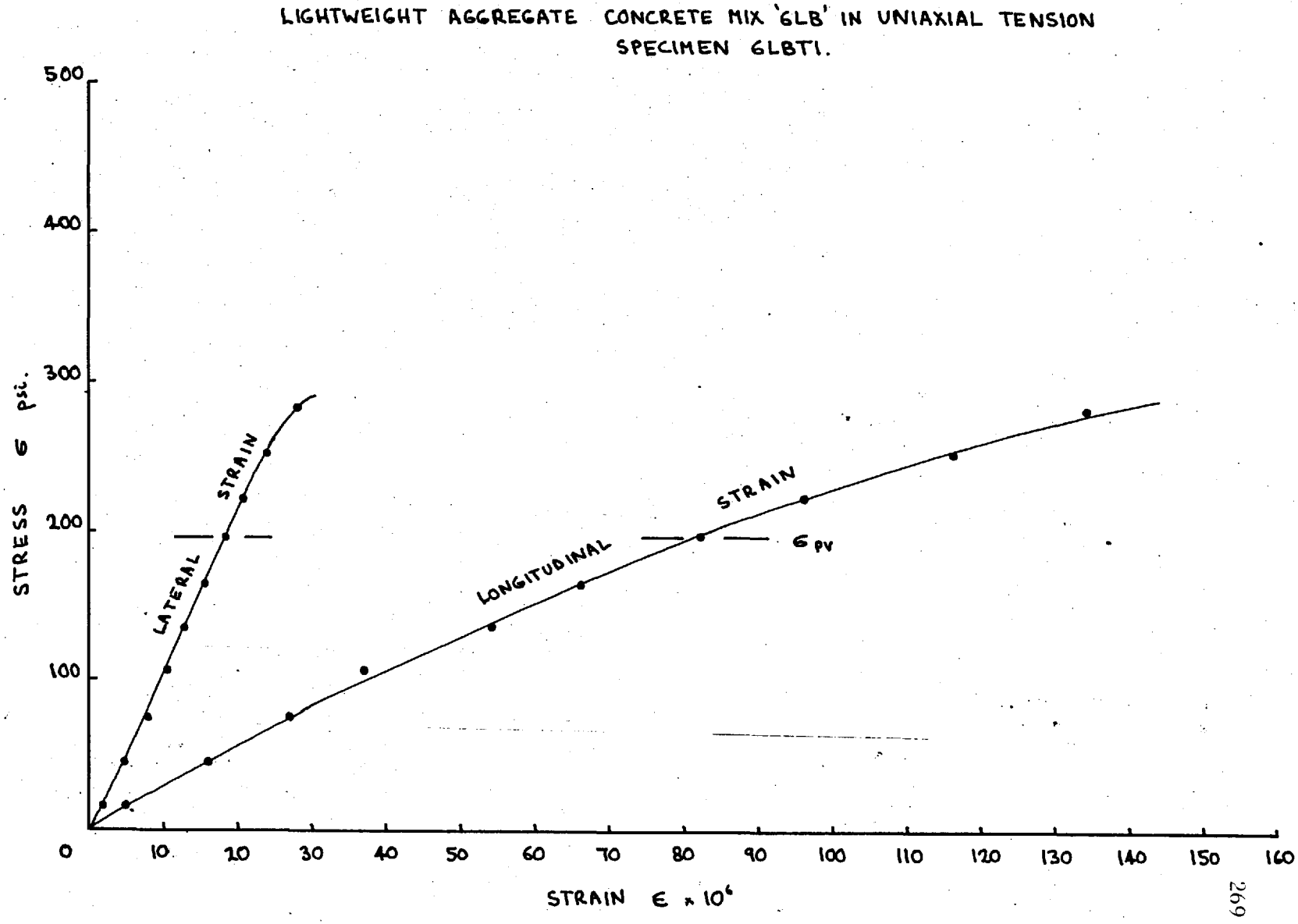


FIG. 7.5.2.6. STRESS-STRAIN CHARACTERISTICS IN UNIAXIAL TENSION OF 6LB.

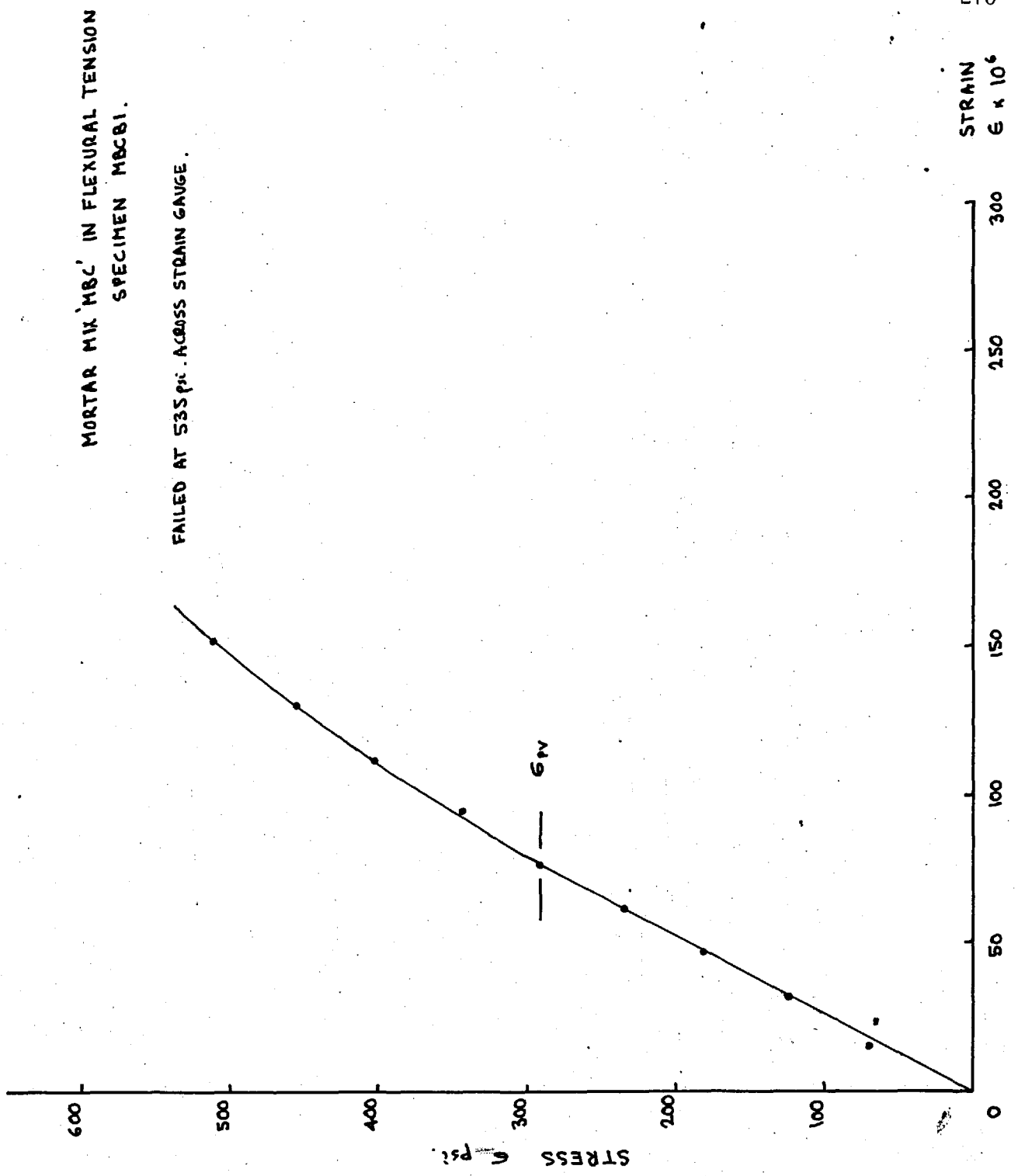


FIG. 7.5.2.7. STRESS-STRAIN CHARACTERISTICS IN FLEXURAL TENSION OF MBC.

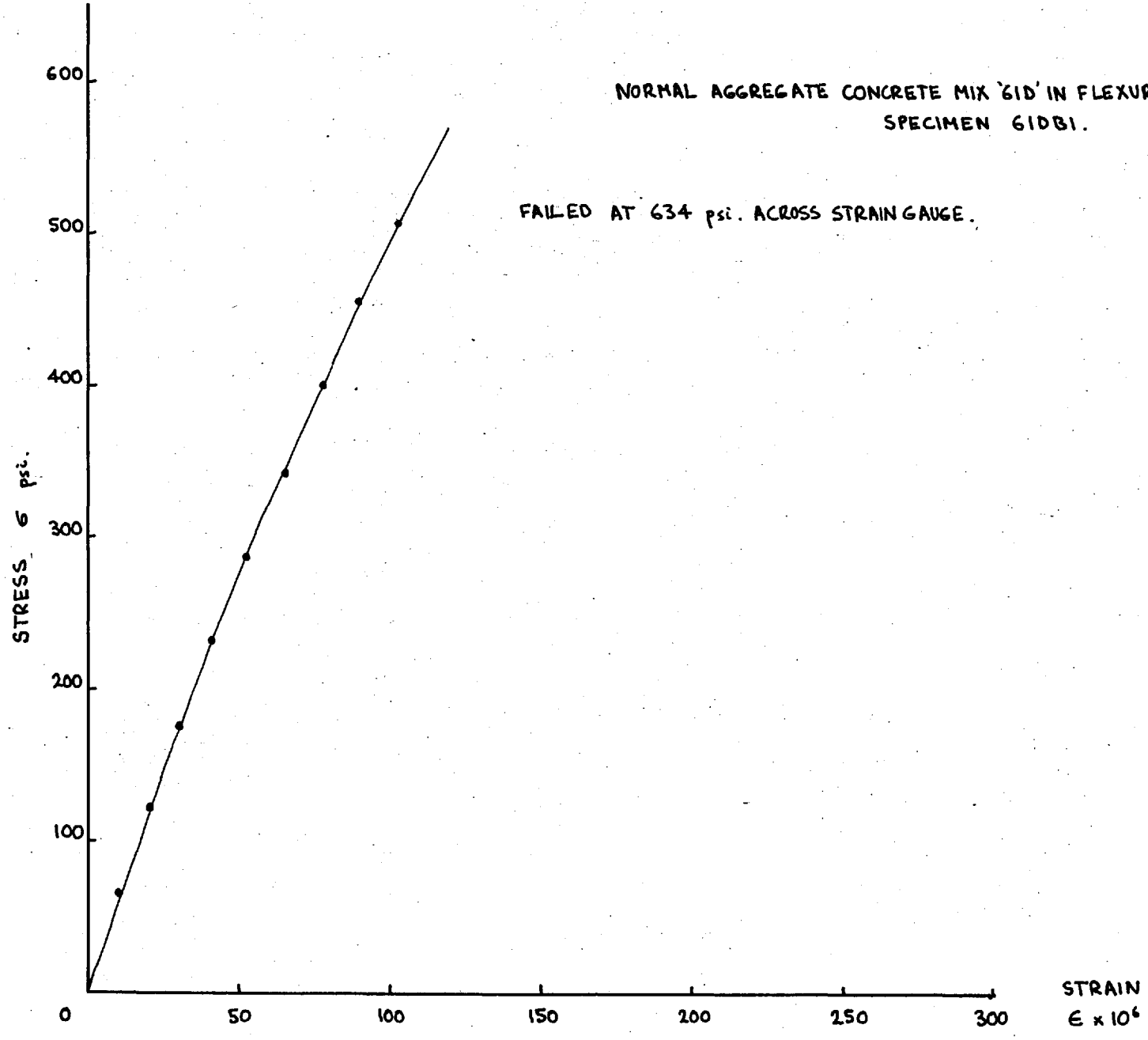


FIG. T.S.2.8. STRESS-STRAIN CHARACTERISTICS IN FLEXURAL TENSION OF 61D.

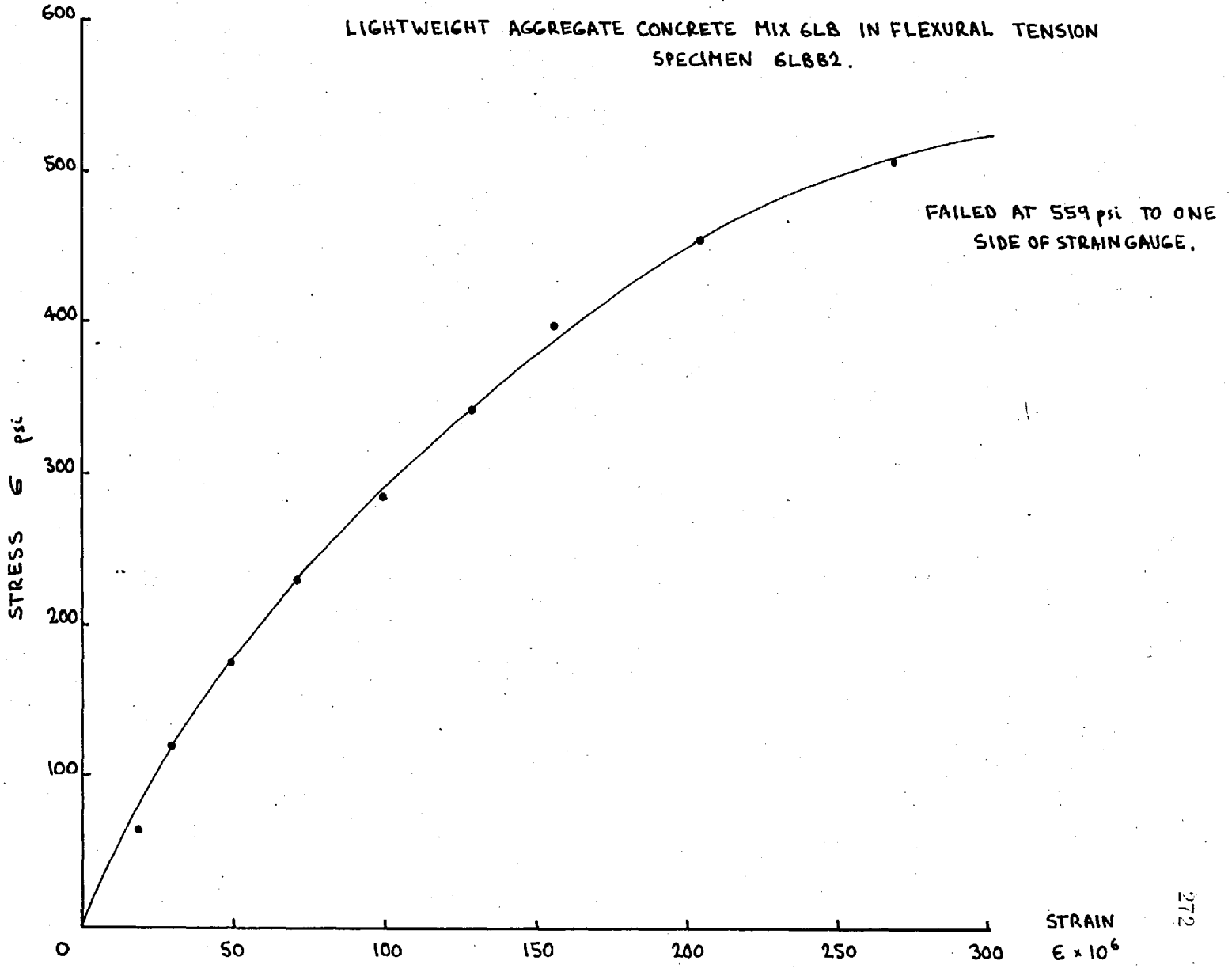


FIG. 7.5.2.9. STRESS-STRAIN CHARACTERISTICS IN FLEXURAL TENSION OF 6LB.

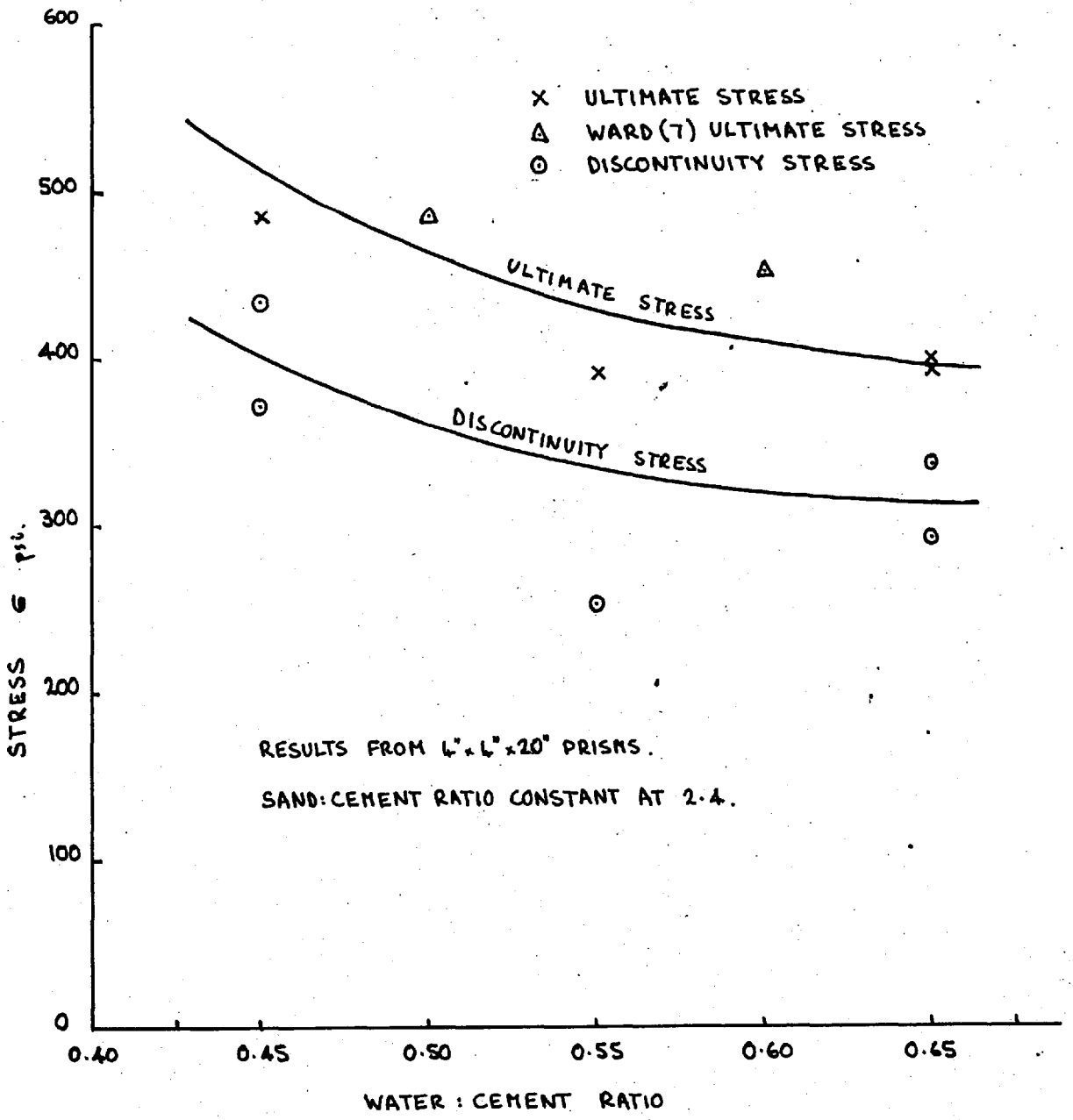


FIG. 7.5.3.1. STRENGTH OF MORTAR MIXES IN UNIAXIAL TENSION.

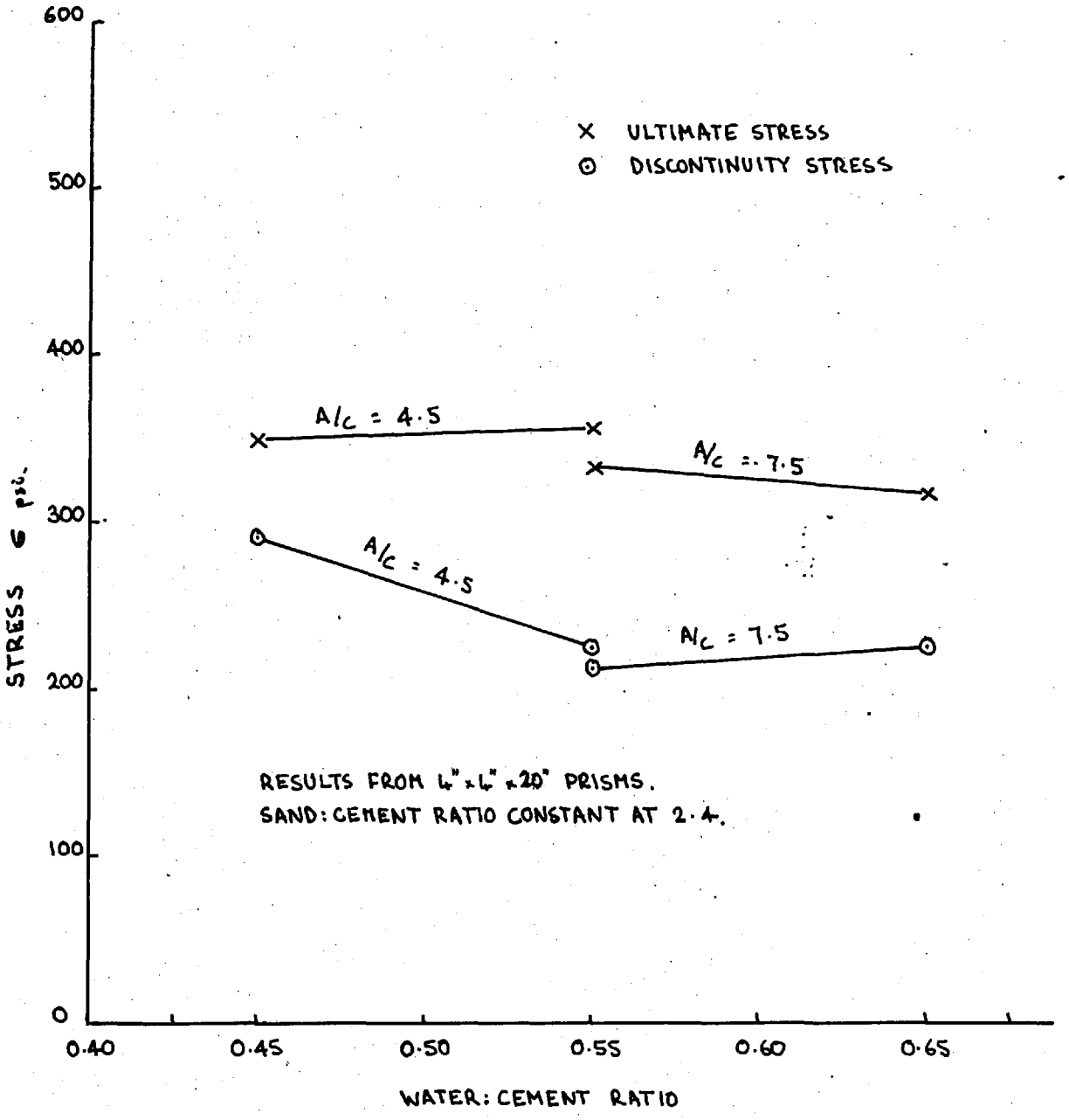


FIG. 7.5.3.2. STRENGTH OF $\frac{3}{8}$" IRREGULAR AGG. CONCRETE MIXES IN UNIAXIAL TENSION.

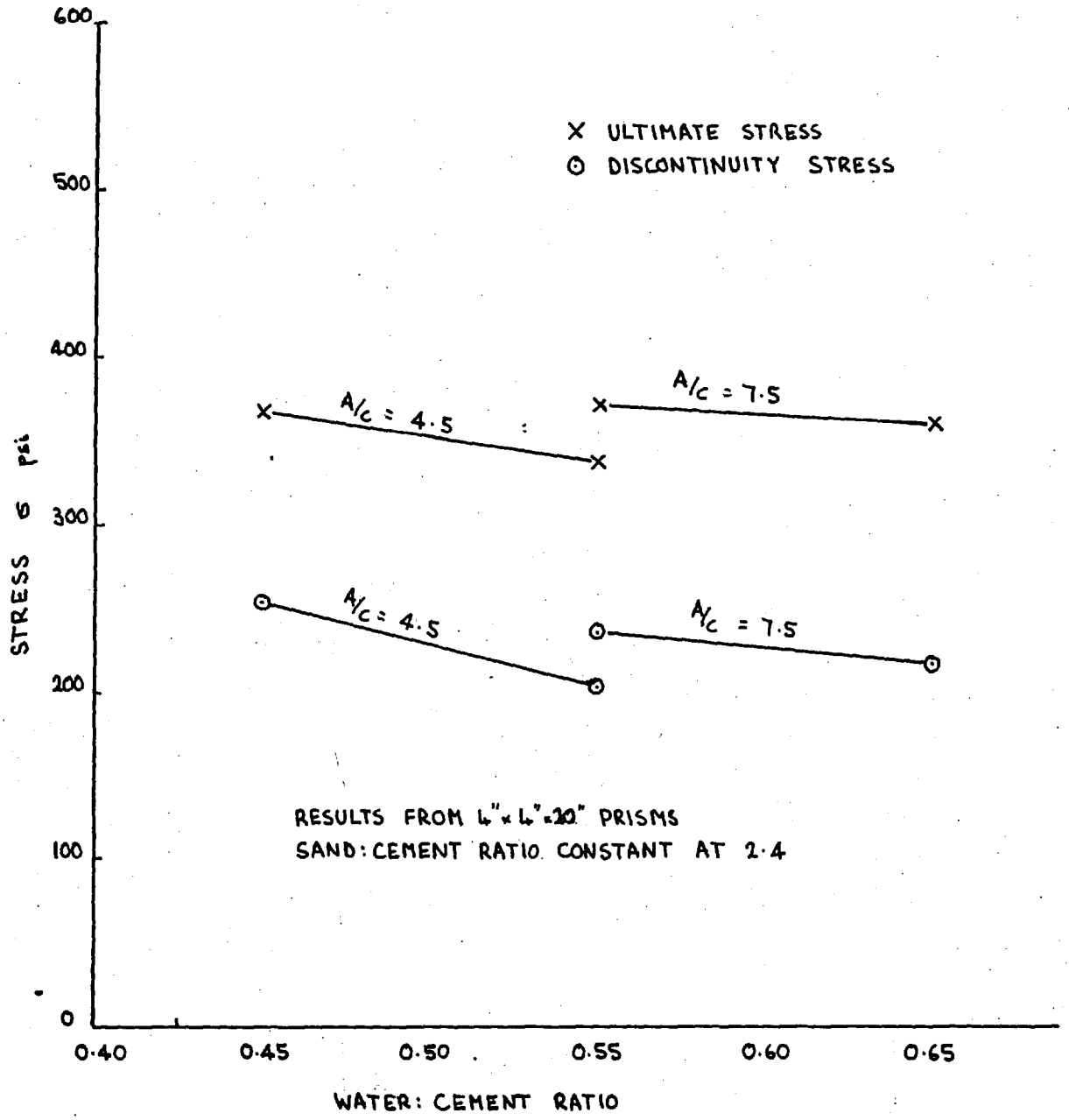


FIG. 7.5.3.3. STRENGTH OF $\frac{3}{4}$" IRREGULAR AGG. CONCRETE MIXES IN UNIAXIAL TENSION.

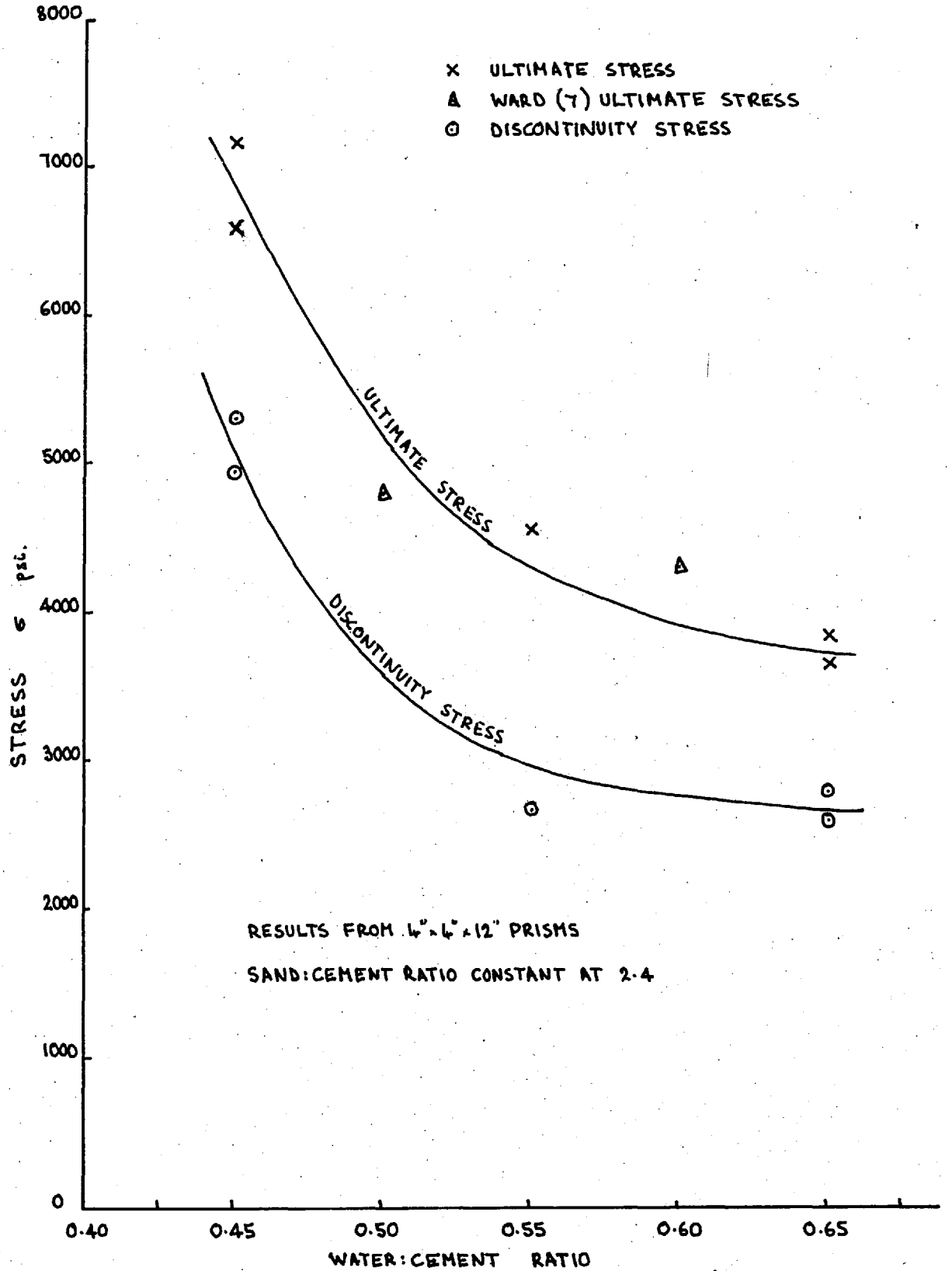


FIG. 7.5.3.4. STRENGTH OF MORTAR MIXES. (MA, MBC, MD) IN UNIAXIAL COMPRESSION.

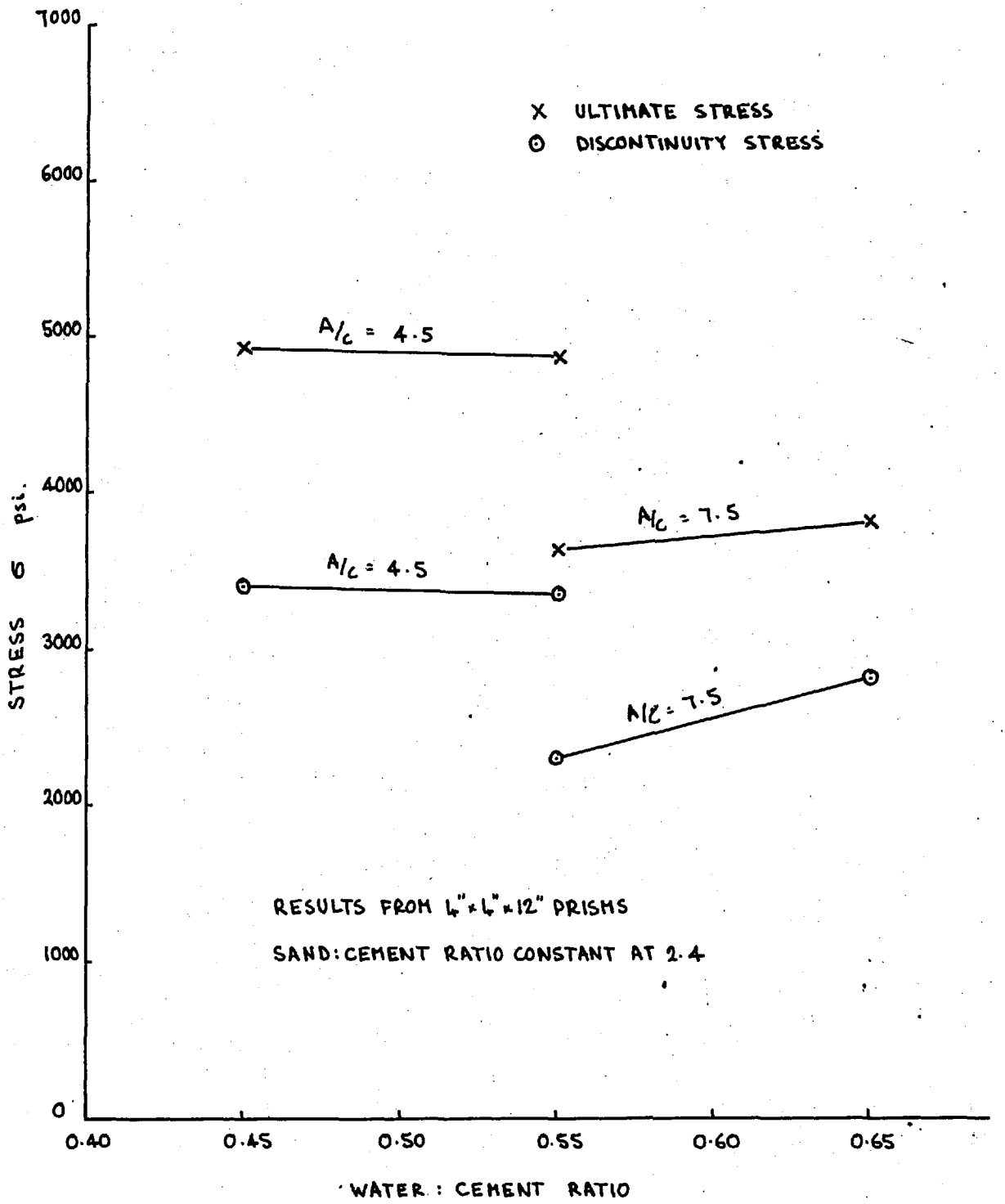


FIG. 7.5.3.5. STRENGTH OF $\frac{3}{8}$" IRREGULAR AGG. CONCRETE MIXES IN UNIAXIAL COMPRESSION.

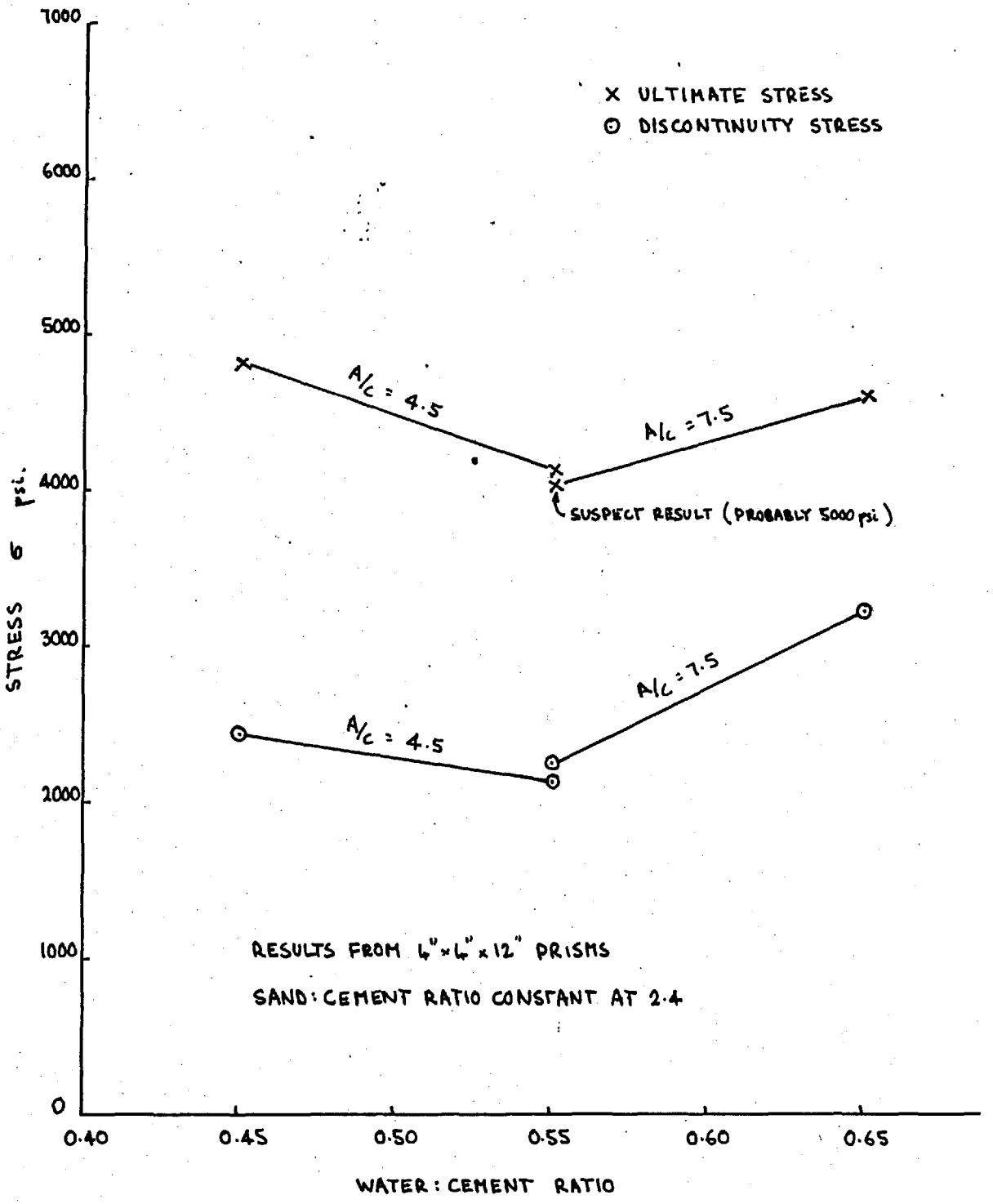


FIG. 7.5.3.6. STRENGTH OF ¾" IRREGULAR AGG. CONCRETE MIXES IN UNIAXIAL COMPRESSION.

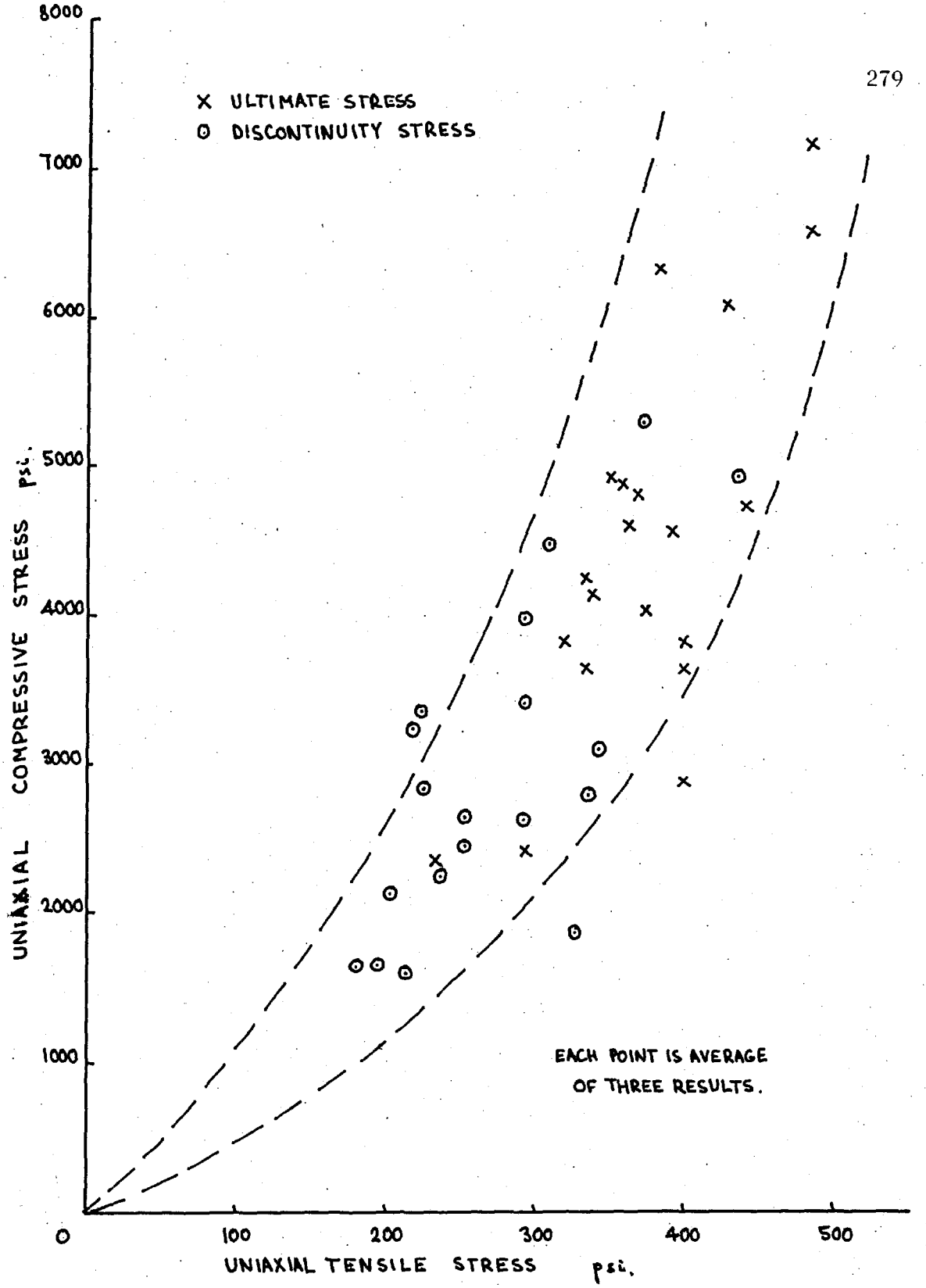


FIG. T.5.3.7. COMPARISON OF TENSILE AND COMPRESSIVE STRENGTH

approximate ratio for tensile to compressive stress of 1:10 appears to be valid for both ultimate and discontinuity stresses.

The percentage of ultimate load at which the discontinuity point occurs and its dependence on the volume fraction of coarse aggregate is shown in fig.7.5.3.8. As the value fraction of coarse aggregate increases the discontinuity point occurs at a lower percentage of the ultimate load under both tensile and compressive stresses.

7.5.4. Discontinuity Strain

The importance of the maximum tensile strain at discontinuity and its possible use as an alternative failurecriterion to ultimate or discontinuity stress has already been pointed out in section 1.6. and elsewhere.

Plots of discontinuity strains (i.e., the strains in the cleavage direction at the point of discontinuity) against volume fraction of coarse aggregate similar to those of Kaplan discussed in section 7.1.4. are shown in figs.7.5.4.1. to 7.5.4.3. in which the variation from the general trend can be seen. Each point plotted is the average of three, or occasionally only two specimens and it can be verified from the results given in tables 7.5.1.1. and 7.5.1.2. that the average standard deviation is 5.0 microstrain for compression and 4.5. microstrain for tension. Kaplan's results are also plotted for comparison.

The lightweight aggregate concretes gave discontinuity strain results similar to their parent mortar (MBC) irrespective of the values fraction of coarse aggregate. This fact is of especial interest when compared later with the biaxial stress characteristics of lightweight aggregate concretes and mortars, and is discussed in detail in Chapter 12.

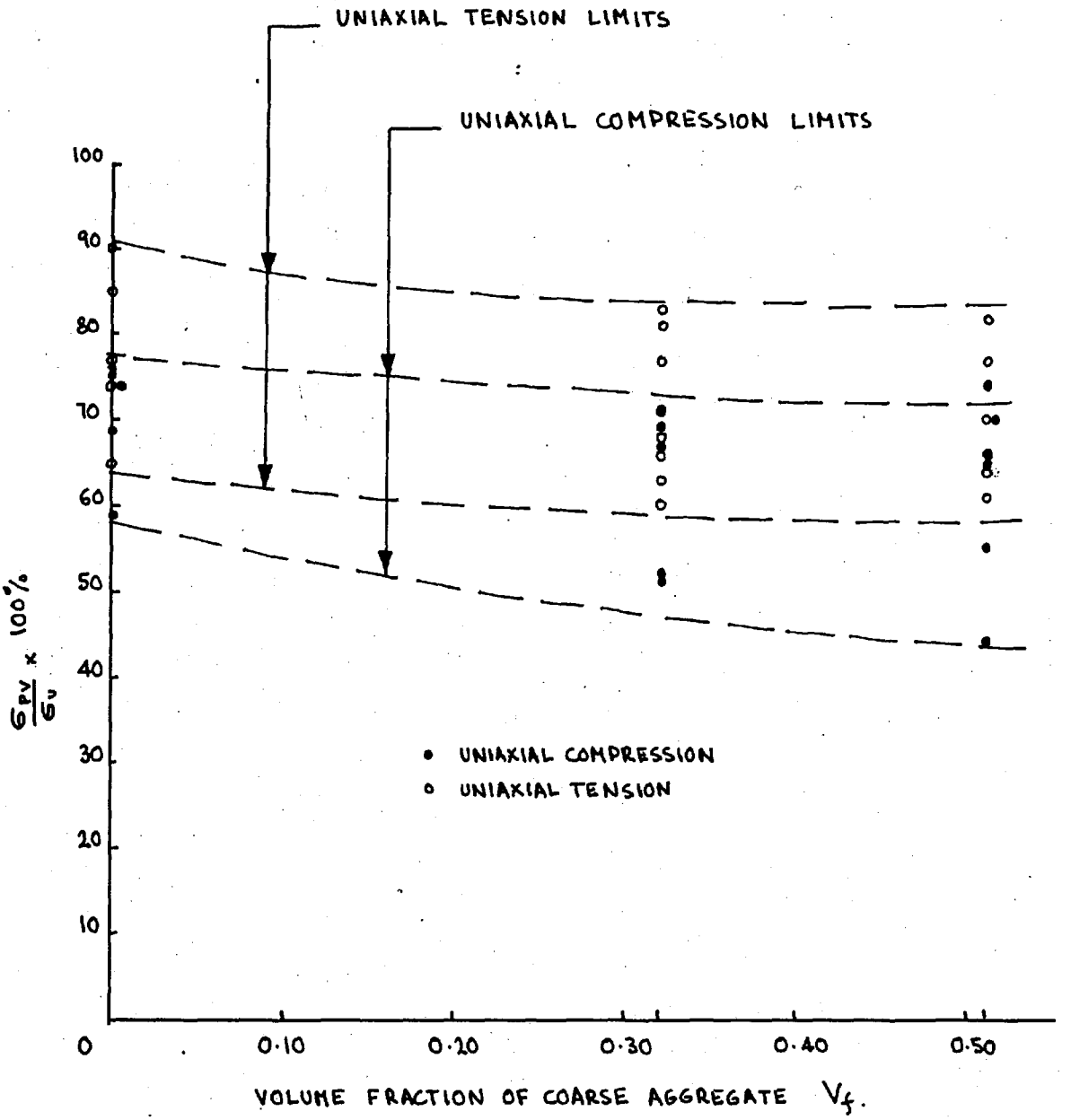


FIG. 7.5.3.8. DISCONTINUITY STRESS LEVEL AS A PERCENTAGE OF ULTIMATE IN UNIAXIAL TENSION AND COMPRESSION.

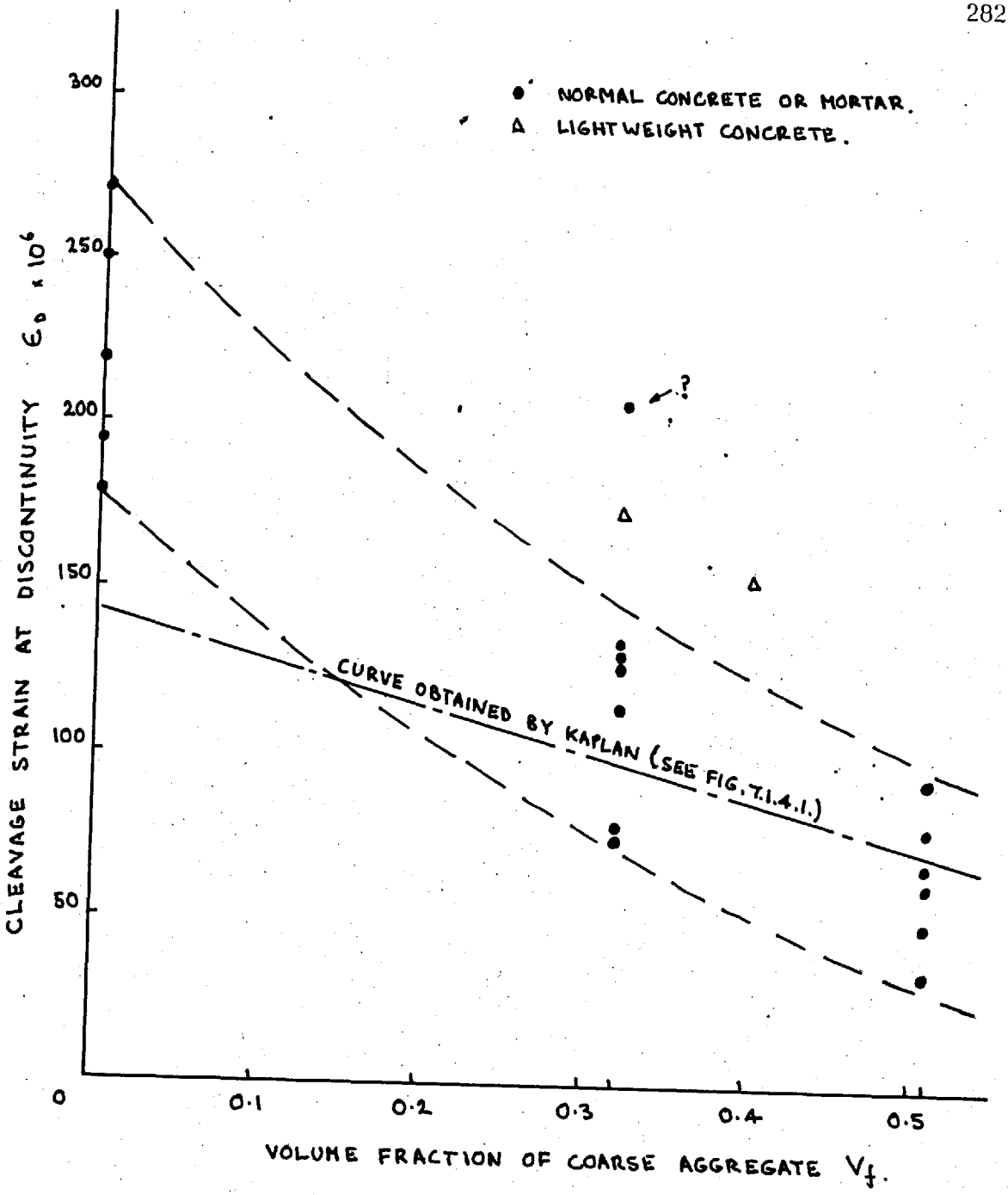


FIG. 7.5.4.1. CLEAVAGE STRAIN AT DISCONTINUITY IN UNIAXIAL COMPRESSION.

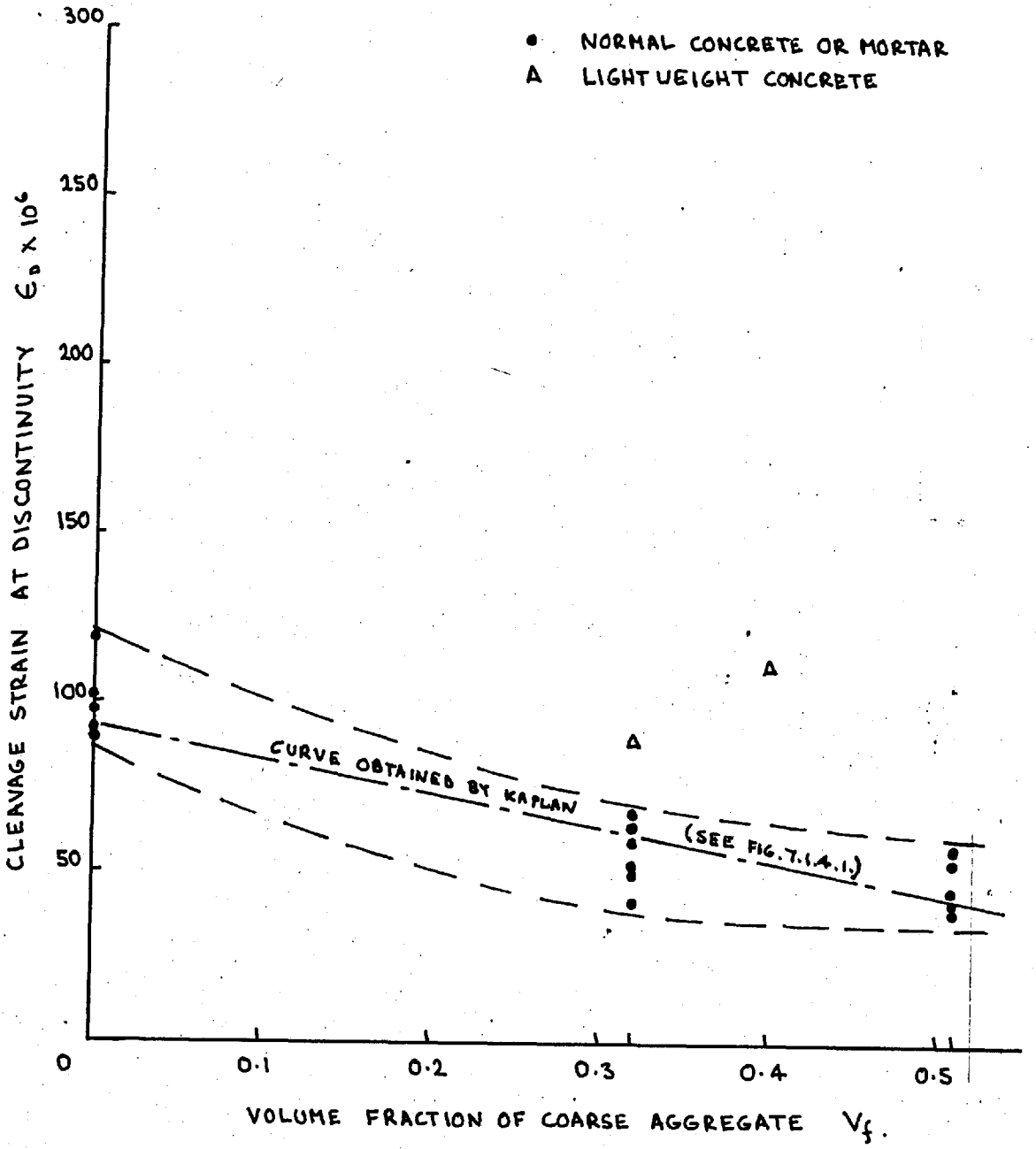


FIG. 7.5.4.2. CLEAVAGE STRAIN AT DISCONTINUITY IN UNIAXIAL TENSION.

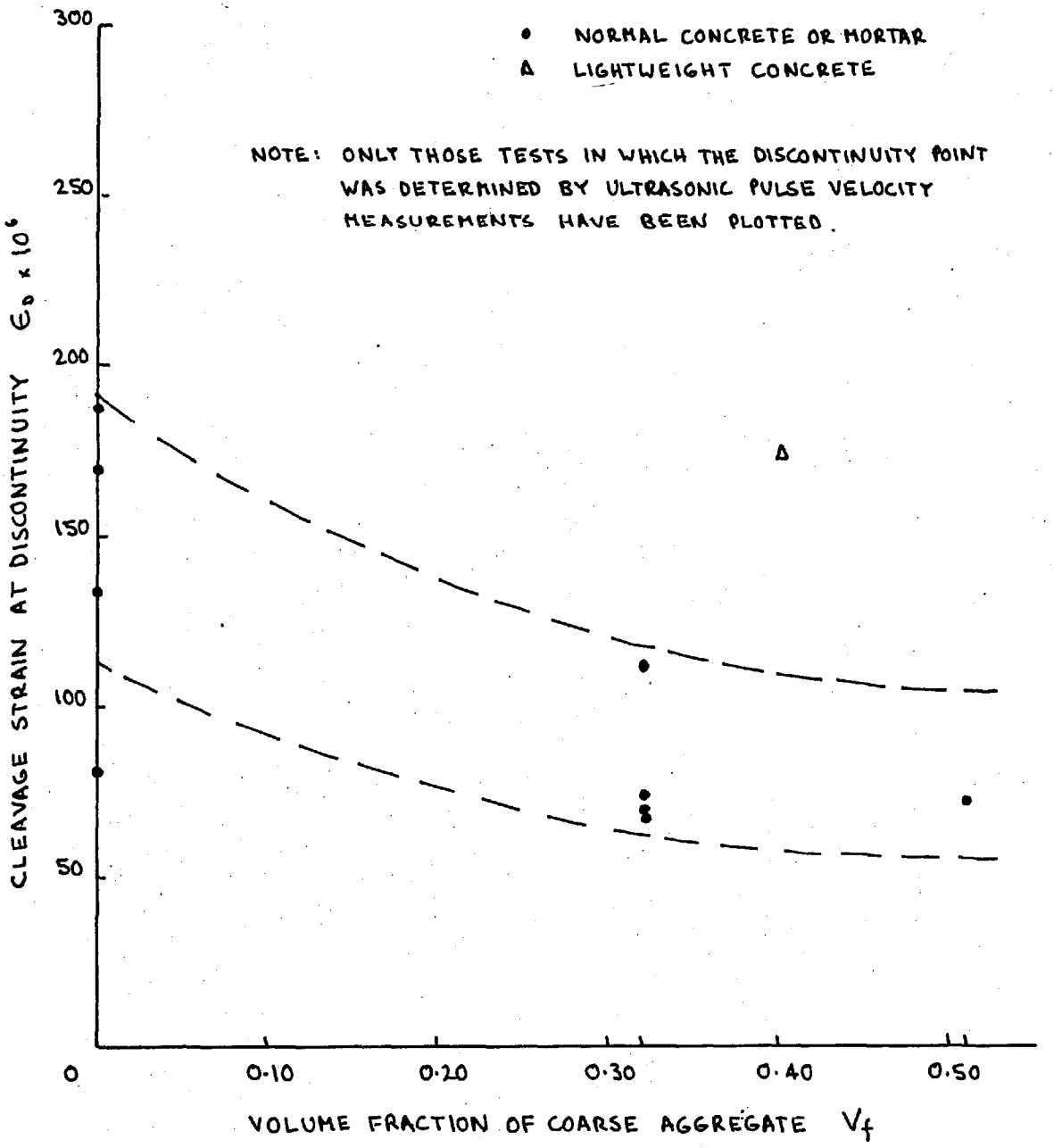


FIG. 7.5.4.3. CLEAVAGE STRAIN AT DISCONTINUITY IN FLEXURAL TENSION.

One possible interpretation of the results of the variation of discontinuity cleavage strain with volume fraction of coarse aggregate has been put forward by Ward. He hypothesised that the overall cleavage strain was directly proportional to the volume fraction of mortar in the concrete.

$$\epsilon_D = K V_{fm}.$$

where ϵ_D = cleavage strain at discontinuity
V_{fm} = volume fraction of mortar
K = constant (value depends on parent mortar and state of stress).

Hence for a given state of stress and a given parent mortar (MBC), a plot of $\epsilon_{D/V_{fm}}$ against the volume fraction of coarse aggregate should give a constant value. Plots of the 'constant' K for the three states of stress investigated are given in fig.7.5.4.4.

As an empirical rule for rough calculations, this hypothesis is probably a useful tool but there remains the possibility of different mechanisms of failure in mortars and concretes.

Lastly a comparison of the cleavage strains at discontinuity for compression and tension (both direct and flexural) stresses are given in fig.7.5.4.7. From the curves obtained it would appear that the failure mechanisms involved undergo a transition depending on the volume fraction of coarse aggregate.

7.5.5. Elastic Constants

Remarkably close agreement between the elastic modulus in tension and compression was obtained as shown in fig.7.5.5.1. confirming the findings of Todd[179] and Ward[7]. This phenomenon suggests that the degree of cracking which takes place on the aggregate bond surfaces under uniaxial tensile and compressive stresses and below the discontinuity stress must be effectively

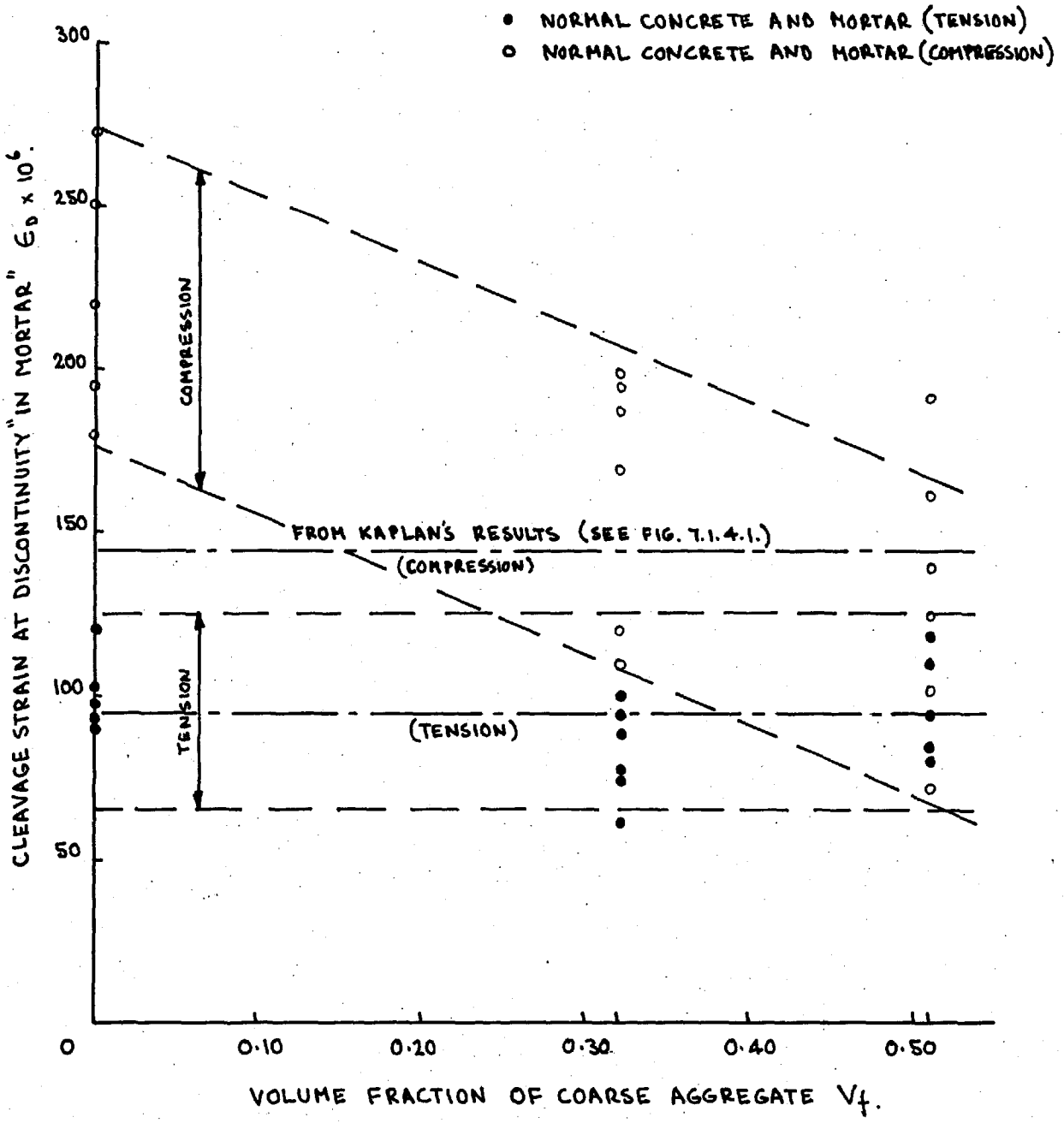


FIG. 7.5.4.4. CLEAVAGE STRAIN AT DISCONTINUITY IF ATTRIBUTED TO NETT VOLUME OF MORTAR.

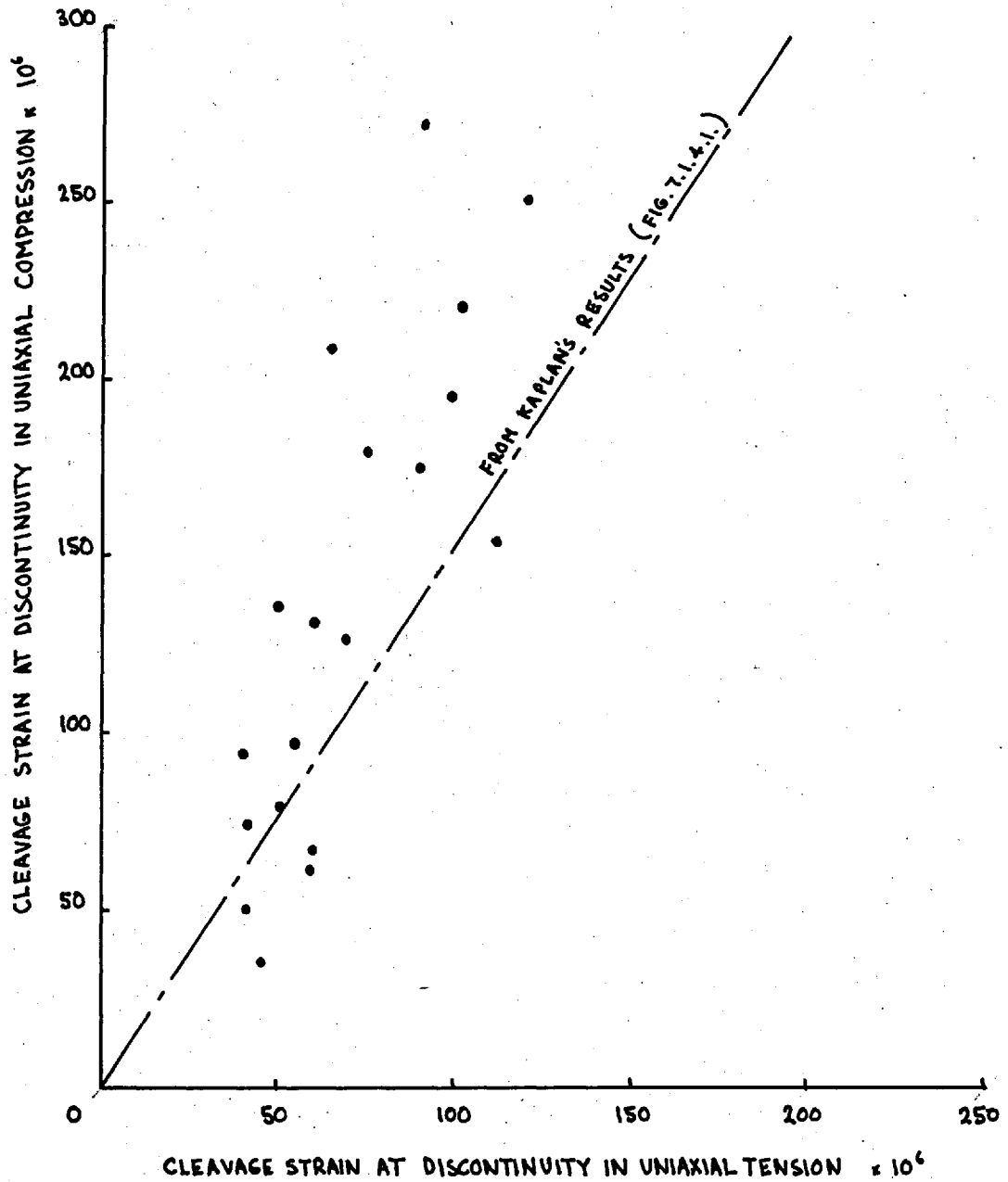


FIG. 7.5.4.5. COMPARISON OF CLEAVAGE STRAIN AT DISCONTINUITY IN UNIAXIAL COMPRESSION AND TENSION.

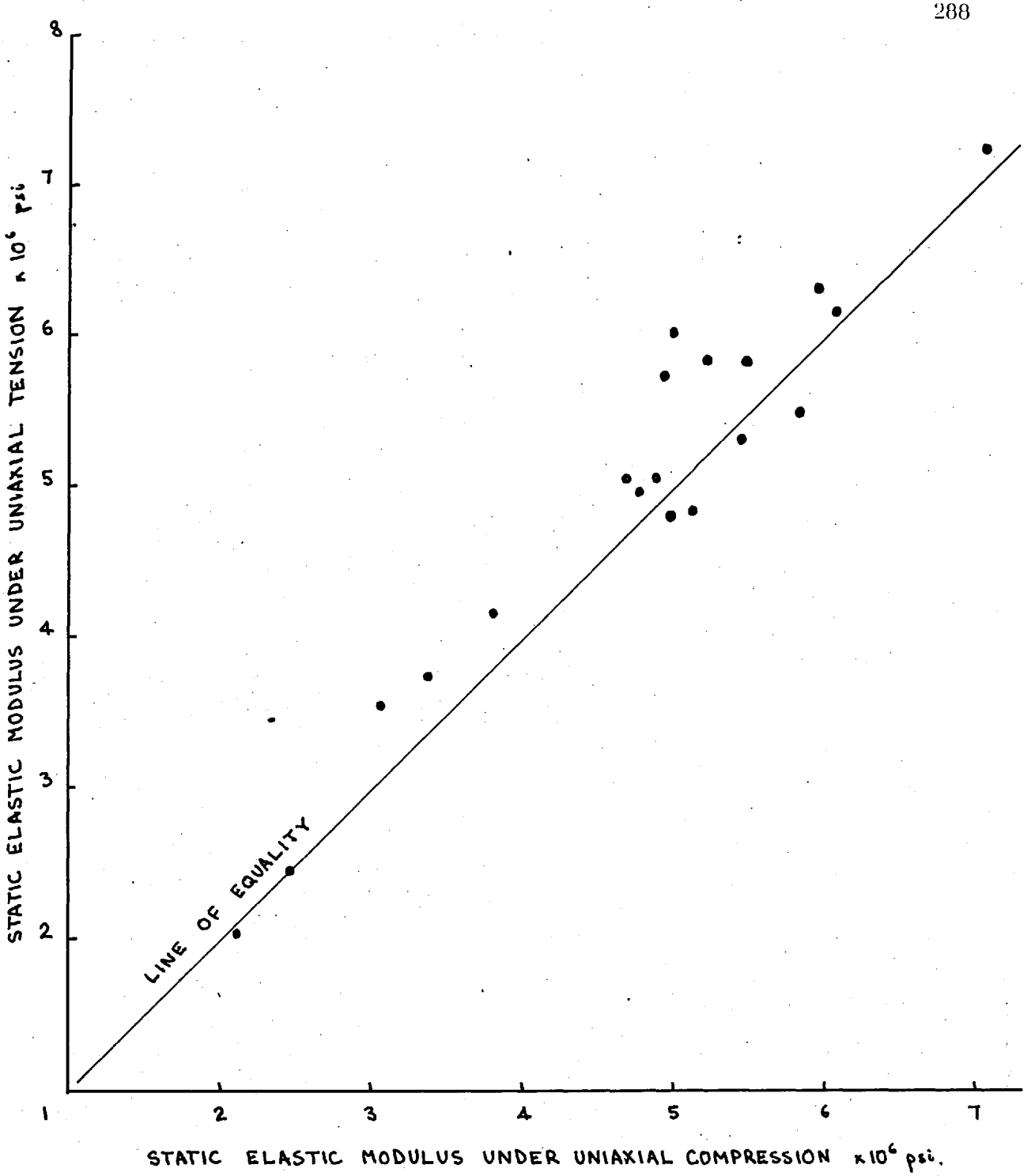


FIG. 7.5.5.1. STATIC ELASTIC MODULUS UNDER UNIAXIAL STRESS

the same. The fact that the same phenomenon is observed with mortars implies that the mortar behaves perfectly elastically in this respect. It must be emphasised that this correlation was observed from specimens which had been preloaded as described in section 7.4. and may not apply to specimens subjected to one loading cycle only.

The Poisson's ratio results when compared in the same way in fig.7.5.5.2. were very scattered but it is possible that with measurements of lateral strain taken at more than one cross-section on the specimens the scatter could be reduced. The difficulties of measuring the very small lateral strains in the uniaxial tension test which are often of the order of 10 microstrain or less add to the problem of reliable Poisson's ratio measurements in tension. However, there appears to be a general trend towards equality.

The relationship between the static and dynamic elastic moduli shown in fig.7.5.5.3. is a consistent one and compares well with the results of Takabayashi[180], Simmons[181] and Ward[7]. The regression analysis equations for the straight line section of the curve was as follows:-

$E_s = E_D - 1.0 \times 10^6$ p.s.i. Simmons and Takabayashi

$E_s = E_D -$ $\times 10^6$ p.s.i. Ward

$E_s = E_D - 0.6 \times 10^6$ p.s.i. Vile

where E_s = static modulus of elasticity

E_D = dynamic modulus of elasticity

Poisson's ratio measurements in compression were more reliable because of the large strains measured and the variation of Poisson's ratio with volume fraction of coarse aggregate shown a regular trend (fig.7.5.5.4.). A similar curve was obtained by Anson[5].

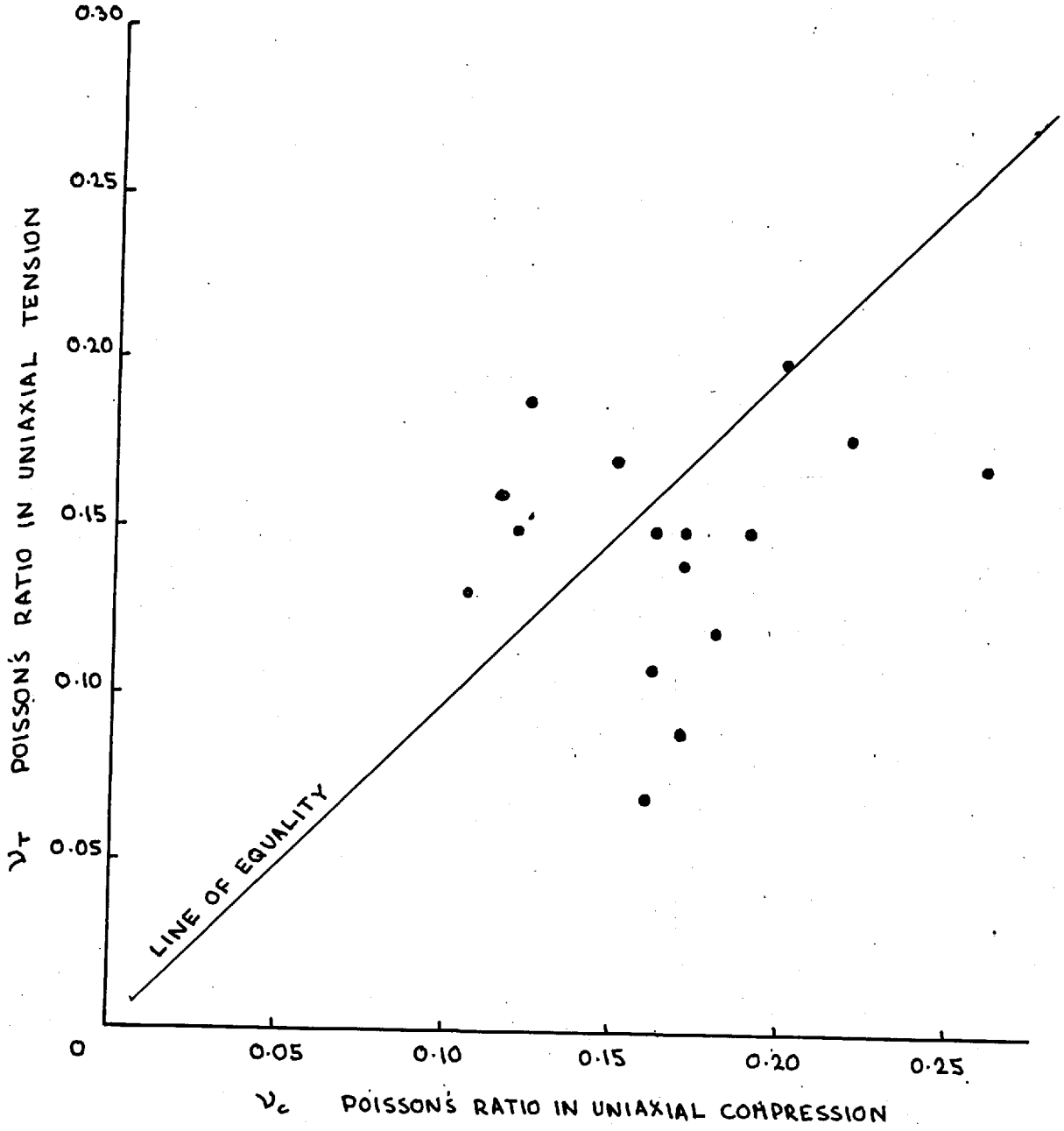


FIG. T.5.5.2. POISSON'S RATIO IN UNIAXIAL TENSION AND COMPRESSION.

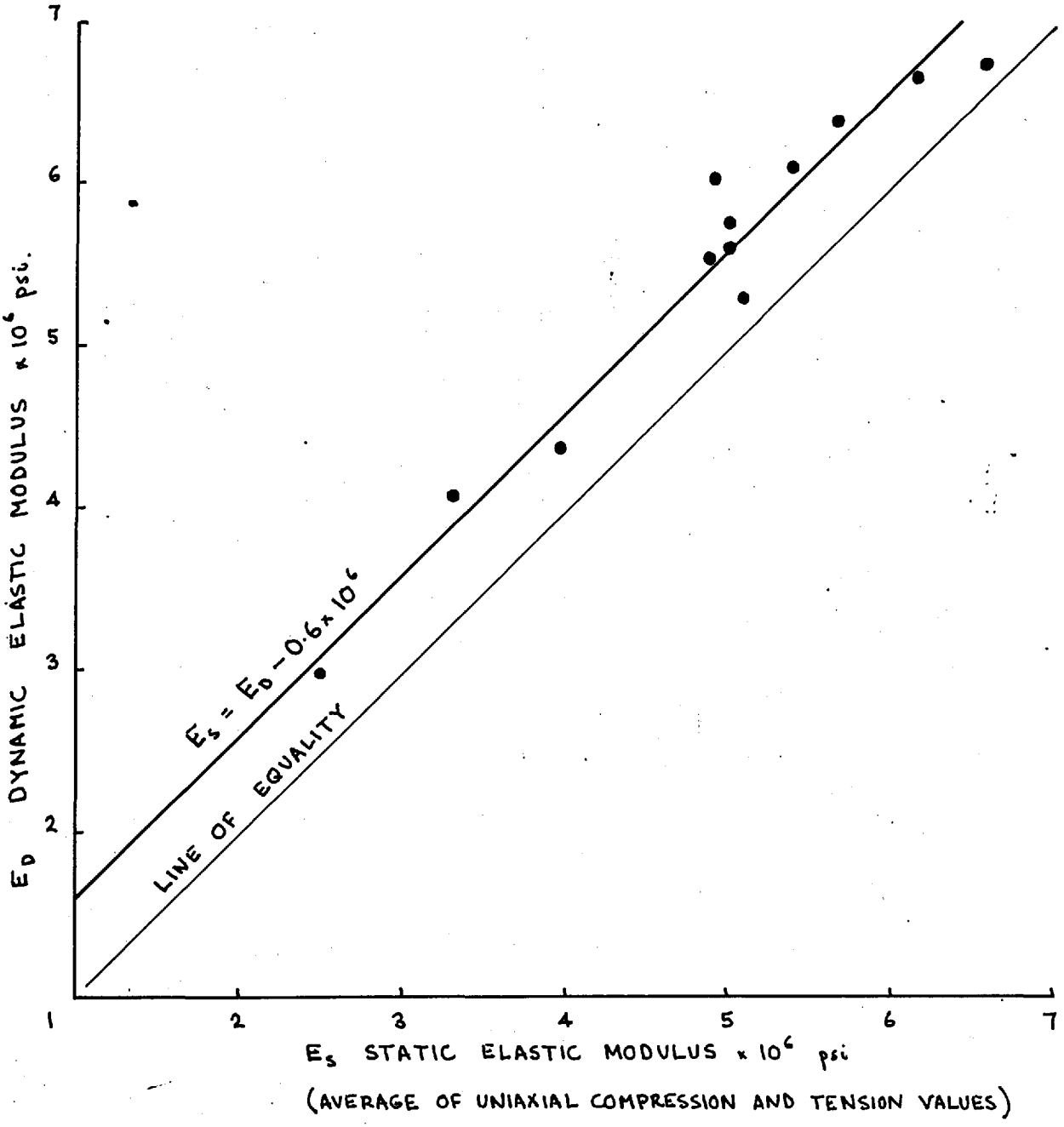


FIG. 7.5.5.3. STATIC AND DYNAMIC ELASTIC MODULI

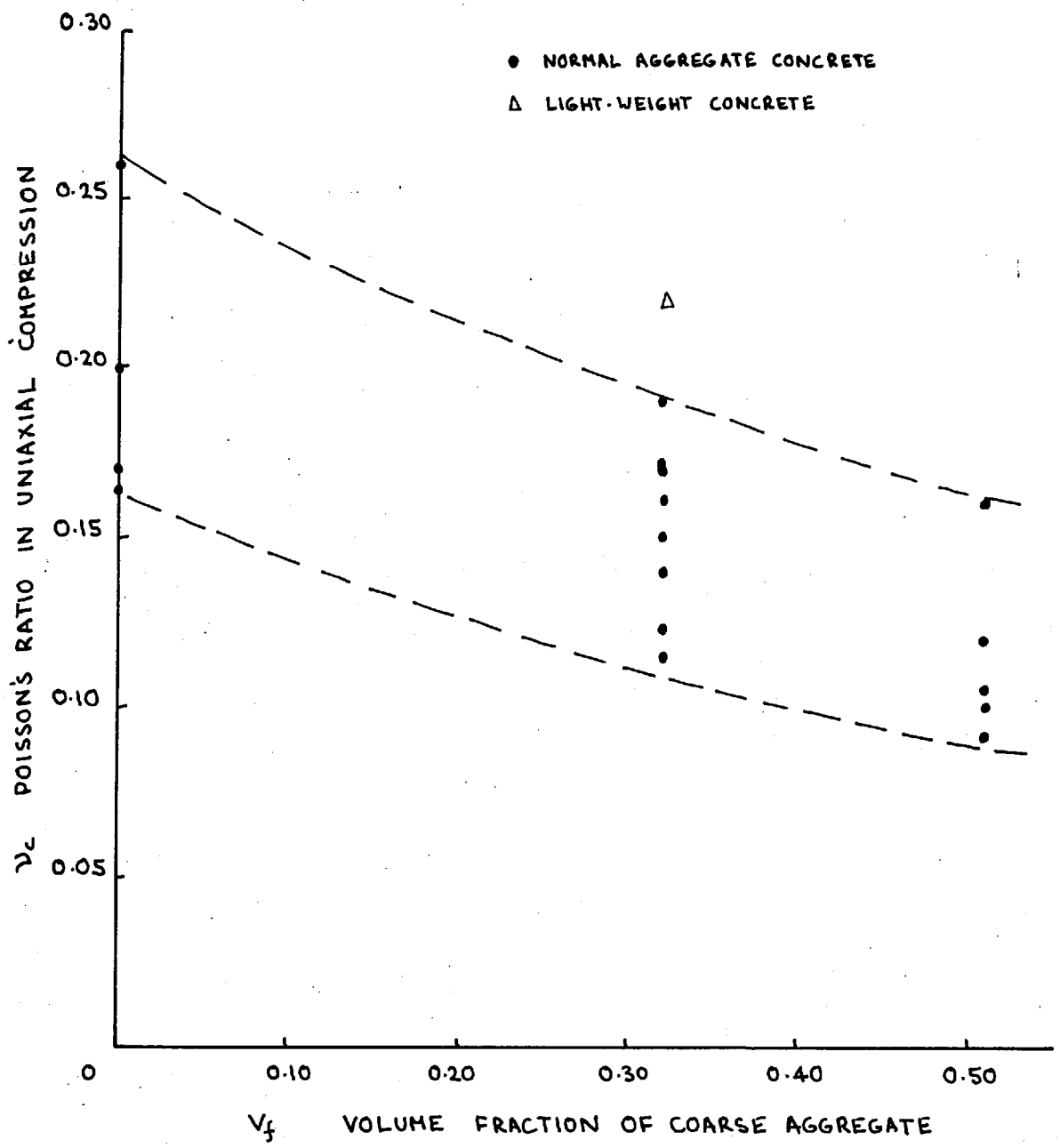


FIG. 7.5.5.4 VARIATION OF POISSON'S RATIO WITH VOLUME FRACTION OF COARSE AGGREGATE.

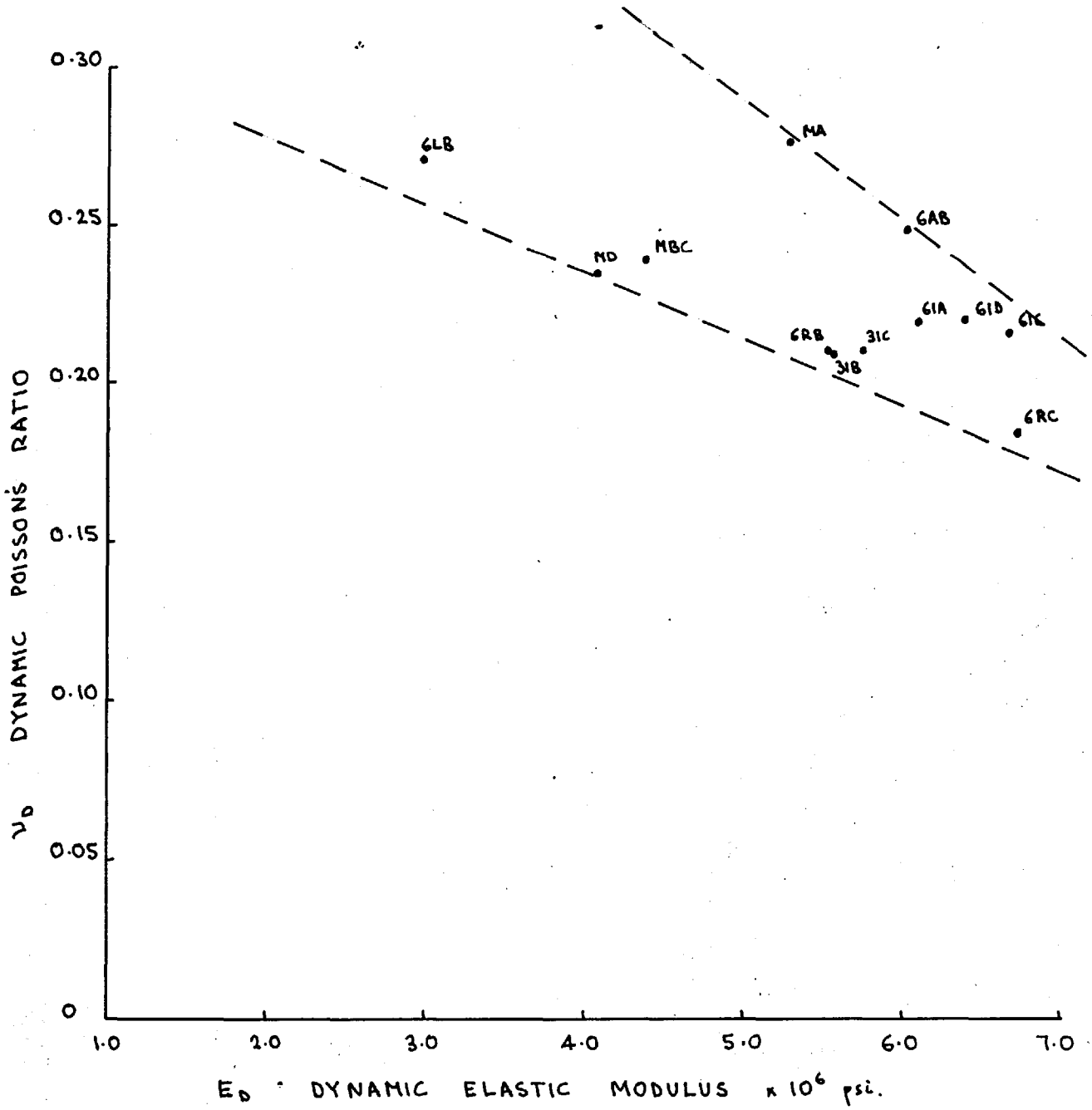


FIG. 7.5.5.5. DYNAMIC ELASTIC MODULUS AND POISSON'S RATIO

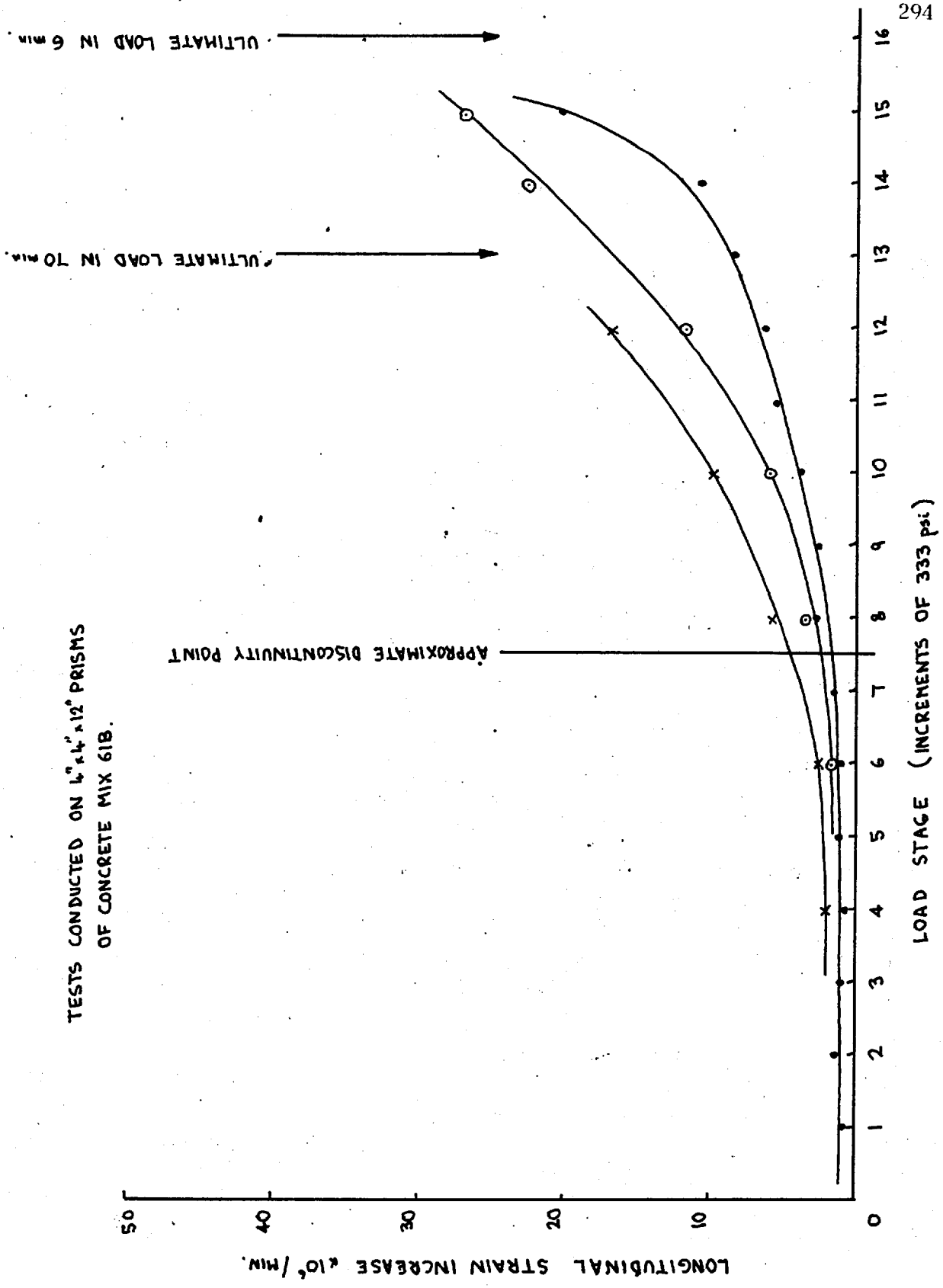


FIG. 7.5.7.1. LONG TERM BEHAVIOUR OF CONCRETE IN TERMS OF STRAIN INCREASE AND ULTIMATE LOAD.

Lastly, the dynamic elastic modulus and dynamic Poisson's ratio values are compared in fig.7.5.5.5. and Ward's results have been superimposed. Both sets of data follow a similar trend.

7.5.6. Long-term Strain Characteristics

The results of all the long-term tests are summarised in fig.7.5.7.1. in which the rate of longitudinal strain increase under uniaxial compression $\frac{d\epsilon}{dt}$ is plotted against applied stress.

The rate of strain increase was calculated by dividing the total increase of strain at a load stage by the five or ten minute interval over which it was measured to give a rate of microstrain per minute.

As can be seen from the results plotted, the rate of strain increase is very small until the discontinuity stress is exceeded when it increases very rapidly. The several tests where the load was maintained constant for one hour or more showed that these rates of longitudinal strain increase, measured above the discontinuity point, are maintained up to failure or until the test is stopped. Therefore, if there is a limit to the longitudinal strain that concrete can sustain before failing, then these results would suggest that at any stress level above the discontinuity stress this failure limit would be reached in finite time.

Hence the results, though by no means conclusive, do suggest a positive link between the discontinuity stress and the long-term ultimate strength of concrete.

CHAPTER 8

BIAXIAL COMPRESSION TEST SERIES

8.1. Introduction

These tests were the main series in this research project and were intended primarily to investigate the effect of the intermediate principal stress σ_2 on the failing stresses, stress-strain characteristics, and elastic constants of mortar and concrete (section 1.8.). The modes of failure are also of interest in the search for a realistic failure criterion.

With the newly designed and proven combined stress testing machine, together with the strain measuring techniques and ultrasonic pulse velocity equipment described in Chapter 5, it was hoped that much more reliable results would be achieved than hitherto and that substantial progress could be made in this difficult field of research.

8.1.1. State of Stress

The 10" x 10" x 4" biaxial compression specimen had already been proved suitable for these tests; the proving trials have already been described in section 4.4. These trials were all carried out under short-term loading as this was the type of test to be carried out in this test series. It is possible that the central zone of 6" x 6" x 4" (minimum), which was shown to be under a uniform state of stress and strain under all stresses, might be even larger under long-term stress as time would be allowed for frictional restraint at the loading platens to be released by slipping.

8.1.2. Strain and Cracking Measurements

Strains in the two principal stress directions were measured on the surface of the unstressed face of the test specimen using electrical resistance strain gauges as described in section 5.2.

Lateral strains were measured using the Sogenique linear displacement transducers mounted on the special frame described in section 5.3.

The ultrasonic pulse velocity equipment was again used for crack detection and, in particular, for determining the point of discontinuity.

8.1.3. Concrete Mixes

Both uniaxial and biaxial test specimens were cast from the same batch for a given mix as noted in section 6.2.2. However, the testing of the uniaxial specimen has already been described and this chapter deals with the biaxial compression specimens only. The mixes selected have been specified in section 6.4.

8.1.4. Cleavage Strains at Discontinuity

In these tests a complete record of the three dimensional stress and strain behaviour was made and particular note was made of the cleavage strain at discontinuity. Robinson's work[10] on biaxial compression suggested that the cleavage strain might vary according to the state of stress though his strain readings were not used to investigate this phenomenon. These measurements were regarded as being complementary to the work under uniaxial states of stress where the cleavage strain at discontinuity was clearly dependent on the state of stress.

8.1.5. Rates of Loading

The general rate of loading for uniaxial compression tests in this research was specified as 700 p.s.i./min. in section 7.1.5. For biaxial compression tests it was decided that the sum of the rates of loading of the two principal stresses should be equal to 700 p.s.i./min divided in the ratio of the magnitudes of the two stresses.

This rate of loading gave adequate time for all measurements to be taken and ensured that the cleavage strain in the unstressed direction increased at approximately the same rate in all uniaxial and biaxial compression tests. In other words, the rate of energy input into all test specimens wes approximately constant.

8.1.6. Age and Condition of Specimens at Test

All biaxial compression specimens were tested at 28 days after casting. The associated uniaxial xpecimens were tested at either 27 or 29 days whichever was the most convenient. This was unavoidable as the total number of specimens from a given mix was fifteen, of which the six biaxial and nine uniaxial specimens took one full day each to test.

The specimens were all tested in a saturated condition which was maintained throughout the test by the use of a curing compound as described in section 6.9.

8.2. Testing Machines and Equipment Used

All tests were conducted in the combined stress testing machine as planned.

The surface strain measurements taken by electrical resistance strain gauges were recorded using the Solartron automatic data logger (section 5.2.) and the lateral strains were recorded manually using the Sogenique strain measuring box.

The Cawkell ultrasonic pulse velocity apparatus was used in the same manner as in the uniaxial tests, but for convenience was mounted in the centre of the machine control panel as shown in section 3.6.

8.3. Testing Arrangement

Care had to be taken in locating all the measuring devices onto the surface of the specimen to ensure that all measurements were taken within the central 6" x 6" x 4" zone of uniform strain.

The linear displacement transducer had to be placed centrally to simplify the setting up of the transducer support frame. This dictated that the ultrasonic pulse velocity transducers should be placed along the opposite diagonal and arranged as described in section 5.4. so that they scanned the central volume of the specimen.

Fortunately these arrangements left sufficient room for two electrical resistance strain gauges for vertical and horizontal strain measurements and so the final arrangement was as shown in fig.8.3.1.

A summary of the strain and pulse velocity measurements taken in these tests is given below:-

1) Specimens S1, S3, S6

Longitudinal Strain in direction 3	2 x 30 mm. E.R.S.6.
Longitudinal Strain in direction 2	2 x 30 mm. E.R.S.6.
Lateral Strain in direction 1	4 in. gauge length-Sogenique
Ultrasonic pulse velocity	8 in. path length.

2) Specimens S2, S4, S5

Longitudinal Strain in direction 3	-
Longitudinal Strain in direction 2	-
Lateral Strain in direction 1	4 in. gauge length - Sogenique
Ultrasonic pulse velocity	8 in. path length.

The directions 1, 2, and 3 are defined in fig.8.3.1. and the specimen numbers S1-S6 are explained in fig.8.4.5.1.

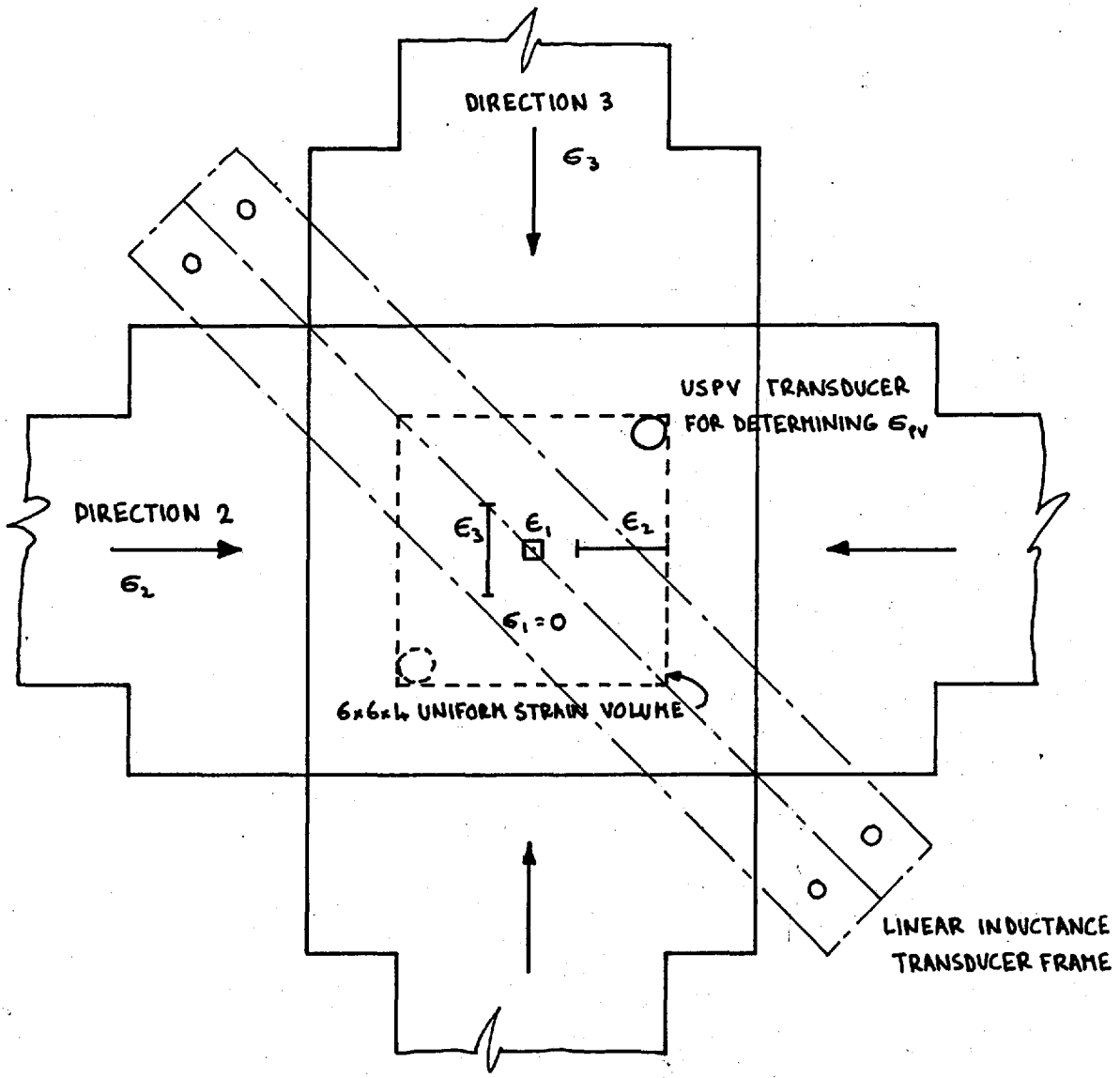


FIG. 8.3.1. GENERAL ARRANGEMENT OF MEASURING EQUIPMENT FOR BIAXIAL COMPRESSION TESTING.

8.4. Test Procedure

8.4.1. Alignment of Specimen

The procedure for the alignment of the specimen is described in detail in this section because much time was saved by adopting a methodical approach.

With the two horizontal sub-frame platens about $\frac{1}{4}$" away from their final position when bearing on the test specimen, the specimen was placed on the bottom platen and carefully aligned on all four faces. The bottom platen and specimen were then jacked upwards until the bottom of the specimen was 1/16" lower than the underside of the horizontal platens as shown in fig.8.4.1.1.

As described in section 3.4. the platen/spherical seating unit was fixed to the ram of the jack by a centrally located davel so that the whole unit could be rotated about the central axis. Consequently a final adjustment of the orientation of the specimen and bottom platen in the horizontal plane had to be made by checking the alignment of the front and back vertical faces of the specimen with the sides of the horizontal platens as shown in fig.8.4.1.2.

The bottom platen was then held in this position by closing the stop-valve on the pressure line to the bottom jack whilst the top jack was released and brought down to bear on the top of the specimen with a pressure of about 200-300 p.s.i. This held the specimen and the top and bottom platens firmly in their aligned positions.

The two horizontal platens were then jacked out to bear on the sides of the specimen. Both jacks were operated together and allowed to bear on the specimen with a nominal pressure of about 200-300 p.s.i. During this operation care was taken to ensure that the horizontal platens were rotated to line up with the sides of the specimen.

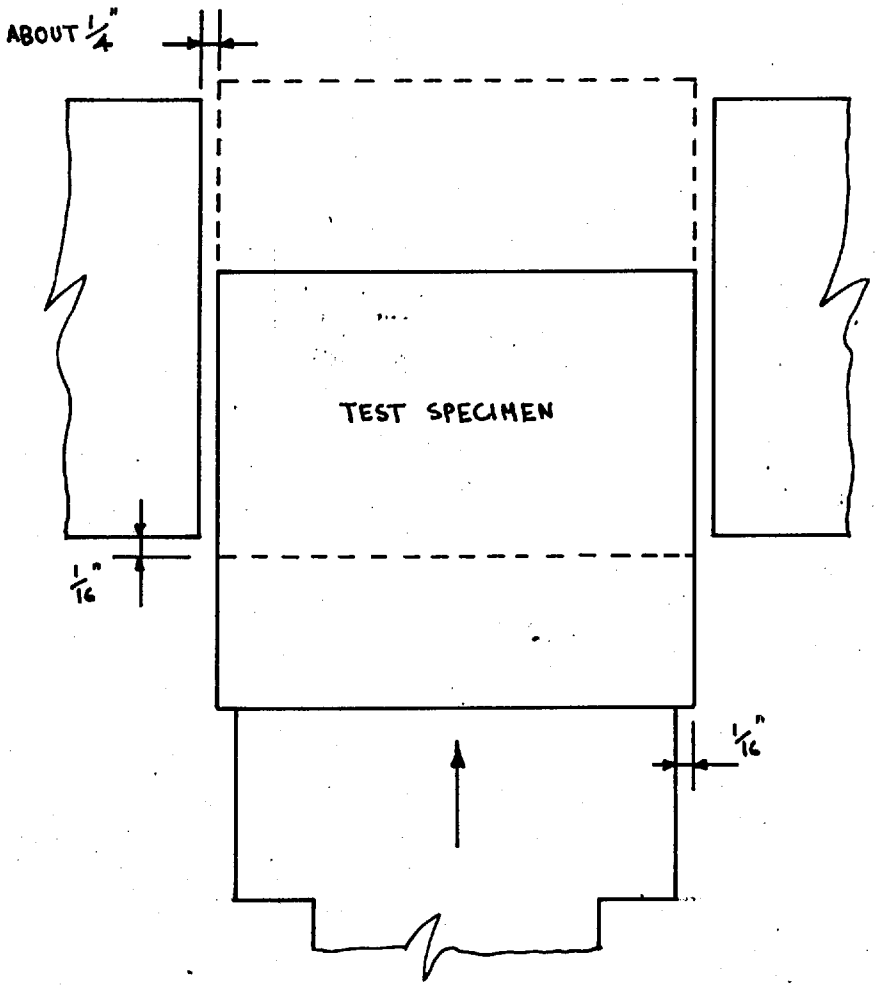


FIG. 8.4.1.1. VERTICAL ALIGNMENT OF TEST SPECIMEN.

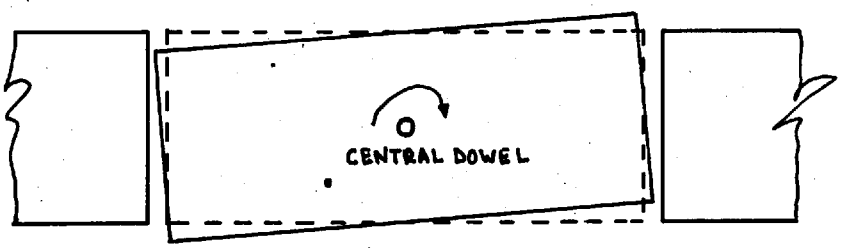


FIG. 8.4.1.2. ROTATION OF TEST SPECIMEN FOR HORIZONTAL ALIGNMENT.

As soon as the horizontal platens were bearing on the specimen, the bottom jack could be released by opening the stop valve. The specimen was then said to be "floating" in the machine meaning that it was under pressure from and held in position only by the hydraulic jacks.

A final check was made on the alignment of the platens on the specimen and if any platen was out of position it was corrected whilst the other jacks were locked in position using the stop valves.

With a little practice, this operation normally took only ten minutes; most of the time spent being due to the rather slow motion of the jacks.

8.4.2. Attacking the Measuring Devices

Although the electrical resistance strain gauges were already stuck onto the surface of the specimen, they still had to be connected to the data logger. The soldering of the leads was normally the first operation after aligning the specimen. The fabric of the gauge and the gauge connecting wires are very delicate and so the leads were taped to the specimen after soldering to avoid their being pulled off accidentally.

The Sogenique linear displacement transducer frame was then clamped onto the specimen. The procedure was to first clamp the frame without the transducers and their housing blocks. This was done by removing the two tie bolts at one end of the frame, threading the remaining U shaped frame diagonally across the specimen, and then replacing the two tie bolts thus compressing the springs and clamping the frame to the specimen. The transducers and their housing blocks were then bolted into the frame so that they bore centrally on the glass pads and were at right angles to the face of the specimen. The Sogenique strain measuring box reading was adjusted to its mid-range of readings, as this is the most accurate section, by tightening up the frame tie bolts.

Lastly, the two ultrasonic pulse velocity transducers were fixed to the faces of the slab using a rubber thong on each, wrapped around the columns of the horizontal sub-frame (fig.8.4.2.1. and 8.4.2.2.). Because of the relative dimensions, the thongs had to be packed out from the columns using small wooden blocks so that a small pressure was exerted on the transducer to hold it firmly in place (fig.8.4.2.3.).

The area of the specimen on which the transducer bore had firstly been scraped clean of curing compound and then Vaseline grease applied to the surface of the transducer to ensure a good surface contact.

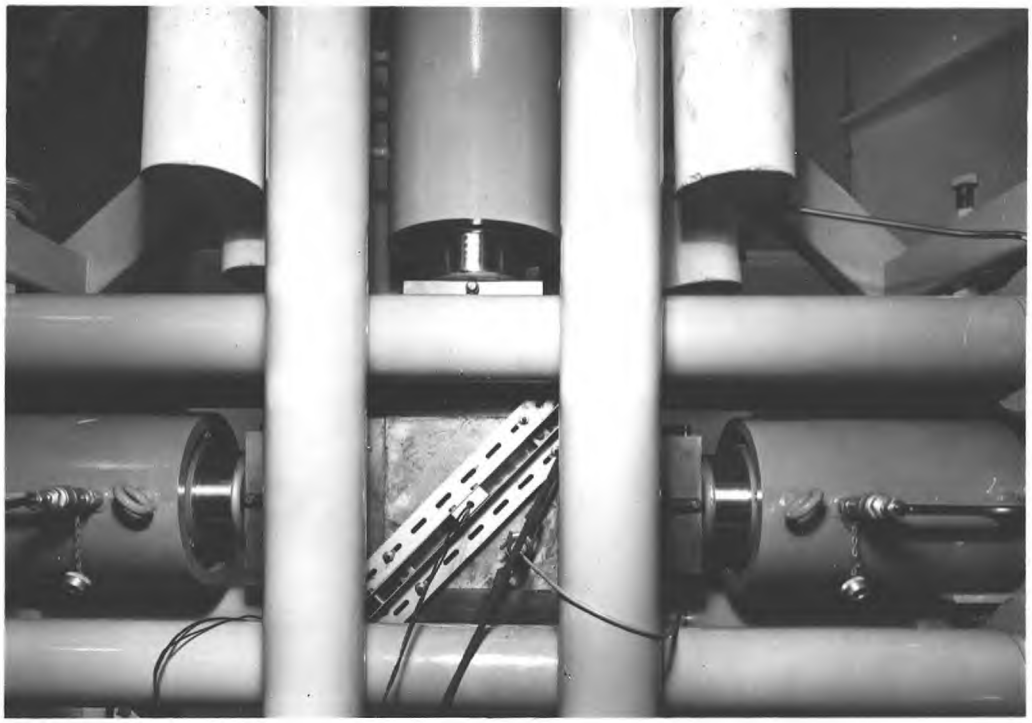
Care had to be taken to ensure that the rubber thong did not foul the linear displacement transducer frame and this was achieved by angling the thong as seen in fig.8.4.2.1.

8.4.3. Exercising the Specimen

A procedure for initial exercising of the specimen prior to testing was adopted similar to that used in the uniaxial test series and described in section 7.4.

Having attached all the measuring devices, the specimen was loaded and unloaded three times at the specified loading rate of 700 p.s.i./min. total (see section 8.1.5.) to a maximum load of about 30% of its estimated ultimate uniaxial compressive strength in both directions simultaneously. As with the uniaxial specimens, this probably had the effect of ironing out the curvature of the stress-strain curve below the discontinuity point. This is peculiar to the first loading only and not really a true reflection of the subsequent behaviour of the material.

This exercising sequence also provided an opportunity to check the workings of the Sogenique and ultrasonic pulse velocity equipment, since any faults in attaching the equipment to the specimen would be reflected in the readings taken on each return to the same stress level.



(14)

Fig. 8.4.2.1. Biaxial Compression specimen showing Linear Inductance and Ultrasonic Pulse Velocity Transducers attached.

15)

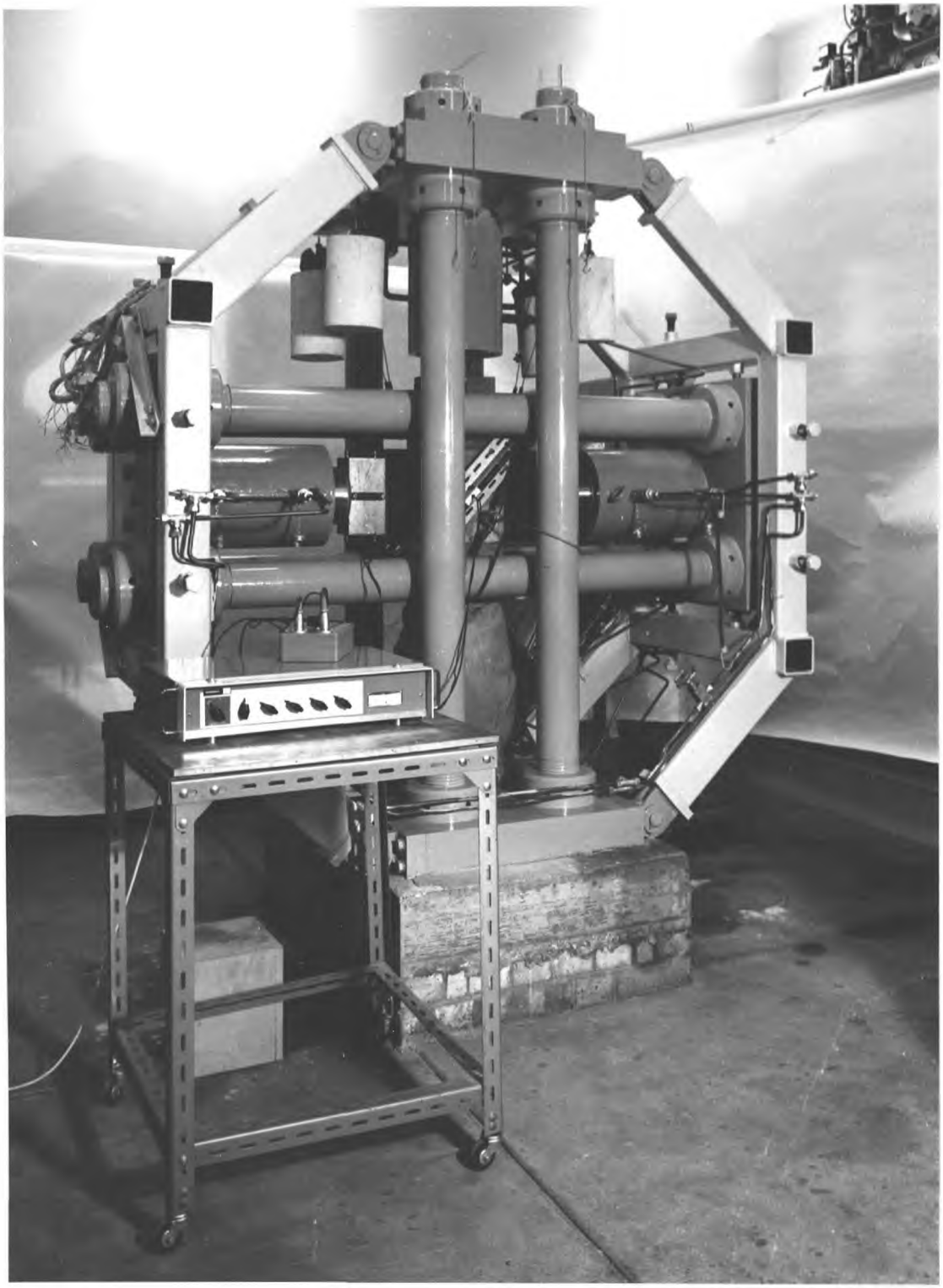


Fig. 8.4.2.2. Complete machine set-up for biaxial compression tests.

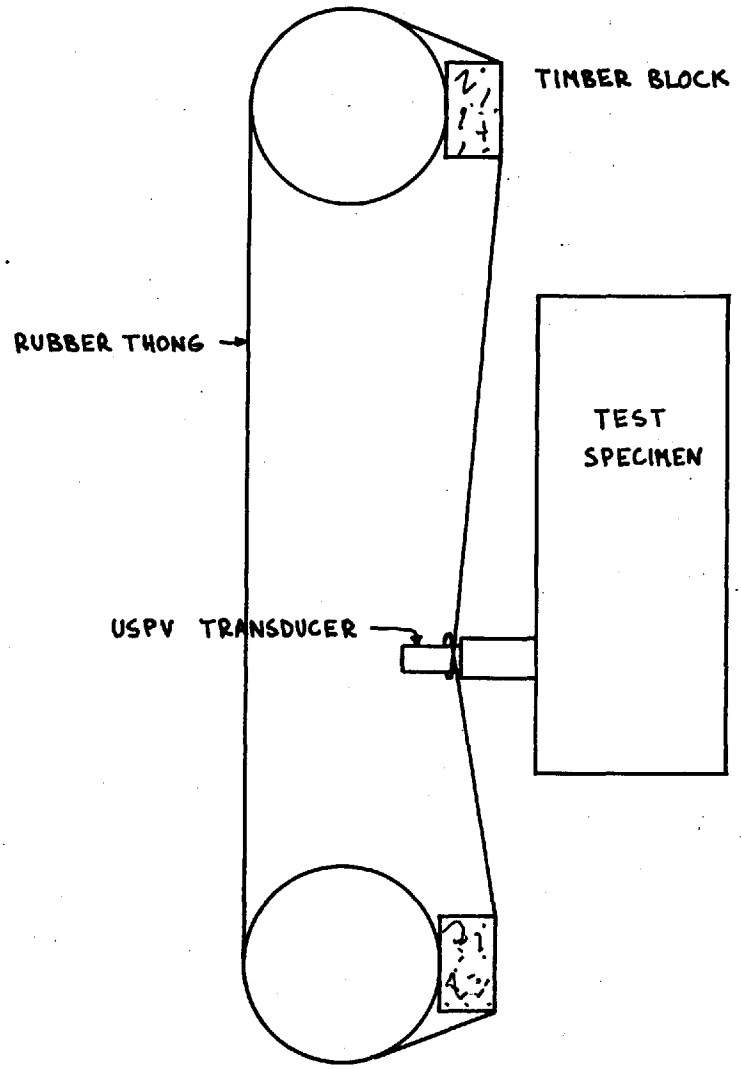


FIG. 8.4.2.3. USE OF RUBBER THONGS AND TIMBER BLOCKS TO FASTEN USPV TRANSDUCER TO FACE OF TEST SPECIMEN.

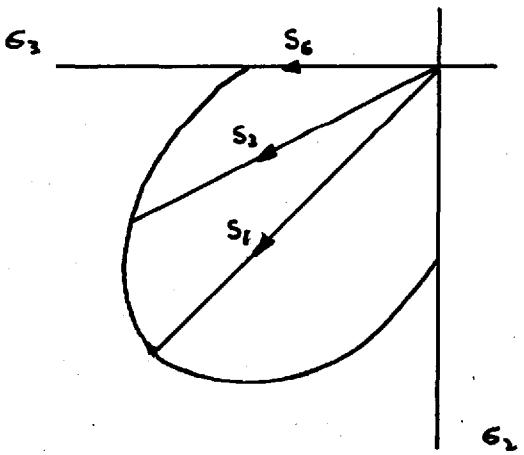
The data logger readings were then set to approximately zero for the initial load stage and the Sogenique readings observed for drift. If all readings were satisfactory, the following procedures were then adopted for loading the specimen.

8.4.4. Elastic Measurements

As mentioned in section 8.3., electrical resistance strain gauges were attached only to the three specimens numbered S1, S3 and S6 from each mix batch of six and on these three specimens were made all the elastic measurements.

These measurements were intended to obtain . repeatable values of strain behaviour, especially elastic modulus and Poisson's ratio, within the "elastic" range of the specimen i.e., at stresses below the point of discontinuity In particular, the influence of the intermediate principal stress σ_2 on the uniaxial elastic parameters obtained under σ_3 alone was to be investigated.

The procedure was to increase both principal stresses together to stress levels of 267, 667, 1333 p.s.i. (and for strong mixes to 2000 p.s.i.) which were equivalent to convenient gauge readings. Then the intermediate principal stress σ_2 (usually the horizontal sub-frame) was maintained c nstant at the chosen stress level whilst the major principal stress was increased and decreased to loading stages either side of this mean. The loading stages usually gave a loading range of 533 p.s.i. which was again a convenient value from the point of view of reading the hydraulic pressure gauges. At both load stages, measurements of all three principal strains were made from which the elastic moduli and Poisson's ratio values could be calculated for varying degrees of biaxial compression. The maximum load stage never exceeded 25% of the estimated ultimate load. If any drift in the strain readings was noted, these loading and unloading cycles were repeated until two consecutive cycles gave repeatable results.



EVENTUAL LOADING TO FAILURE

SPECIMENS					
S_6		S_3		S_1	
σ_3	σ_2	σ_3	σ_2	σ_3	σ_2
①		①		①	
267	0	267	267	267	267
667		667		667	
267		267		267	
667		667		667	
267		267		267	
①		②		①	
400	0	400	667	400	667
933		933		933	
400		400		400	
933		933		933	
400		400		400	
				③	
				1067	1333
				1600	
				1067	
				1600	
				1067	

ALL STRESSES IN psi.

FIG. 8.4.4.1. ELASTIC MEASUREMENT PROGRAMME.

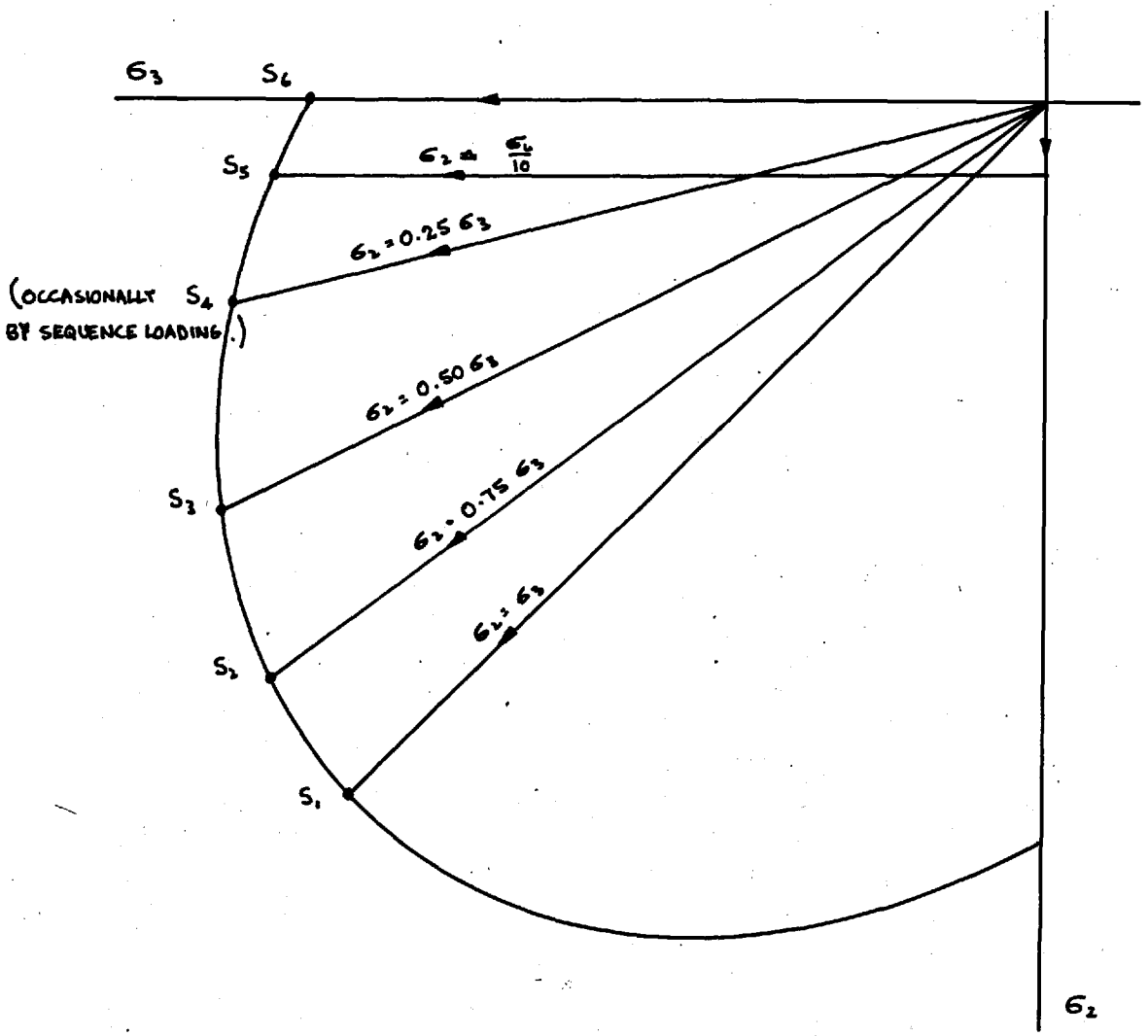


FIG. 8.4.5.1. LOADING SEQUENCES TO FAILURE IN BIAXIAL COMPRESSION.

Unfortunately due to occasional failure or non-availability of equipment, the complete programme of elastic measurements could not be carried out. However, all the results obtained are given in section 8.5.

The specimen numbering is significant as it indicates the subsequent loading to failure of the specimen as shown in the diagram in fig.8.4.5.1.

A schedule of the elastic measurements programme giving the stress combinations to be used is given in fig.8.4.4.1.

8.4.5. Loading to Failure

The loading procedure both for specimens used for elastic measurements and those loaded to ultimate failure was basically the same.

In all cases the load was increased in increments of 267 p.s.i. on the major principal stress with an initial load stage also at 267 p.s.i. The figure of 267 p.s.i. was chosen as it corresponded to the convenient pressure gauge reading of 200 and hence made the process of proportional loading shown in fig.8.4.5.1. much easier for the operator. The loading sequence is specified in fig.8.4.5.1. and with one exception (specimen S5) was always proportional loading.

By proportional loading is meant the steady increase of the magnitude of both principal loads in a fixed proportion to each other. Sequence loading refers to a loading sequence in which one load is applied until it reaches its final value and is then held constant at this value whilst the other load is increased to its final value. However, as was demonstrated in section 4.4., no significant difference could be detected between the results of proportional and sequence loading. Hence in some of the earlier tests the conditions of strict proportional loading specified above were relaxed until experience had been gained with the testing machine.

The rate of loading specified in section 8.1.5. was that the sum of the two principal stresses should be at a rate of 700 p.s.i./min. For example, with specimen S1 the two principal stresses σ_2 and σ_3 would each be applied at a rate of 350 p.s.i./min.

This means that if the elastic modulus and Poisson's ratio remain constant under biaxial states of stress, the maximum tensile or lateral strain increases atthe same rate for all specimens. It was later found from these tests that the elastic modulus tends to increase with the degree of biaxial compression as the volume fraction of coarse aggregate was increased whereas the Poisson's ratio remains effectively constant up to the discontinuity stress. Hence for concrete mixes, the actual rate of increase of the maximum tensile strain tended to be lower than that for mortars.

Measurements of all three principal strains and the ultrasonic pulse velocity were taken at each load stage which involved a pause in the loading of about six seconds. The linear displacement transducers and their housings were removed three load stages after the discontinuity point had been reached as indicated by the ultrasonic pulse velocity readings. This precaution was necessary because some of the failures were explosive and might have damaged the transducers.

After the specimen had failed, the bottom platen was locked in position to save time in the alignment sequence for the next test whilst the other three platens were withdrawn about $\frac{1}{2}$". This allowed the linear displacement transducer frame and then the test specimen to be removed from the machine.

The failed specimen was examined for failure crack patterns and the results of these examinations are described in detail in section 11.5.

The loading platen faces were then cleaned with acetone-soaked soap in preparation for the next test.

8.4.6. Special Series on Elastic Modulus

Once it was recognised from the elastic measurements described above that concretes exhibited an increase in elastic modulus under biaxial compression, it was decided to investigate this variation in more detail by initiating a separate series of tests on an intermediate concrete (mix 61B). This mix was chosen as it has a nominal 30% volume fraction of coarse aggregate and is therefore intermediate between the mortars and the 50% volume fraction concretes.

Two test specimens were cast and the opportunity of checking the stress state in a concrete specimen was taken to provide a comparison with the standard mortar proving trial specimens. This was achieved by using 13 electrical resistance strain gauges on each face of the test specimens with the layout given in fig.4.4.1.1.

The loading sequence was different for the two specimens and is given in detail in table 8.4.6.1. However, the general sequence of loading was such that the elastic modulus could be calculated for a series of load stages starting at zero load and increasing in 267 p.s.i. intervals up to failure. All 26 gauges were read at each load stage so that a picture of the strain distribution could be obtained. This necessitated a reduction in the loading rate from the specified rate of 700 p.s.i./min. to about 500 p.s.i./min. to allow this large number of readings to be taken. However, it is unlikely that the results were significantly affected by the change in loading rate.

The results from the strain distribution point of view have already been described in section 4.4.1. and the results of the elastic modulus variation are given in section 8.5.

8.5. Results

The summarised results of the test series are given in tables 8.5.1. to 8.5.3. whilst the graphical presentation of these results and their significance are discussed below.

SPECIMEN			
E_1		E_2	
σ_2	σ_3	σ_2	σ_3
0	0	0	0
	133		133
	0		0
	133		133
	0		0
267	133	267	133
	400		400
	133		133
	400		400
	133		133
533	400	533	400
	667		667
UNTIL		UNTIL	
1333	1200	1333	1200
	1467		1467
	1200		1200
	1467		1467
	1200		1200
UNTIL		UNTIL	
1600	1467	1600	1600
	1733	1867	1867
	1467	UNTIL	
UNTIL		2667	2667
2667	2533		2933
	2800		3200
	2533	UNTIL	
2667	2667	2667	6775
2933	2667	FAILURE	
2933	2933		
3200	2933		
UNTIL			
4000	4000		
4267	4267		
UNTIL			
6610	6610		
FAILURE			

TABLE. 8.4.6.1. LOADING SEQUENCE FOR SPECIAL SERIES ON ELASTIC MODULUS.

MIX	PRINCIPAL STRESS	ULTIMATE STRESS psi.						DISCONTINUITY STRESS psi					
		S_1	S_2	S_3	S_4	S_5	S_6	S_1	S_2	S_3	S_4	S_5	S_6
MA	σ_2	8630	6660	4640	2050	450	0	7200	6660	3670	2030	450	0
	σ_3	8750	8950	9100	8750	7280	7300	7200	7700	7450	7960	5590	4310
MBC	σ_2	5650	3950	2560	1250	200	0	3400	2050	1380	1250	200	0
	σ_3	5730	5800	5520	5450	5100	4350	3400	2850	2850	3400	3000	2250
MD	σ_2	4520	3350	2160	1050	450	0	3000	2300	1600	700	450	0
	σ_3	4660	4760	4500	4480	4600	3650	3000	3130	3250	3060	2850	1370
31B	σ_2	6750	5050	3250	1250	450	0	4350	3690	2300	1250	450	0
	σ_3	6800	7450	7380	6420	6100	5360	4350	4500	4750	4500	3690	3000
31C	σ_2	5610	4220	2560	1250	450	0	3680	3400	2040	1250	450	0
	σ_3	5620	6340	6060	5840	5650	4220	3680	3400	3400	3400	3120	2300
61A	σ_2	7130	5300	3250	1250	600	0	5050	4630	2710	1250	580	0
	σ_3	7130	7650	7960	6800	6600	5400	5050	5450	5580	4350	3400	2570
61B	σ_2	5720	4760	3540	2300	1100	0	3950	4000	3350	2200	1050	0
	σ_3	5850	6050	6350	6000	5600	4220	3950	4560	4700	4570	3800	3000
61C	σ_2	7200	5300	3680	2040	700	0	4050	4730	3680	2040	700	0
	σ_3	7200	7500	7450	6760	5920	5050[X]	4050	4730	5300	4630	3940	2570
61D	σ_2	6120	4760	3120	1780	600	0	3950	3700	3120	1800	600	0
	σ_3	6120	6420	6520	6300	5650	4600	3950	4500	4750	4750	3950	3120
6RB	σ_2	5250	3950	2560	1250	450	0	3950	2580	1900	1250	450	0
	σ_3	5300	5130	5700	4850	4700	3950	3950	3400	3950	3130	2850	2580
6RC	σ_2	6520	4760	3250	1770	600	0	3400	3950	1700[X]	1780	600	0
	σ_3	6600	7150	6730	6550	5900	4350	3400	4750	3530	4750	3950	3400
6AB	σ_2	8125	5300	3250	1250	450	0	6000	5320	2710	1250	450	0
	σ_3	8125	9220	8120	8000	7670	6400	6000	6000	6400	5600	4750	4220
6LB	σ_2	3700	2700	1780	600	200	0	2300	1500	1100	600	190	0
	σ_3	3700	3840	3700	3630	3290	2700	2300	2030	2300	2030	2030	1780

X DENOTES SUSPECT RESULT.

TABLE 8.5.1. BIAXIAL COMPRESSION TEST RESULTS (PART 1 & 3 — STRESS RESULTS).

MIX	PRINCIPAL STRAIN	DISCONTINUITY CLEAVAGE STRAIN x 10^6						DISCONTINUITY LONGITUDINAL STRAIN x 10^6					
		S_1	S_2	S_3	S_4	S_5	S_6	S_1	S_2	S_3	S_4	S_5	S_6
4A	ϵ_3 ϵ_2 ϵ_1	936	1060	848	778	–	267	1471 1455	–	1793 354	–	–	1158 -346
4BC	ϵ_3 ϵ_2 ϵ_1	399	259	236	171	–	133	921 780	–	864 265	–	–	776 -162
4D	ϵ_3 ϵ_2 ϵ_1	466	360	422	343	250	106	857 767	–	1271 309	–	–	486 -89
3IB	ϵ_3 ϵ_2 ϵ_1	405	428	309	207	167	120	921 743	–	1327 193	–	–	703 -128
3IC	ϵ_3 ϵ_2 ϵ_1	–	–	–	–	–	–	–	–	–	–	–	–
6IA	ϵ_3 ϵ_2 ϵ_1	–	–	–	–	–	–	723 666	967 483	866 308	847 63	684 -62	460 -80
6IB	ϵ_3 ϵ_2 ϵ_1	140	–	–	–	–	104	2140 1889	1283 963	1432 778	1120 358	1016 111	752 -104
6IC	ϵ_3 ϵ_2 ϵ_1	115	128	–	95	82	63	–	–	–	–	–	–
6ID	ϵ_3 ϵ_2 ϵ_1	302	293	325	320	233	131	647 646	–	825 558	–	–	612 -107
6RB	ϵ_3 ϵ_2 ϵ_1	391	254	326	221	223	183	1010 852	–	1075 191	–	–	372 -49
6RC	ϵ_3 ϵ_2 ϵ_1	167	282	147	248	141	107	478 452	–	626 149	–	–	607 -44
6AB	ϵ_3 ϵ_2 ϵ_1	756	533	403	394	246	296 X	1243 1243	–	–	–	–	933 –
6LB	ϵ_3 ϵ_2 ϵ_1	467	297	360	206	215	177	518 546	–	872 221	–	–	716 -54

X DENOTES SUSPECT RESULT

TABLE 8.5.2. BIAXIAL COMPRESSION TEST RESULTS (PART 2 & 3 – STRAIN RESULTS).

MIX	ELASTIC MODULUS -								
	S_1			S_3			S_6		
σ_2 psi	267	667	1333	267	667	1333	0	0	0
MA	5·07	5·33	5·22	—	—	—	—	—	—
MBC	4·26	4·38	4·37	3·32	3·34	3·85	3·36	3·24	3·46
61A	6·37	6·71	7·14	4·14	4·29	4·89 (5·62)	—	—	—
61B	4·61	5·73	5·60	—	—	—	—	—	—
61C	—	—	—	3·92	4·33	4·98	—	—	—
6LB	2·92	2·26	2·43	—	—	—	3·01	—	2·48

(5·62) WHEN σ_2 = 2000 psi.

(REFER TO FIG. 8.4.6.1. FOR LOADING SEQUENCES)

TABLE. 8.5.3. BIAXIAL COMPRESSION TEST RESULTS (PART 3 of 3 – ELASTIC MODULUS).

8.5.1. Stress-Strain Characteristics

The general shape of the stress-strain curves obtained for mortars and concretes under biaxial compression were very similar to those obtained under uniaxial compression. Three typical curves for a mortar (MBC), a normal aggregate concrete (61D), and a light-weight aggregate concrete (6LB) are shown in figs.8.5.1.1. to 8.5.1.3. with a corresponding uniaxial curve superimposed for comparison.

The stress-strain curves for equal biaxial compression were selected for comparison with the uniaxial compression curves from the same mix batch as this enables the two extreme cases to be considered; other degrees of biaxial compression affording intermediate cases. Even though the test specimens had all been exercised in accordance with the procedure laid down in sections 7.4 and 8.4, some curvature in the stress-strain curve is evident below the discontinuity point.

Biaxial compression stress-strain curves will inevitably be steeper than the uniaxial curves because of the reduction in longitudinal strain due to the effect of the other principal stress. In the curves presented, one principal stress has been plotted against the longitudinal strain in its direction and against half the lateral strain as the other half of this strain is due to the other principal stress.

A comparison between the behaviour of mortars and concretes below the discontinuity point with that of an ideally elastic material has already been made by noting the curvature in the stress-strain curve below the discontinuity point indicating a variation in elastic modulus with stress level. A further comparison has been made on the curves presented by endeavouring to predict the biaxial stress-strain behaviour from the uniaxial stress-strain behaviour using the basic elastic theory equations.

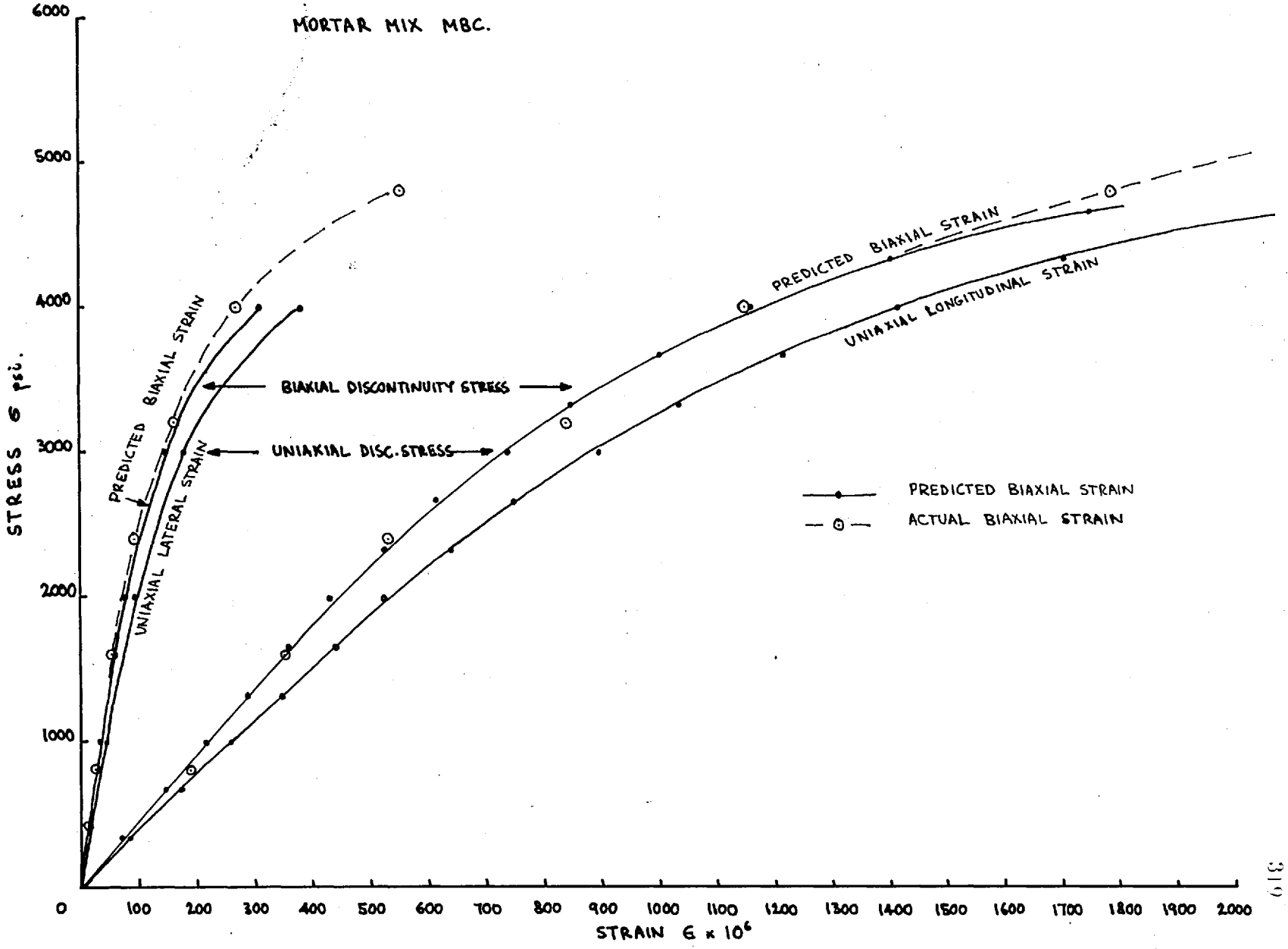


FIG. 8.5.1.1. COMPARISON OF UNIAXIAL AND BIAXIAL COMPRESSION BEHAVIOUR.

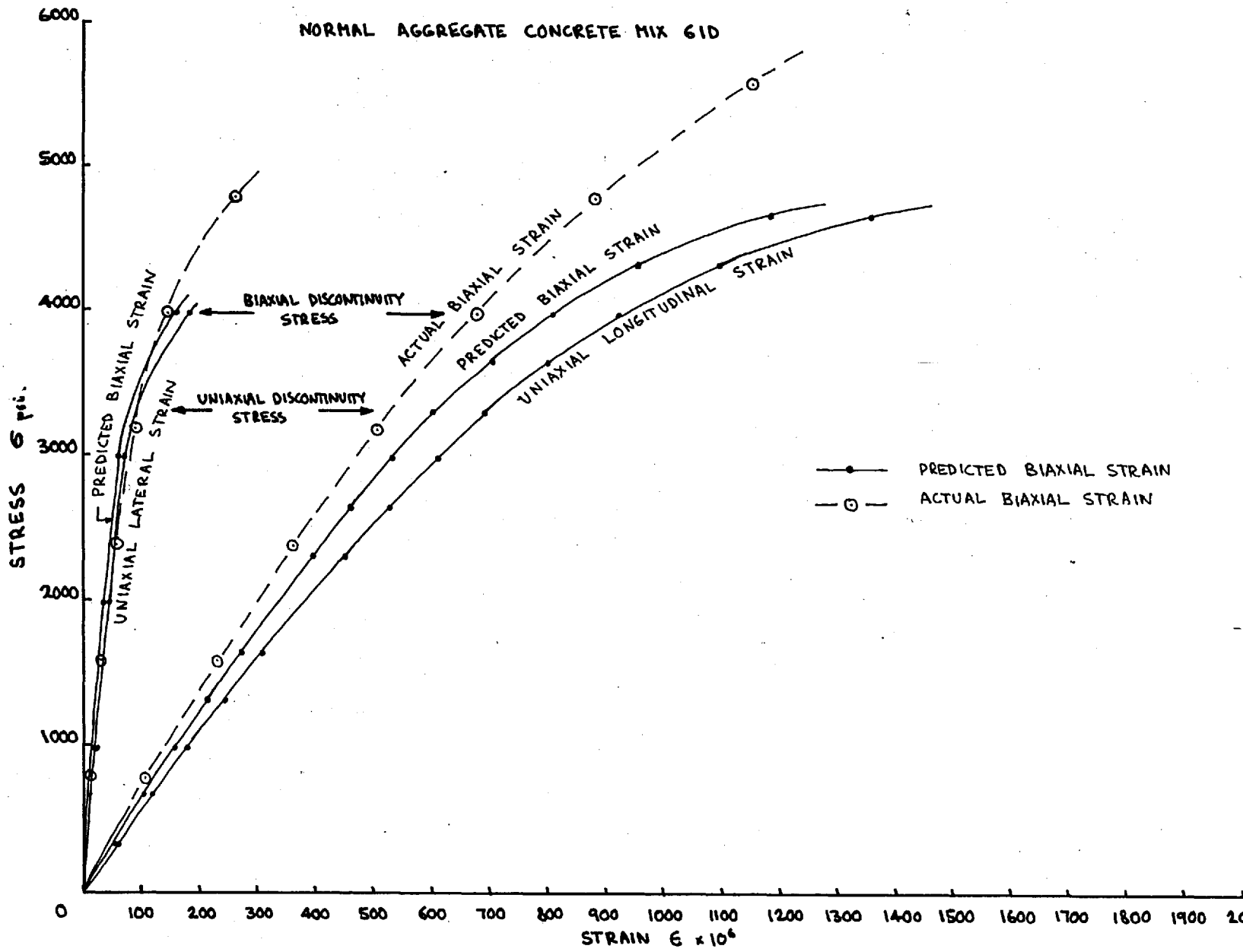


FIG. 8.5.1.2. COMPARISON OF UNIAXIAL AND BIAXIAL COMPRESSION BEHAVIOUR.

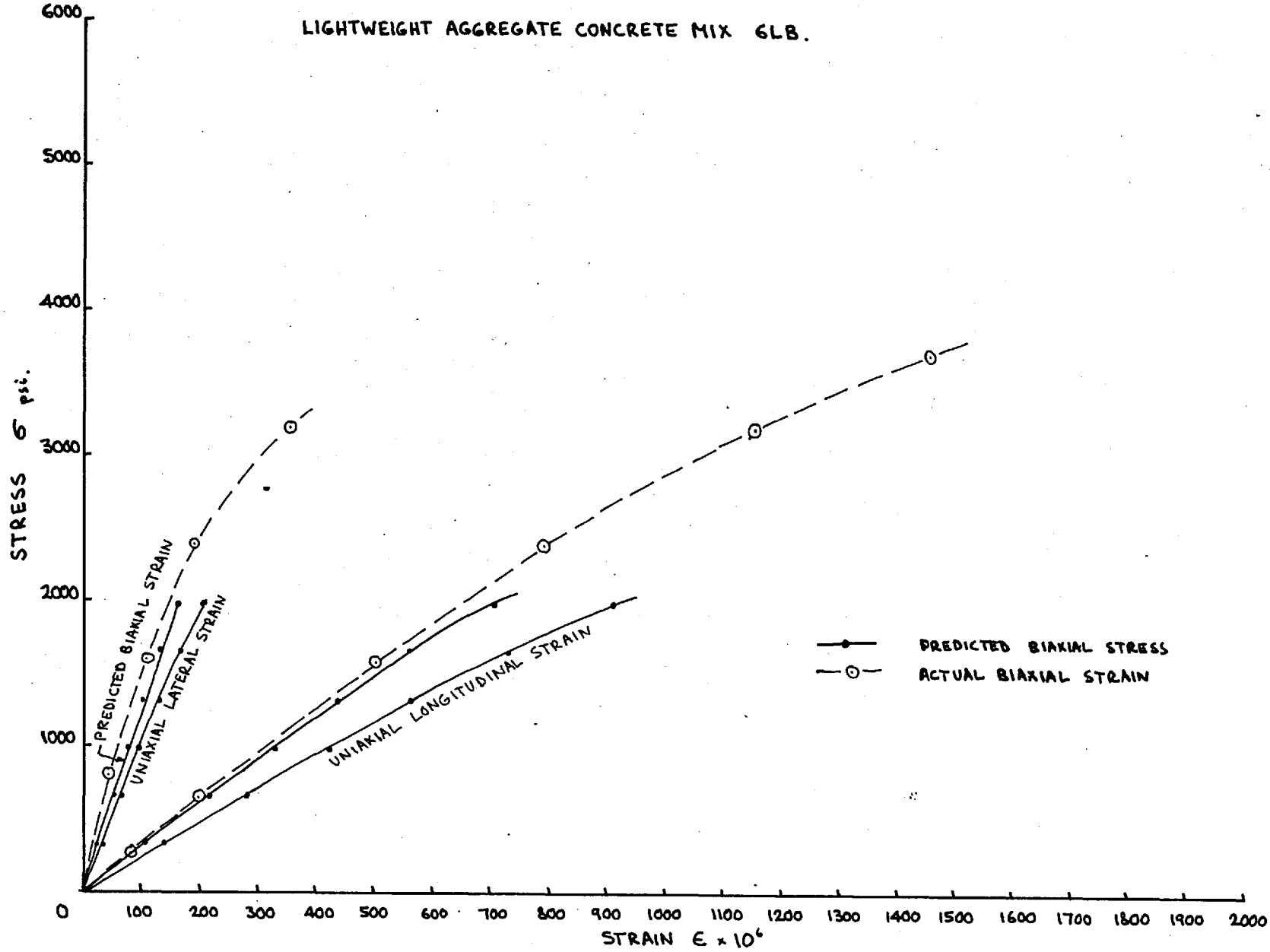


FIG. 8.5.1.3. COMPARISON OF UNIAXIAL AND BIAXIAL COMPRESSION BEHAVIOUR.

Under uniaxial compression:-

$$\epsilon_3 = \frac{\sigma_3}{E}$$

whilst under biaxial compression:-

$$\epsilon_3 = \frac{\sigma_3}{E} - \frac{\nu\sigma_2}{E}$$

Now if the case of equal biaxial compression plotted in the stress-strain curves presented in figs.8.5.1.1. to 8.5.1.3. is considered:-

$$\sigma_3 = \sigma_2 \qquad \epsilon_3 = (1-\nu)\frac{\sigma_3}{E}$$

This latter equality lessens the problem presented by the elastic modulus varying with stress level. Hence by making the above conversion of the uniaxial strain readings, the biaxial stress-strain curve up to the discontinuity point on the uniaxial curve may be predicted. Note that in the curves actually plotted, Poisson's ratio was assumed constant and this assumption is discussed below.

The results of this comparison of behaviour are very interesting. In the cases of mortar and lightweight aggregate concrete the prediction of biaxial behaviour is very close to the actual measured behaviour being never more than 2% in error for the mortar or 4% for the lightweight concrete. However, with the normal aggregate concrete, the error is substantial being about 15% for both longitudinal and lateral strains and indicating an increase in elastic modulus under biaxial compression (see figs.8.5.1.1.-3.).

These curves are typical for the results obtained with mortars and concretes. The normal aggregate concrete mix chosen in these graphs was 61D as it has a nominal volume fraction of coarse aggregate of 0.50 and therefore would be expected to give the greatest deviation from elastic behaviour if such deviation

existed. The other normal aggregate concretes show similar trends though of a magnitude which appears to be relative to their volume fraction of coarse aggregate. This behaviour is best illustrated in fig.8.5.1.4. in which the variation of elastic modulus with mean normal stress is shown for a parent mortar (MBC), two normal aggregate concretes (61B with 0.30 and 61C with 0.50 nominal volume fraction of coarse aggregate), and one lightweight aggregate concrete (6LB with 0.30 nominal value fraction of coarse aggregate).

In this graph, the variations noted in the normal stress-strain curves discussed above are again evident. Whilst the elastic modulus for mortar and lightweight concrete show variations of less than 5% under varying degrees of biaxial compression, the concretes exhibit variations of up to 40% depending on the volume fraction of coarse aggregate in the mix.

This behaviour of normal aggregate concrete suggests that some form of particle interaction is responsible for the variation of elastic modulus with stress level and may be coupled with a mechanism of internal crack closure under confining pressure.

Note that the parameter of mean normal stress was used in fig.8.5.1.4. and is defined as;-

$$\sigma_m = \frac{\sigma_1 + \sigma_2 + \sigma_3}{3}$$

The use of this parameter is convenient because it is a basic quantity in elasticity being related to the first invariant of the stress tensor by the equation $I_1 = 3\sigma_m$. Although it may be misleading when biaxial compression behaviour is under consideration, as it implies triaxial or volumetric stress, the alternative parameter of degree of biaxial compression defined as:-

$$\sigma_a = \frac{\sigma_2 + \sigma_3}{2}$$

is rarely used and the simple relationship between these two parameters is:

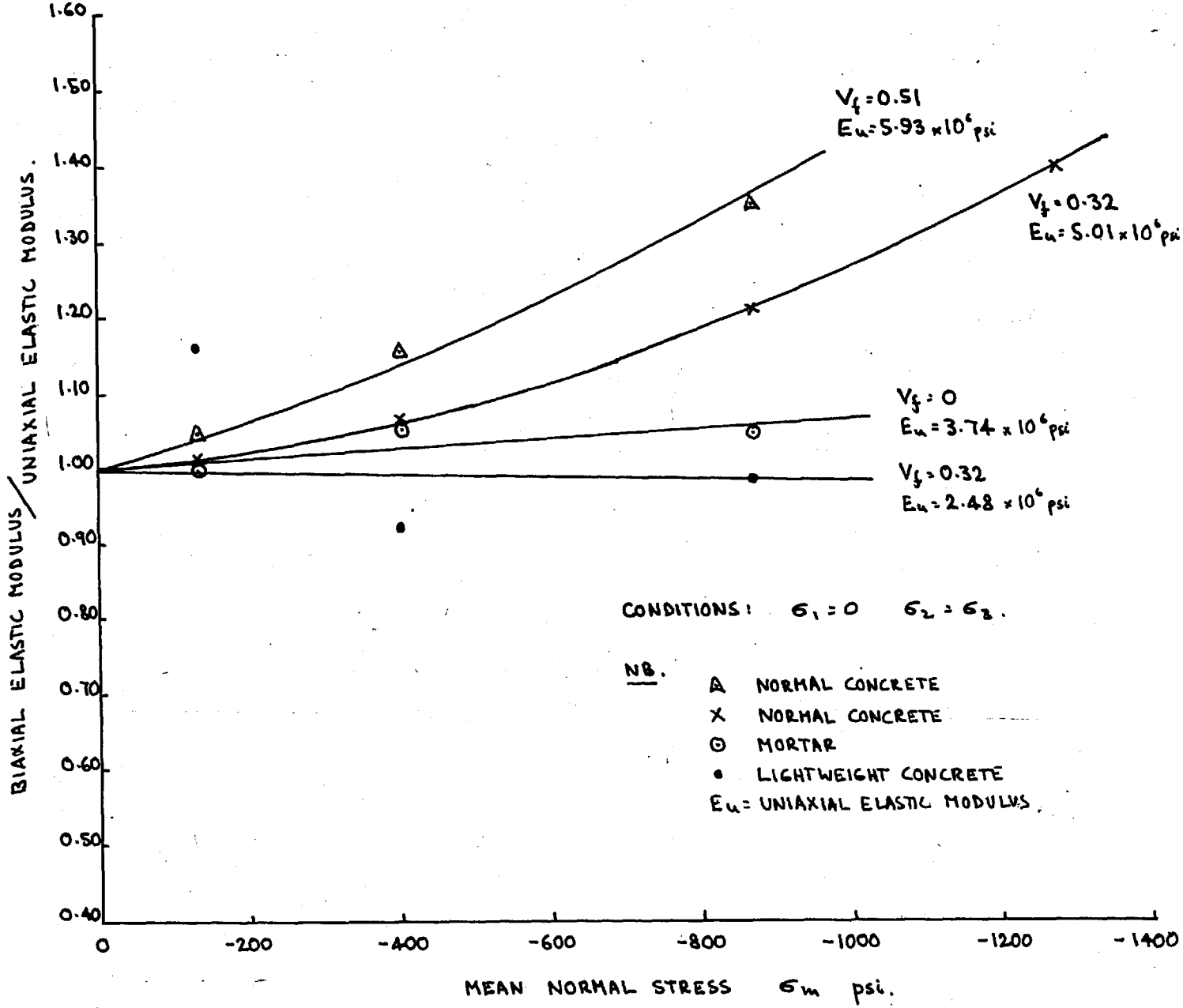


FIG. 8.5.1.4. VARIATION OF ELASTIC MODULUS WITH BIAXIAL STRESS.

$$\sigma_a = \frac{3}{2}\sigma_m. \qquad \text{when} \quad \sigma_1 = 0$$

Therefore a change of parameters simply means a change of scale and has no effect on the shape of the resulting curves. Hence the parameter of mean normal stress is used throughout.

The results of the more detailed investigation into the variation of elastic modulus with degree of biaxial compression described in section 8.4.6. is shown in fig.8.5.1.5. and simply confirms the findings of the main test series given in fig.8.5.1.4.

An assumption made in using the elastic theory equations to predict the biaxial stress-strain curves above was that Poisson's ratio remained effectively constant under all states of stress below the discontinuity point. The validity of this assumption can be assessed from the variation of Poisson's ratio with the major principal compressive stress σ_3 for the three representative mixes used in the stress-strain curves under uniaxial and biaxial compression given in fig.8.5.1.6.; Poisson's ratio being calculated from the three dimensional equations of elasticity.

Although for most practical purposes the value of the Poisson's ratio below the discontinuity point is constant and almost equal under biaxial and uniaxial compression there is a small yet gradual increase in its value up to the discontinuity point. This increase, which at its maximum is of the order of 10%, may be due to the opening up of small cracks and fissures around aggregate particles. It is noteworthy that this effect is more pronounced with the concrete mixes than with the mortars. However, it is clear from the relative orders of magnitude that the trends regarding the variation of elastic modulus noted above cannot be invalidated by these relatively small variations in Poisson's ratio. Note that the Poisson's ratio values are based on the total longitudinal and lateral strains measured from zero load up to the stress level considered.

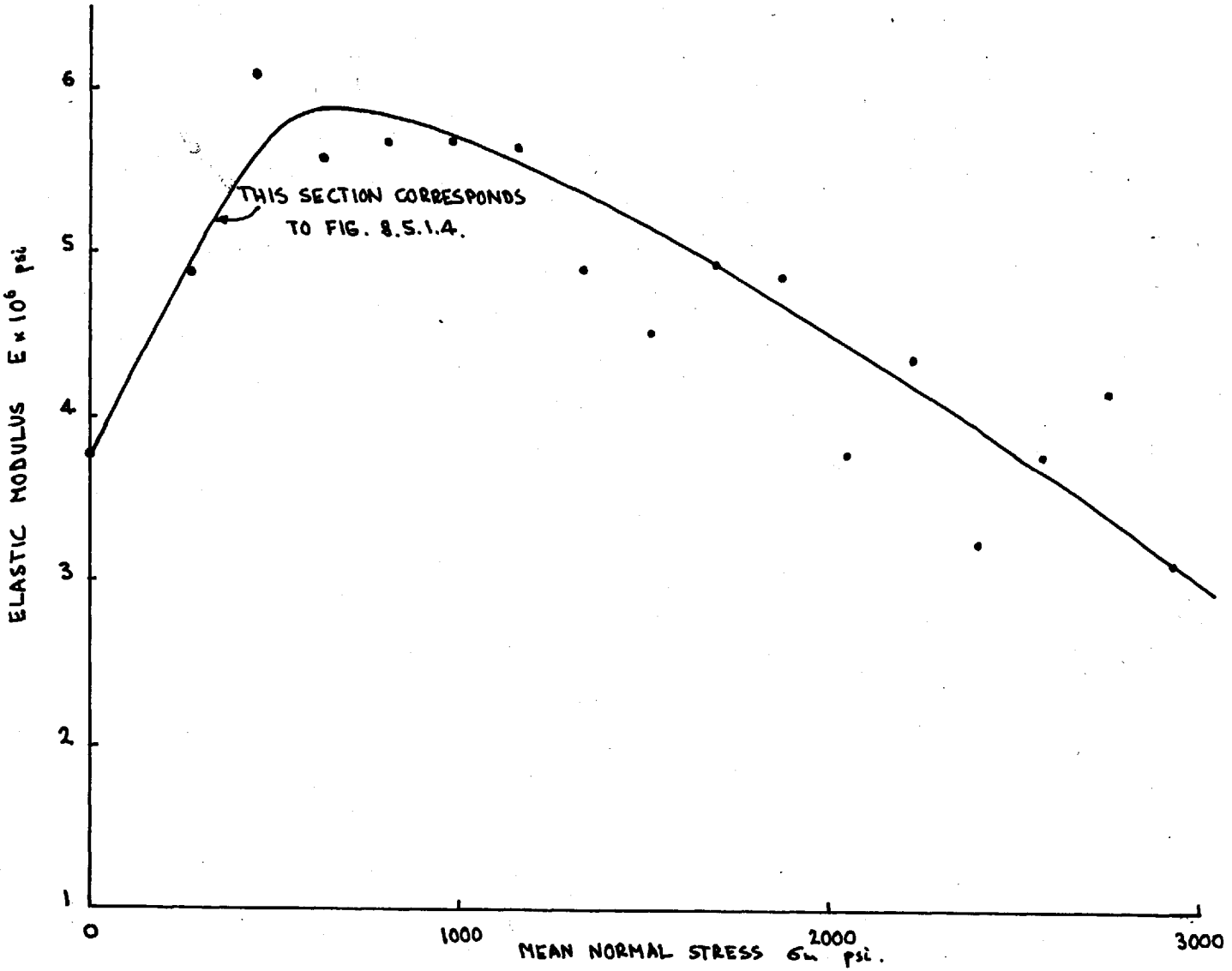


FIG. 8.5.1.5 OVERALL VARIATION OF ELASTIC MODULUS IN CONCRETE

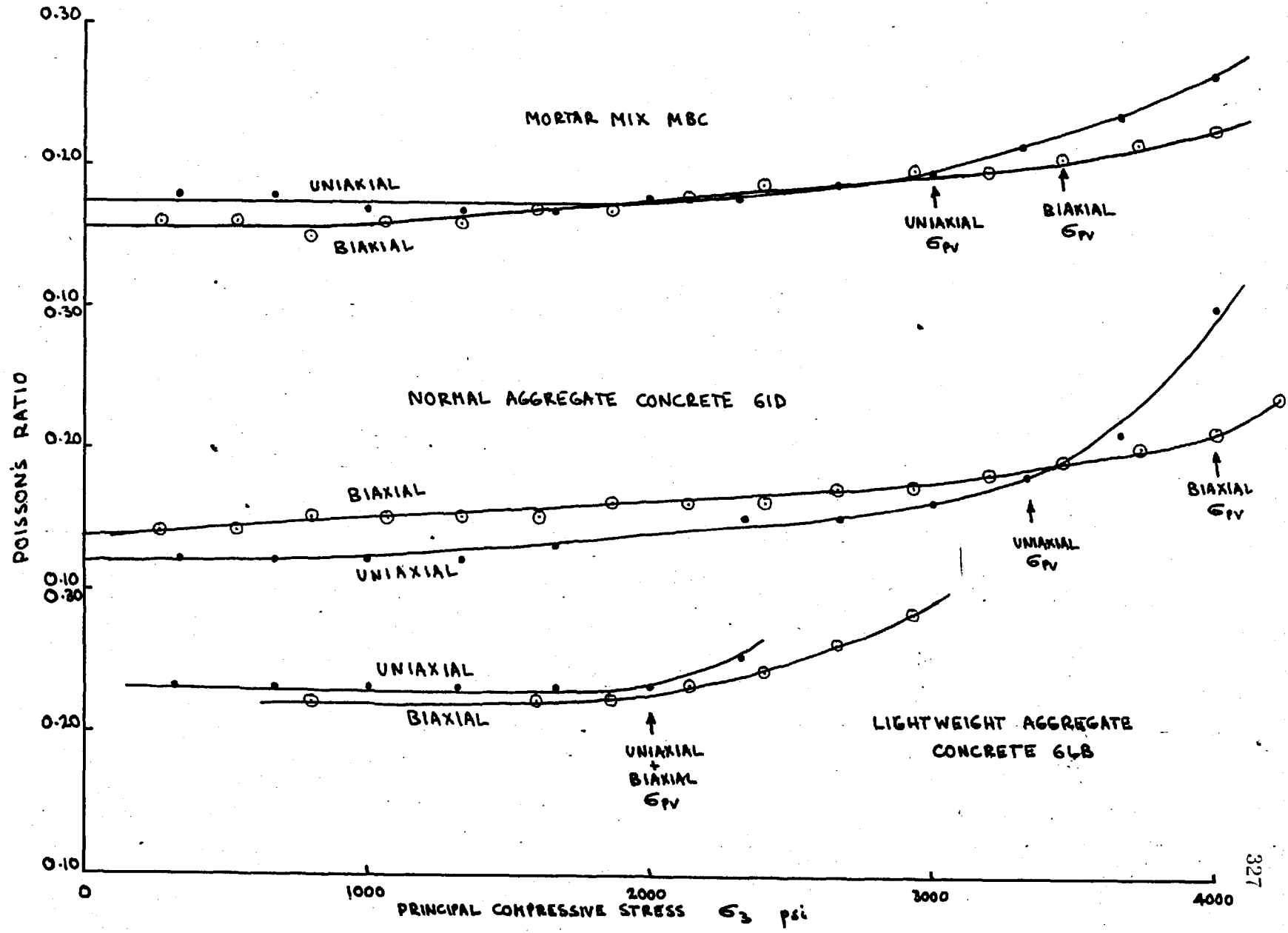


FIG. 8.S.1.6. VARIATION OF POISSON'S RATIO IN UNI- AND BIAXIAL COMPRESSION.

In conclusion it may be said that mortars and lightweight concretes obey the elastic theory for all practical purposes, the greatest deviation being a slight curvature of the stress-strain curve below the discontinuity point. Normal aggregate concretes on the other hand exhibit an increase in elastic modulus with mean normal stress of up to 40%. Poisson's ratio is effectively constant below the discontinuity point but rises rapidly above it.

8.5.2. Discontinuity and Ultimate Stress

The discontinuity and ultimate stresses recorded under biaxial compression have been plotted in the form of envelopes. They have been projected symetrically either side of the equal stress axis in view of the isotropic nature of carefully compacted concrete and each mix tested has been shown separately in figs. 8.5.2.1. ao 8.5.2.13.

The first feature of the envelopes to note is the emergence of two apparently different shapes of envelope. On the one hand, the normal aggregate concretes produce a rounded envelope whilst on the other, the mortars and the lightweight aggregate concrete produce a more square shaped envelope. Some evidence of similar trends is present in the results of Robinson[10] which have been replotted in figs.1.3.19.2. and 1.3.19.3. though the results were by no means conclusive and one mortar curve (Mix 3) contradicted the other two.

The second significant feature is the similarity between the discontinuity stress envelope and its corresponding ultimate strength envelope regardless of whether the envelopes are of the rounded or square shaped variety. This feature suggests, though by no means establishes, that the basic mechanism of failure for any one mortar or concrete is similar under all states of biaxial compression.

MIX No. MA $^{S}/_{C}$ 2.4 $^{W}/_{C}$ 0.45

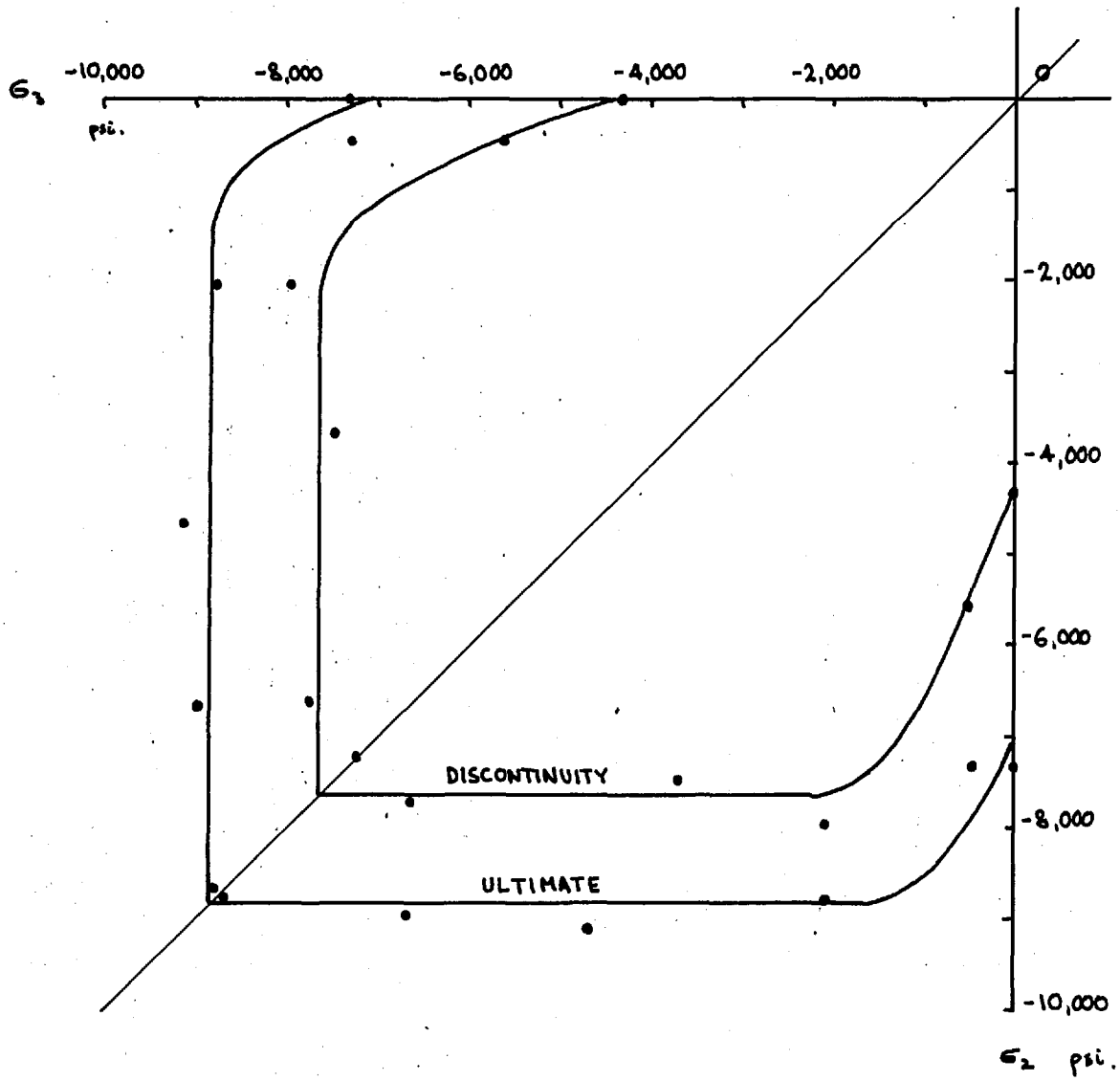


FIG. 8.5.2.1. BIAXIAL COMPRESSION STRESS ENVELOPES FOR MIX MA.

MIX No. MBC $^{S}/_{C}$ 2.4 $^{W}/_{C}$ 0.55

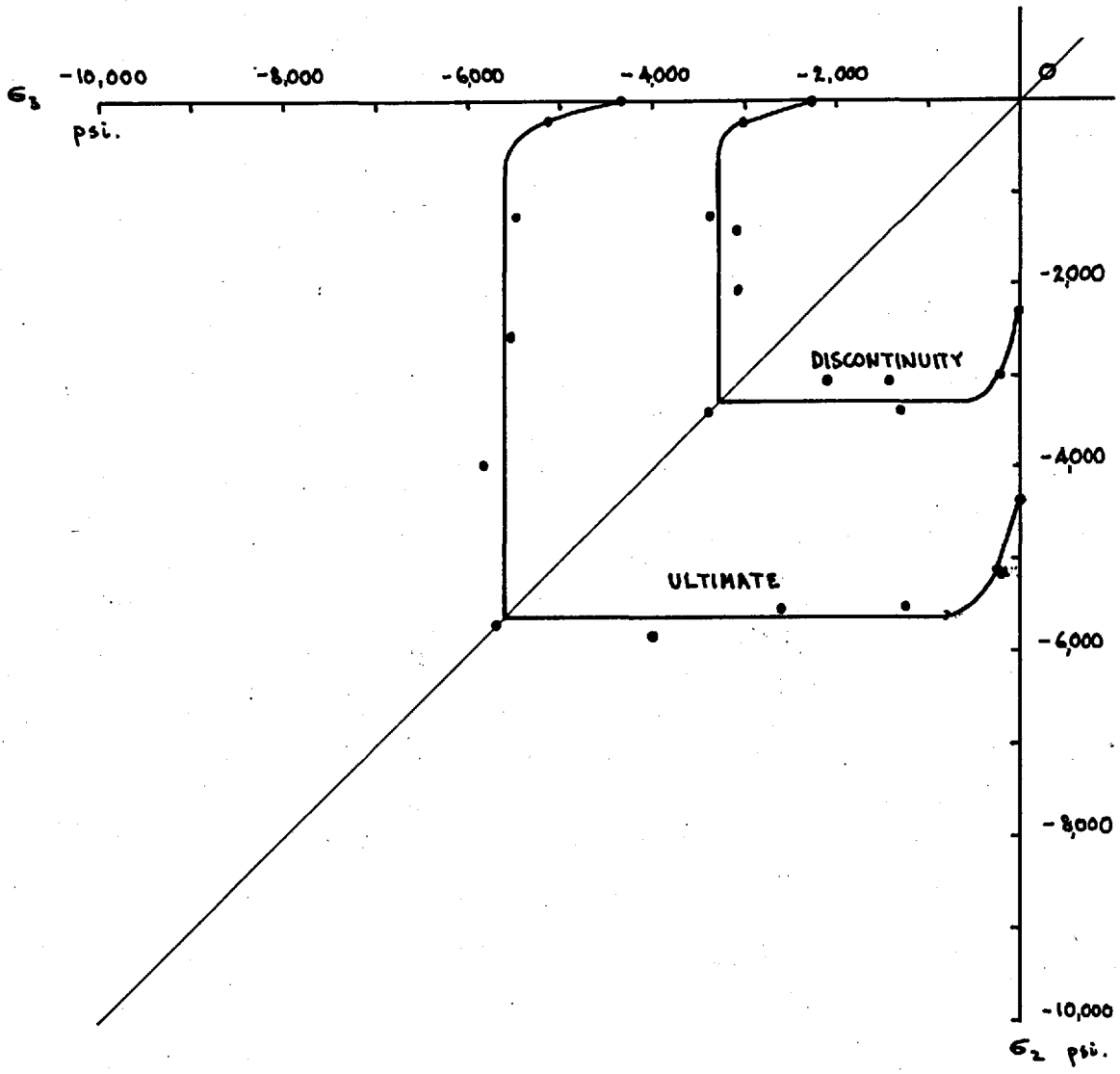


FIG. 8.5.1.1. BIAXIAL COMPRESSION STRESS ENVELOPES FOR MIX MBC.

MIX No. MD $^{S}/_{C}$ 2.4 $^{W}/_{C}$ 0.65

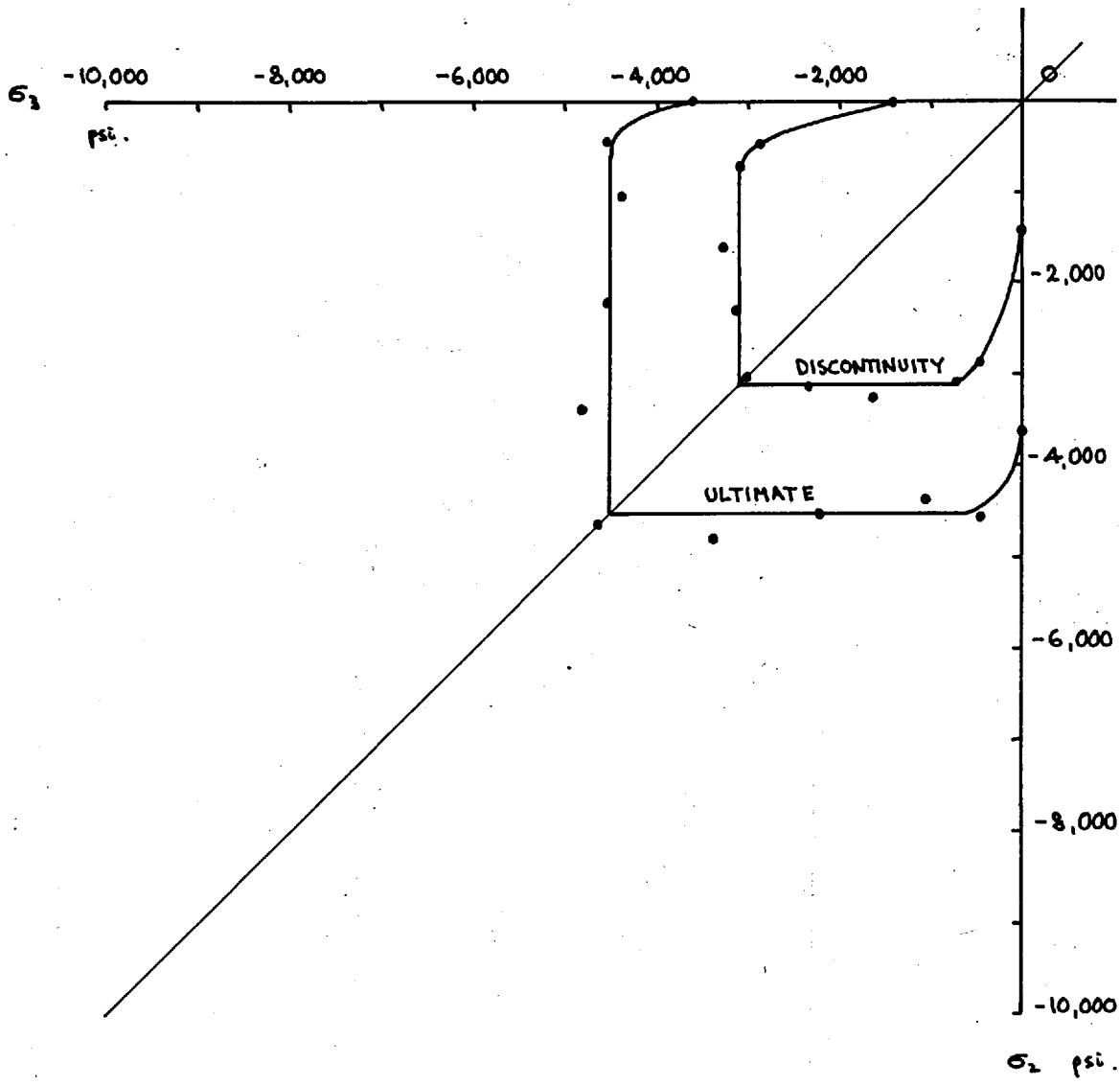


FIG. 8.5.2.3 BIAXIAL COMPRESSION STRESS ENVELOPES FOR MIX MD.

MIX No. 31B $^{A}/_{C}$ 4.5 $^{S}/_{C}$ 2.4 $^{W}/_{C}$ 0.55 $^{3}/_{8}$" IRREGULAR COARSE AGGREGATE

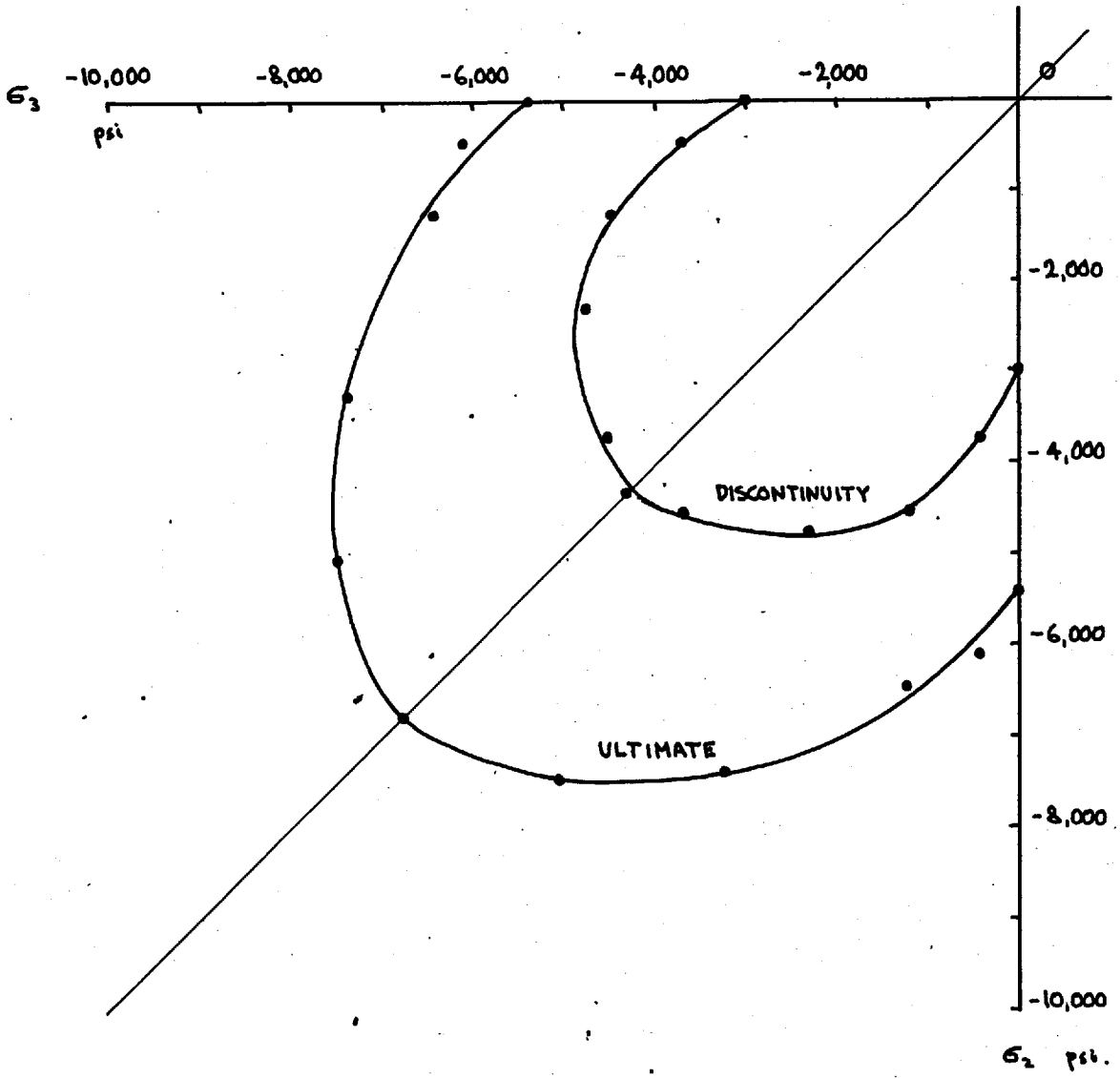


FIG. 8.5.2.4. BIAXIAL COMPRESSION STRESS ENVELOPES FOR MIX 31B.

MIX No. 31C $^{A}/_{C}$ 7.5 $^{S}/_{C}$ 2.4 $^{W}/_{C}$ 0.55 $^{3}/_{8}$" IRREGULAR COARSE AGGREGATE

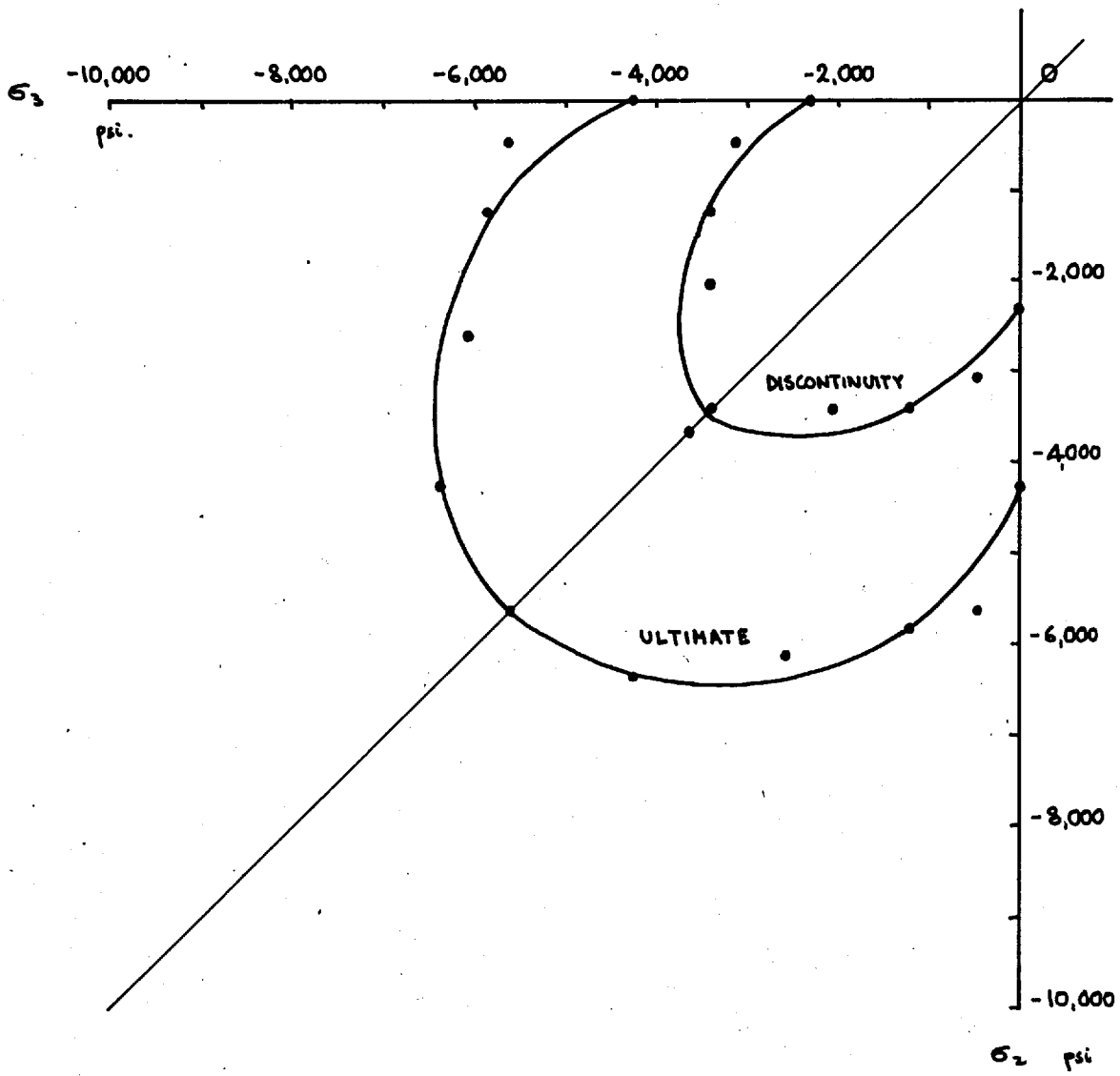


FIG. 8.5.2.5. BIAXIAL COMPRESSION STRESS ENVELOPES FOR MIX 31C.

MIX No. 61A $^{A}/_{C}$ 4.5 $^{S}/_{C}$ 2.4 $^{W}/_{C}$ 0.45 $^{3}/_{4}$" IRREGULAR COARSE AGGREGATE

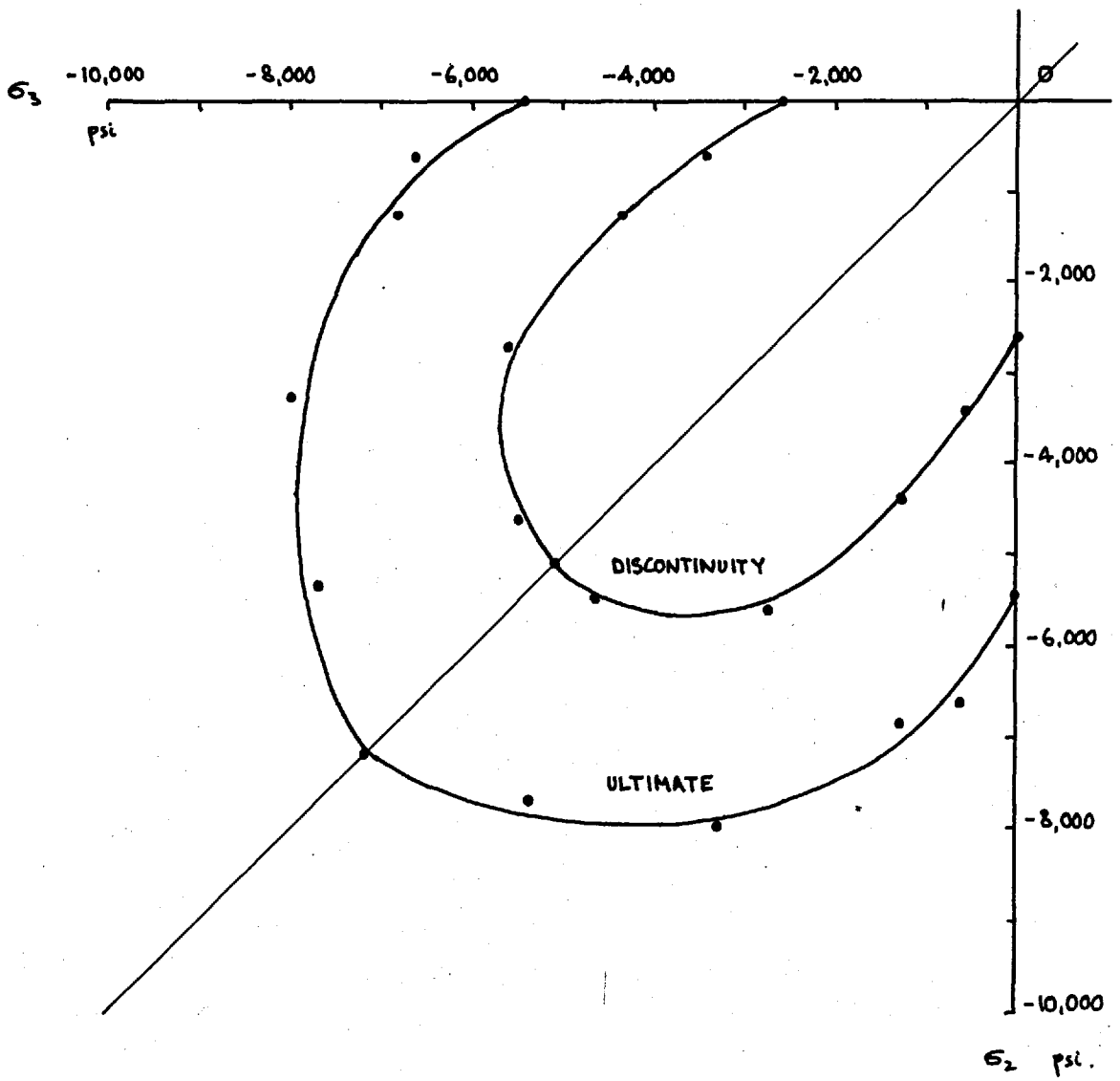


FIG. 8.5.2.6. BIAXIAL COMPRESSION STRESS ENVELOPES FOR MIX 61A.

MIX No. 61B $^{A}/_{C}$ 4·5 $^{S}/_{C}$ 2·4 $^{W}/_{C}$ 0·55 $\frac{3}{4}$" IRREGULAR COARSE AGGREGATE

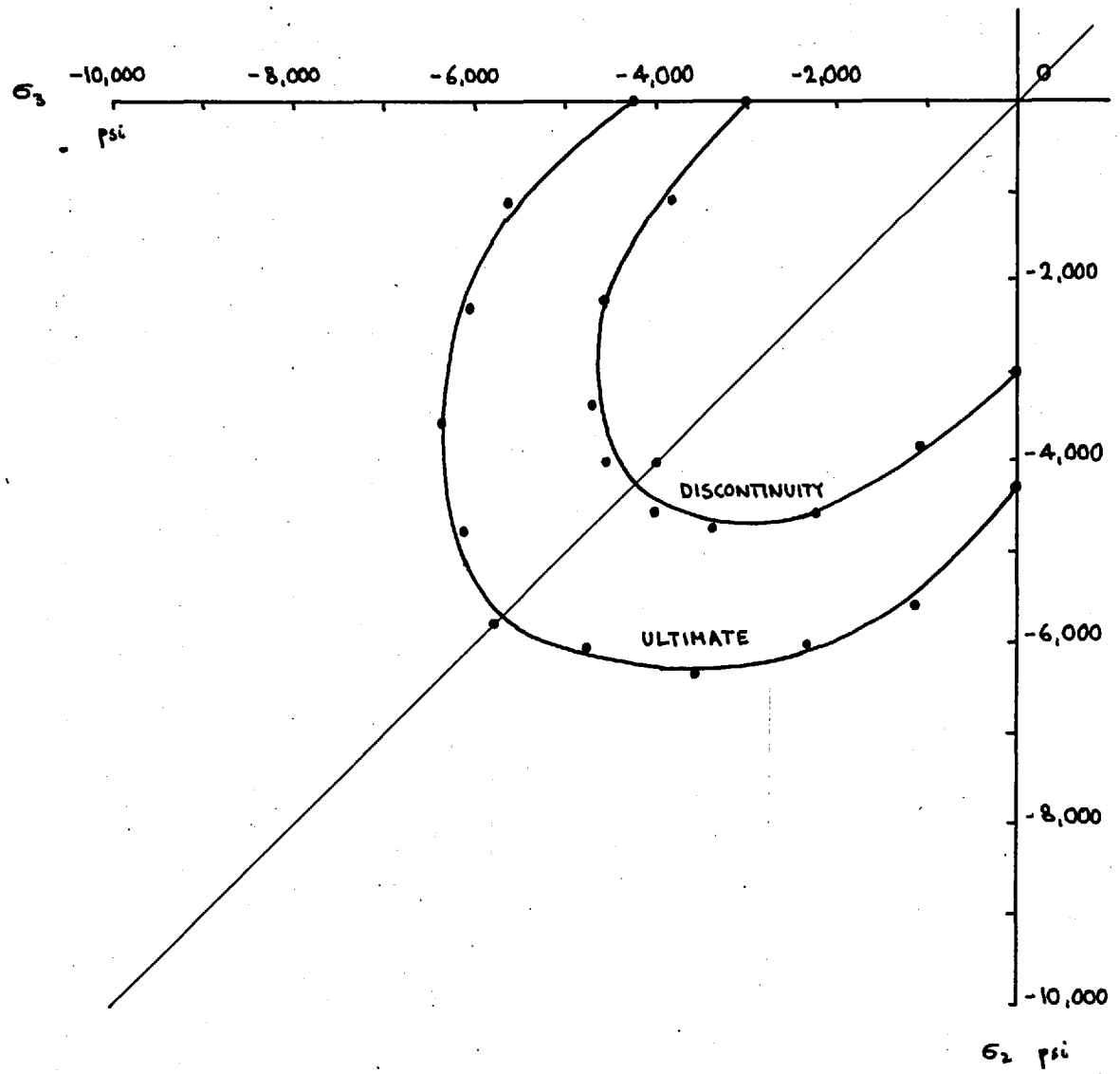


FIG. 8.5.2.7. BIAXIAL COMPRESSION STRESS ENVELOPES FOR MIX 61B.

MIX No. 61C $^{A}/_{C}$ 1.5 $^{S}/_{C}$ 2.4 $^{W}/_{C}$ 0.55 $^{3}/_{4}"$ IRREGULAR COARSE AGGREGATE

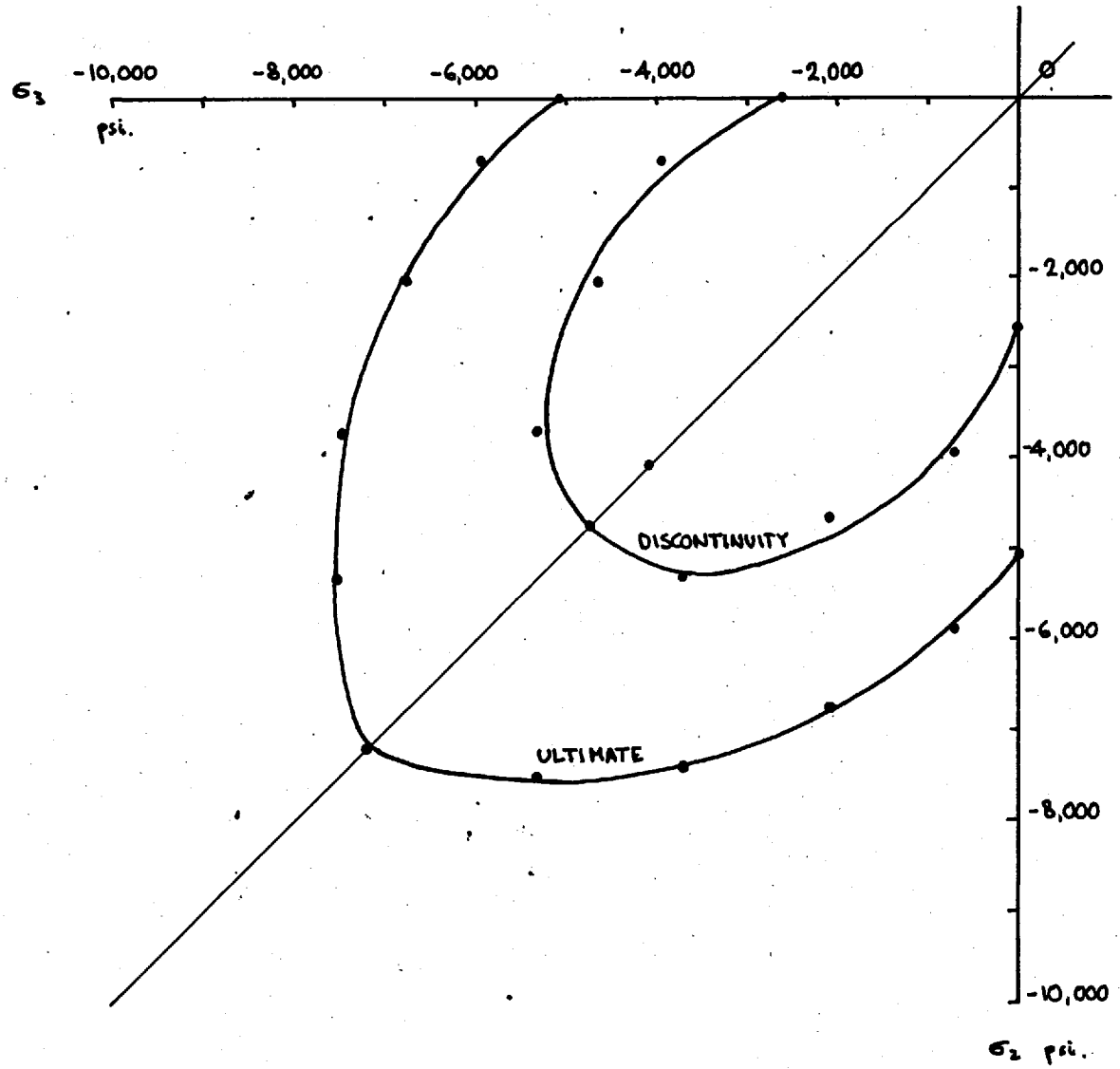


FIG. 8.5.2.8. BIAXIAL COMPRESSION STRESS ENVELOPES FOR MIX 61C.

MIX No. 61D $^{A}/_{C}$ 7·5 $^{S}/_{C}$ 2·4 $^{W}/_{C}$ 0·65 $^{3}/_{4}$" IRREGULAR COARSE AGGREGATE

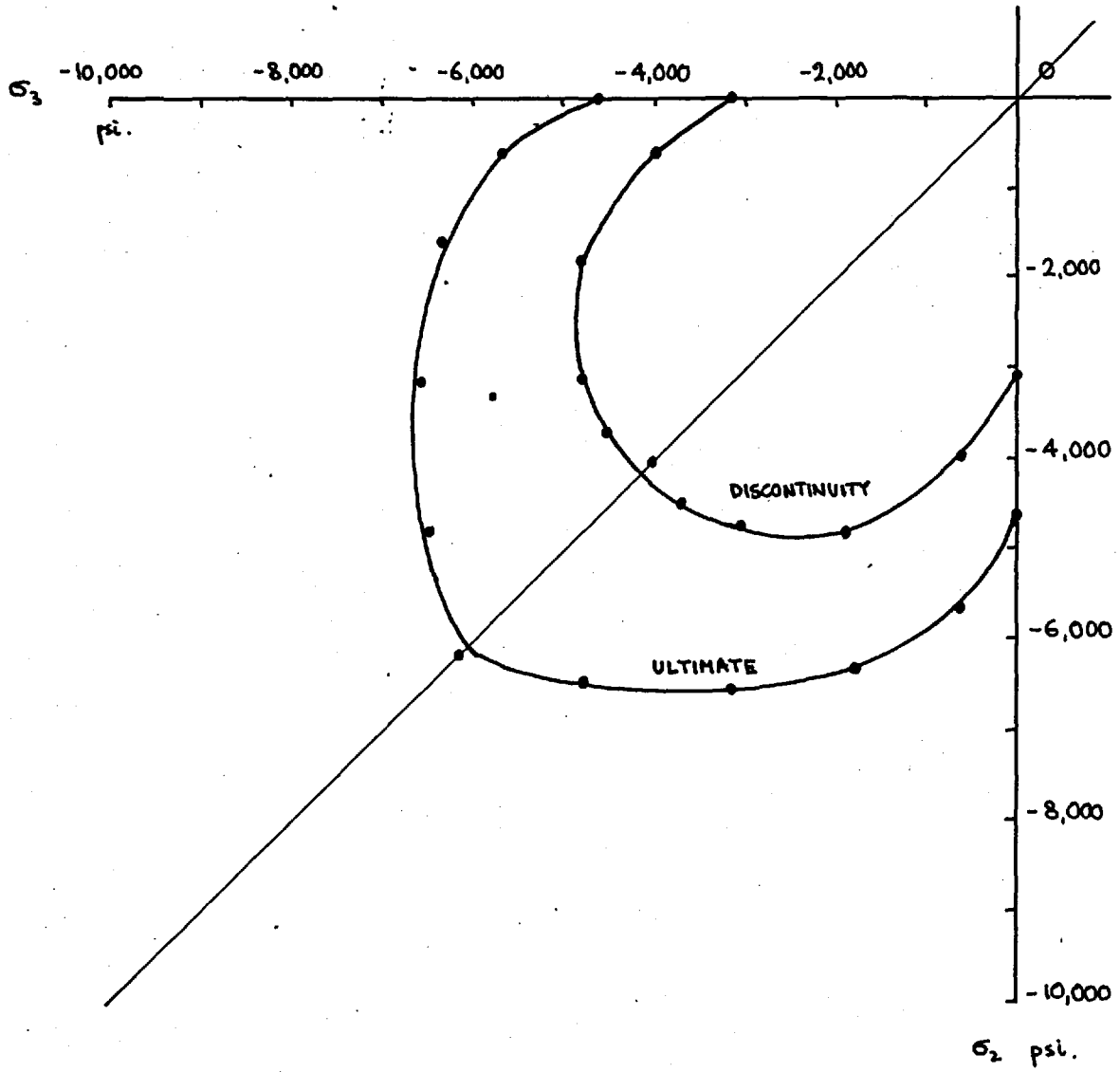


FIG. 8.5.2.9. BIAXIAL COMPRESSION STRESS ENVELOPES FOR MIX 61D.

MIX No. 6RB $^{A}/_{C}$ 4.5 $^{S}/_{C}$ 2.4 $^{W}/_{C}$ 0.55 $^{3}/_{4}$" ROUNDED COARSE AGGREGATE

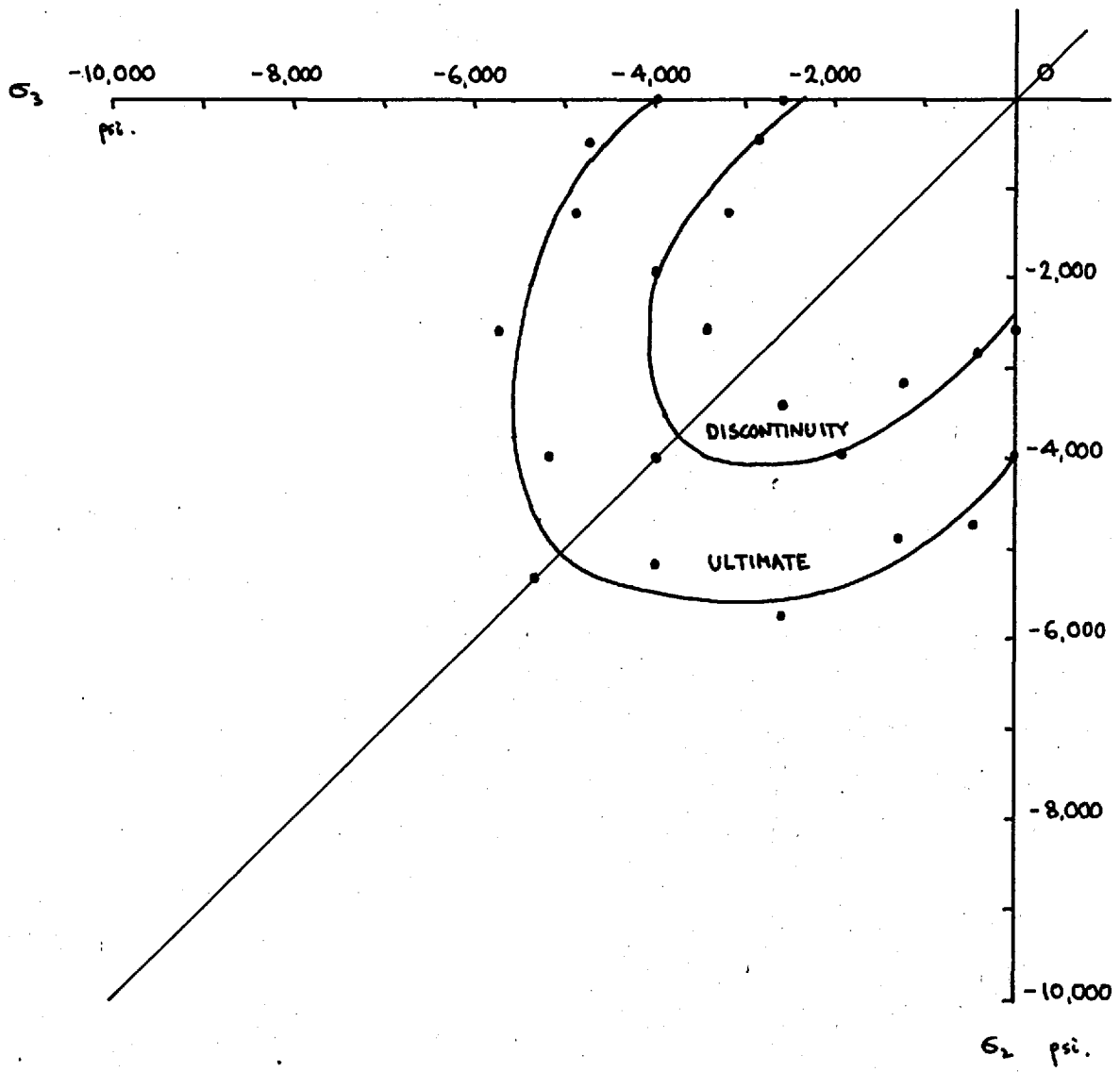


FIG. 8.5.2.10. BIAXIAL COMPRESSION STRESS ENVELOPES FOR MIX 6RB.

MIX No. 6RC $^{A}/_{C}$ 7·5 $^{S}/_{C}$ 2·4 $^{W}/_{C}$ 0·55 $\frac{3}{4}$" ROUNDED COARSE AGGREGATE

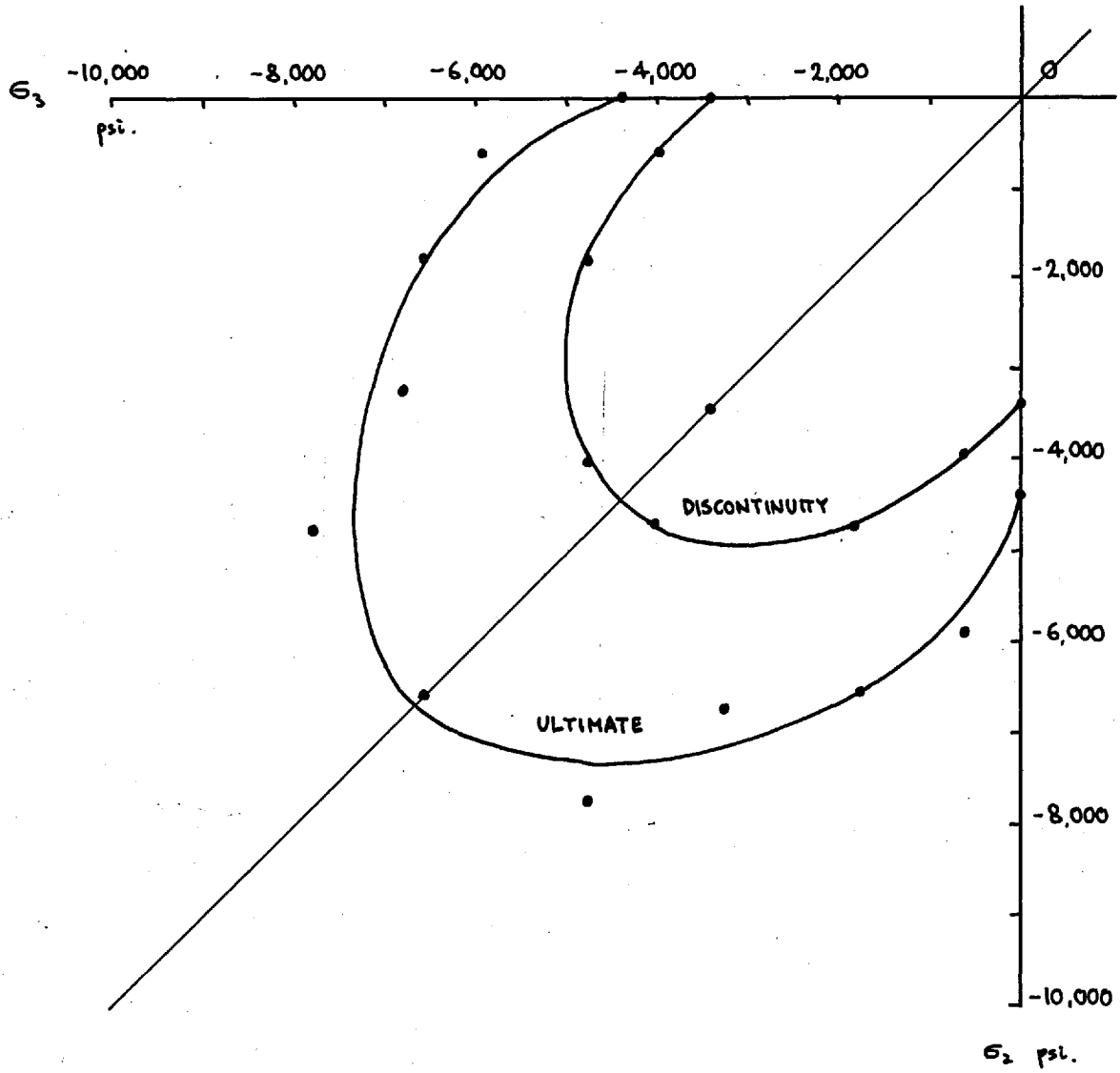


FIG. 8.5.2.11. BIAXIAL COMPRESSION STRESS ENVELOPES FOR MIX 6RC.

MIX No. 6AB $^{A}/_{C}$ 4.5 $^{S}/_{C}$ 2.4 $^{W}/_{C}$ 0.55 $^{3}/_{4}$" ANGULAR COARSE AGGREGATE

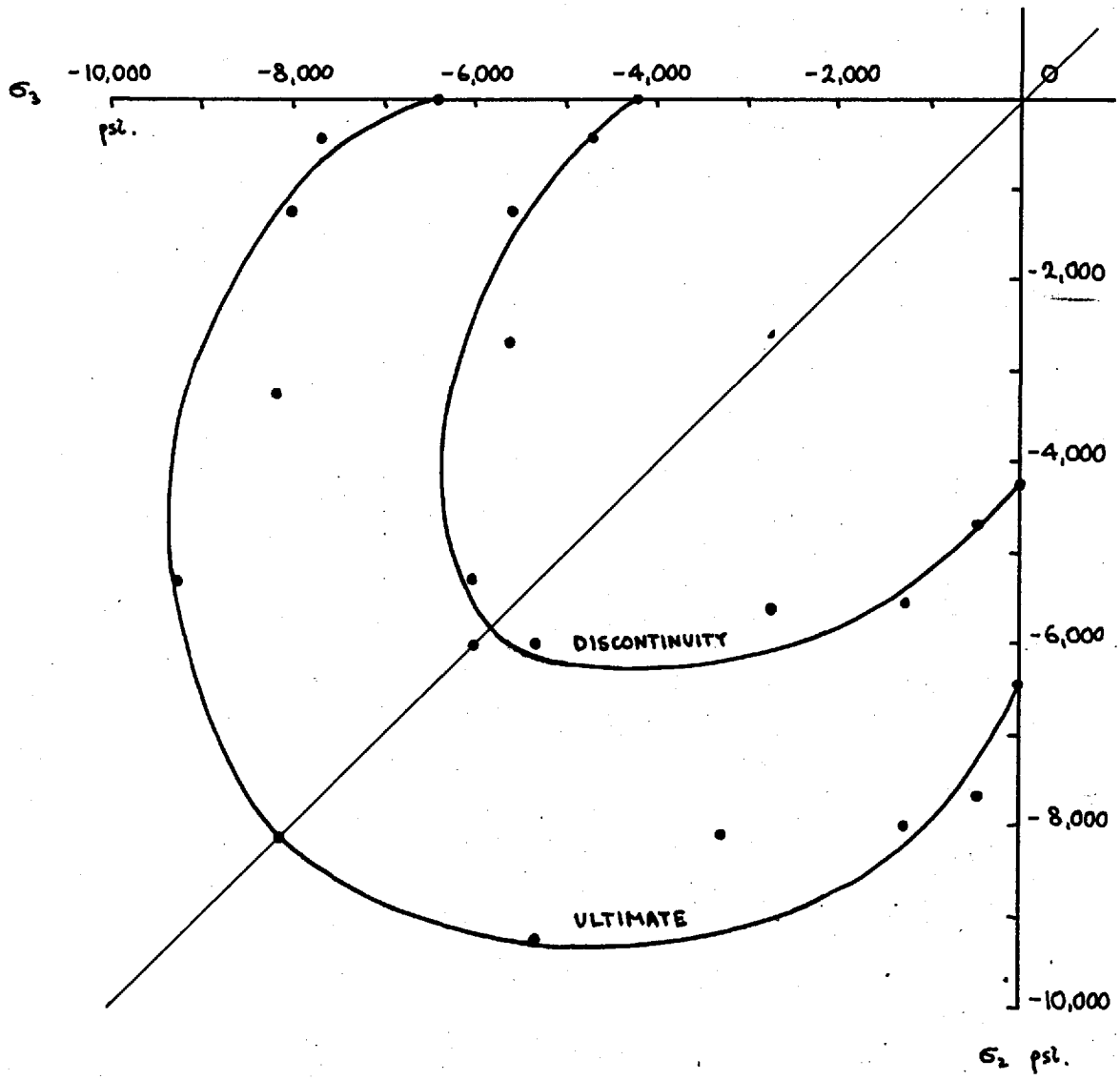


FIG. 8.5.2.12. BIAXIAL COMPRESSION STRESS ENVELOPES FOR MIX 6AB.

MIX No. 6LB $^{A}/_{C}$ 4.5 $^{S}/_{C}$ 2.4 $^{W}/_{C}$ 0.55 $^{3}/_{4}"$ LIGHTWEIGHT COARSE AGGREGATE

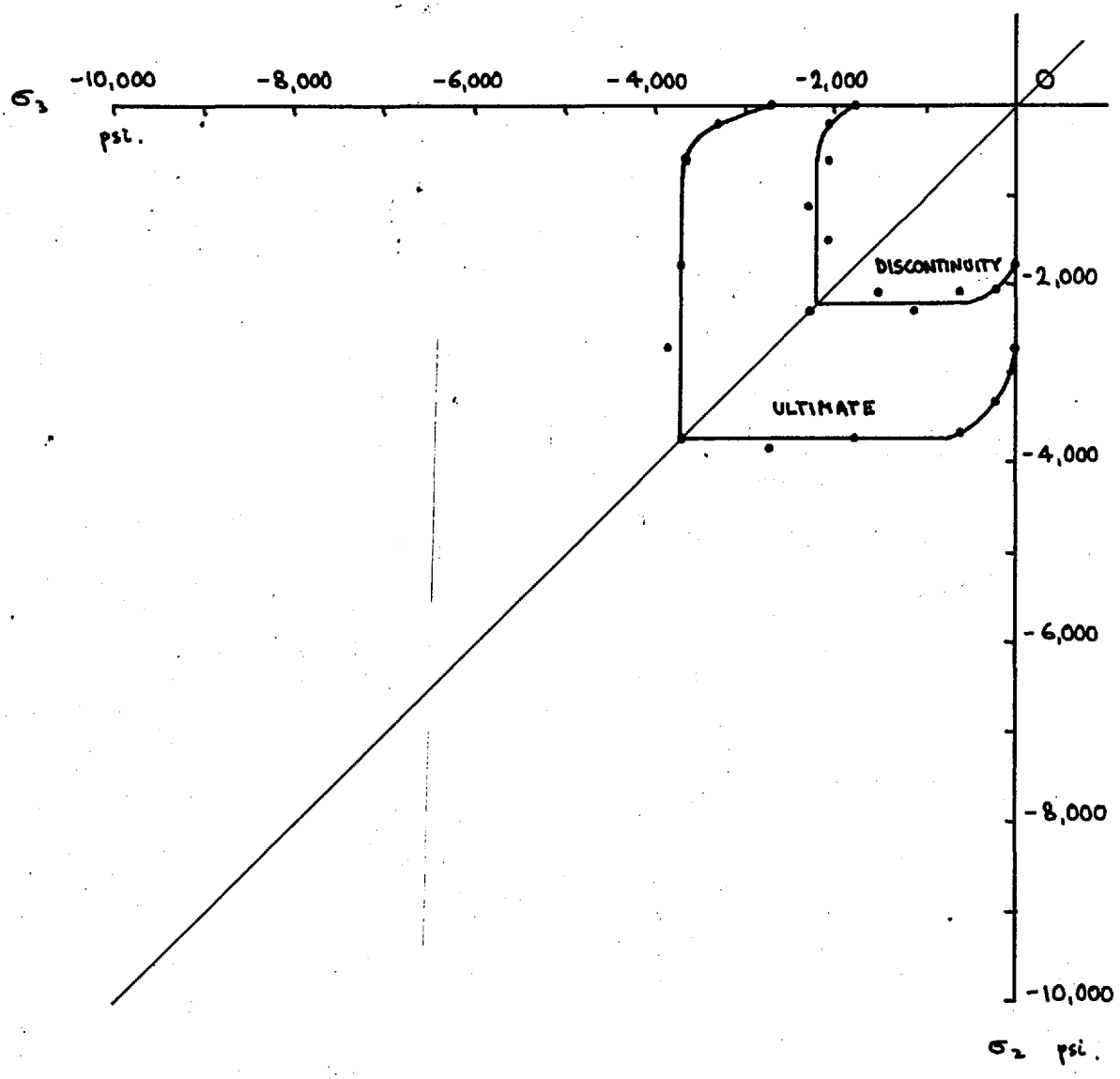


FIG. 8.5.2.13. BIAXIAL COMPRESSION STRESS ENVELOPES FOR MIX 6LB.

Lastly the similarity of the curves obtained is shown in the unit diagrams of figs.8.5.2.14 and 8.5.2.15. in which the ultimate and discontinuity stresses are plotted for the two varieties of envelope on unit diagrams with the ultimate uniaxial compressive strength as unity. The similarity of the mortar and lightweight concrete envelopes is remarkable and even the wide range of normal aggregate concretes used follow a fairly narrow band.

The discontinuity stress results both in the unit diagrams and in the individual mix diagrams described above showed more variation or scatter about their envelope curve than the ultimate stress results. This is the opposite of what one would expect as the ultimate stresses are time dependent whereas it is thought that the discontinuity stresses are not. The explanation of this result is probably due to the care with which the ultimate stress is determined - it being simply the maximum load recorded - whereas the discontinuity stress has to be assessed from a curve as shown in Appendix D.

Unexpected scatter was obtained with the rounded aggregate concretes (6RB and 6RC) on both ultimate and discontinuity stress envelopes which was not characteristic of the remainder of the experimental work. This may be a feature of the aggregate texture and could be explained by the reasonings put forward in Chapter 12, or it may be that the aggregate was very variable or was not washed and sieved properly. It is difficult to put the results down to any one reason but the resulting envelopes are a guide to the behaviour of these two mixes.

Good correlation was achieved, however, between the stresses and strains obtained from the uniaxial compression tests on prisms and the uniaxial compression test on one slab from each mix. From table 8.5.1. it can be seen that the average prism strength and uniaxial slab strength never differed by more than 12%.

σ_u : UNIAXIAL COMPRESION PRISM ULTIMATE STRESS

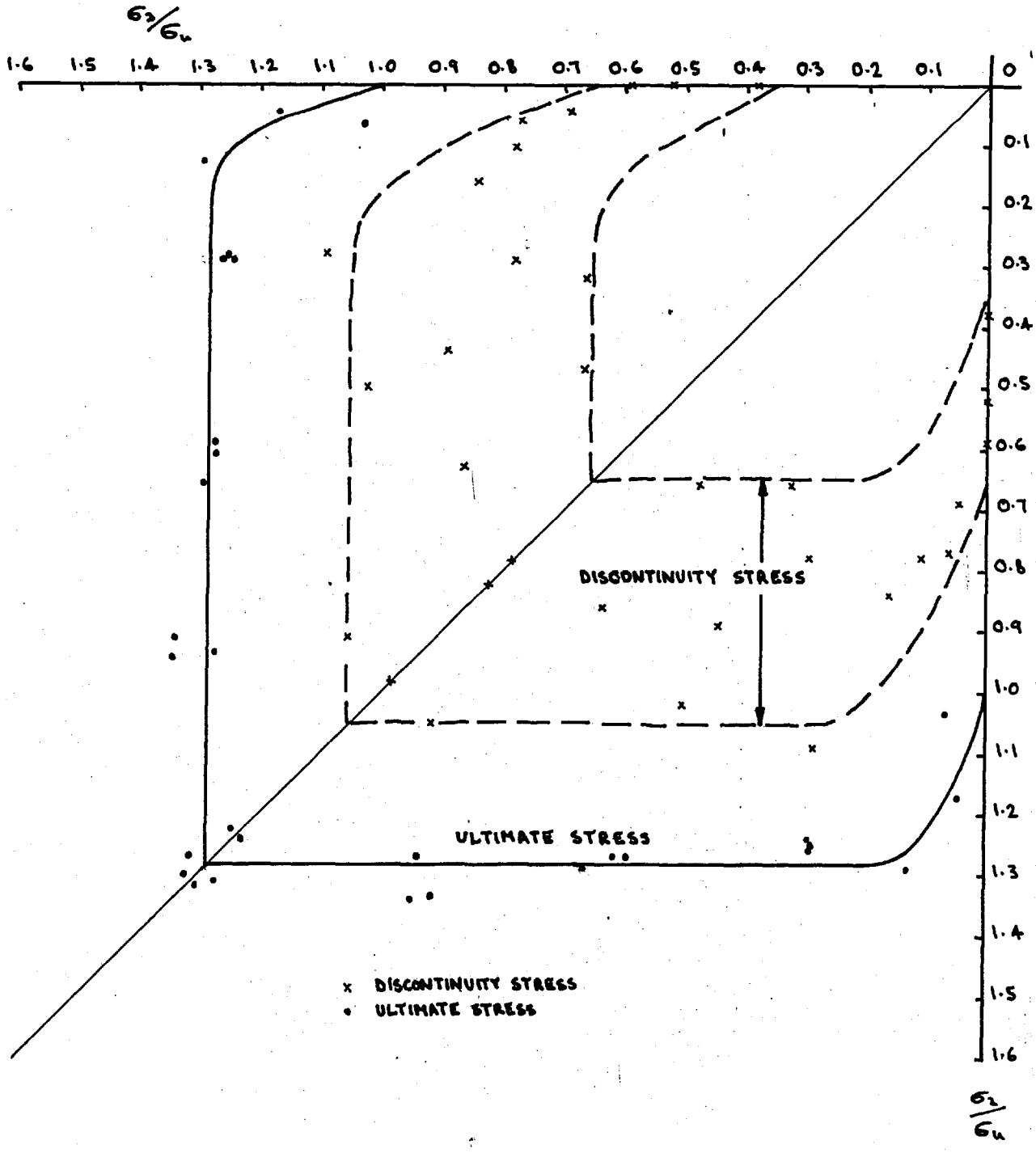


FIG. 8.5.2.14 UNIT DIAGRAM FOR MORTAR STRESS ENVELOPES.

σ_u = ULTIMATE UNIAXIAL COMPRESSION PRISM STRESS

• ULTIMATE STRESS

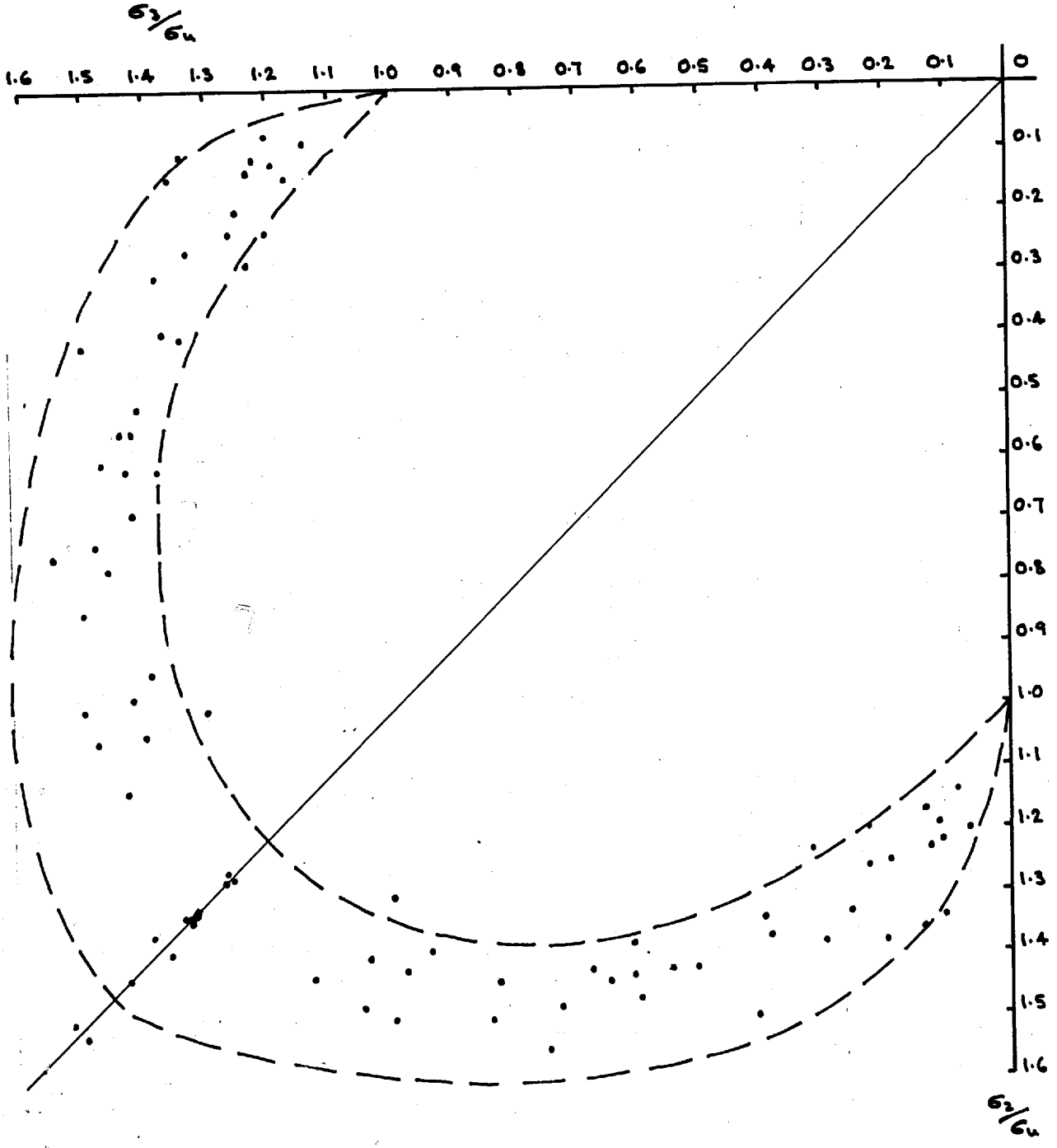


FIG. 8.5.2.15 UNIT DIAGRAM FOR NORMAL AGGREGATE STRESS CONCRETE ENVELOPES.

One other surprising feature of the results which is bfought out in the unit stress diagram for concrete shown in fig.8.5.2.15 is that aggregate volume fraction, aggregate sizes of both $\frac{3}{8}$" and $\frac{3}{4}$", aggregate texture, aggregate stiffness, and water-cement ratio appear to have little effect on the shape of the stress envelopes. As was pointed out in Chapter 7, all these factors have an influence on the uniaxial strength of concrete but it seems that they play no additional significant role in the biaxial strength of concrete. Indeed the main parameter for biaxial strength behaviour appears to be a maximum aggregate size limit of $^{3}/_{16}$".

An alternative presentation of the stress results is given in figs.8.5.2.16. to 8.5.2.19. in which the Mohr's circle construction has been used and the envelopes drawn for stresses at discontinuity and ultimate. The straight line envelopes exhibited by the mortars (in contrast with the curved envelopes of the concretes) suggest some form of shear or internal friction failure theory.

Note that all the results in this chapter have been combined with the results of the biaxial compress-on-tension tests in Chapter 11 for comments on the overall biaxial behaviour of concretes and mortars.

8.5.3. Discontinuity Strain

One of the main suggestions arising from the uniaxial test series coupled with the results of previous work was that the cleavage strain at discontinuity might prove a more realistic basis for a criterion of failure for concrete than the more usual approach via the stresses. It was demonstrated that within reasonable bounds, the cleavage strain at discontinuity was dependent upon the volume fraction of coarse aggregate and the state of stress but effectively independent of aggregate texture, size, stiffness or the water-cement

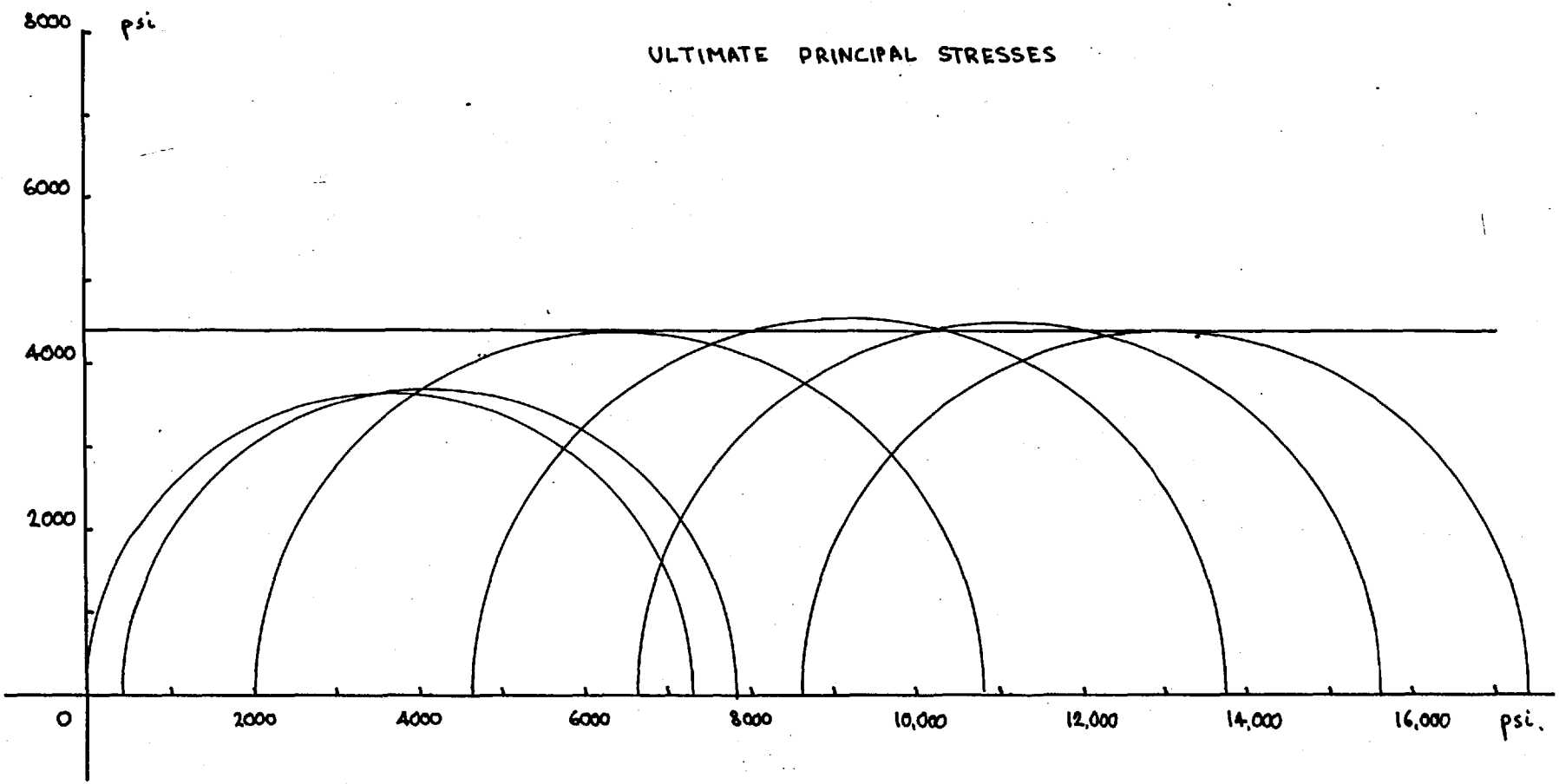


FIG. 8.5.2.16. MOHR'S CIRCLE PRESENTATION OF STRENGTH OF MORTAR MA.

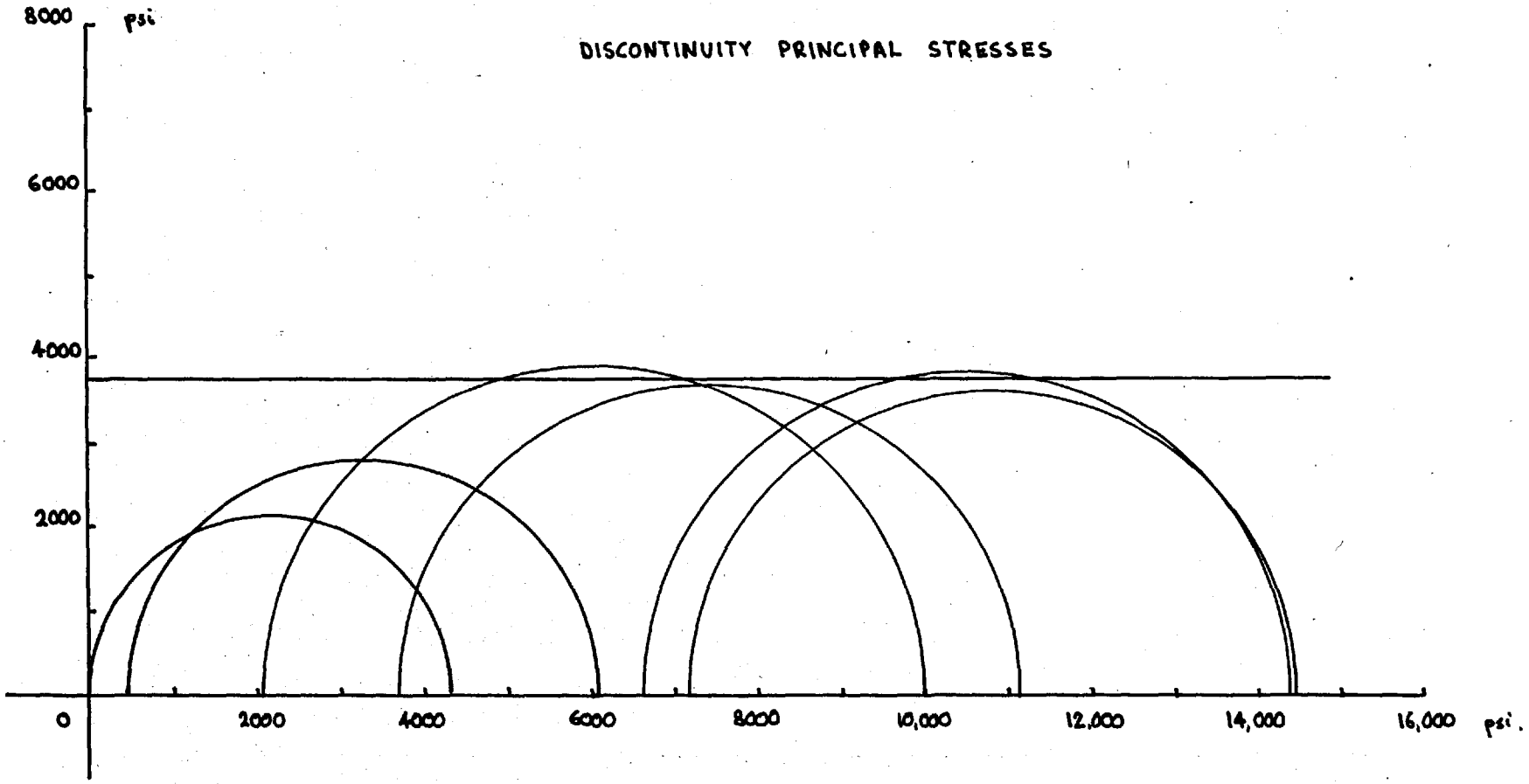


FIG. 8.5.2.17. MOHR'S CIRCLE PRESENTATION OF DISCONTINUITY STRENGTH OF MORTAR MA.

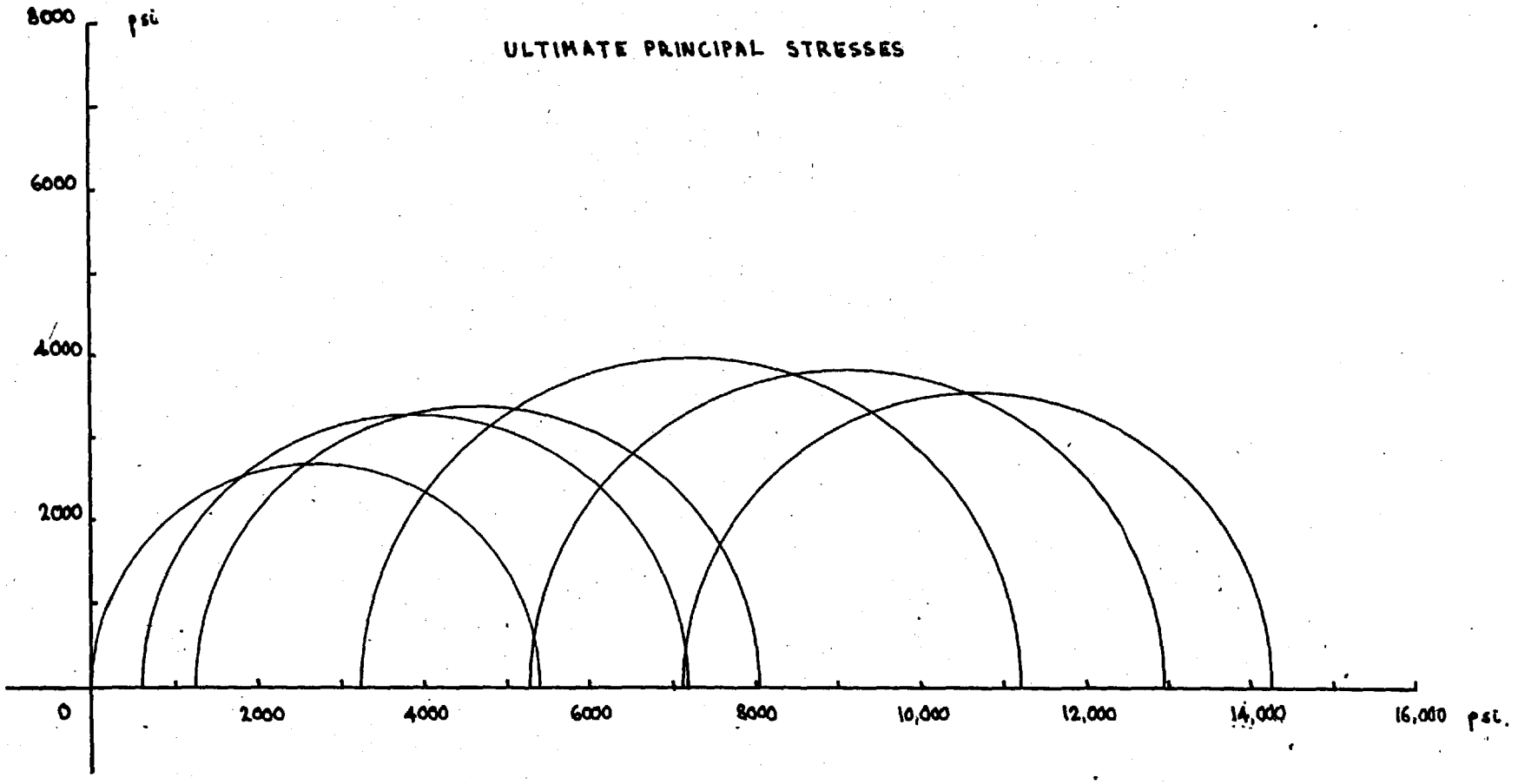


FIG. 8.5.2.18. MOHR'S CIRCLE PRESENTATION OF ULTIMATE STRENGTH OF CONCRETE 61A.

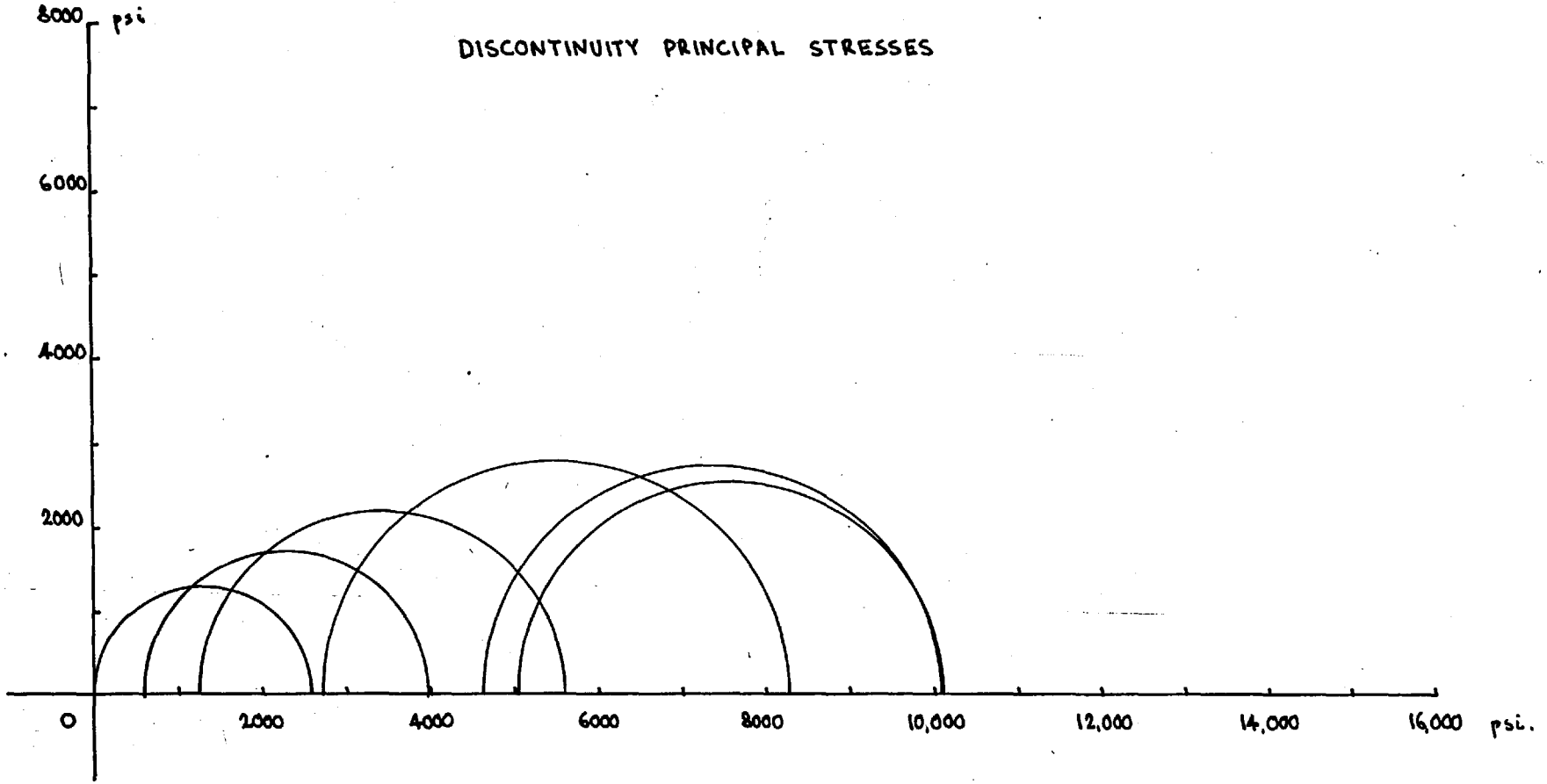


FIG. 8.5.2.19. MOHR'S CIRCLE PRESENTATION OF DISCONTINUITY STRENGTH OF CONCRETE 61A.

ratio. The sand-cement ratio was constant in these tests and so its effect on the cleavage strain cannot be calculated.

From the stress-strain characteristics it appears that both concretes and mortars are capable of undergoing considerably more distortion prior to failure under biaxial compression than under uniaxial compression and this is especially true of the strain in the cleavage direction. In fig.8.5.3.1. the cleavage strain at discontinuity has been plotted for the uniaxial and biaxial compression tests against the mean normal stress. The trend is clear with the strain capacity in the unstressed direction increasing with the mean normal stress; the softer mortars exhibiting larger strains than the stiffer concretes. For the sake of clarity only four typical mixes have been shown though an inspection of the results given in tables 8.5.1. to 8.5.3. show that the trend is general.

8.5.4. Volumetric Strain, Shear and Bulk Modulus

Much has been said about the variations of the elastic modulus and Poisson's ratio in section 8.5.1. above but it must be realised that these are uni-direction effects in a basically three-dimensional strain pattern. As complete three dimensional strain measurements were made in this research, the other elastic parameters should be considered.

The variation of volumetric strain with the major principal compressive stress is shown in fig.8.5.4.1. for a uniaxial and equal biaxial compression test on a typical concrete (61D). The discontinuity points, as found from ultrasonic pulse velocity measurements, have been marked for comparison. The two curves have no distinguishing features being similar in shape but with a greater volumetric strain under uniaxial compression.

The two elastic parameters most common in three dimensional elasticity are the shear rigidity modulus G and the bulk modulus K.

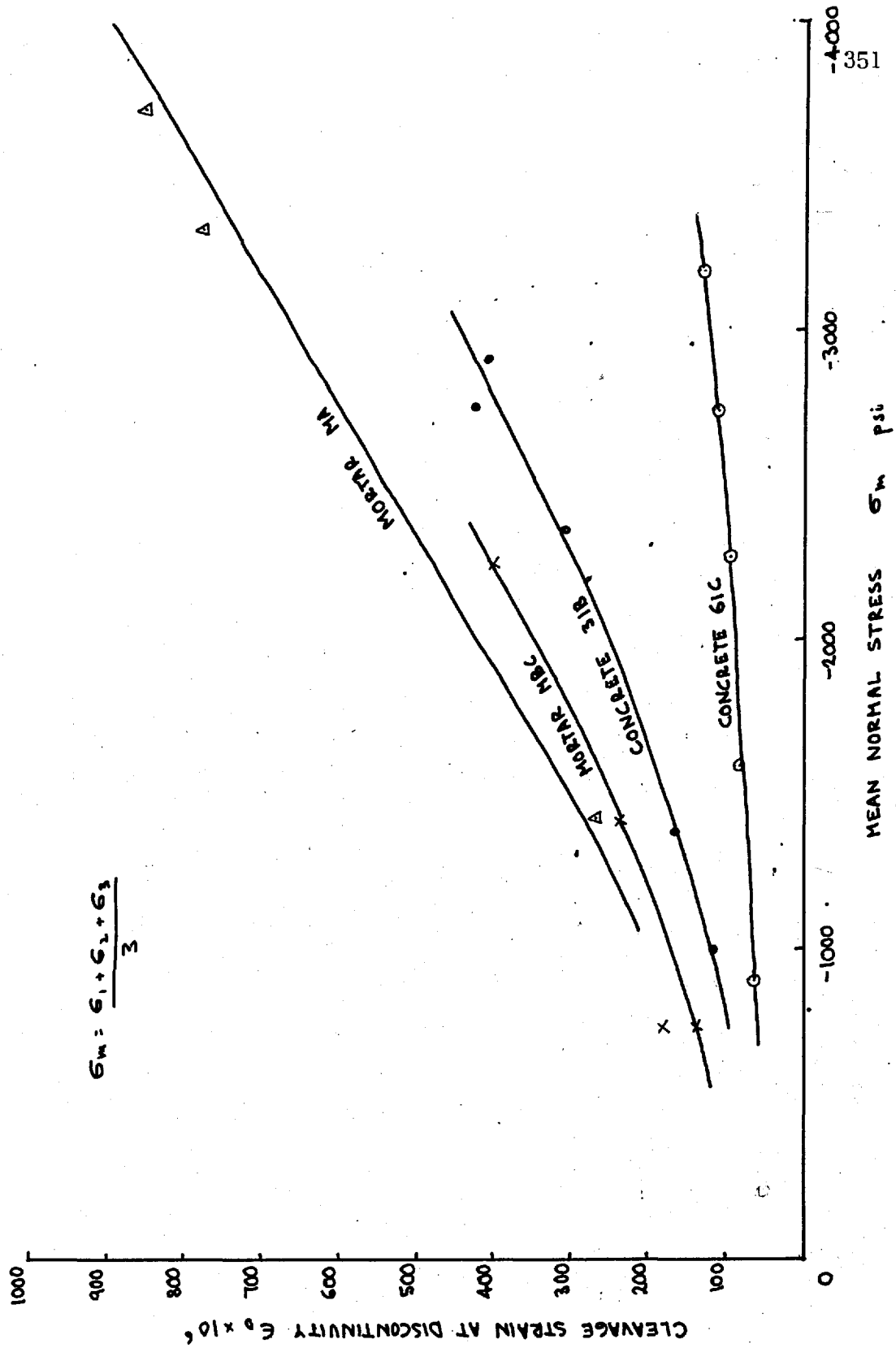


FIG. 8.5.3.1. VARIATION OF CLEAVAGE STRAIN AT DISCONTINUITY WITH MEAN NORMAL STRESS

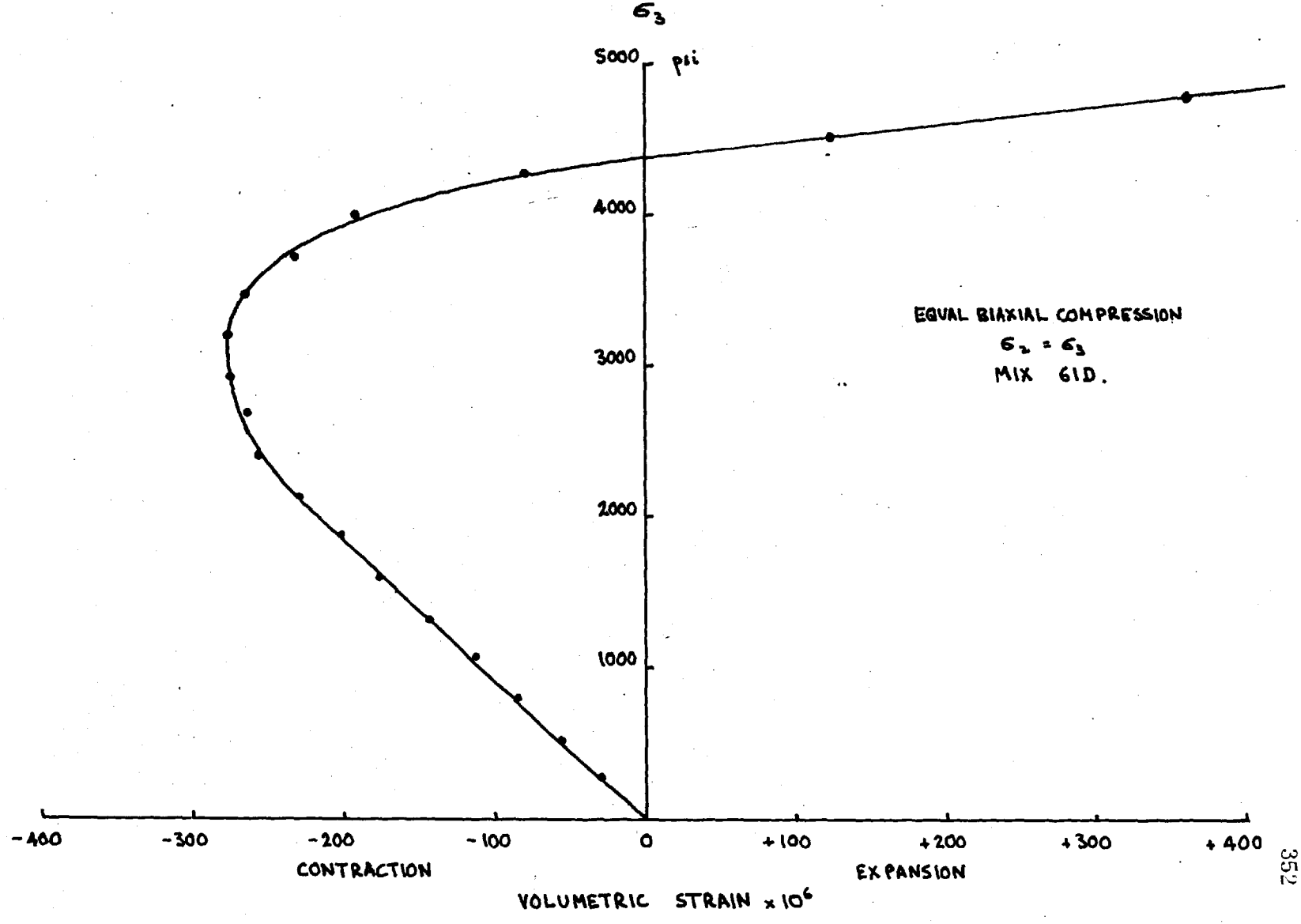


FIG. 8.5.4.1. VARIATION OF VOLUMETRIC STRAIN WITH PRINCIPAL STRESS

These two parameters are constant for an ideally elastic material and are related to the linear elastic modulus E and Poisson's ratio by the following equations:-

$$G = \frac{E}{2(1+\nu)} \qquad K = \frac{E}{3(1-2\nu)}$$

Now if Poisson's ratio is effectively constant over the elastic range considered below the discontinuity point, then:-

$$G \propto E \propto K$$

Hence the shear rigidity and bulk modulus will be constant values for all stress states in mortars and lightweight concretes but will increase with mean normal stress in normal aggregate concretes. Therefore, for normal aggregate concretes, all the elastic constants vary with the mean normal stress.

Note that the small variations in the values of Poisson's ratio reported in section 8.5.1. will not affect this result significantly.

This result is discussed further in Chapter 11.

8.5.5. Failure Modes

A general cleavage mode of failure was observed in all uniaxial and biaxial compressive tests with more numerous and well aligned cracks in concretes than in mortars. The cleavage planes of the mortars tended to be slightly oblique to the general direction of cleavage which was always in the third unstressed direction.

All photographs and observations of crack patterns have been retained for discussion in Chapter 11 where the failure modes under all biaxial states of stress can be discussed together.

8.6. Conclusions

8.6.1. State of Stress in Biaxial Compression Specimen

Arguments were put forward in section 4.4.6. for the state of uniform strain in the central 6" x 6" x 4" volume of the specimen and the conclusion that the stress acting on this volume was the same as the average applied stress obtained by dividing the total load applied by the cross-sectional area of the specimen. These arguments were based on the results of the extensive proving trials carried out on the state of stress induced in the biaxial compression specimen and it was concluded that this data alone was sufficient to confirm tha above arguments. It is of interest to refer back to this original discussion, and its rejection of Hilsdorf's contention (57) that the stress on the central zone is not the average applied stress, in the light of the experimental results obtained in this test series.

All the results quoted so far have assumed that the analysis of section 4.4.6. was valid and that the stress acting on the central uniformly strained volume of the biaxial compression specimen is the average applied stress. If this assumption is wrong and the actual stress in the central volume is less than the applied stress, then mortar cannot obey the elastic theory within the limits laid down in section 8.5.1. This conclusion must follow because the elastic modulus of mortar would have to reduce under increasing degree of biaxial compression. No known brittle material exhibits this phenomenon and therefore it must be regarded as a very remote possibility; the mechanism for such behaviour would contradict the known characteristics of mortar such as its brittle failure mode.

Furthermore the biaxial stress envelope obtained would hardly exhibit such a regular shape if the stresses calculated were in error. Its co-incidence with certain classical failure theories for the failure of homogeneous materials is further evidence for its accuracy. As discussed later in Chapter 11, the initial curvature of the envelope is probably due to a statistical effect concerned with the alignment of critical flows and both this curvature and the subsequent square slope of the biaxial compression stress envelope are also obtained with cast iron (Chapter 12) which is a remarkably similar material.

In conclusion it is contended that these results support the claim of section 4.4.6. that the stress on the uniformly strained central volume of the biaxial compression specimen is equal to the average applied stress.

8.6.2. Behaviour related to Elastic Theory

It was noted that the stress-strain curve for both mortar and concrete was slightly non-linear below the discontinuity point. Given this deviation, however, it was found that mortars and lightweight aggregate concretes obeyed the equations of the elastic theory whilst normal aggregate concretes exhibited an increase of elastic modulus with mean normal stress. Hence, whilst Poisson's ratio remained effectively constant for all mixes under biaxial compression, the elastic modulus remained constant for mortars and lightweight concretes whereas it increased with degree of biaxial compression for normal aggregate concretes.