THE TESTING OF CONCRETE MATERIALS BY PRECISELY CONTROLLED UNI-AXIAL TENSION

by

M.A. Ward, B.Sc.(C.E.), D.I.C.

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Imperial College of Science and Technology, London.

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ABSTRACT

This thesis describes the design and construction of a uni-axial tension testing machine for concrete materials. The design criteria is given for such a machine, as established from the consideration of past attempts to test concrete in uni-axial tension. Tests are described which established the ability of the machine to induce a controlled uniform state of stress in a specimen, free from stress concentrations and eccentricity effects.

Four test series were completed with the uni-axial tension machine:

(1) Using a standard mortar mix, it was established that the variance of the new test was similar to that obtained from two other tension tests, and two compression tests. Furthermore, it was shown that the residual ends of a failed tension specimen could be used for further destructive tests.

(2) A destructive testing program was carried out to establish the relationship of the ultimate failing strength obtained from the new test with the ultimate strengths obtained from 5 other destructive tests, for a wide range of mix parameters and 4 different types of coarse aggregate. In general, it was shown that the granite coarse aggregate concretes gave the highest compression strengths, and the mortars of the concretes, the highest recorded tensile strengths.

(3) Strain measurements were taken up to the ultimate load on uni-axial tension and compression specimens, at 7 and 28 days, on Thames Valley river gravel concretes and their constituent mortars and cement pastes, for a variety of mix parameters. It was shown that in general, a limiting tensile or cohesive strain limit exists for concretes and mortars loaded in uni-axial tension. Correlation was obtained between ultimate longitudinal tensile strains and lateral compressive 'tensile' strains at 60 - 70 per cent of the ultimate compressive strength. Relationships were obtained between the compression and tension moduli and the dynamic modulus of elasticity.

(4) Pilot tests were carried out to determine the effect of moisture loss of concrete materials on their apparent uni-axial tension strengths. It was shown that in general, the tensile strength was increased if the specimens were allowed to dry for periods up to 24 hours before testing. Longer periods of drying resulted in a decrease in apparent tensile strength.

Finally, a rational explanation is given to differences in ultimate failing strength of concretes with the same volumetric mix proportions but containing different aggregate types, and other related topics. In general, the theories developed correlate the observed failing strengths to the shape and surface texture of the coarse aggregate.

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CHAPTER 1

INTRODUCTION AND SCOPE

1.1 INTRODUCTION

The work to be described in this thesis forms part of a general programme of research instigated by K. Newman at Imperial College with the ultimate aim of understanding more fully the nature of concrete, its behaviour under various states of stress, and the mechanism of fracture and failure.

An important part of such a programme is the establishment of reliable and accurate testing techniques[38]. It is often not realised that the results obtained from many standard tests are not necessarily basic physical properties of the material under test. A test method, ideally, should produce the required stress or strain conditions in the test specimen independently of any effects of specimen shape and size, design of machine, etc. The state of stress should be uniform, and exist not only in the elastic range but also up to failure of the specimen. The state of stress should exist completely throughout the specimen. If this is not possible, the specimen should be loaded in such a manner that the desired stress state exists in the critical section under consideration.

The first stage in this programme was concerned with an investigation into the so-called simple compression or crushing test. The result of the work of Lachance[78] has been the standardization of a specimen shape and testing technique which enables a state of uni-axial compression to be induced in a test specimen. As the next stage in the research programme, the Writer was set the task of producing a satisfactory method of testing concrete materials in uni-axial tension ($\sigma_y = \sigma_x = 0$; $\sigma_z \neq 0$).

The properties of concrete in uni-axial tension are not only of considerable importance in the development of theories concerning its mode of deformation and failure, but are of great interest to the practicing engineer. Current design practice usually ignores the tensile strength of concrete and so a knowledge of the safe tensile working load it can withstand could lead to a much more efficient use to be made of concrete.

In the past, a considerable number of investigators have attempted to test mortars and concretes in uni-axial tension. That their successes have been severely limited is shown by the fact that no acceptable standard test yet exists for uni-axial tension. It is suggested that one reason for this is that few investigators have ascertained whether their testing technique induced the required state of uni-axial tensile stress. The uni-axial tension testing of brittle materials which have a much smaller strength in tension than compression is extremely difficult and the test method is critical. The tensile testing of ductile materials is much simpler, since stress concentrations and non-uniform loading are of less importance due to the ready ability of these materials to redistribute stress.

Because of the problems associated with the testing of concrete materials in uni-axial tension, the introduction of the cylinder splitting test to establish the 'tensile strength' of concrete materials has been greeted with much enthusiasm. This method has the obvious attraction of obtaining a tensile strength value from a compression test on a simple specimen form. However, it is considered that the adoption by so many authorities of the splitting tensile strength as representative of the uni-axial tension strength of concrete materials without thoroughly investigating the method of test, can be misleading and dangerous. This state of affairs emphasized the importance of that part of the general research programme of the Concrete Materials Research Group of Imperial College concerned with the development of testing methods which produced the desired state of stress or strain.

1.2 SCOPE

The first object of the present work was to develop a suitable uni-axial tension test. Design criteria for the testing machine were able to be formulated after a critical review of previous tension testing methods. The tension test had to satisfy these established design criteria to produce a testing method wherein a state of uni-axial tensile stress could be developed in a test specimen. Furthermore, the testing technique had to be suitable for use both as a research and as a standard machine. On this basis a machine was designed and constructed. Using this testing machine, an investigation was undertaken to study some of the properties of concrete materials in uni-axial tension.

This present work can be divided into three parts: (1) the development of the uni-axial tension testing method and associated calibration tests, (2) an experimental programme to study the relative failing strengths obtained from the uni-axial tension test and other destructive and non-destructive tests, and (3) an experimental programme to examine the deformational behaviour of concrete materials tested to failure in uni-axial tension and compression.

<u>(1) Development of the uni-axial tension testing method</u>:

A critical review of the work of past researchers enabled a list to be formulated of general design criteria for an ideal uni-axial tension testing machine. A testing machine was designed, developed, and constructed to satisfy these design criteria. Various tests were then carried out to determine if the new tension testing method satisfied the design criteria.

<u>(2) Relationship of uni-axial tension strengths with those obtained from other testing methods:</u>

A testing programme was carried out to establish the relationship between failing stresses obtained in uni-axial tension and other methods of test. In this programme, a variety of mix parameters and aggregate types were employed in an attempt to establish quantitative data on the relative behaviour of concrete in uni-axial tension.

<u>(3) The deformation of concrete materials</u>:

A study was made of the stress-strain behaviour of concrete materials loaded to failure in uni-axial tension and compression. External strain measurements were taken while the test specimens were loaded to failure in an attempt to provide information on the phenomena associated with the relative deformational and failure behaviour of concretes under the two uni-axial states of stress.

CHAPTER 2

REVIEW OF PAST RESEARCH

2.1 INTRODUCTION

The first objective in this work was the selection from previous investigations of the most suitable tensile testing method, compatible with the requirements of the proposed investigation. It was decided that if an adequate system could not be found, the only alternative was to develop a suitable method. Either of the above approaches demanded a comprehensive survey of the history of tensile testing. Such a survey is given in the first part of this chapter. The second part of Chapter 2, i.e. 2 - 4, is concerned with an examination of some of the properties exhibited by concrete materials under uni-axial tensile loading reported by previous investigations.

2.2 GENERAL

Before attempting to review the work of previous investigations, it is necessary to define terms which will be introduced into this work.

Uni-axial or direct tension test:

This is a test in which only a uni-axial state of tensile stress ($\sigma_1 = \sigma_2 = 0$; $\sigma_3 \neq 0$) is assumed to exist in a specimen, i.e. the specimen is loaded in tension uniformly across a plane in one direction and is free to move in the perpendicular directions.

Indirect tension or splitting test:

This is a test in which the loading system induces a biaxial state of stress in the specimen such that one of the principle stresses is tensile ($\sigma_1 = 0$, $\sigma_2 \neq 0$, $\sigma_3 \neq 0$). It is called an indirect tension test because failure in brittle material such as concrete usually occurs along the plane where the principle stress is tensile. (see Fig. 2.1(e)).

Hoop tension test:

In a hoop tension test the tensile stress is developed in the wall of a hollow cylindrical or annular ring specimen by an internal hydrostatic pressure. A tensile stress is developed everywhere in the hoop. However, there is a variable, not uniform, stress distribution across the hoop (see Fig.2.1(d)).

Stress concentration:

In a true uni-axial tension test, it is assumed that a uniform stress distribution exists across the section subjected to the tensile force. When,due to abrupt changes in cross section of the specimen or the application of concentrated loads, a highly irregular stress distribution is obtained, stress concentration will occur. (e.g. tensile briquet specimen Fig.2.1(a)). Irregularity of the distribution of the stress across the plane under load implies that at certain points, stress values exist which are far above the average value for the section.

Eccentricity:

In a perfect uni-axial tension test, eccentricity cannot exist since the definition of the test implies that the direction of application of the tensile force must pass through the centre of the specimen section. However, when a uni-axial tension test is carried out on a homogeneous material, eccentricity can be developed in three different ways: (1) assuming a perfect loading system, a poorly designed specimen can give an eccentric load due to its asymmetrical shape, (2) assuming a perfect loading system and specimen, an eccentric load can be developed due to misalignment of the specimen with the loading system during the setting up operation, (3) assuming a perfect specimen and accurate alignment of the specimen with the loading system during setting up, a poorly constructed and misaligned loading system can produce eccentric loading if the applied load does not pass through the centre of the specimen cross section.

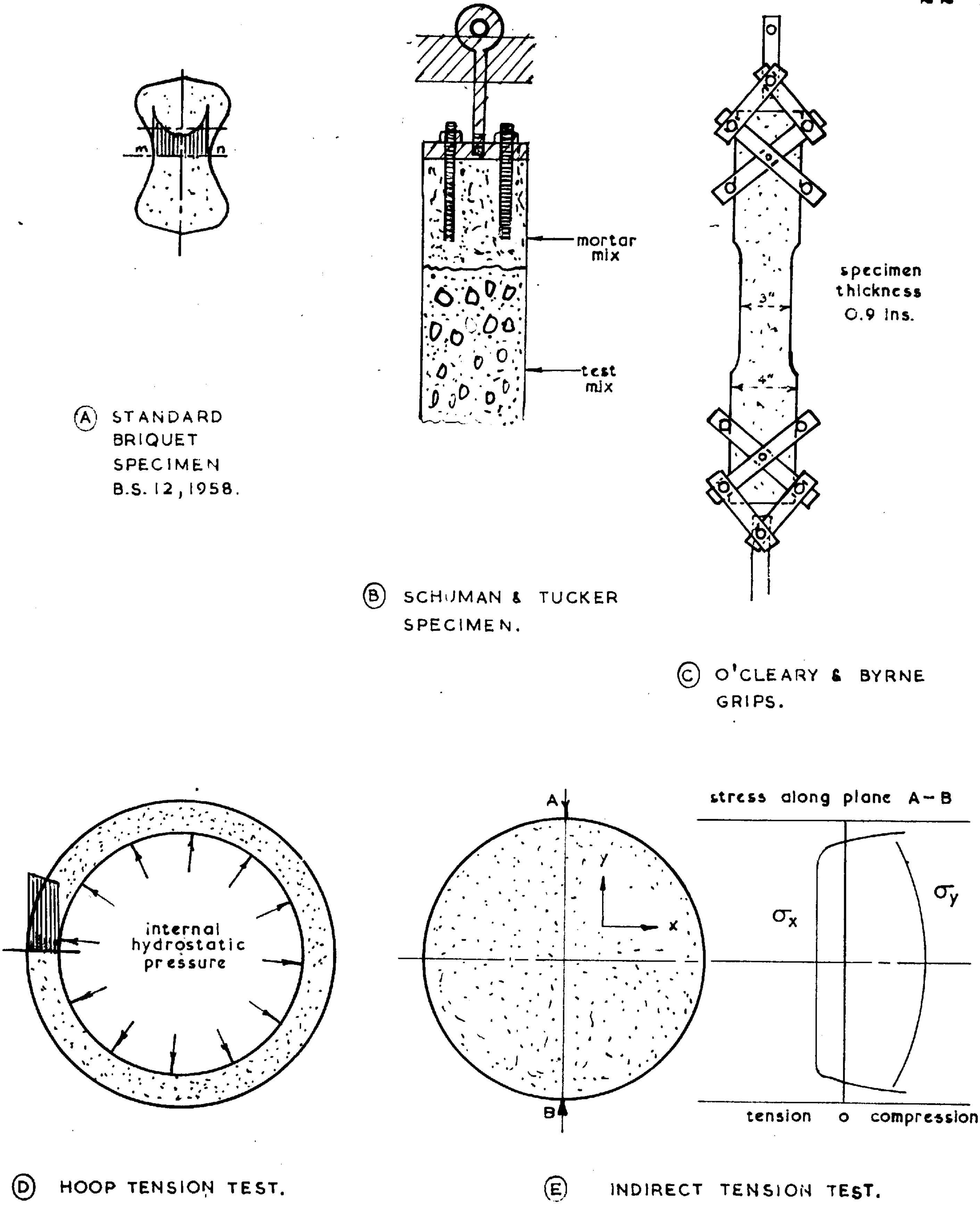

m
n
(A) STANDARD BRIQUET SPECIMEN B.S. 12, 1958.
mortar mix
test mix
(B) SCHUMAN & TUCKER SPECIMEN.
specimen thickness 0.9 ins.
3"
4"
(C) O'CLEARY & BYRNE GRIPS.
internal hydrostatic pressure
(D) HOOP TENSION TEST.
A
B
y
x
stress along plane A – B
σ_x
σ_y
tension
o
compression
(E) INDIRECT TENSION TEST.

FIG. 2.1

2.3 EARLY TENSILE TESTING METHODS

It is very difficult to establish when the first tests on the mechanical properties of cementitious materials were carries out. Gonnerman[(1)] states that there is little record of any methods employed to test hydraulic cements prior to 1750. Moreover, it appears that the main purpose of these early tests was to establish the durability of the materials under various service conditions rather than their strength or mechanical behaviour under load.

The first strength tests were carried out on beams or prisms subjected to flexure. Gonnerman names three early experimenters in this field. Bengt Quist of Sweden published results of bending tests of mortar prisms manufactured from artificial water limes, Cessart in 1787 carried out similar tests on 'concrete' prisms, and General C.L. Treussart of France carried out systematic bending tests on mortar beams during the period 1816 - 1825.

Kirby and Laurson[(2)] record that the first English experimenter of any distinction was General C.W.Pasley. In 1838, Pasley published a comprehensive book in which he described a progressive cantilever test for determining the bending strength of cementitious materials. In addition, he introduced a form of direct tension test. In order to determine the strength of a mortar joint, special grips were attached to two bricks, which had been 'cemented' or mortared together, and a tensile force applied. It would appear that this direct tension test was the forerunner of the universal cement briquet test.

Up to this time the use of the various cementitious materials was not extensive, since their strength was neither high nor dependable. However, on December 14, 1824, Joseph Aspdin obtained a patent for a new cementitious material which he called Portland Cement. The introduction of this new material, whose quality was far superior to all previous hydraulic cements, created the need for a standard testing method which could be used as an acceptability test.

The period after the discovery of Portland Cement saw the development of a multitude of different tensile briquet tests. It is sufficient to mention the work of the English engineer, John Grant[4] who in 1858 carried out the first extensive tests to investigate the potential of cement for use on a more widespread scale. After considerable experimentation with different briquet specimen shapes and gripping methods, he decided on a specimen with a reduced cross-sectional area of $2\frac{1}{4}$ sq. ins. (Fig 2.2). The shape of this specimen was similar to that used in the current British Standard 12 (1958) (see Fig 2.1(a)) with the exception that it had sharp corners rather than smooth introductory curves.

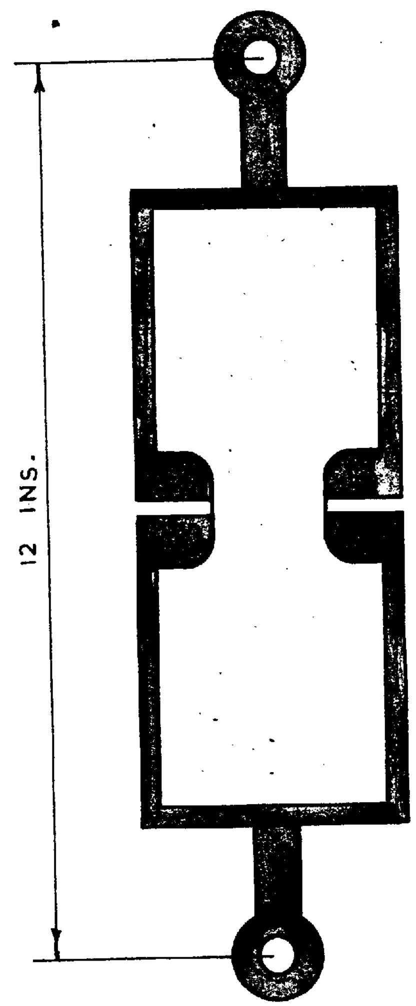


Fig. 2.2 Tensile Specimen and grip of John Grant.

The popularity of the briquet test for cement was such that by 1860, it had displaced all other methods for determining the quality of cement pastes. It was probably this popularity which influenced the choice of the briquet or its modification by the first investigations which studied the properties of mortars and concretes in tension. The period 1860 - 1890 saw the emergence of concrete and especially reinforced concrete as reliable structural materials. This increased greatly the interest in the tensile properties of plain concrete since the cracking of reinforced concrete beams and slabs is primarily a tensile failure.

2.4 UNI-AXIAL OR DIRECT TENSION TESTING METHODS

Since the first real interest in the properties of mortars and concrete in uni-axial tension at the turn of the century, more than 30 testing techniques have been employed to apply a uni-axial tension force to test specimens. However, detailed examination of these various techniques indicated that they can be divided into four major methods as follows:

(1) Specimen with enlarged ends.
   a) tensile briquets
   b) large tension specimens
(2) Specimens with embedded bars
(3) Frictional gripping techniques
(4) Adhesives

The above four methods are **aimed** at satisfying the requirements of uni-axial tension as defined in 2.2. However, other test methods have been introduced in which either one of the principle stresses is tensile (indirect tension, see 2.2), or a variable tensile stress distribution exists across the section (hoop tension, see 2.2). These cannot be uni-axial tension tests by definition. It is felt however, that the methods are important and have enough bearing on this work to be considered.

(5) Hydrostatic pressure

(6) Indirect tension tests

A critical examination of the various methods will now be given. Particular attention is paid in any group to those testing techniques which are considered to be the most significant contribution of the particular method.

(1) Specimens with enlarged ends:

This appears to be the first method employed for testing concrete materials. It would seem that after the tensile briquet test had been accepted as the standard cement performance test, it was logical to adopt such a specimen shape for testing mortars, and with enlarged specimens, concretes.

In 1897, Féret[5], using a briquet with a 5 cm. square cross-section, studied the relationship between the strength of mortars tested in flexure and direct tension. He concluded that in general the properties of mortars in flexure and direct tension were similar. The flexural strength was stated to be 1.89 times the direct tensile strength. Mills and Dawson[3] recorded that a year later De Joly had completed a series of static and fatigue tests using briquet specimens. De Joly concluded that the fatigue strength of mortars in tension was 55 per cent of their static strength.

One obvious disadvantage of the briquet specimen was its small size. Contemporary investigators attempted to overcome this short-coming, which limited the maximum size of coarse aggregates, by enlarging the specimen [6,7,8]. In general, inconsistent results were obtained.

It was not until 1926 that an effective tension test on concretes was performed using this method. A.N. Johnson[10] was obviously aware of the problems of eccentricity due to poor alignment resulting either from the grips or the specimen geometry. His direct tension test used a cylindrical specimen having a diameter of $4\frac{1}{2}$ ins. and an effective height of 9 ins. The specimen was increased conically to a diameter

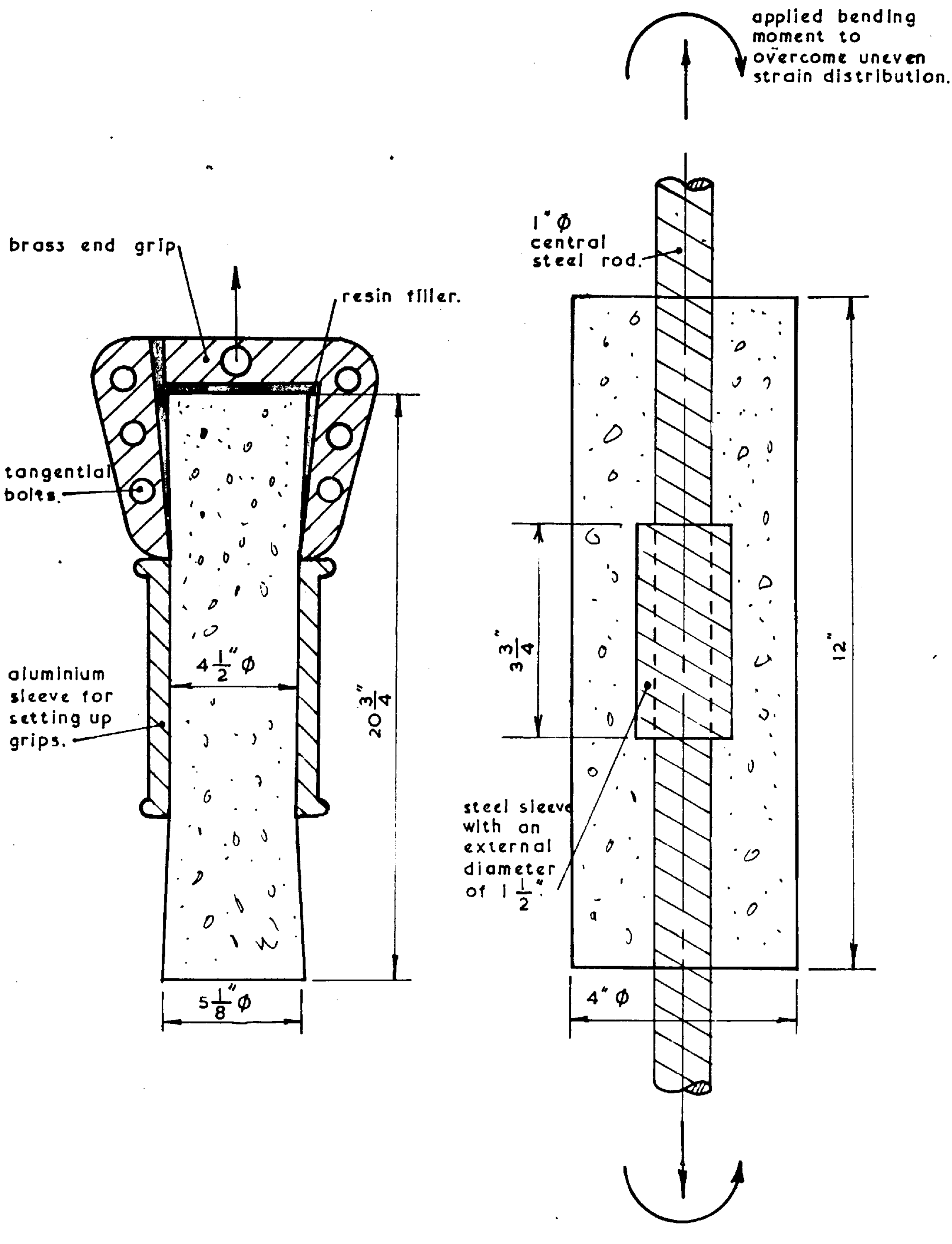


FIG. 2.3. a. JOHNSON'S SPECIMEN SHOWING METHOD USED FOR SETTING UP EACH GRIP.

FIG. 2.3. b. TODD'S SPECIMEN.

FIG. 2.3

of $5\frac{1}{8}$ ins. at each end (Fig.2.3(a)). Special conical brass grips applied the tensile force to the cylindrical part of the specimen through the conical ends. An aluminium guide clamped to the central cylindrical portion of the specimen was used to locate each brass grip in turn. Once the first grip had been accurately located, a resin adhesive was poured into the space between the grip and the specimen. This secured the specimen in a fixed position relative to the grip. After the resin had hardened, the whole was inverted and the second grip was fixed. The method, although slow and tedious, was noteworthy for its effectiveness in reducing the degree of eccentricity due to non-alignment of the centre line of the specimen with the applied load, and stress concentration from specimen shape.

Schuman and Tucker[11] report the work of Dutran and Guttman who also used grips to hold cylindrical specimens with enlarged ends. However, little work of any significance was performed with this testing method until 1939 when McNeilley[90] redesigned the standard cement briquet grips and increased the reduced mid-height portion of the specimen. These modifications alleviated somewhat the problems associated with the tensile briquet test since a considerable increase in tensile strength was obtained compared with previous methods.

In 1946, Evans[12] carried out strain measurements on opposite sides of several different sizes of briquet specimen loaded through modified grips. From these tests he concluded that the strain distribution on these specimens depended to a large extent on the original location of the specimen in the grips. He established that it was possible to adjust the specimens in the grips until a uniform strain condition existed. It is suggested by the writer that this work indicated the severe limitations of the briquet tests so disqualifying it as a suitable general direct tension testing technique.

Ten years later, Blackman, Smith, and Young[13] tested an increased mid-section length briquet specimen with a 1 sq.ins. cross-section. Only a small number of tests were performed in which the tensile loading system was varied from uni-axial tension to flexure.

Recently at the University of Birmingham[31], large 'I' shaped specimens have been used to test concrete in direct tension. The load was applied through jacks placed on either side of the main centre section and connected to a single manifold. Unfortunately, considerable eccentricity was developed in the specimen during loading due to different frictional characteristics of the separate jacks.

A similar method is currently being used with greater success at the Portland Cement Association Laboratories, Skokie, Illinois[29]. In this work, jacks with P.T.F.E. (polytetrafluoroethylene) linings have overcome the problem of differential friction previouly encountered at Birmingham. These linings have produced with this method significantly reduced and very similar frictional characteristics for each jack.

(2) Specimens with embedded bars:

The use of embedded bars to apply a tensile load to a specimen is probably the simplest and most widely used form of direct tension test. Conventional testing machines can be used and for small testing programs, the method had obvious economic advantages. With the desire to test larger specimen volumes of concrete materials in direct tension, the embedded bars method was an obvious development from the standard briquet specimens.

According to Schuman and Tucker[10], the first investigators using embedded bars were Talbot in 1904 and Withey in 1907. Both of these investigators used short smooth steel rods embedded in each end of the test specimen to apply the load. Apart from the efforts of Mills[14] in 1916 and Davis[15] in 1930, this method does not appear to have been used extensively until Schuman and Tucker[10] developed the method further for their work in 1943.

Basically, the technique adopted was similar to that used previously by Talbot and Withey. The significant difference was that each cylindrical specimen, 4 ins.diameter and 16 ins. long, was composed of three separate layers of material (Fig.2.1(b)). The top and bottom of each specimen consisted of a layer of high strength mortar 4 ins.deep. Four threaded rods fixed to each end cap were set 3 ins. deep into this mortar. In effect, the material to be tested in direct tension in the central portion of the specimen was loaded through a mortar interfacial layer. This system proved reasonably successful since consistent results were obtained, and the majority of the specimens failed in their central portion. The method of Schuman and Tucker was used subsequently by Wright[16] in 1955 and Kaplan[17] in 1963.

Evans[12] also used several specimens with embedded bars but recorded a considerable degree of eccentricity during loading with strains on opposite faces sometimes differing by a factor of 2. In his paper, Evans criticized the inclusion of a ball seating mechanism which would allow the specimen to move under load due to the forces created by misalignment or specimen anisotropy. This point is amplified and discussed in 4.2.2.

In 1954, an ingenious and rather complex tension test was developed by Todd[18] working at Oxford University. The test consisted of a 4 ins. diameter cylinder of concrete, 12 ins. long reinforced by a 1 ins. diameter rod 28 ins. long, passing through the centre of the specimen. The ends projected from each end of the concrete cylinder so that the tensile force could be applied to the rod. A steel sleeve $1\frac{1}{2}$ ins. diameter and $3\frac{3}{4}$ ins. long was fitted over the rod at its centre so that when the concrete was cast the sleeve in effect reduced the cross sectional area of the specimen. A thin coat of wax was put on the sleeve before casting to ensure that the concrete did not adhere to it. The first 4 ins. of the rod which protruded from each end of the sleeve was knurled to increase the bond of the rod to the concrete. specimen. A strain gauge was fixed onto the rod under the sleeve. Since the total load on the machine was known and the load carried by the rod in the central portion could be calculated, it was a simple matter to calculate the load carried by the annular ring of concrete in the central reduced section.

An interesting feature of Todd's technique was the method adopted to overcome any eccentricity that might occur due to non-axial loading. A separate external loading system was incorporated which produced a constant bending moment over the specimen section. At each load stage during a test, a bending moment was applied of suitable magnitude and direction so that the three extensometers which were mounted on the specimen 120° to each other, gave the same reading. Todd noticed that the required direction of the applied bending moment to counterbalance the eccentricity of any specimen remained much the same from small loads right to failure.

In 1956, R.G. Robertson and D.C. Robertson[19] used two types of tensile prisms, 4 x 4 x 48 ins. with short embedded bars and a 4 x 4 x 30 ins. with one long centrally located 3/4 ins. bar. They obtained similar tensile strength results from both techniques. A year later, Humphries[20] reported results of direct tension tests using 5 ins. diameter and 33 ins. long cylindrical specimens with a mild steel rod embedded 8 ins. at each end. In his analysis of the results, Humphries discarded low 'detached values' which he attributed to eccentricity. This approach was suggested by Evans[12] to produce a meaningful average from briquet tests. It was adopted by Humphries in order to reduce the otherwise large variation in test results obtained with his method.

Recently, Halabi[21], using the embedded bar test of M. Peltier, obtained interesting results. His were the first reported direct tension results which were higher and less variable than those obtained from the cylindrical indirect tension test.

<u>(3) Frictional gripping methods:</u>

The briquet specimen used with method (1) was generally quite small and suffered from local stress concentrations due to the specimen shape. Method (2) was tedious and the number of specimens that could be cast and tested at any one time was limited. An obvious improvement to these methods was to use a system with a specimen which was easy to cast and set up as in Group 1, but which did not induce stress concentrations due to its geometrical shape as in Group 2. Such a development is apparent in the frictional gripping methods applied to suitably shaped test specimens.

In 1926, D.Abrams[22], of the Lewis Institute, produced the first of such grips. These grips comprised pieces of 6 ins. steel pipe split into quarters and lined with leather. Friction was produced by tightening the tangential connecting bolts. The unique feature of this method was that after failure, the 6 ins.diameter 21 ins.long cylindrical specimen could be gripped and tested again over a shorter length. In general, these subsequent tests produced higher results. This particular design of grips was also used by the Bureau of Public Roads[22,23,24].

Similar grips were adopted by Gonnerman and Schuman[25] of the Portland Cement Association in 1928 for their comprehensive study on the properties of concrete materials in compression and tension. Although a high percentage of the specimens tested using such bolted frictional grips failed in the vicinity of the grip, it was assumed by the investigators that the failing stress very nearly represented the true tensile strength of the material.

It is considered that the most effective method for gripping a direct tension specimen is that designed by O'Cleary and Byrne[26] in 1960 (Fig.2.1(c)). Self-centering 'lazy tongs' grips were used to test a specimen with a reduced central section of 3 ins. by 0.9 ins. The coefficient of variation of the tests reported was high at 8 per cent. However, considering the size of the specimens and the probable lack of compaction, the test results were promising. The major disadvantage was that steel mesh had to be cast into the specimen ends to ensure that local compression failures did not occur under the gripping bars.

Recently, C.D. Johnston[27] of Queen's University of Belfast has used a modified O'Cleary and Byrne system which loads through plates instead of bars. These grips can accomodate prismatic specimens where the maximum dimension of a cross-section can be 6 ins.

(4) <u>Adhesives:</u>

A relatively new method of applying a direct tensile force is to attach the test specimen to the loading system by means of a suitable adhesive. Such a method depends on the bond between the specimen and the loading system being stronger than the material to be tested.

The introduction of fast setting epoxy resins of high strength in recent years has made such a technique possible. Unfortunately, the problem with the majority of the organic resins materials is that they do not adhere well to moist surfaces. Therefore their use to date has been limited to the testing of concrete specimens which are dry or partially dry.

Weigler and Becker[28] in 1961, during an investigation concerned with combined stresses in tension and compression, developed a uniaxial tension test incorporating an epoxy resin adhesive. Good results appear to have been obtained.

Since these first attempts, several testing laboratories[29,30] and individual investigators[31,32] have carried out direct tension tests using adhesives to apply the load. Recently, Broms[32] of Cornell has reported the use of an epoxy adhesive tension test for concrete. However, his technique was apparently suspect as the specimens with a reduced mid-height cross section of 6 x 3 ins., all failed away from the reduced section.

Other recent tests have been carried out by Monfore[29] and Chapman[31] in 1963 with more promising results. Rüsch and Hilsdorf[30] of Germany have also perfected a technique for direct tension testing with adhesives. In addition, their report on pilot tests where loading was by constant strain rate, indicated that a descending portion of the stress-strain relationship exists in tension after the specimen has reached the maximum stress. This is similar to results obtained in compression[91] (see also 4.2). The results of Rüsch and Hilsdorf were obtained in the direct tension test when the axial load was maintained truly axial to the original specimen cross section, even after cracking.

<u>(5) Hoop tension</u>:(see 2.2)

In general, the investigators who have tested concrete materials in tension using hoop tension loading methods have been interested in the tensile properties of concrete supplementary to an investigation concerned with biaxial states of stress.

McHenry and Karni[33], and Bresler and Pister[34] in 1958 obtained a tensile state of stress by loading hollow cylinders with internal hydrostatic pressure as part of their work on combined stress systems. Recently, a British Government Defence Research Establishment has developed an ingenious hoop tension test. The hoop stress is developed by an air bag which is restrained within the concrete hoop, perpendicular to the direction of applied loading on the hoop, by two metal plates mounted on either side of the hoop. The pilot tests with this method have been promising.

(6) Indirect tension tests:

An indirect tension test (see 2.2) is in general a test in which a principal tension stress or strain is developed using some form of compressive loading. The general advantage of this method is the comparatively simple test specimen and test procedure.

It is recorded[102] that the indirect tension test was first used by the Japanese in the 1940's. However, it was not until 1953 that Carneiro and Barcellos[35] of Brazil and Akazawa[36] of Japan independently introduced the cylinder splitting test for concrete. However, further development of the test, involving the selection of suitable loading strips, the comparison of the variance of the test with that obtained from other forms of destructive testing, etc., has been carried out by Wright[38] and Mitchell[39]. From comparison tests, Wright suggested that the strength results obtained from the cylinder splitting test are higher in magnitude and less variable than results from direct tension tests using a test from method (2). Recently, Halabi[21] has stated that the cylinder splitting test gives results that are both lower and more variable than the direct tension test used in his investigation, which also used method (2).

A. Nillson[41] in 1961 showed that an indirect tension test could be carried out on cubes. Good correlation was obtained between the tensile strength from cubes and cylinders, which have dimensions such that the cube circumscribes the cylinder dimensions. (see Fig.2.4)

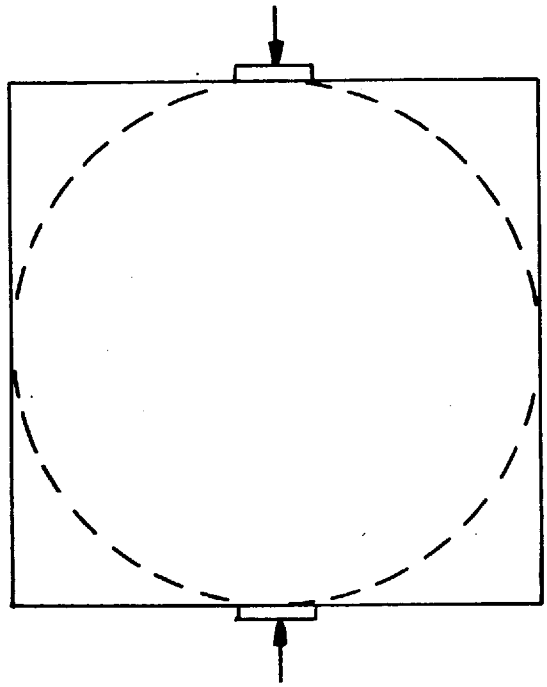


Fig.2.4 Indirect tension cylinder circumscribed by an indirect tension cube.

An interesting development in indirect tension testing was recently suggested by Durelli, Morse and Parks[40]. Here the test specimen is in the form of the greek letter <u>theta</u>, Θ. When the specimen is loaded in compression at right angles to the central bar, a uniform tensile stress is induced in the central bar. It has been shown experimentally that a small amount of misalignment does not affect the results, but the stress in the central bar has to be computed from measured deflections using the questionable assumption that brittle materials exhibit a linear stress-strain relationship to failure. This would appear to be the main limitation of this technique.

2.5 DISCUSSION AND CRITICAL SUMMARY OF TENSION TESTING METHODS

It is useful to reconsider each of the types of direct tension testing techniques,reviewed in the previous section, in order to establish their merits and disadvantages.

(1) Specimens with enlarged ends:

It has been shown using photoelastic techniques[42,43] that the standard and modified tensile briquets suffer from three major faults: (1) the stress system in the central section is biaxial with a compressive stress due to the grips occuring at right angles to the tensile force, (2) the tensile stress distribution is of a parabolic form with the maximum stress at the edges 1.75 times the average stress (Fig.2.1(a)), and (3) it has been shown that the briquet specimen cannot be aligned with sufficient accuracy in the grips which results in considerable eccentricity in the specimen[12].

It would appear that the larger specimen used by Johnson[10] overcame many of these difficulties, and his results were the most consistent and meaningful with this method. The main advantages of a large specimen are the ability to test concrete with large aggregate size and less effect from small eccentricities on the stress distribution within the specimen.

Unfortunately, the practical difficulties inherent in his complex testing technique severely limited the number of specimens that could be tested in any one day. This factor probably discouraged later investigators from using his method. It can be concluded that an efficient direct tensile test requires a simple test specimen of the nature of a briquet, but without the briquet's testing faults.

(2) Specimens with embedded bars:

The direct tension test developed by Schuman and Tucker[11] has the distinction of being copied by two later investigators, Wright and Kaplan. Kaplan[17] reported that the difference in strain on opposite faces of the tension specimens rarely exceeded 10 microstrain. Wright[38] obtained an acceptable coefficient of variation of ultimate tensile strength of 7 per cent. Considering the foregoing and the lack of stress concentrations due to the specimen shape and design, the test would appear to be most satisfactory. However, the method has three disadvantages: (1) the specimens are cast vertically which both Wright and Kaplan suggest as the reason for the low ultimate strengths obtained with the test, (2) the test method is rather complex since two mixes have to be batched for each casting, firstly, a mortar mix to bond the embedded bars and secondly, the mix under test (Fig.2.1(b)), and (3) the locating of the steel bars in the moulds and the casting of the specimens has to be carried out with extreme care.

The method used by Todd has the advantage that the eccentricity can be controlled and reduced to a negligible level. Again, the disadvantage is that the preparation of a specimen for casting is complicated, requiring great care. In addition, the testing procedure is extremely complex since a singular test requires the operation of three separate counterbalance systems.

(3) Frictional gripping methods:

The earlier gripping methods suffered to a large extent from the same fault, i.e. the majority of the specimens failed in the vicinity of the grips. This indicated that there were secondary effects due to the method of applying the gripping force. Furthermore, with the type of grips used by the earlier investigators, it would be difficult to maintain an axial loading condition from test to test.

O'Cleary and Byrne made the most significant contribution to this type of testing method. Their self-centering grips (Fig.2.1(c)) would appear to override the past objections due to alignment, but the specimens had to be reinforced under the grips and the practical application of their test was limited. However, the modification of O'Cleary and Byrne's method by Johnston[27] is an improvement. He added loading plates to overcome the local stress concentrations obtained under the gripping pins in the original method.

(4) Adhesives:

For the purpose of applying an even direct tensile load to a test specimen, flat plates bonded uniformly to the ends of the specimen by an adhesive theoretically is the best method. If the specimen shape is designed properly, no problems are encountered due to connection failures. When care is taken, eccentricity due to mis-alignment of the loading plate during the pretest preparation may not be a problem. It is possible to obtain a stiff connection between the specimen and the testing machine, which is necessary if loading by constant strain rate to failure is desired (see 4.2.1). There are two disadvantages; firstly, steel loading plates have to be stuck on each end of the specimen some time before testing[30,31], secondly, the majority of the available and suitable adhesives do not adhere to moist surfaces, therefore limiting the use of the test method to investigations using dry or partially dry specimens.

(5) and (6) Hoop tension and Indirect tension tests:

The advantages of the split-cylinder and the hoop tension tests in general are casting and testing simplicity.

The major disadvantage with all of the methods (with one exception, the theta specimen[40]) is that the stress or strain condition is not one of uni-axial stress or strain (see 2.2). In the hoop tests, there is a stress gradient across the section (Fig. 2.1(d)). In the split cylinder and split cube tests, the tensile stress on the diametral plane between loading points is accompanied by a significant compression stress perpendicular to it[44] (Fig. 2.1(e)). Further, results obtained from the foregoing tests can be a function of the loading strip materials which also detracts from their values.

Conclusions:

It is obvious from the review of tension testing methods and the critical assessment of the review that a satisfactory direct tension testing method for general laboratory and research use has not yet been developed. This is apparent when it is considered that (1) none of the specifications and standards agencies throughout the world have a standard or tentative specification for a direct tension test, (2) a very large number of different methods of tests have been developed, and (3) it has been seen that there has been active interest in this subject since the middle of the nineteenth century.

In general, any method which might satisfy the conditions of uni-axial stress usually falls short of acceptance as a standard test procedure because of practical limitations imposed due to the method of casting or testing. Conversely, the test methods which can be shown to be practical, suffer from a multitude of faults which make them unacceptable either as a research test or as a general laboratory test.

2.6 EFFECT OF TESTING TECHNIQUE ON THE DIRECT TENSION STRENGTH OF CONCRETE

Unfortunately a comprehensive testing program has not yet been carried out to assess the effect on the direct tensile strength of concrete materials of such testing conditions as (1) size of test specimen, (2) rate of application of load, (3) direction of loading, (4) sustained loading, (5) frequency of loading, and (6) moisture condition of specimen at time of test. However, the following discussion examines the available information on these effects from the direct tension investigations mentioned previously. When little evidence is available, reference is made to the behaviour of concrete when tested in compression, flexure, and indirect tension, for which there is considerably more information.

(1) Size of test specimen:

Concerning the size of the test sample, conflicting evidence is available. Evans[12] observed a decrease in ultimate direct tension strength with an increase in specimen size whereas Gonnerman and Schuman[25] observed no such effect. However, the results of both flexural and compressive tests, [47, 106] tend to confirm Evan's conclusions as do the theoretical considerations of Tucker[46] and Weibull[45] on the basis of the statistical distribution of critical flaws. Furthermore, since 60 per cent of Gonnerman and Schuman's test specimens failed at the grips, it is suggested that this factor would override any effect due to an increase in the total length or size of specimen.

(2) Rate of loading:

The only investigation which has examined the effect of the rate of loading on the direct tension test is that of Rüsch and Hilsdorf[30]. They concluded that the failing strengths of test specimens are in general proportional to the rate of loading, faster loading rates producing higher strengths. Such an effect has long been known for concretes tested in flexure and compression from the work of Wright[47] and Watstein[48] respectively. It is suggested that irrespective of the testing method, the strength of concrete specimens is proportional to the rate of loading.

(3) Direction of loading:

It has been suggested by Wright[16] and Kaplan[17] that the direct tension specimens used in their investigations gave lower results than expected because the direction of direct tensile loading was perpendicular to the casting layers, i.e. across possible planes of weakness. This is supported indirectly by the work of L'Hermite[50] who reported that the strength of a test cube loaded parallel with the layers as cast is some 13 per cent less than when the load is applied perpendicular to the casting layers.

(4) Sustained loading:

The results of tests performed by Price[52], Rüsch[53], and Berg[54], all agree that the strengths of test specimens under a sustained load are only 60 - 70 per cent of the static strength, i.e. that obtained at normal testing rates. Although no such evidence is available for direct tension tests, it is reasonable to assume that the same effect would occur in concrete specimens tested in direct tension.

(5) Repeated loading:

The only investigation of the fatigue behaviour of concrete materials in uni-axial tension was that conducted by De Joly[3] as early as 1898. He concluded from uni-axial tests on briquet specimens that a fatigue limit existed for concrete which was 50 per cent of the static strength. In comparison, the majority of investigations in uni-axial compression have reported that a fatigue limit of 55 per cent exists[55]. The same order of fatigue limit has been obtained also from flexural tests[55].

(6) Moisture conditions:

Loss of moisture before testing would appear to reduce the ultimate tensile strength obtained. No information is available from uni-axial tension tests but it is recorded that significant reductions are obtained in the flexural[56,57] and indirect tension[57] strengths when a degree of drying is allowed before testing. The flexural strength suffers the greatest reduction in strength since the associated shrinkage stresses initially are only a surface phenomena. Alexander[59] stated that a loss in flexural strength was observed in half-inch square prisms which were exposed to dry air for as little as 30 seconds. On the other hand, Mills[58] has observed an apparent increase of as much as 80 per cent in the compressive strength of concrete after oven drying.

2.7 PROPERTIES OF CONCRETE IN UNI-AXIAL TENSION

The majority of investigations into the properties of concrete in uni-axial tension have been concerned with the ultimate strength obtained from the test methodsand their relation to other destructive tests. Very little work has been reported where both longitudinal and lateral strain measurements were recorded during the test. This is not surprising when the magnitude of these strains is considered, maximum values being of the order of 100 and 20 microstrain approximately in the longitudinal and lateral directions respectively. Furthermore, with the exception of Todd's[18] work at Oxford, the majority of the tension testing methods did not allow strain measurements to be made easily.

(1) Variance of ultimate strengths in direct tension:

In Table 2.1, a comparison is given of the coefficients of variation of the ultimate strengths obtained from the direct tension and other destructive tests. It is interesting to note that the available evidence is conflicting. Wright and Ali imply that their direct tension testing method gives more variable results than either the indirect tension test or the flexural test. In Lingham's[61] work the variances obtained with the flexural test is less than the indirect tension and direct tension tests, which gave similar results. Recently, Halabi[21] concluded that Peltier's direct tension test was less variable than the cylindrical indirect tension test. To further confuse the issue, Wright and others have reported results in which the variance of the flexural tests is greater than the indirect tension tests, which disagreed with the work of Lingham, Rüsch, and Ali. Of the above investigators, only Ali and Lingham appear to have used an acceptable uni-axial tension test, in which variation of the strength results were similar to those obtained from the indirect tension test. As the majority of uni-axial tension tests were of secondary importance to the investigation reported and with the probable relative lack of attention paid to the test method, the high coefficients of variation are not surprising. Considering this, it would be expected that the direct tension test with proper care could give a similar distribution of results to that obtained from other tension testing methods.

Table 2.1 Coefficients of variation obtained from various compression and tension tests

Strength test	coefficient of variation obtained by:			
	Wright[16]	Rüsch[62]	Ali[63]	Lingham[61]
Tension:				
(1)indirect tension	5.0	6.0	8.1	8.6
(2)flexure tension	6.0	4.5	6.3	5.8
(3)uni-axial tension	7.0	-	12.2	8.7
Compression				
(a)cylinder	3.5	3.5	-	-
(b)cube	-	2.5	-	-

(2) Modulus of elasticity:

Previous investigators disagree as to whether the modulus of elasticity in flexural or direct tension in terms of longitudinal strains is greater than or less than the modulus of elasticity in compression. Hatt[5], Todd[18], and Blakely and Beresford[65] suggest that they are equal. Evans[66], A.N. Johnson[10], J.W. Johnson[9], and Schuman and Tucker[11] all concluded that the modulus in tension is less than the modulus in compression whereas Robertson et al[19], and Mörsh[66] hold contrary views.

It has long been known that the stress-strain curve in compression and tension is curvilinear[18] from zero load. Thus the modulus of elasticity either in tension or compression is a secant value which depends upon the stress level at which it is measured. It is suggested that the apparent contradictions reported above are a result of this phenomena. This being so, the available evidence would suggest that the modulus of elasticity in tension is of the same order as the modulus in compression at comparable stress levels.

(3) Comparison of ultimate uni-axial tensile strength with the strength from other testing methods:

The results of many investigators [10,11,12,19,25,39.52] agree that the value of the ratio between the uni-axial tension and the compression strength from cubes and cylinders varies in general from 0.07 to 0.15. The highest ratio is obtained from the lowest strength mixes.

It would seem that any factor which increases the strength of a concrete mix, i.e. a decrease in the water/cement ratio, an increase in age, etc., will decrease the ratio of tension to compression. This phenomena was observed by Féret [5] as early as 1897.

It is interesting to note that Price [67] indicates that the uni-axial tension strengths of concrete are from 50 - 63 per cent of their flexural strengths.

CHAPTER 3

DESIGN CONSIDERATIONS FOR THE UNI-AXIAL TENSION TESTING MACHINE

3.1 INTRODUCTION

From the examination in Chapter 2 of the tension testing methods used by previous experimenters, it was possible to formulate the requirements in general terms, for an ideal universal tension testing machine. This chapter outlines the general design criteria for the ideal machine and reference is made to previous work where necessary to establish any point. The testing method developed in this investigation to satisfy these design criteria is then presented in general terms. A detailed analysis of the design criteria and of the testing machine developed to satisfy these criteria will be given in Chapter 4.

3.2 GENERAL REQUIREMENTS FOR THE DESIGN OF A UNI-AXIAL OR DIRECT TENSION TESTING MACHINE

It is apparent when viewing the achievements of previous experimenters who have tested concrete in uni-axial tension that some compromise was necessary when either the testing method was designed, the tests conducted, or the test results analysed. When the general requirements were being tabulated for the uni-axial testing machine developed in this investigation, it was decided from the beginning that the aim would be for a testing method which demanded a minimum of such compromise. What was required essentially was an accurate and precise[95] (see also 7.2) testing machine from which results could be obtained which were not in any way influenced by the method of test or the shape of the test specimen.

The following basic design requirements were considered:

1. Moisture Condition of Specimen:

The loading system adopted must be independent of the moisture condition in the specimen. This criteria eliminates the possible use of epoxy resin adhesives to fix the specimen to a loading device through which the tensile load is applied. The adhesives available at the moment do not bond adequately to moist concrete surfaces.

2. Eccentricity of Load:

The loading system should induce a state of uni-axial stress or uni-axial strain in the test piece within strict tolerances. Since this test was developed as a standard laboratory test as well as a basic research tool, it was essential that eccentricity due to the loading system be kept to a minimum. Corrective techniques such as Todd's[18] although effective, were not practical for a standard method of testing. Rejection of low results where eccentricity is suspected, as adopted by Humphries[20] and proposed originally by Evans[12], was considered unacceptable. This point also cast doubt on the use of epoxy adhesive materials as it was thought that a major problem with these materials would be the control of eccentricity.

3. Stress Concentrations:

The uni-axial tension test specimen and associated loading system should be designed so that a uni-axial state of stress exists in the portion of the test specimen where stress or strain results are to be obtained. It was considered very important to avoid the problems associated with the use of cement briquet specimens. Due to their shape and the loading method, such specimens develop parabolic stress distributions across a plane at their mid-sections.[42,43,44]

4. Pre-Test Preparations:

Obviously, the casting requirements and the setting up operation prior to testing should require time and skills similar to those required in other standard destructive tests. Bars cast in the ends of test specimens, the fixing of end plates on both ends of a test specimen with epoxy adhesives, or the use of ancillary loading systems or other special techniques, are complex and therefore undesirable. The above techniques, by reducing the number of specimens that can be manufactured or tested in any one day, give a test method suitable only for research instead of a more generally acceptable laboratory test.

5. Specimen Size:

The section of the specimen at which the tensile stress is developed should be of such a dimension as to enable concretes with 3/4 ins. maximum size aggregate to be tested. It is also desirable to have a section size subjected to the uni-axial stress which can accomodate strain measuring instruments with gauge lengths of 4 ins. This gauge length is necessary since (1) the maximum aggregate size is 3/4 ins.[67], and (2) the strains to be measured, particularly the lateral strains, are very small, i.e. less than 30 microstrain. Furthermore, to limit size effects, the volume of concrete subjected to the uni-axial stress should be similar to that volume under stress in companion compression and flexural test specimens, especially when comparisons between the properties of concrete under various types of loading are to be made.

6. Test Variance:

The statistical distribution of failing stresses in uni-axial tension should be similar to that obtained from other tensile tests. It is important that the within-batch and the between-batch standard deviations and coefficients of variation of the failing stresses obtained with the test method should be similar to those obtained from other tensile test methods. In general, the portion of the total variance of the results obtained from a uni-axial tension test which can be attributed to the testing machine is due to (1) the change of eccentricity from specimen to specimen, and (2) the size of the specimen. It is important that consideration be given to any factor which will minimize the effect of the testing method on the variability of the test results obtained from the machine. For example, the practice of rejecting low results because of suspected eccentricity is in principle, unacceptable[20].

7. Longitudinal Stiffness of Machine:

It is ideal to have a machine capable of testing both by constant stress or constant strain increments. In order to satisfy loading by a constant rate of stress, it is necessary to ensure that the machine has sufficient longitudinal stiffness. Then the loading system can apply continuous load increments, while overcoming the large deformations that occur in the specimen as it approaches failure, as well as the deformation that occurs in the testing machine. To satisfy loading by constant strain rate, it is necessary to have a longitudinal stiffness of the machine and loading system significantly greater than the expected stiffness of any specimen[38,91].

8. Lateral Stiffness Considerations:

In a uni-axial test, segregation of the material can cause a variation of stiffness through the specimen. This leads to bending of the specimen under load. In a compression test, the bending is restrained by the lateral stiffness of the machine which induces lateral forces and moments into the specimen resulting in a variable rather than the constant stress distribution that is generally assumed. In a tension test, any moments induced in this manner could have an appreciable effect on reducing the ultimate strength results obtained from the machine. It is considered important that an attempt be made in tension testing machine design to reduce as far as possible, induced lateral forces and moments.

9. Stability Considerations:

It had been indicated that a tension machine can buckle if the length of the specimen is greater than the length of the machine[100]. A tension specimen effectively pinned at both ends in a machine comprising columns fixed to the machine cross-heads such that the distance between the pins is less than the column length, is an example of a perfectly stable machine. This condition therefore, should be pursued.

3.3 SUMMARY

It is considered that the order of relative importance of the foregoing criteria is as given in Table 3.1. During the development of the uni-axial testing method, an attempt was made to satisfy the design criteria in this order.

Table 3.1 Design criteria in order of considered relative importance.

Order	Design Criteria
1	Ability to control eccentricity.
2	Specimen shape and loading system must not induce stress concentrations.
3	Low test variance, i.e. high precision.
4	Specimen must be a reasonable size to cast and test.
5	Pre-testing operation must be practical.
6	Ability to test saturated specimens.
7	Sufficient longitudinal stiffness.
8	Sufficient lateral stiffness.
9	Sufficient stability.

3.4 GENERAL DESCRIPTION AND DISCUSSION OF THE UNI-AXIAL OR DIRECT TENSION TESTING METHOD

At this point, a general description of the tension testing machine used in this investigation will be useful to indicate how the design satisfies to a large extent the stipulated design criteria. Plate 3.1 shows the testing machine with a typical specimen inserted.

The machine loads the tensile specimens by means of a mechanical gripping technique in which the amount of grip is directly proportional to the load applied by the machine. The idea of this so called "lazy tongs" gripping technique for holding specimens to be tested in tension originates from the work of O'Cleary and Byrne[26].

The main features of the testing machine are the self centering grips, the two pin universal joints at each end of the machine, the hydraulically operated centre pull jack, and the reduced mid-section flexural beam test specimen.

The gripping method appears to be the most satisfactory solution for a research and general laboratory uni-axial tension test of concrete materials. The selection of this technique will now be justified in terms of the design criteria of 3.2.

1. Moisture Condition of Specimen:

With this method af applying the tensile load the moisture condition in the specimen is of no importance. The gripping platens will hold on any specimen where the coefficient of friction is greater than 0.3 (see 4.3). There is therefore no possibility of slipping during the testing of any concrete materials.

2. Eccentricity of Load:

Since strict tolerances were achieved in the construction of the grips, and furthermore, since it is possible to locate the test piece very accurately in the grips, it is possible to keep to a minimum eccentricity due to the misalignment of the applied load with the centre of the specimen. As will be shown, a difference in strain on opposite sides of the test piece of less than 2 per cent was easily attainable.

3. Stress Concentrations:

The transitions from the reduced sections to the full 4 ins. square section is achieved by means of a transition curve with a constant radius of curvature. It has been calculated that the stress concentration factor in the vicinity of the necked portion is 1.01, which is not significant[99].

4. Pre-Test Preparations:

In general, the time taken to place a specimen in the uni-axial tension testing machine is approximately four minutes. Care and a certain amount of skill is required for this operation. However, several operators have used this apparatus during the course of this work and no setting up problems arose when the set procedure was adhered to.

5. Specimen Size:

The test piece used in the machine is a standard flexural beam (4 x 4 x 20 ins., B.S.1881)[74] with a reduced mid-section. The volume of concrete in which the uni-axial state of stress in induced is similar to that obtained in the uni-axial compression test, ~~and~~ the 4 x 4 x 12 ins. prism developed by Lachance[78].

6. Test Variance:

The variance of the test is comparable to that obtained from other destructive tests. On a standard mortar, the coefficient of variation obtained from between-batch results was a low as 3.5 per cent (see Table 7.5) which is related to a high degree of precision(94).

7. Longitudinal Stiffness Considerations:

A criticism of the uni-axial testing machine is that considerable strain energy is stored in the grips and the rest of the loading system during loading. It will be shown that the machine has sufficient overall stiffness to test specimens by constant load increments, but the machine is basically a 'soft' machine(38) and loading by constant strain increments is impossible with the present equipment. The above point will be amplified in Chapter 4.

8. Lateral Stiffness Considerations:

The loading system is connected to the grips and specimen through universal joints at each end of the machine (see Plate 3.1). This allows complete lateral flexibility in the loading system which effectively excludes external moments from the test specimen. In general, the line of force passing through the centre of the universal joints passes also through the centre of the test specimen. This point will be amplified in Chapter 4.

9. Stability Considerations:

Since (1) the specimen is shorter than the columns of the testing machine, (2) the specimen is pinned at both ends, and (3) the columns of the machine are fixed to the cross-heads, this machine is an example of a perfectly stable machine.

3.5 SUMMARY

The general testing method adopted for this investigation has been compared with the general design criteria which were formulated from the review of the advantages and disadvantages of the test methods used by previous investigators. A tabular comparison is given in Table 3.2 in which the general methods reviewed in Chapter 2 and the new uni-axial tension machine are compared to the general design criteria for an ideal uni-axial tension machine. From this information, it was concluded that the adopted gripping method and specimen is the most suitable testing method.

Table 3.2 General comparison of writer's uni-axial tension test and the 6 general methods of direct tension testing with the general design criteria.

Design criteria in order of importance (see Table 3.1)	writer's uni-axial tension test (Plate 3.1)	Uni-axial tension tests (2.3)			Other tension tests (2.3)		
		(1) briquet specimens	(2) embedded bars	(3) gripping techniques	(4) adhesives	(5) hoop tension	(6) indirect tension
1. Eccentricity	S	U	L	L	L	U	S
2. Stress concentrations	S	U	S	S	L	S	L
3. Low variance	S	U	L	S	S	S	S
4. Specimen size	S	L	S	S	S	S	S
5. Pre-test preparation	S	S	U	S	U	U	S
6. Saturated specimens	S	S	S	S	U	S	S
7. Longitudinal stiffness	L (see 3.2(7))	S	L	L	S	–	S
8. Lateral stiffness	S	S	S	S	S	–	S
9. Stability	S	S	S	S	S	–	S

S - satisfactory (method satisfies criteria)
L - limitations (method partially satisfies criteria)
U - unsatisfactory (method does not satisfy criteria)

CHAPTER 4

THE DETAILED DESIGN, CONSTRUCTION, AND TESTING OF THE UNI-AXIAL TESTING MACHINE

4.1 INTRODUCTION

On the basis of the design criteria and considerations discussed in the previous chapter, the general outline and principles of the testing technique were established. In this chapter, a more detailed account is given of the design and development of the gripping method, testing frame and tensile specimen. The tests designed to investigate the behaviour of the complete testing machine are also described.

4.2 THE TESTING FRAME

In figure 4.1, the general arrangement of the four column loading frame is shown. The frame was designed to accomodate in its present form, 40 ins. long specimens. Specimens up to 60 ins. long can be tested with column extensions available in the laboratory.

It is estimated that the writer personally performed one half of the machining operations required for the construction of the frame, and supervised the remainder.

4.2.1 Longitudinal Rigidity of the Frame

Figures 4.1 and 4.2 show the overall dimensions of the frame components. As stated in 3.2, it is desirable to have a testing frame with a relatively large longitudinal stiffness. The dimensions of the columns and the cross heads were chosen with this in mind.

The relations for the deformation of each component of the machine for a given applied load, 'P' lbs., are given in Table 4.1.

The relationships between the various factors considered in the following discussion are as follows:

$$\text{deformation} = \Delta = KP \text{ ins.}$$

$$\text{stiffness} = X = \frac{1}{K} = \frac{P}{\Delta} \text{ lbs./ins.}$$

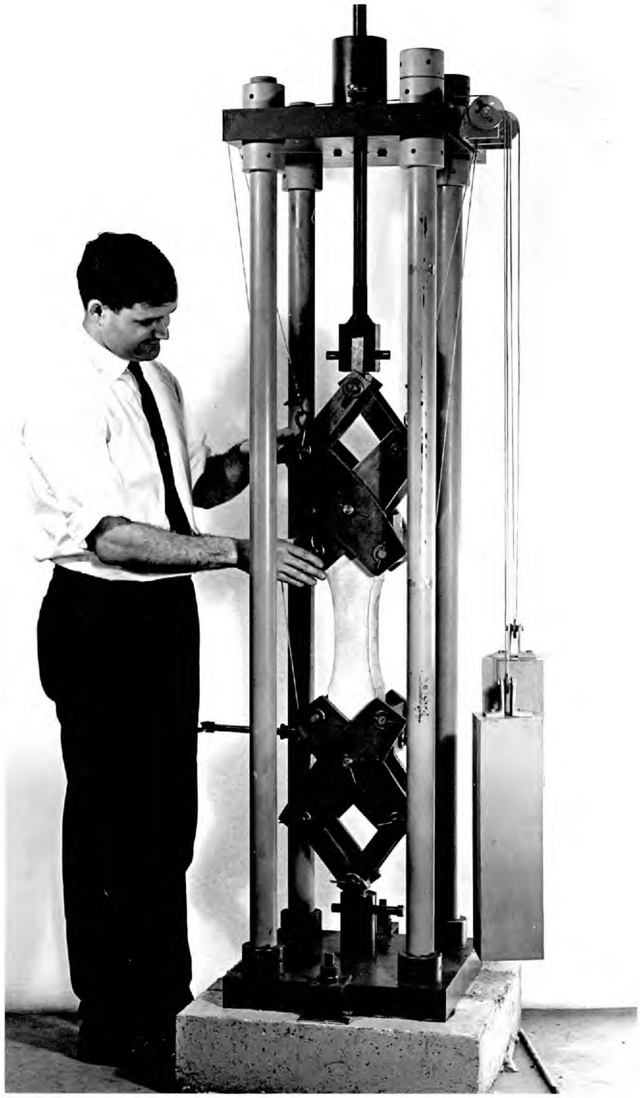


Plate 3.1 Uni-axial tension testing machine with typical specimen.

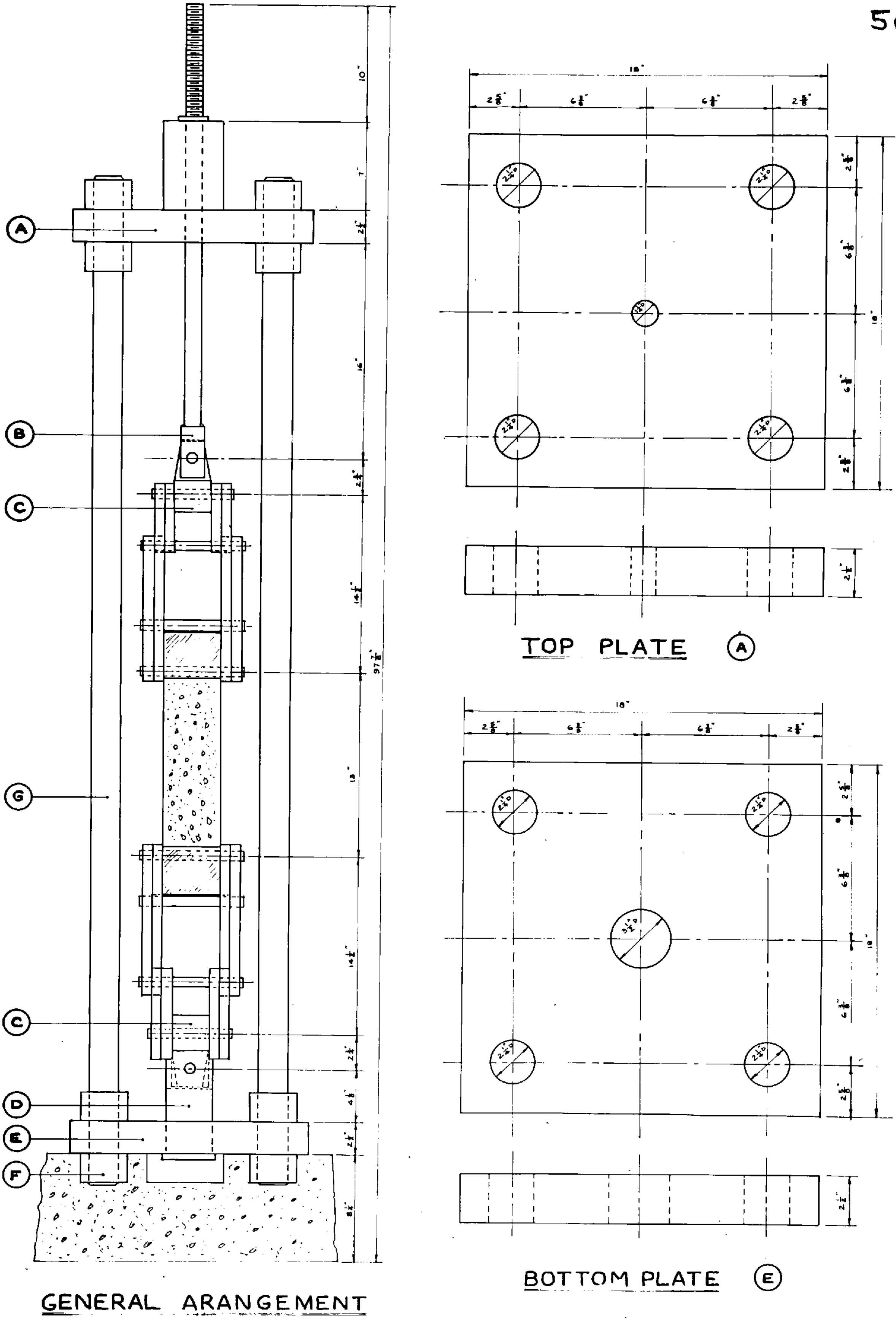


FIG. 4.1

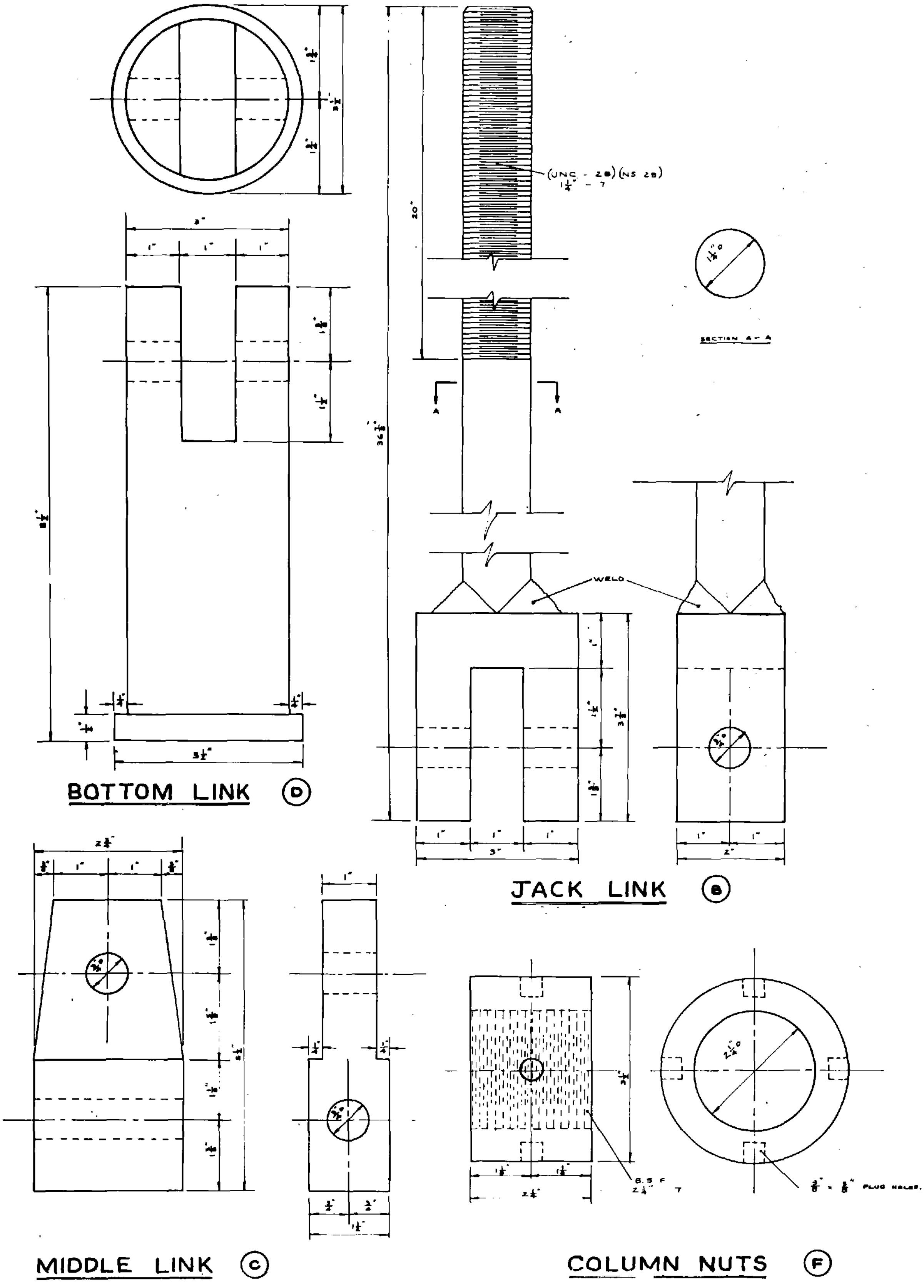

BOTTOM LINK (D)
JACK LINK (B)
MIDDLE LINK (C)
COLUMN NUTS (F)
WELD
SECTION A - A

FIG. 4.2

Table 4.1 Deformation relations for uni-axial tension machine components.

machine component	ref. fig. 4.1	Δ = deformation relation (ins.)	factors considered in assessing the deformation relation.
columns	G	$1.51 \times 10^{-7}P$	4 columns
cross-heads	A and E	$1.76 \times 10^{-7}P$	2 end plates
centre pull-rod	B	$6.53 \times 10^{-7}P$	3 ft. centre rod
grips (2^{nd} set)	-	$P(85.9\times10^{-6} + \frac{2.52}{E_c})$	both grips
grips (3^{rd} set)	-	$P(36.4\times10^{-7} + \frac{2.52}{E_c})$	both grips
loading jack	-	$2.10 \times 10^{-7}P$	1 ins. column of oil under jack ram.
hydraulic lines	-	$2.45 \times 10^{-9}P$	10 ft. of 1/4 ins.-diameter steel pipe.
hydraulic fluid	-	$4.60 \times 10^{-7}P$	fluid within above pipe.
typical specimen	-	$2.2 \times 10^{-7}P$	20 ins. specimen with $E_c = 4 \times 10^6$ lb/in^2

It is evident that the deformation of the machine components are of a larger order than those of the specimen, i.e. the longitudinal stiffness of the specimen is greater than the longitudinal stiffness of the machine. The uni-axial tension testing machine is a 'soft' machine.[38] In order to perform a constant strain rate test, it is necessary to have a testing machine with a longitudinal stiffness greater than that of the specimen, i.e. it should be a 'hard'machine[38,91]. Thus, in its present form, the machine cannot be used to carry out a test at a constant strain rate.

It is interesting to compare the stiffness of the tension machine with that of the constant strain rate compression testing machine developed recently at Cambridge by Turner and Barnard[91].

Cambridge compression machine:

$$X_m = 2 \times 10^6 \text{ lbs/ins.}$$

Uni-axial tension machine:

$$X_m = 2 \times 10^5 \text{ lbs/ins.}$$

It can be seen that the Cambridge machine is ten times as stiff as the tension machine. It is shown in Table 4.1 that the major portion of the deformation of the tension machine results from the flexibility of the grips and the centre pull-rod. It should be possible to stiffen these pieces and other parts of the machine to develop a uni-axial tension machine suitable for testing at a constant strain rate.

It is necessary to examine the deformation relations for the machine to see how the longitudinal stiffness affects its ability to carry out a test at a constant loading rate (stress instead of strain increments). Basically, the hydraulic system should be able to supply sufficient fluid, as the specimen approaches its ultimate load, to overcome both the increasingly large deformations that occur in the specimen as well as the deformation of the testing machine.

If X_m is the stiffness of the testing machine and X_s is the corresponding stiffness of the specimen, then it can be shown(79) that

$$\frac{dD}{dt} = \frac{dL}{dt}\left[\frac{1}{X_m} + \frac{1}{X_s}\right] \qquad \text{.......................................4.1}$$

where $\frac{dL}{dt}$ is the rate of increase of load with time and $\frac{dD}{dt}$ is the rate of extension of the jack with time.

For the jack used (area 7.22 ins^2.) and the available hydraulic loading rate at maximum working pressure (2.10 ins^3/min. at 1000 p.s.i.g.),

$$\frac{dD}{dt} = \frac{2.10}{7.22} = 0.29 \text{ ins/min.}$$

Assuming a typical concrete ($E_c = 4 \times 10^6$ lbs/in.^{2}), taking $X_m = 2 \times 10^5$ lbs/ins., and with a constant stress rate of 170 p.s.i.g./min. (which corresponds to a constant load rate of 1950 lbs/min. $= \frac{dL}{dt}$) by solving equation 4.1 for X_s, we obtain

$$X_s = 7.0 \times 10^3 \text{ lbs/ins.}$$

This corresponds to a specimen modulus of elasticity of

$$E_s = 1.1 \times 10^4 \text{ lbs/in.}^2$$

Such a low instantaneous value of E_s would only be obtained very close to the ultimate load of a uni-axial tension specimen tested by constant stress rate. It can be concluded therefore that the present machine is capable of loading a specimen by a constant loading rate to within at least 1 per cent of the ultimate load.

4.2.2 Lateral Rigidity and Stability of the Testing Frame

As indicated in the last chapter, induced lateral moments and forces from the testing machine produce a variation in stresses during loading which may cause premature failure of a test specimen. Consequently, it was decided to adopt a system which would allow the resultant reaction to always pass through the centre of the specimen. This was achieved by the introduction of universal joints at the end of each grip (see Fig.4.1). These create a two force member with the resultant reaction acting along the line connecting the centres of the universal joints.

As no pinned ends can be completely friction-free, a theoretical investigation was conducted to determine the magnitude of induced moments. The pins were initially assumed to be in an unlubricated condition. Assuming a static coeffient of friction of 0.6 for steel on steel, then the magnitude of the moment capable of being produced at the 3/4 ins. diameter universal joint pins is 0.225 P ins-lbs. A severe testing condition was analysed. A very segregated specimen with a strain distribution of 1[illegible] $\times 10^{-6}$ at one face and 200×10^{-6} at the opposite face was assumed. The assumed load on the test specimen was 7,500 lbs.(i.e. corresponding to a stress at the reduced mid-section of 650 psi.). The moment of resistance of the pin is therefore 1685 ins-lbs. Assuming complete fixity of the pins and the connection between the centre pull-rod and the jack, it is possible to solve for the unknown moments and forces. The moments developed are 690 ins-lbs. at the bottom pin, 225 ins-lbs. at the pin above the top grips and 160 ins-lbs in the centre pull-rod - jack connection. This will produce a stress variation across the critical section of $\pm$ 55 psi., that is approximately $\pm$ 10 per cent. It was found that the unlubricated pins remained fixed at this load since the frictional moment of resistance of the pins (1685 ins-lbs. calculated above) is greater than the amount calculated due to the specimen anisotropy (690 ins-lbs. developed above).

A second calculation, again at a load of 7500 lbs. and assuming a surface at the pins lubricated by molybdenum disulphide with a coefficient of friction of 0.04, indicated that the moments developed are 112 ins-lbs. at the bottom pin, 48 ins-lbs. at the pin above the top grips, and 7 ins-lbs. at the centre pull-rod - jack connection. This will produce a stress variation across the critical section of $\pm$ 9 psi. that is approximately $\pm$ 2 per cent. The bottom lubricated pin was considered not to remain fixed as the internal moment obtained in the first calculation (690 ins-lbs.) was greater than the fixing moment at the lubricated pin due to friction (112 ins-lbs.) As a result of these calculations, molybdenum disulphide was adopted as the universal joint pin lubricant.

In view of the fact that the above analysis is for a particularly severe degree of specimen segregation, it is considered that moments produced in the pins due to slight segregation of the material will result in variations considerably lower than the $\pm$ 2 per cent calculated above. Great care was taken while casting to ensure a specimen of uniformity from the bottom of the mould to the top. It is therefore possible to conclude that the resultant load reaction deviates a negligible amount from the centre line of the specimen due to induced moments from specimen segregation.

In the above analysis, the horizontal forces were also calculated. In the first instance, i.e. no lubrication on the pins, the horizontal force is 12 lbs. and the second calculation, i.e. lubrication on the pins, the horizontal force is 2 lbs. These are negligible.

Stability:

A tensile testing machine is inherently stable[100] if the effective length of the specimen is less than or equal to the length of the testing machine. In the developed tensile machine, the distance between the bottom pin of the specimen and the centre pull-rod - jack connection was less than the length of the frame. Consequently, the machine is inherently stable.

4.2.3 Alignment of Frame Components

It is appreciated that any slight eccentric loading can produce a surprisingly large variation in stress distribution instead of the assumed constant stress distribution (see 4.4.1). In order to eliminate any eccentric loading due to misalignment of components, great care was taken in the machining operations. All boring operations were carried out on a vertical milling machine and wherever possible, the location dimensions of the holes were measured using the feeds in the mill which were graduated in 0.0005 ins. divisions. Extreme care was taken in the manufacture of the pieces which formed the universal joints connecting each grip to the frame. As with the main parts of the frame, all finishing operations during the manufacture of the pieces were performed on a milling machine. The machine shop drawings for the frame and components are given in Appendix A.

4.3 THE DESIGN AND DEVELOPMENT OF THE GRIPS AND SPECIMENS

It had been concluded in the previous chapter that some form of mechanical gripping method should be used to apply the load in the tension test.

It was decided that the double 'lazy tong' system of O'Cleary and Byrne(Fig.2.1(c)) suffered from two basic faults: (1) a large length of specimen would have to be included within each grip and then the total size of the specimen required would become unmanageable,and (2) reinforcement was required in the specimen ends to prevent local compression failures under the gripping pins.

As may be seen from the photographs of the prototypes and final grips, a single mechanical leverage system was used in this work (Plates 4.1,4.2,3.1). However, abandoning the double lazy tong system did introduce a new problem. The double system of O'Cleary and Byrne acted basically as a self-aligning grip during the setting up procedure and there was no skill or special technique required in order to locate the specimen accurately in relation to the grip. When one unit of this system is removed, the self-aligning property of the grip disappears. Consequently, it was obvious from the outset that the success or failure of this new method depended on the following factors: (1) the degree of accuracy to which the grips could be manufactured and the ease and accuracy with which the specimen could be aligned in these grips, (2) the load had to be applied so that failure was induced in the central reduced portion of the specimenand not at the grips, without requiring reinforcement in the ends of the specimen, and (3) there should be no slipping of the grips on the specimen during loading.

4.3.1 The Grips

The basic design parameter for any gripping method is the value of the static coefficient of friction for steel on concrete. This was obtained experimentally by determining the force required to promote sliding for various normally applied loads of a smooth steel platen on a moist smooth concrete surface. The static coefficient of friction obtained from these tests was 0.6. The value used in arriving at the geometry of the grip members, which produce the normal force, was 0.2 (see Fig.4.3). This is a conservative value since the grip plates in practice were grooved and consequently developed a very high static coefficient of friction. Furthermore, the pins in the grip plates were located off-centre so that an uneven normal stress distribution

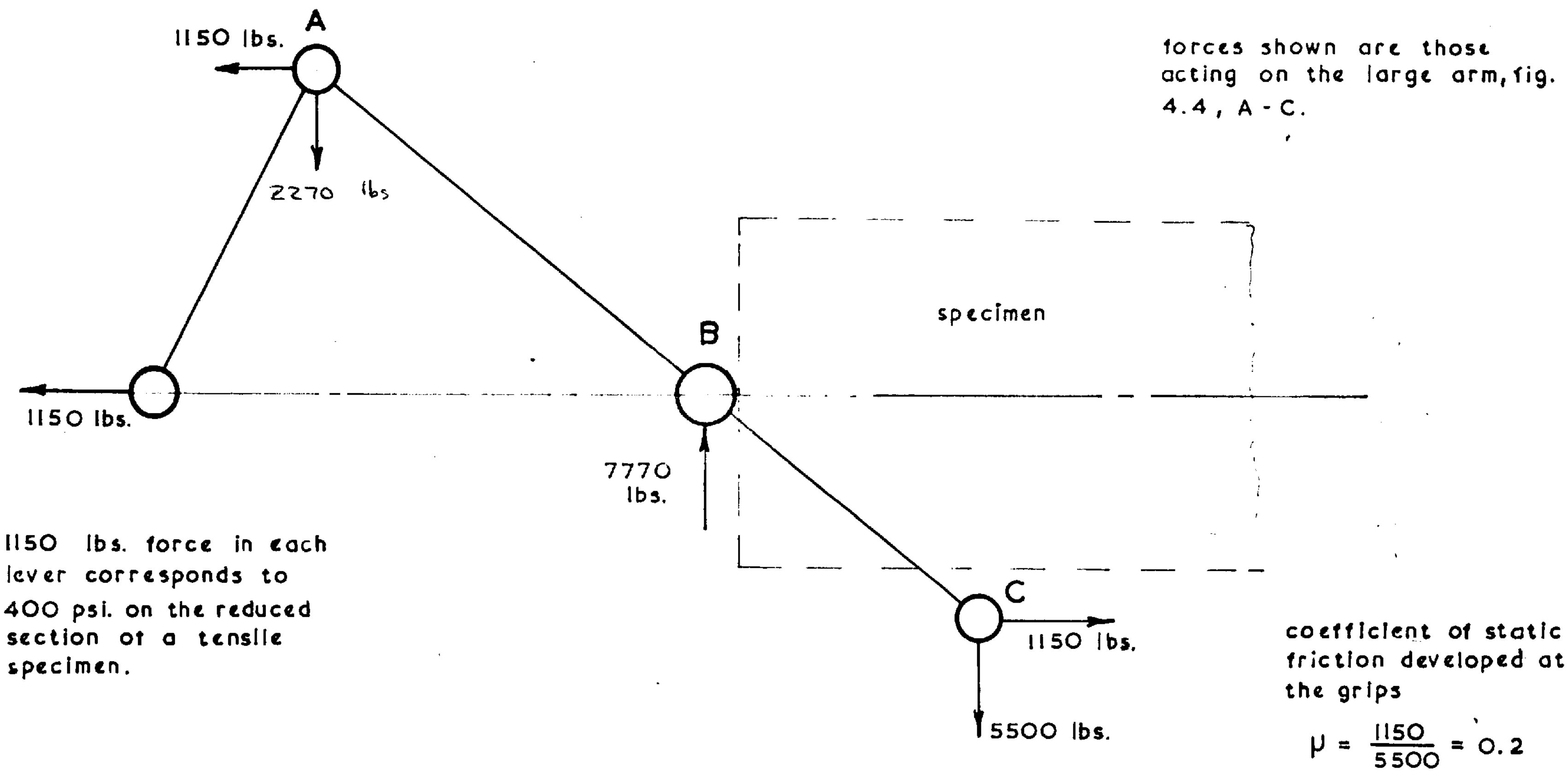


FIG. 4.3 FORCE DIAGRAM FOR ONE GRIP LEVER.

was developed in the specimen beneath the grip platen. The resulting stress distribution, in effect, was analogous to a specimen with enlarged ends, as the material under the edge of the platen nearest the centre of the specimen would deform more than the material under the platen near the end of the specimen. The dimensions of the grips were also chosen so that when a specimen was located against the main pin of the grip and the grip plates were placed firmly against the specimen sides, the specimen would be projecting approximately 0.01 ins. above the ground ends of both grip plates. This was required so that the test specimen can be located accurately and easily in each grip (see 4.4.1).

The first prototype grip, Plate 4.1, was built entirely by the author. Its purpose was to investigate the general feasability of the method to grip the specimen up to the ultimate load without slipping, and to see if the method of holding the test piece was practicable. Several specimens were cast with embedded steel rods at one end to enable a tensile force to be applied at both ends. These pilot tests did show that the method possessed promise and a design was produced for two complete grips which were larger, more accurately made and which, for the first time, accepted a 4 x 4 x 20 ins. flexural beam with a reduced section (Plate 4.2).

The second set of grips were constructed entirely by the author, and were the first to be fitted into the direct tension frame. Two separate counterbalance systems, which were part of the main frame, were devised so that the grips could be easily manipulated. The bottom counterbalance system was designed to balance the weight of the lower grip, a test specimen and one clamp, together with a small reserve which held the grips firmly against the specimen (Plate 4.3). With this system it was an easy matter to place and then accurately locate the specimen in the grip. The upper counterbalance system was only designed to balance the upper grip weight.

Experience was obtained with this complete system by testing various concretes and mortars. It was during this period that the degree of reduction for the reduced flexural beam was determined to ensure that failure was induced in the reduced section and not in the vicinity of the grips.

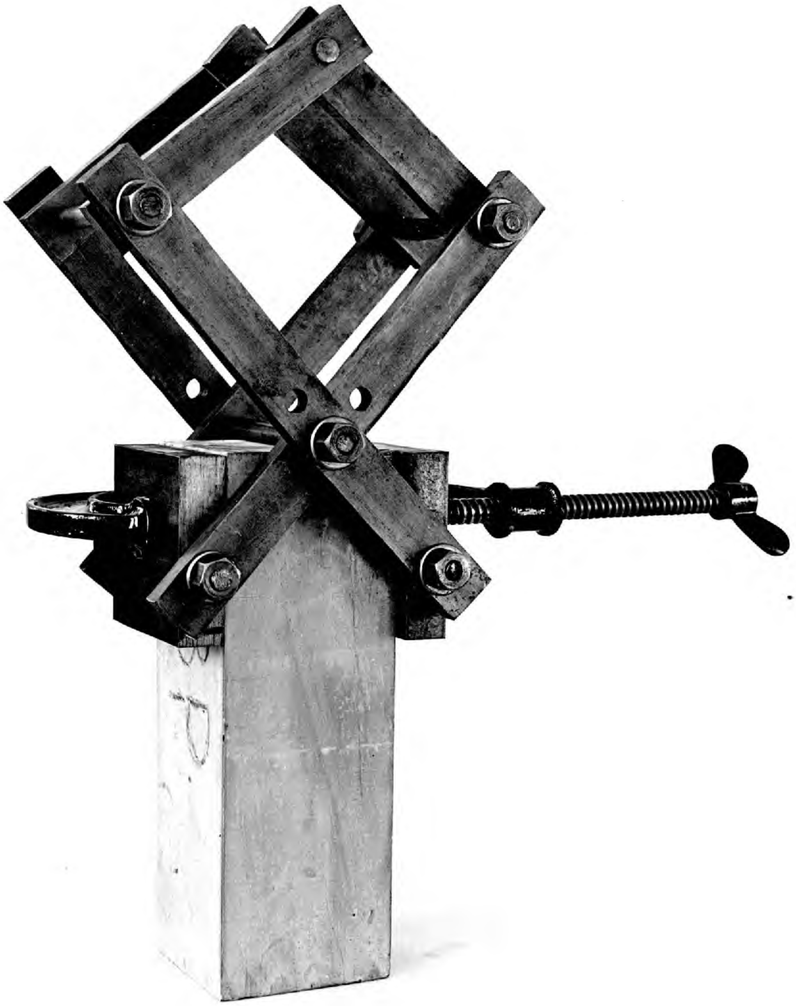


Plate 4.1 First prototype grip.

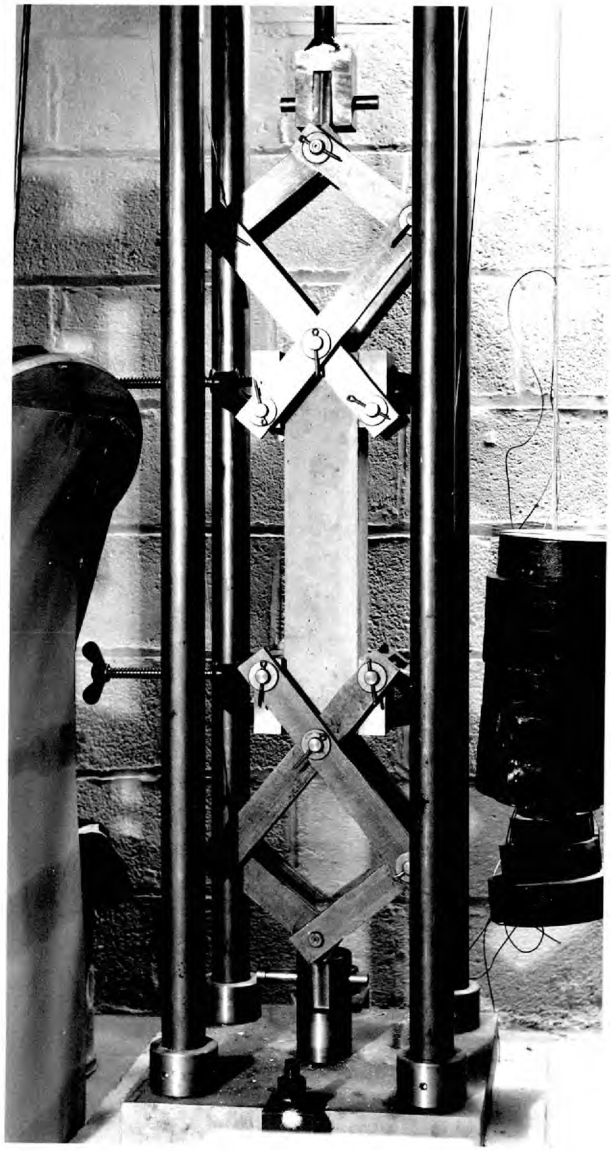


Plate 4.2 Second prototype grips and testing frame.

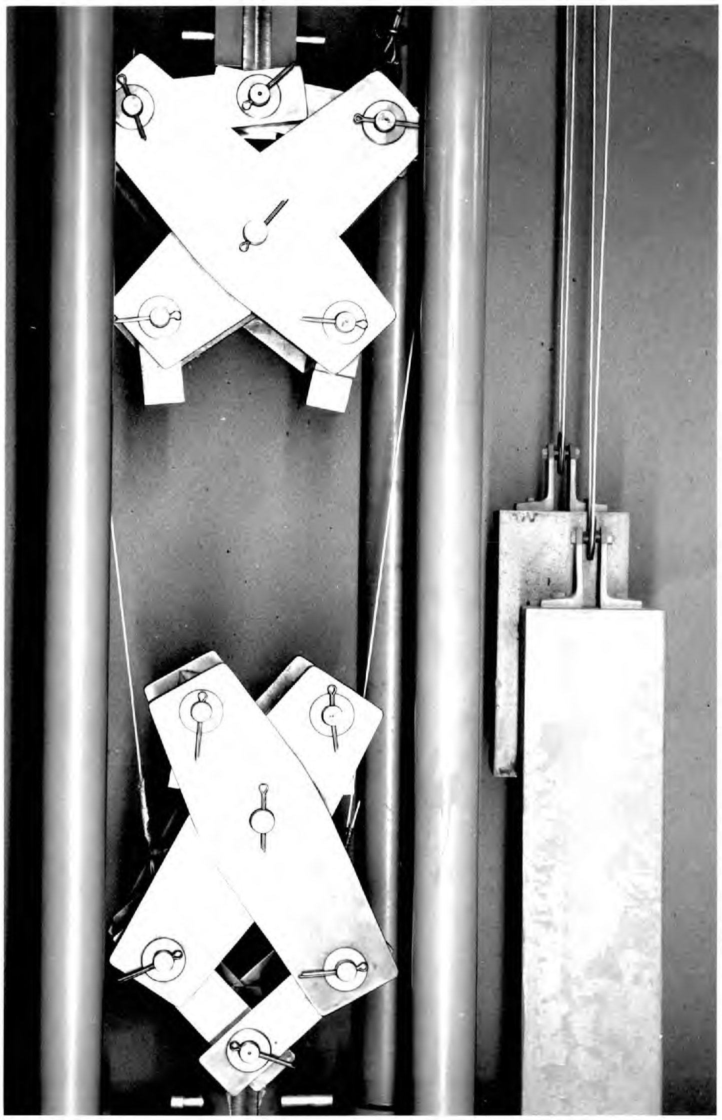


Plate 4.3 Third set of grips showing effect of counterbalance system.

During the various pilot tests conducted with the second set of grips, it was apparent that although the grips were well made there was room for improvement with regard to the construction working tolerances. Furthermore, the overall stiffness (see Table 4.1) of this system was not large and on several occasions quite violent failures were observed, particularly with the higher strength concretes. This was attributed to the large amount of strain energy stored in the grips. The final design for a third set of grips was undertaken with these points in mind. The stiffness of the grips was greatly increased and before starting their manufacture and construction considerable thought was given to the problems of alignment and to the ways in which the eccentricity developed in the specimen due to construction errors could be reduced to a minimum.

Manufacturing technique:

As a result, the following machining and assembly procedures were adopted for the construction of the third set of grips:

All similar arms in both grips (Fig.4.4, 4.5) were clamped together and the shaping and boring operations on the arms were performed simultaneously. The arms were numbered and separated from each other only after all major machining operations had been completed. The only operations performed on the arms after this were deburring and the filing of small radii on the edges of the arms to facilitate handling. As may be seen in Figs.4.6, 4.7, the grip plates had to be constructed with all the faces which were important to the operation of the grip accurately related to the main pin. When the final surface grinding was performed, all four grip platens were placed on the magnetic machine table together and were supported on their respective pins and precision blocks and all grinding on similar surfaces performed simultaneously.

The pins of the grips were made from bright steel stock and were turned so that a sliding fit (corresponding to a H7-G6 fit, B.S.S.1916) was obtained in the holes of the grip platens and arms. The exception was the main universal joint pins which were turned from oversize high

LARGE ARM

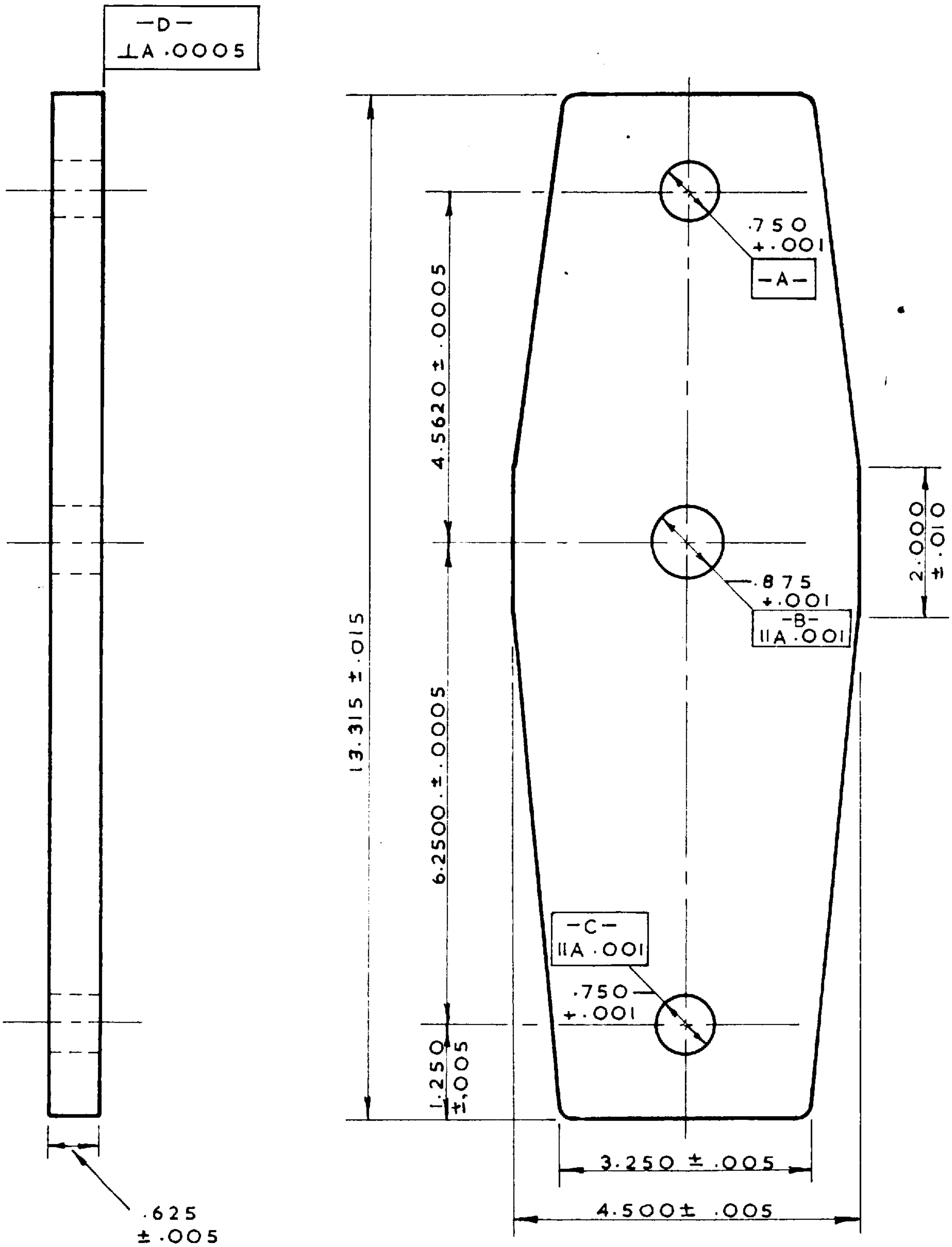


FIG. 4.4

half scale

SMALL ARM

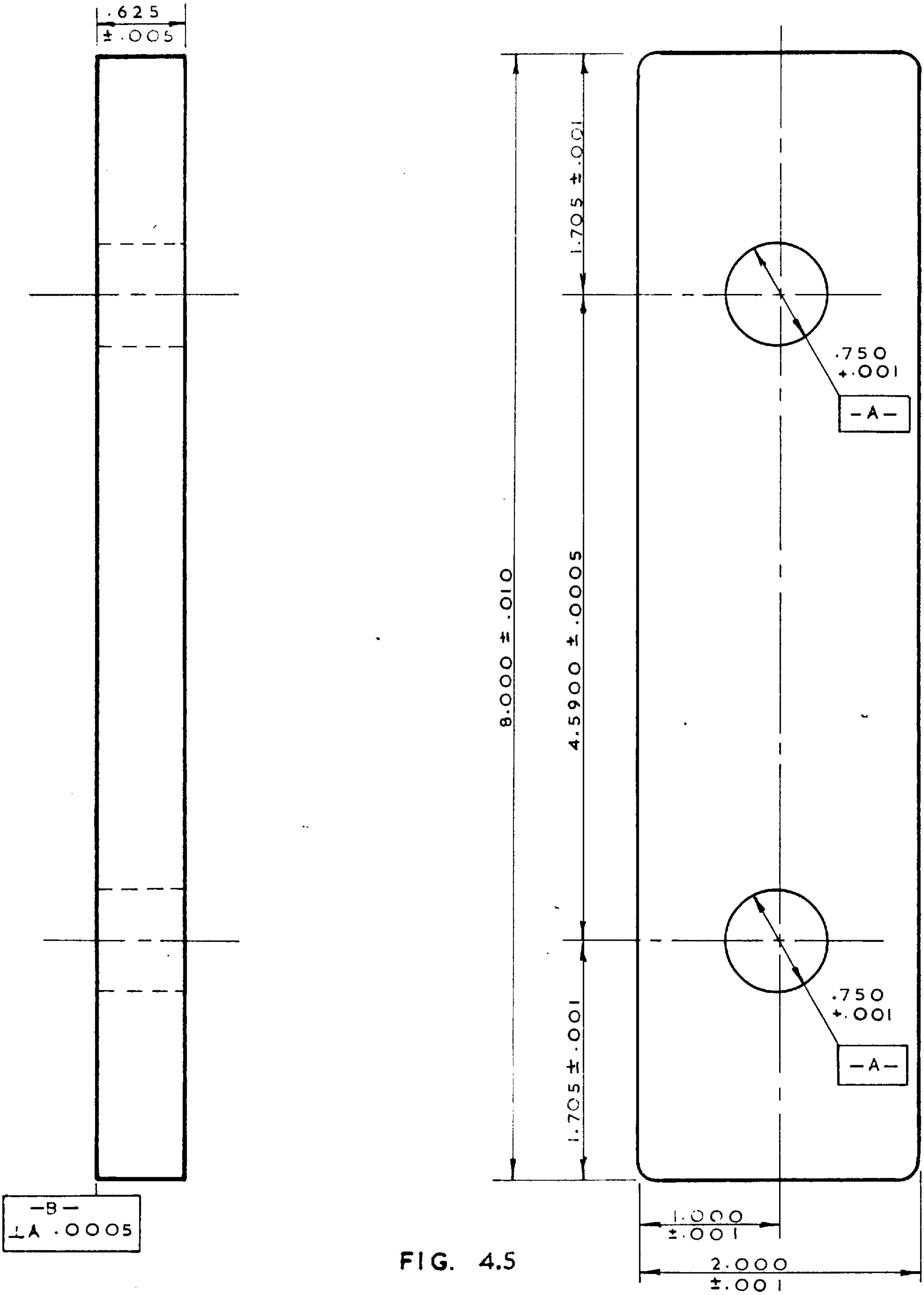


FIG. 4.5

full scale

tensile steel and then ground. The arms of the grips were maintained in their respective positions using a series of washers and cotter pins.

After all the various parts of the grips had been manufactured, each grip was assembled according to a specific plan. As mentioned previously, the arms had been numbered according to their position during machining and because of the method of alignment during the surface grinding operations, the grip plates were interchangeable. No.1 and No.3 of the large arms were connected to the small grip plate (Fig.4.6) and arms No.2 and No.4 were reversed, then connected to the opposite large grip plate (Fig.4.7) and so on (see Fig.4.9 and 4.10). This procedure ensured that there would be no inaccuracies in the test piece due to (1) slight tapering of the holes in the grip arms, and (2) non-alignment of the three holes on the large grip arms. During the final assembly, molybdenum disulphide was applied to all bearing surfaces to ensure free movement under load.

It was considered that the careful machining and assembly operations specified for the manufacture of the third set of grips enabled any possible eccentric loading effects introduced by the non-alignment of components to be ignored.

4.3.2 The Specimen

As stated in the previous chapter, one of the principal objectives of this investigation was the development of a general laboratory test, as well as a pure research test. Consequently, it was decided to adopt some form of a reduced mid-section flexural beam so that standard 4 x 4 x 20 ins. beam moulds, with only a minimum of modifications, could be utilized. These moulds are of a sturdy construction and once bolted together, maintain their dimensions. The four moulds to be modified for use in this investigation were new moulds selected from the beam moulds in the laboratory.

The grips incorporate four inches of the specimen at one end, the remaining twelve inches compared favourably in size with the 4 x 4 x 12 ins. prism compression specimen used in this investigation. This is an important consideration when comparisons are to be made between compressive and tensile strength results.

LARGE GRIP PLATEN

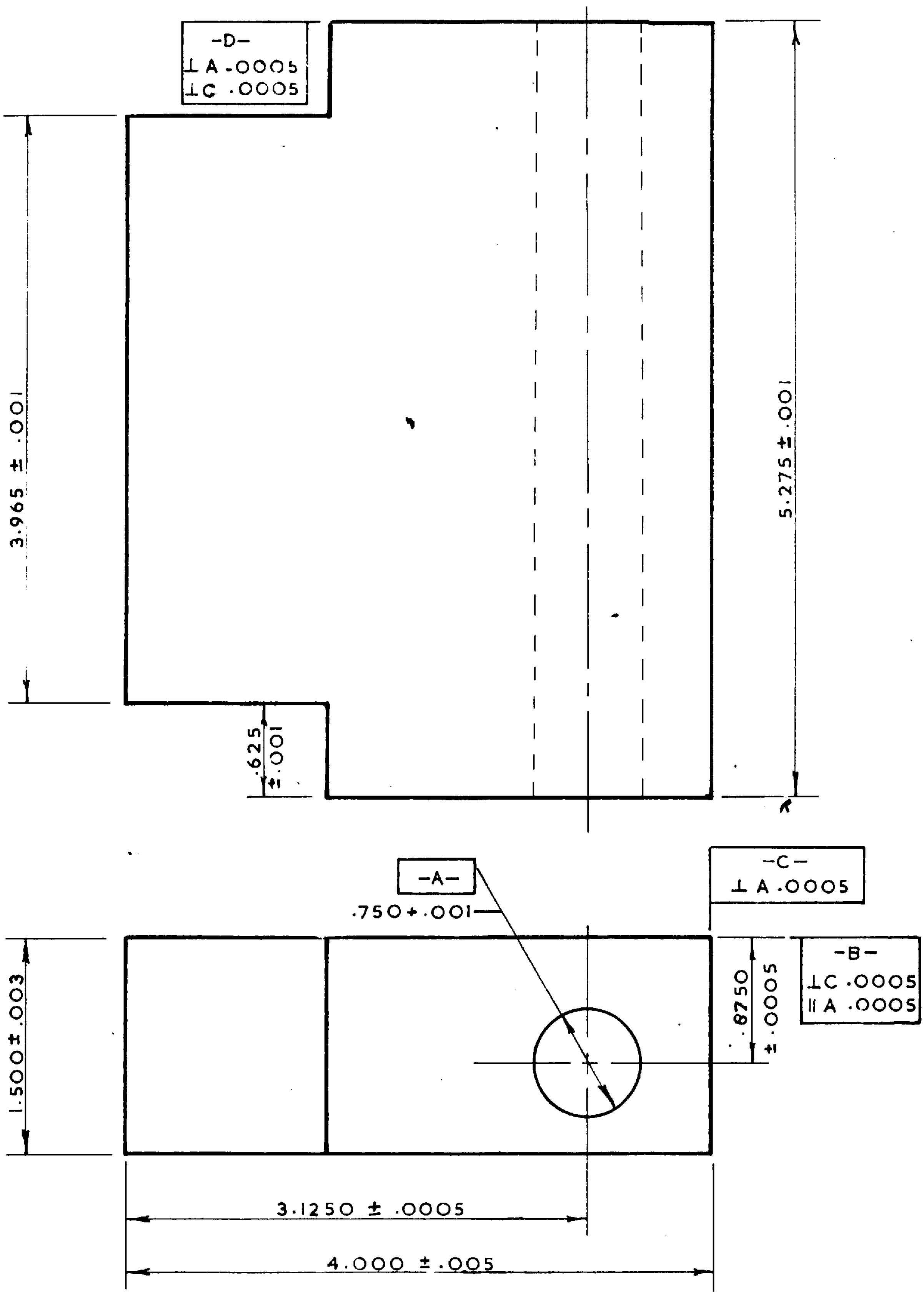


FIG. 4.6

full scale

SMALL GRIP
PLATEN

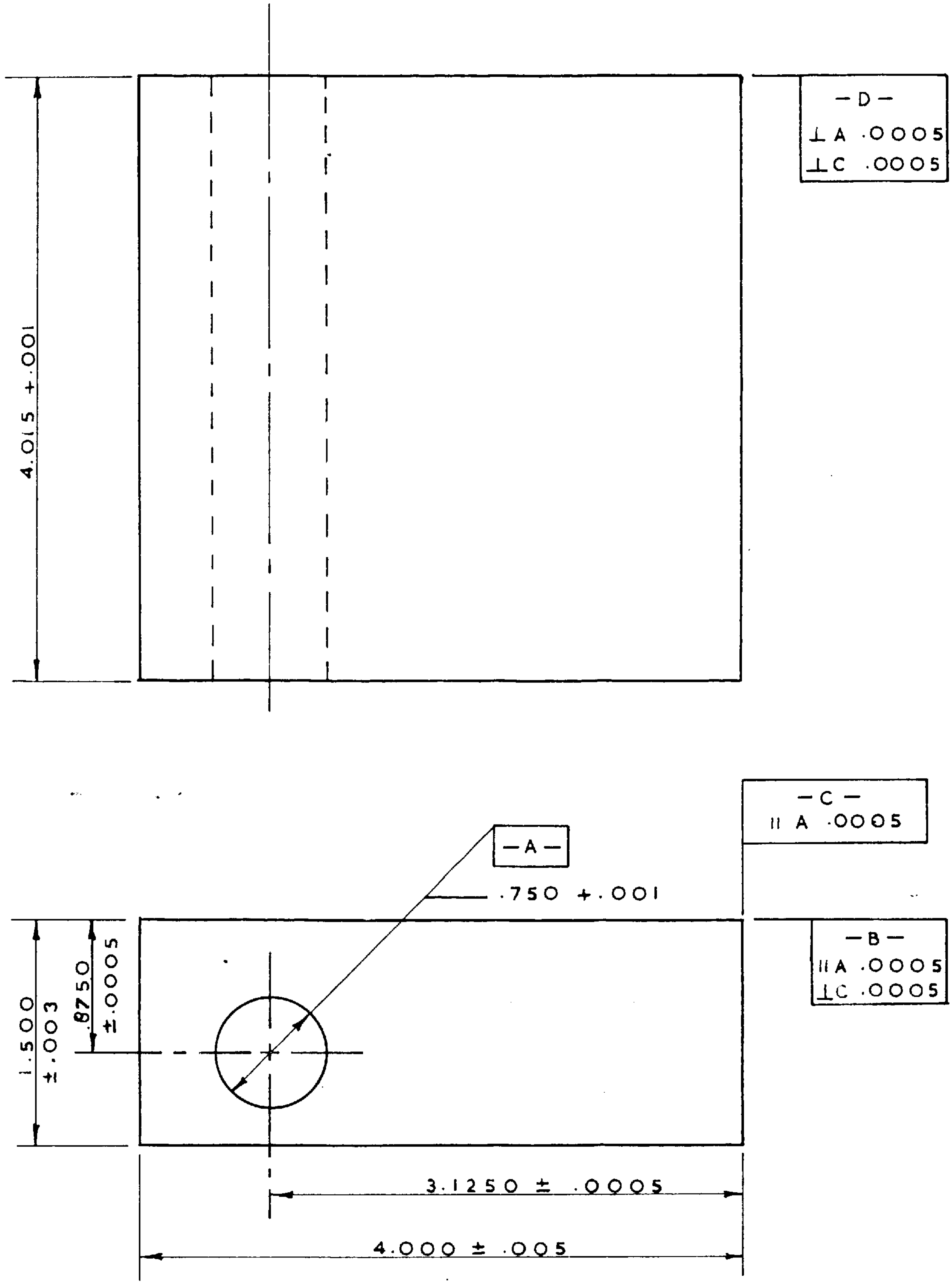


FIG. 4.7

full scale

The problem on how to reduce the central section of the specimen was considered in two parts; firstly, the cross-section of the specimen should be reduced sufficiently so that failure is ensured in the central portion; secondly, there should be no stress concentrations due to the form of the reduction. Both of these points were satisfied by the specimen shape shown in Fig.4.8. Using this shape, all the specimens tested failed in the central portion with the exception of some of the seven day mortars and all of the cement pastes in Series 3.

The reason for these failures of the gripping technique can be seen by considering Plate 4.2. It is apparent that deformation of the specimen directly under the grips is effectively restrained by the grip platens. However, the restraint effect is reduced towards the centre of the specimen. Due to the Poisson's ratio effect, the lateral normal force produces a longitudinal strain which reduces the tensile strain across the section by a variable amount due to the above mentioned restraint effect. It is evident that if the Poisson's ratio of the material or the ratio of the grip normal force to tensile force is increased, the strain relief at the central portion of the section under the grip increases. Since the total strain over the whole section remains the same, this produces an increase in strain at both edges of the specimen. It was observed from the test programme that the limiting condition for failure to occur at the grip section was reached with the third set of grips when the Poisson's ratio of the specimen was greater than 0.2. To test such materials it would be necessary to (1) change the geometry of the grips, i.e. lower the normal grip force to tension force ratio, or (2) reduce the central section of the specimen further, or (3) reinforce the specimens in the critical section at the grips. It is considered that the above points are given in their order of practicability. It is suggested however, that cement paste specimens should always be reinforced due to the high tension to compression ultimate strength ratios of these materials.

TENSILE SPECIMEN

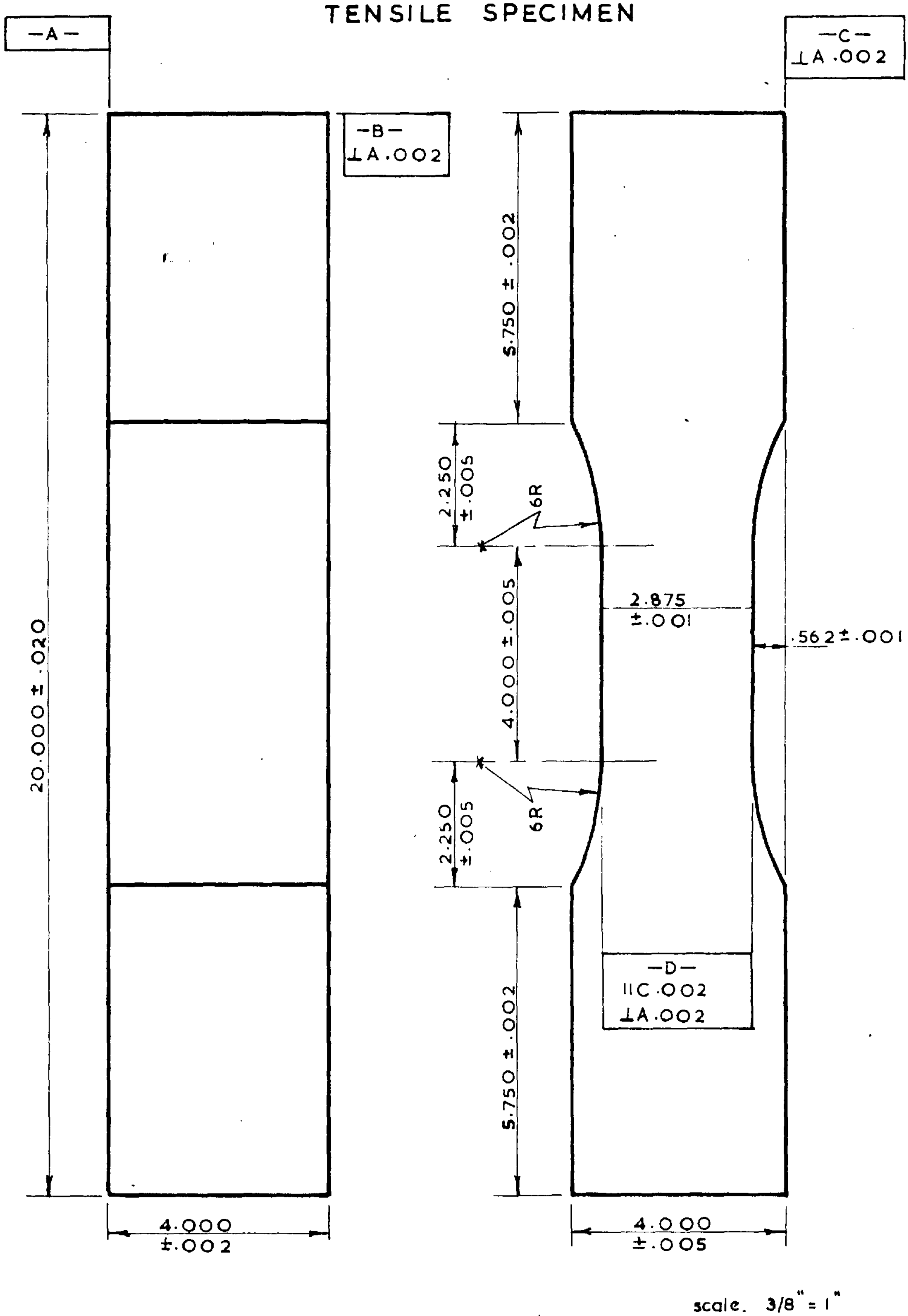


FIG. 4.8

4.4 OPERATION AND TESTING OF THE COMPLETED MACHINE

4.4.1 Operational Procedure

An important consideration for this type of machine is the ability of the operator to place the specimen into the machine easily but accurately so that there is little chance of any non-alignment of the specimen in the grips. With the gripping method used, the above criterion was satisfied by having two measurable contact checks and one visual check. The general arrangement of the specimen in the grips is shown in Fig. 4.9 and Fig.4.10.

Firstly, the specimen is lowered into the lower grip with the two small smooth and parallel faces (these are the faces which were stripped from the mould sides) resting against the grooved face of the grip plates. The lower end of the specimen now rests on the large central pin of the lower grip which locates the specimen in that direction. This is the first contact check (Fig.4.11.1).

Secondly, the bottom grip plates are pulled upwards until they come into contact with the specimen, and the counterbalance wires are centralized. The specimen is still located in one direction only, i.e. perpendicular to the axis of the central pin, and the grip plates are firmly located on the specimen sides due to the effect of the counterbalances. By tapping the specimen lightly in a direction parallel to the central pin, it is possible with either a slip gauge or visually, to locate the specimen in the centre of the main pin, i.e. with equal spaces between the specimen and the large grip arms which are connected to the small grip plate. This is the visual check (Fig.4.11.2). It should be noted that these spaces do not always exist, as they are entirely dependent on the depth of the specimen as cast.

Thirdly, the specimen is now checked for alignment of the bottom of the grip plates and the specimen, in the direction perpendicular to the axis of the main pin. This is the most important operation as any misalignment in this direction will have an effect across the section of the test piece with the lower section modulus. Because of this, the setting up procedure has been designed so that the most accurate method of checking alignment is employed in this, the

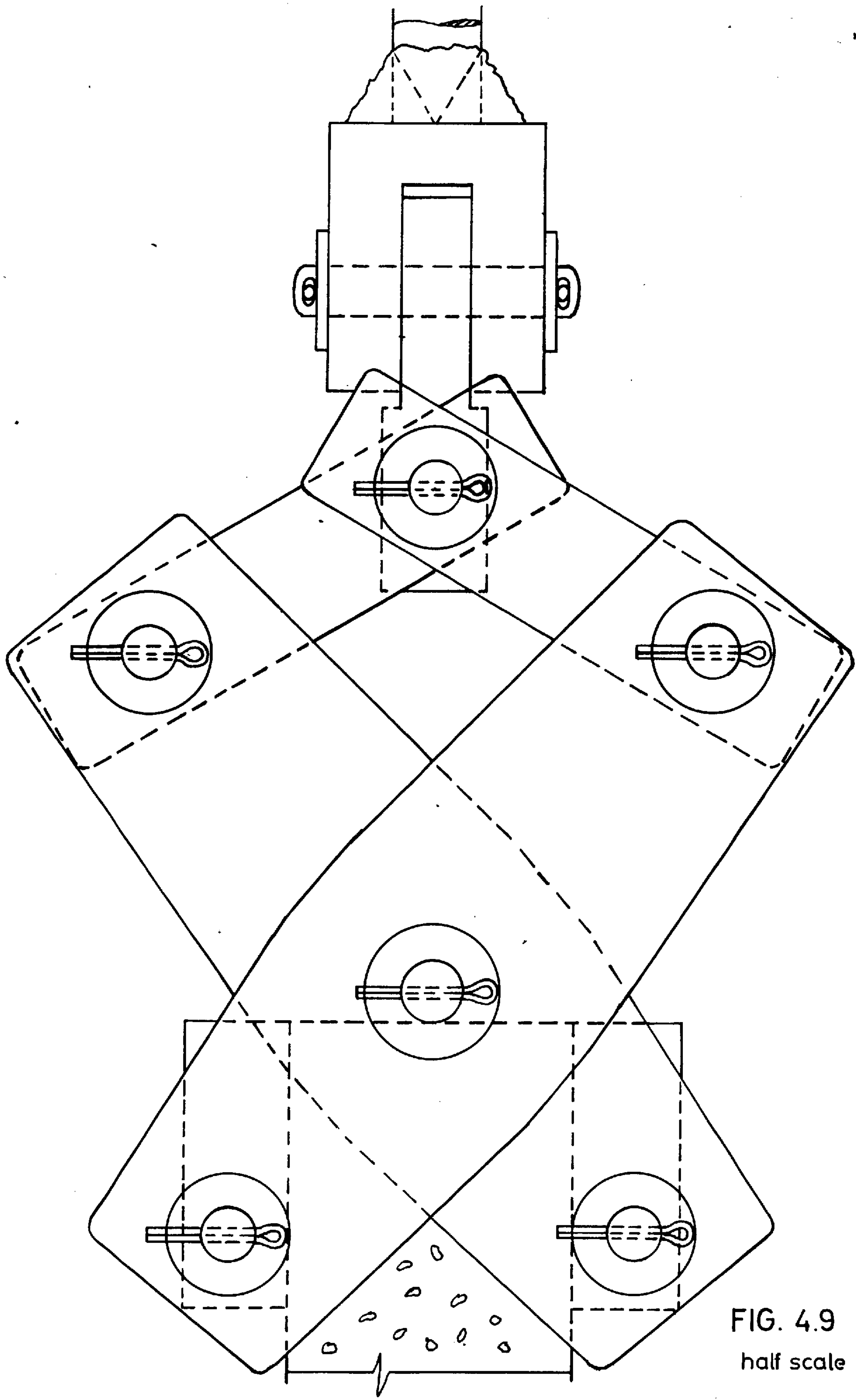


FIG. 4.9
half scale

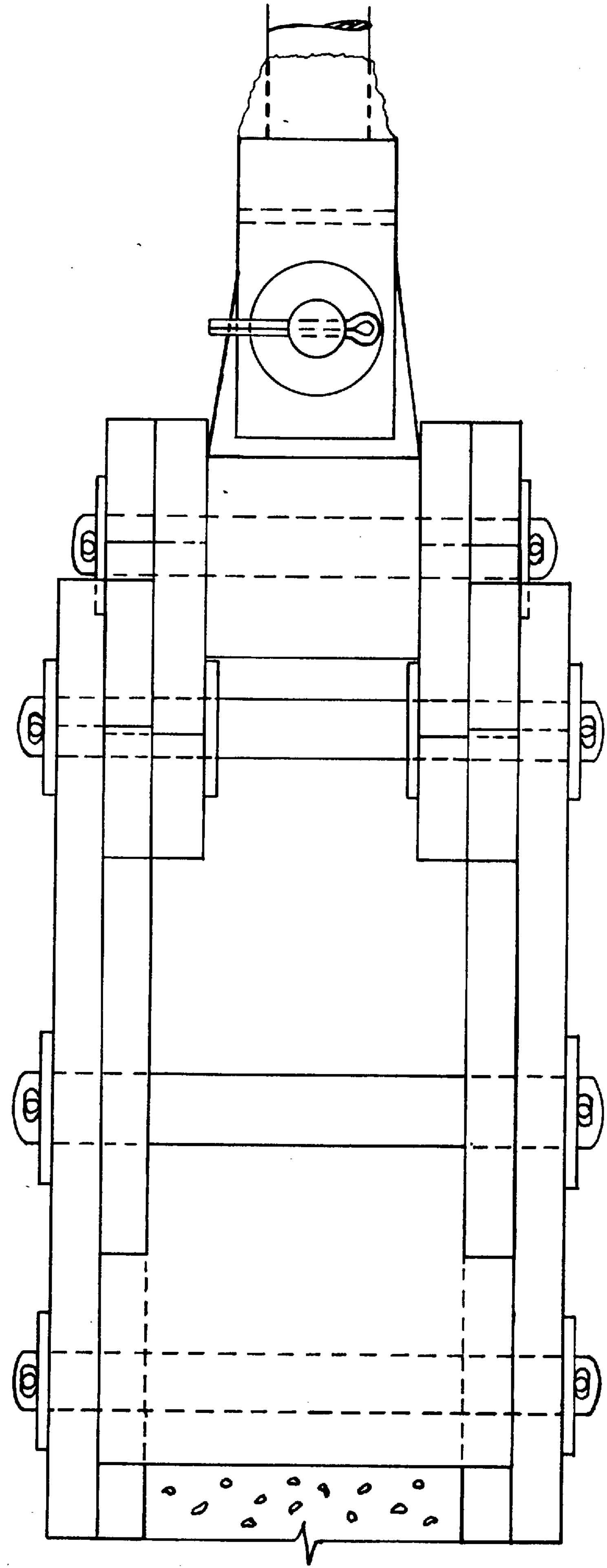


FIG. 4.10
half scale

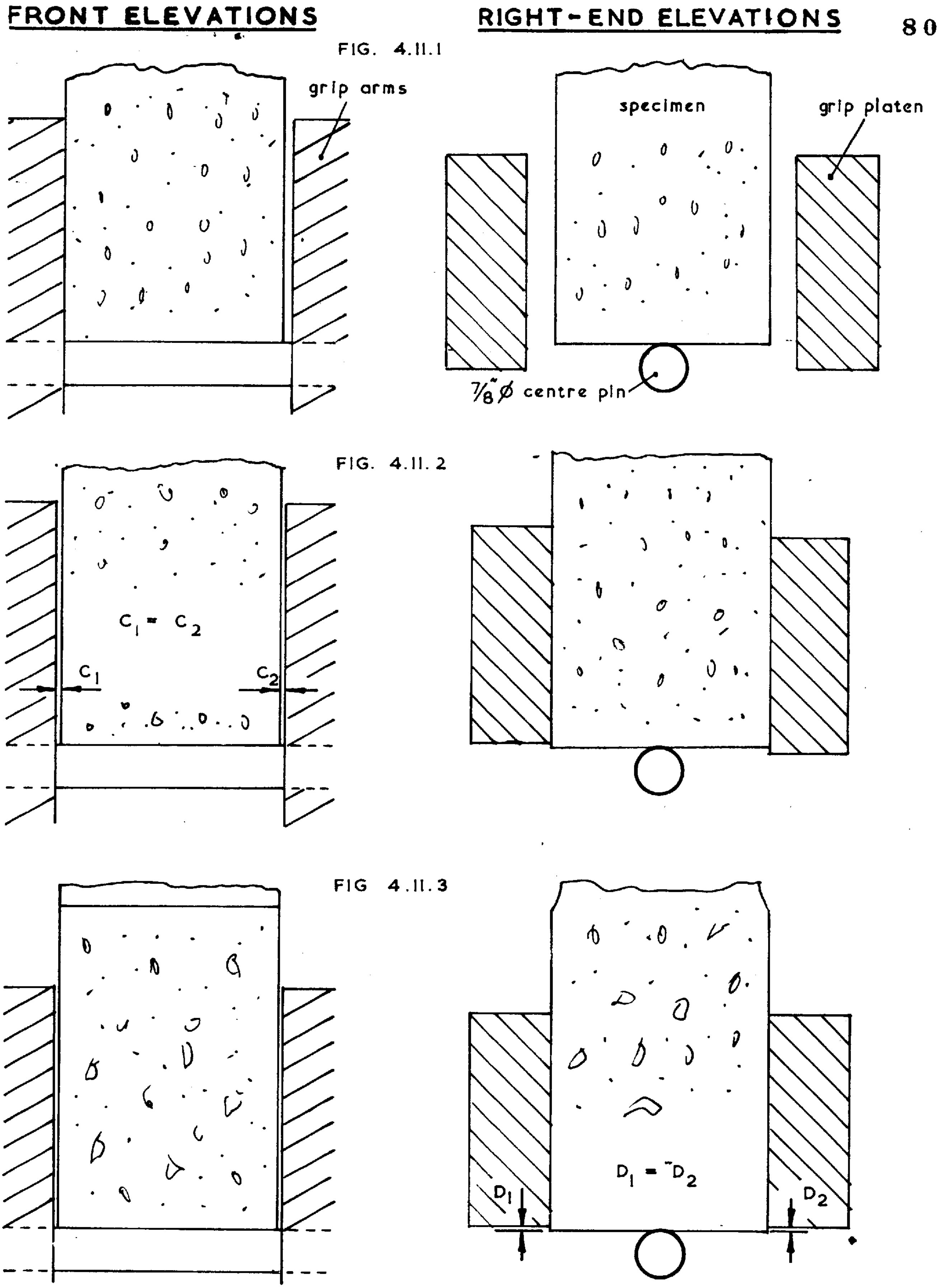


FIG. 4.11 SEQUENCE: PLACING A SPECIMEN IN THE UNI-AXIAL TENSION MACHINE.

second contact check (Fig.4.11.2). The object is to have the same distance between the surface of the specimen, resting on the main pin, and the two ground surfaces of the grip plates. This is accomplished by feeling simultaneously both sides of the projecting edge of the specimen. Adjustments are made by tapping lightly on the specimen in a direction perpendicular to the central pin. With very little practice it is possible to obtain consistent positioning of the specimen such that with this method, the difference between d_1 and d_2 (see Fig.4.11.3) is always less than 0.005 ins.

Finally, the three checks are repeated to ensure that the specimen is located accurately. A clamp is then fixed to hold the two grip plates rigidly against the specimen. A similar procedure is adopted for the top grip. Both clamps can be removed after a small initial load has been applied to the specimen. The total time required to place the specimen in the machine and locate both grips is between three and four minutes.

Using the above procedure, it is possible to locate the specimen very accurately in relation to the applied load. Calculations based on an assumed eccentricity of 0.005 ins., a value which is easily detected by the method of setting up, give a maximum difference in strain of 4 per cent between opposite faces of the specimen in the direction with the lower section modulus. As will be seen, this strain difference is twice as large as any obtained experimentally when due care was taken in placing the specimen in the testing machine.

4.4.2 Testing of the Completed Machine

Eccentricity:

As has been shown theoretically, the loading grips and the method of placing the specimen in the machine should not produce any significant eccentric loading in the test pieces. A series of tests were carried out on the direct tension testing machine to verify experimentally these theoretical conclusions.

Plate 4.4 shows the tensile apparatus fitted with an aluminium 4 x 4 x 20 ins. calibration beam ready for test. One inch electrical

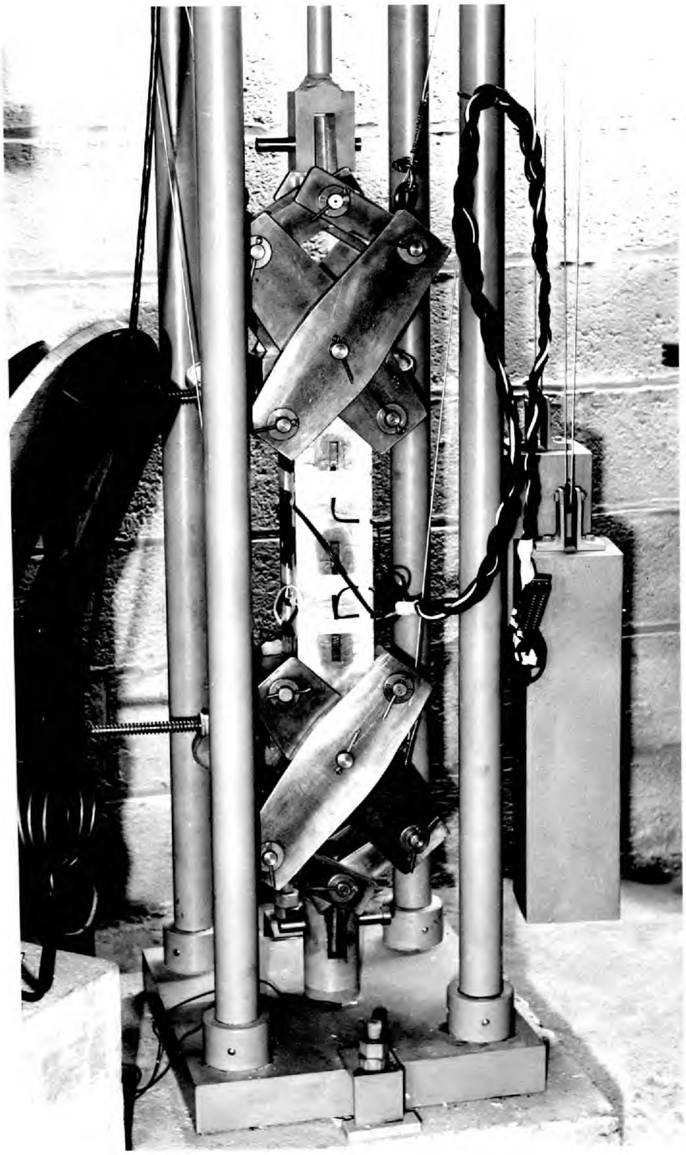


Plate 4.4 Third set of grips holding the aluminium calibration specimen.

resistance strain gauges mounted around the central portion of the beam registered the peripheral longitudinal strains. The specimen was loaded to 1600 p.s.i.g.(which is equivalent to approximately 970 p.s.i. on the reduced section of a concrete specimen) eight times over a ten day period. During this period, the beam was rotated through 180 degrees and inverted between tests to nullify gauge effects. When the specimen was located in the machine with due care, it was observed from the strain readings that the maximum difference in strain recorded between any two faces was 3.4 per cent (see Table 4.2).

In a second calibration test, a one year old dry mortar specimen with a 1 ins. electrical resistance strain gauge mounted on each of the four sides on a plane at the mid-section of the reduced section gave similar results to those obtained using the aluminium calibration beam. It was observed from the test results that the maximum difference in strain between any two faces, when the correct setting up procedure was followed, was 2.7 per cent (see Table 4.3). The test piece was rotated and inverted between tests.

For a third test, two 28 day old saturated mortar specimens from the same mix were tested to failure with no preloading. A one inch electrical resistance strain gauge was mounted on each of the two faces corresponding to the extreme fibres of the mid-section of the reduced specimen with the lower section modulus. Using the Solartron Data Logger and its continuous scan facility which was engaged at 80 per cent of the ultimate load of the specimens, the strain was recorded continuously to failure.

It can be seen from Table 4.4 that for both specimens the strain on opposite faces was essentially the same until about 85 per cent of the ultimate load had been reached. In addition to these tests, similar results were obtained from the pilot tests in Series 4 where electrical resistance strain gauges were mounted on opposite faces of various concrete and mortar tension test specimens. Furthermore, on tests for other research students conducted by the writer, the limits of eccentricity described herein were never exceeded.

Table 4.2 Typical results obtained from aluminium specimen with careful set-up procedure.

Load psig	No.5		No.6		No.7		No.8	
	(1)	(2)	(1)	(2)	(1)	(2)	(1)	(2)
May 28,1962:								
0	0	0	0	0	0	0	0	0
200	9.5	9.0	8.5	9.0	9.0	8.0	8.0	8.5
400	17.0	17.5	17.0	17.5	17.0	16.5	16.0	17.0
600	25.5	26.0	25.0	26.5	25.5	25.5	24.5	25.0
800	34.0	35.0	34.0	35.0	34.0	33.5	33.0	34.0
1000	42.5	43.5	42.0	44.0	42.5	42.5	41.5	42.5
1200	51.5	52.5	51.0	52.5	52.0	51.5	49.5	50.5
1400	60.5	61.5	60.0	62.0	61.0	60.5	59.0	60.0
May 25,1962:								
0	0		0		0		0	
200	10.0		9.0		9.0		8.5	
400	18.0		17.5		17.5		17.5	
600	26.0		25.5		25.0		25.5	
800	35.0		35.0		34.0		34.5	
1000	43.5		43.5		42.5		43.0	
1200	52.0		52.5		51.5		52.0	
1400	61.0		62.0		60.5		61.0	

All above gauge readings are in microstrain.

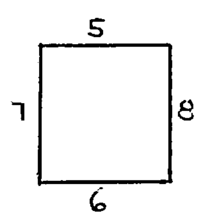


Table 4.3 Typical results with mortar calibration specimen with careful set-up

Nov.5, 1962

Mortar specimen No.1, specimen loaded 3 times

Load(p.s.i.g.)	1	2	3
Gauge No.1:			
0	0	0	0
100	15.0	15.0	15.0
200	26.8	26.5	26.5
300	39.0	38.8	39.0
400	51.5	51.0	51.5
500	64.2	64.0	64.2
Gauge No.2:			
0	0	0	0
100	15.0	14.0	14.5
200	26.5	26.2	26.5
300	39.0	38.8	39.0
400	52.0	51.8	52.0
500	64.8	64.8	64.8
Gauge No.3:			
0	0	0	0
100	16.0	15.5	16.2
200	27.5	27.2	27.5
300	39.5	40.0	40.0
400	52.0	52.5	52.5
500	64.8	65.0	65.0
Gauge No.4:			
0	0	0	0
100	15.0	15.0	15.0
200	26.5	26.2	26.8
300	39.5	39.2	39.8
400	52.5	52.2	53.0
500	66.5	65.8	66.2

All above gauge readings are in microstrain.

Check on maximum readings at 500 p.s.i.g. -

Gauge No.1 : 0 to 64.1
No.2 : 0 to 64.8
No.3 : 0 to 64.9
No.4 : 0 to 66.1

Table 4.4 Stress-strain relationships to failure of two mortar tensile specimens.

Strain gauges were mounted on opposite extreme surfaces corresponding to the lowest section modulus of the specimen. Results obtained using Solartron Data Logger.

stress	Specimen No.1		Specimen No.2	
p.s.i.	Gauge No.1	Gauge No.2	Gauge No.1	Gauge No.2
0	0	0	0	0
100	22	22	20	22
131	28	30	30	32
162	40	40	36	38
190	42	44	42	44
220	52	52	50	50
250	56	56	52	54
280	64	64	60	62
310	74	76	74	72
340	80	82	76	76
370	86	88	80	84
400	92	96	91	92
430	104	106	104	100
460	112	112	110	110
490	122	122	116	116
520	124	128	130	128
526	122	122	-	-
532	122	124	-	-
538	124	124	-	-
544	124	126	-	-
550	126	128	140	138
556	130	130	134	138
562	130	132	134	138
568	132	132	136	146
574	134	138	136	148
580	134	140	140	150
586	140	142	142	154
592	140	146	146	158
598	142	148	146	162
604	146	148	150	166
610	146	150	156	170
616	150	152	156	174
622	152	156	160	178
628	154	158	164	180
634	158	162	170	186
640	162	162	172	188
646	164	164	176	194
652	166	162	180	198
658	170	160	184	204
664	172	156	182	214
670	168	106	-	-

Rigidity:

In order to check for any horizontal movement in the frame, an ancillary support framework was rigidly connected to the bottom base plate of the testing machine. Five 0.0001 ins. dial gauges were mounted on this framework to check for any horizontal movement of the top plate in relation to the bottom plate. Two high strength mortar specimens were loaded to failure under these conditions. No perceptible movement was detected on any of the five gauges. This confirms that the machine has sufficient lateral rigidity.

4.5 SUMMARY

The design and construction of the tension machine frame, the grips, and specimens have been described in detail. The longitudinal and lateral rigidity of the testing machine has been analysed, from which it has been shown that the machine and loading system is capable of loading to failure by constant stress rate but not constant strain rate. The introduction of lubricated universal joints to connect each grip to the loading system results generally in the line of the applied force passing through the centre of the specimen, even with moderately anisotropic specimens.

It has been shown theoretically that as a result of the careful machining and assembly operations specified for the manufacture of the components of the tension testing machine, the investigation could ignore eccentric loading effects introduced by the non-alignment of components. Furthermore, tests have been described which verified experimentally this theoretical conclusion.

The test specimen, which is a reduced mid-section flexural beam, has been shown to be of a suitable form in that failure is induced in the reduced central portion and stress concentration effects are negligible. Moreover, the overall size is adequate but not unwieldly.

CHAPTER 5

OUTLINE OF EXPERIMENTAL WORK

5.1 INTRODUCTION

The tests carried out by the writer in this work were designed to provide the following information:

(1) the variance of the new direct tension testing method when compared with destructive and non-destructive tests,

(2) quantitative information on the behaviour of concretes containing different coarse aggregates with a variety of aggregate/cement ratios and water/cement ratios,

(3) the effect of varying some of the mix parameters on the longitudinal and lateral stress-strain relationships in the uni-axial tension and compression tests,

(4) the effect of rate of loading and moisture condition on the behaviour of concretes and mortars tested to failure in uni-axial tension.

This chapter describes the type of specimens used and the tests performed on them. A complete description of the notation used in numbering the specimens in each series and a detailed description of the mix design and test method for all test series is given in Chapter 6.

5.2 TEST SERIES 1

The first test series was carried out primarily to establish the variance of the new direct tension test in terms of the ultimate strengths of mortar specimens, in comparison with the variance observed from results of other destructive tests. During this series, equivalent cube and indirect tension tests were carried out on the broken ends of the direct tension and flexural specimens, to establish the usefulness of such a technique.

The mix chosen was a mortar with a sand/cement ratio at 2.0 and a water/cement ratio of 0.45 by weight. This mix has been adopted as the standard 'calibration' mix in the laboratory for this and later investigations. Since it is a very cohesive mortar which can be easily compacted without segregation, the test results obtained from it are very consistent from batch to batch.

Thirteen batches, each containing eighteen specimens, were cast and tested over a period of four months. The tests performed on the specimens were as follows:

<u>Destructive tests</u>:

3 direct tension tests on reduced section specimens 4 x 4 x 20 ins.

3 indirect tension tests on one broken direct tension end.

3 equivalent cube tests on the other broken direct tension end.

3 flexural tests on 4 x 4 x 20 ins. beams.

3 indirect tension on one broken flexural beam end.

3 equivalent cube tests on the other broken flexural beam end.

3 indirect tension tests on 4 ins. cubes.

3 compression tests on 4 ins. cubes.

<u>Non-Destructive tests:</u>

The following non-destructive tests were performed on the 4 x 4 x 20 ins. beams before they were failed in flexure.

3 density measurements.

3 longitudinal resonant frequency measurements.

3 ultrasonic pulse velocity measurements.

Tests were carried out at 7 and 28 days and all specimens were tested in a saturated condition. In this series, a total of 429 destructive and non-destructive tests were performed.

<u>5.3 TEST SERIES 2</u>

The object of this, the largest test series undertaken in the present work, was to obtain quantitative results on the inter-relation of various testing methods using concretes with different coarse aggregates and a variety of mix parameters.

Four coarse aggregates were used: Montsorrel Granite, Crushed Gravel, Leca lightweight aggregate, and Thames Valley Natural River Gravel. Mixes with three different aggregate/cement ratios and two water/cement ratios at each aggregate/cement ratio were made for each aggregate type. The mortars from the concretes were also tested as separate mixes. Compacting factor and slump tests were performed generally in accordance with B.S.1881 at each casting. In all, thirty batches of concrete and mortar were cast and tested over a period of four and a half months. Each casting contained eighteen specimens and the tests performed on these specimens were as follows:

Destructive tests:

3 direct tension tests on 4 x 4 x 20 ins. reduced section specimens.

3 flexural tests on 4 x 4 x 20 ins. beams.

3 indirect tension tests on 6 ins. diameter x 9 ins. cylindrical specimens.

3 compression tests on 4 ins. cubes.

3 compression tests on 6 ins. cubes.

3 compression tests on 4 x 4 x 12 ins. prisms.

Non-Destructive tests:

These tests were performed on the 4 x 4 x 20 ins. beams before they were failed in flexure.

3 density measurements.

3 longitudinal resonant frequency measurements.

3 ultrasonic pulse velocity measurements.

All the above tests were carried out at 28 days on saturated specimens. A total of 810 destructive and non-destructive tests were performed during the test series.

5.4 TEST SERIES 3

In the third test series, longitudinal and lateral strain measurements were taken up to failure using an overall gauge length of 4 ins. on 4 x 4 x 12 ins. prism compression specimens and on 4 x 4 x 20 ins. reduced mid-section direct tension specimens. Non-destructive tests including density determination, ultrasonic pulse velocity, and the resonant frequency tests were carried out on standard beam specimens before they were failed in flexure.

The mix parameters were chosen from those used in test series 2. The two lower aggregate/cement ratios and the corresponding water/cement ratios of this earlier test series were adopted with the addition of a new water/cement ratio midway between the two used previously. The investigation however, was restricted to the Thames Valley River Gravel coarse aggregate, the mortar of each concrete mix, and the cement paste of each mortar.

Tests were performed at 7 and 28 days on saturated specimens. Each casting contained 3 flexural beams, 4 reduced section direct tension specimens, and 4-4 x 4 x 12 ins. prism specimens. Two direct tension and two compression specimens were tested ~~and two compression specimens were tested~~ at each age. The non-destructive test specimens were replaced in the curing tank after the 7 day tests for further use at 28 days.

A total of 289 destructive and non-destructive tests were performed during the course of the third test series, over a period of three and one half months.

5.5 OTHER TESTS

Further pilot tests were performed after the completion of the main test series in order to obtain some information on the effect of the moisture condition of the specimen and the rate of loading on the stress-strain relationships and failing loads of concrete and mortars. Although only tentative conclusions can be drawn from these preliminary tests, the results are of interest and are therefore included in this work.

Firstly, 8 direct tension mortar specimens were tested to failure at age 28 days after various systems of curing. Secondly, 4 direct tension mortar specimens and 4 direct tension concrete specimens were tested in a saturated condition at 28 days at different rates of loading up to failure.

Strain measurements were made with Lamb's roller extensometer for the moisture condition tests, whereas electrical resistance strain gauges and the Solartron Data logger were used for the rate of loading tests.

CHAPTER 6

MANUFACTURE AND TESTING OF SPECIMENS

6.1 INTRODUCTION

The source and a description of the materials used in the manufacture of the test specimens for this investigation is given. The manufacturing methods and the curing techniques employed are also described. The description of the testing methods used for both non-destructive and destructive tests and the relation of these methods to the relevent British Standard and American Society for Testing Materials Specifications is discussed. The calibration of equipment used is described and the possible errors involved in the measurement of load and strain stated.

Considerable amount of detail will be given but it is suggested that this detail is justified since several of the techniques adopted for testing and calibration are not standard and were being used in the Imperial College laboratory for the first time.

6.2 MATERIALS

6.2.1 Cement

Ordinary Portland Cement supplied by the Cement Marketing Company from their Kent works, was used for all mixes.

Enough bagged cement for each test series was blended, then stored in air tight steel drums until used. Care was taken that the bagged cement for each series came from the same cement batch,- identification being made from the batch tag on each cement bag.

Cement Quality:

The quality of cement used in this investigation was assessed by means of 2.78 ins. mortar cubes tested in compression in accordance with B.S.12:1958, the average values of which are given in Table 6.0. The tests were carried out on cement representative of the cement used in all test series.

Table 6.0 Compressive Strength of 2.78 ins. Cement Cubes

Age (days)	cement lot 1 test series 1 and 2	cement lot 2 test series 3
7	5700	5600
14	6650	6750
28	7500	7650

each result is the average of three specimens.

It is seen that no significant difference was obtained between lots.

6.2.2 Water

Potable water drawn from the Imperial College mains supply was used for all mixes.

Since a 24 hour aggregate pre-soaking period was used (see 6.3.1), the temperature of the mix water had stabalized to the ambient temperature of the laboratory by the time it was incorporated with the cement in the mixer. This temperature was 70°F. ± 3°.

6.2.3 Fine Aggregate

The sand used for all mixes was a natural Thames Valley river sand supplied by the Stone Court and Ballast Company from their Rickmansworth Pit. Sufficient sand for each test series was in storage at the start of each casting program.

To prepare the sand for each mix, the following procedure was adopted:

> The sand was heated slowly on a 2 Kw. drying pan to a temperature less than 100°C. and subsequently brought

to a condition approaching "bone dry"[68]. Then the sand was passed through a 3/16" sieve and stored in steel bins with lids.

As a quantity of the prepared material was stored for a period before it was required, the temperature of the material was that of the ambient temperature of the laboratory (70°F.± 3°) when finally used.

A sieve analysis was made once during each mix series and a negligible change in grading was recorded between series. A sample analysis taken from the second test series is given in Table 6.1. The sand conforms to the Zone 2 limits in the current British Standard sand gradings[69].

Table 6.1 Sand Grading - Series 2

B.S. sieve size	Cumulative per cent passing
3/16	100
7	84
14	60
25	39
52	10
100	1

F.M. = 3.06

6.2.4 Coarse Aggregates

A similar procedure to that adopted for the fine aggregates was used in the preparation of the coarse aggregates.

After drying, the coarse aggregates were sieved through a 3/4" sieve before being stored in steel bins with lids.

A sieve analysis was made during each series for each aggregate type. An exception was the light weight aggregate. This material which consisted of particles with different densities at each particle size did not lend itself to such an analysis. The aggregate was received in two sizes, coarse - 3/4" to 3/8", and medium - 3/8" to 3/16", which was recombined to provide the light weight coarse aggregate using two bags of coarse mixed with one bag of medium. These proportions were recommended by the manufacturer, W.C. French Ltd., to give a volumetric grading similar to that in use for normal coarse aggregates in concrete.

(a) Thames Valley Natural River Gravel

This aggregate is a natural river gravel, typical of that obtainable in the London area, containing little crushed material and classified as to particle shape as a rounded and irregular material. It was supplied from the Rickmansworth Pit of the Stone Court and Ballast Company.(92)

Table 6.2 - Sieve Analysis Natural Thames Valley River Gravel

B.S. sieve size	Cummulative per cent passing
3/4	99
3/8	7
3/16	0

(b) Granite Crushed Stone

Supplied by Mountsorrel Granite Co. Ltd. of Leicester from their Mountsorrel Quarry.(92) This aggregate was 100 per cent crushed material, and would be classified as to particle shape as angular.

Table 6.3 - Sieve Analysis Granite Crushed Stone.

B.S. sieve size	Cummulative per cent passing
3/4	100
3/8	3
3/16	0

(c) Crushed Gravel

The crushed gravel was supplied by J. Cross and Sons Ltd. of Wolverhampton from their Four Ashes Pit[92] and contained angular quartz and quartzite.

Table 6.4 - Sieve Analysis Quartzite Crushed Stone

B.S. sieve size	Cummulative per cent passing
3/4	100
3/8	6
3/16	0

(d) Lightweight Aggregate

W.C. French Co. Ltd. supplied the "Leca" expanded clay lightweight aggregates. Two sizes were supplied and were blended before using to give a nominal 3/4" - 3/16" grading. This was accomplished by combining one bag of medium aggregate (3/8 - 3/16) to two bags of coarse aggregate on the manufacturers instructions.

6.2.5 Description of Mixes

Throughout this next section, continual reference will be made to the water/cement ratio of a mix. This is defined as the ratio between the water in a mix over and above that required for the aggregate absorption and the total amount of cement.[68]

(a) Series 1

The first test series was undertaken primarily to ascertain the relative variance of the different destructive testing techniques, including the direct tension test. Since the properties of a low workability mortar are more easily reproduced from mix to mix, a mortar was chosen as the standard mix of this study. Using this mortar, thirteen mixes were cast.

Standard Mix:

Aggregate/cement ratio = 2.0 (by weight)

Water/cement ratio = 0.45 (by weight)

All aggregate passing 3/16" sieve.

Specimens Cast:

4 flexural beams, 4 x 4 x 20 ins.

3 direct tension beams; reduced flexural beams

3 6" cubes

4 4 x 4 x 12" prisms

6 4" cubes

3 6"ϕ x 9" cylinders

All specimens were loaded perpendicular to the direction of casting.

Identification:

No special identification system was used for this series.

(b) Series 2

The main object of the second series was to investigate the effect of varying aggregate/cement ratios, water/cement ratios and aggregate types on the relationships between the various destructive tests including the direct tension test.

Table 6.5 - Mixes for Thames Valley River Gravel aggregate test Series 2

W/C ratio \ A/C ratio	4.5	6.0	7.5
0.425	1C1	-	-
0.500	2C1	3C1	-
0.600	-	4C1	5C1
0.675	-	-	6C1

Specimens Cast:

3 flexural beams, 4 x 4 x 20 ins.

3 direct tension beams; reduced flexural beams

6 6" cubes

6 4" cubes

3 4 x 4 x 12" prisms

3 indirect tension specimens, 6"Ø x 9" cylinders.

2 12 x 12 x 18" density specimens.

Identification of Specimens:

Table 6.5 is an example of the specimen marking for this test series. All specimens were identified by a letter preceeded and followed by a number. The letter denotes the coarse aggregate type.

A - crushed gravel   C - Thames Valley River Gravel

B - granite   D - lightweight aggregate

The number prefix denotes the mix parameters:

Table 6.6 - Specimen notation for the concretes of test series 2

number prefix	A/C ratio	W/C ratio
1	4.5	0.425
2	4.5	0.500
3	6.0	0.500
4	6.0	0.600
5	7.5	0.600
6	7.5	0.675

The number suffix denotes the specimen number in the mix. The specimens were numbered consecutively from 1 - 27, e.g. 19, 20, and 21 are the flexural beams.

Mix E represents the mortar mix of all the concretes, i.e. the concrete with the coarse aggregate omitted. The number prefix to the E mixes designates the same water/cement ratio as above but different aggregate/cement ratios.

Table 6.7 - Specimen notation for the mortars of test series 2

number prefix	A/C ratio	W/C ratio
1	1.8	0.425
2	1.8	0.500
3	2.4	0.500
4	2.4	0.600
5	3.0	0.600
6	3.0	0.675

(c) Series 3

The mixes in Series 3, which wee used to investigate the deformational behaviour of concrete materials, are similar to mixes C in Series 2, that is, they contained only Thames Valley River Gravel as a coarse aggregate. However, additional mixes were added by an interpolation between the water/cement ratios at the given aggregate/cement ratios and the higher aggregate/cement ratio mixes of Series 2 were dropped.

Mixes:

Table 6.8 - Mixes for Thames Valley Natural River Gravel aggregate of test Series 3

W/C ratio \ A/C ratio:	4.5	6.0
0.425	1C	-
0.4625	MC	-
0.500	2C	3C
0.550	-	NC
0.600	-	4C

Specimens cast:

4 direct tension specimens, reduced flexural beams

4 4"x 4"x 12" prisms

4 flexural beams, 4 x 4 x 20 ins

Identification of Specimens:

The same basic system was used as with Series 2, Table 6.5. The additional water/cement ratios are indicated using the prefixes M and N as shown in Table 6.9

In addition, cement paste mixes of the same water/cement ratio as that used in the mortars were also made. These mixes were denoted by the letter P.

Table 6.9 - Complete mixes test Series 3

mix designation	A/C	W/C
1C	4.5	0.425
MC	4.5	0.4625
2C	4.5	0.500
3C	6.0	0.500
NC	6.0	0.550
4C	6.0	0.600
1E	1.8	0.425
ME	1.8	0.4625
2E	1.8	0.500
3E	2.4	0.500
NE	2.4	0.550
4E	2.4	0.600
1P	-	0.425
2P	-	0.4625
3P	-	0.500
4P	-	0.550
5P	-	0.600

6.3 MANUFACTURE OF SPECIMENS

6.3.1 Preparation of Aggregates

To obtain an effective and repeatable water/cement ratio, all the previously dried aggregates were totally immersed in water for 24 hours before casting. This ensured that complete saturation of the aggregates had occurred before mixing.[68,71]

A secondary advantage of the pre-wetting procedure was that the temperature of the constituent materials at the time of mixing was controlled. The batch water was mixed with the material 24 hours before mixing and casting and it can be assumed that the temperature of the concrete at the time of mixing was essentially the ambient temperature of the laboratory, which was 70°F. ± 3°, even for slight variations in the original temperature of the mixing water.

The procedure adopted for the adjustment of the batch water to maintain the water/cement ratios specified by the mix design was as follows:

A companion sample of aggregate was weighed out - usually 3000 gms. - and immersed in water at the same time as the aggregate for the mix. i.e. 24 hours before casting.

Before proceeding with the casting on the following day, the comparison sample was brought to a saturated surface dry[68] condition using a blotting action with dry towels, after the excess water had been drained away. The difference in weight between the S.S.D.[68] sample and the original dry sample, expressed as a per cent, was the amount of water retained by the aggregate to reach the S.S.D. condition. The amount of water retained by the aggregate which was to be used for the mix, could then be calculated and added to the water required to provide the batch water/cement ratio. This sum was the total water required by the batch.

In Series 2, the Thames Valley River Gravel mix (C mix) was chosen as the datum mix and the mix parameters by weight chosen are only true for the mixes containing this aggregate; i.e.

aggregate/cement ratio and water/cement ratio by weight. All other mixes containing coarse aggregates with different densities were adjusted so that these mixes contained the same relative mix proportions by volume as the base mix.

The S.S.D. specific gravity of the different coarse aggregates was determined at the same time as the 24 hour absorption capacity tests. The average of the specific gravity and absorption capacity measurements are given in Table 6.10.

Table 6.10 - Properties of Coarse Aggregates

Aggregate Type	S.G.(S.S.D.)	24 hour absorption capacity	No. of tests
T.V.R.G.	2.52	1.1 per cent	6
Granite	2.65	0.1 per cent	6
Limestone	2.63	0.3 per cent	6
Leca	0.84	5.4 per cent	6
Sand	2.58	1.02 per cent	(Newman[16])

It has been suggested (D. Llewellyn[72]) that lightweight aggregate which has absorbed a relatively high percentage of water will give up some of this 'retained' water to the mix during mixing, subsequently changing the effective water/cement ratio.

In order to make an estimate of the amount of water added in this manner, an experiment was conducted on a lightweight aggregate sample which had been immersed for 24 hours in water. After bringing the sample to a S.S.D. condition as described above, the mixing process was simulated by giving the sample a rolling and shaking motion while wrapped in a damp towel. A plot was made of weight loss of the sample against time. From this graph, it was estimated that approximately 2.5 per cent should be deducted from the 24 hour absorbtive capacity figures for the leca aggregate to approximate conditions in the actual mix. The figures given in the table for Leca have been adjusted in this manner.

6.3.2 Mixing

A standard 3 cubic foot tilting drum laboratory mixer was used for each casting, the only exception being the cement paste mixes which were mixed in a $1\frac{1}{2}$ cubic ft. removable horizontal pan mixer.

The following procedure was adopted for the mixing of all concretes and mortars. Prior to mixing, the batch design calculations were adjusted to allow for absorbed water. The excess water was then poured from the aggregate bins. The aggregate and water were tipped into the dry mixer and mixed for 3 minutes; then the cement was added and the batch mixed for a further 3 minutes. To ensure good mixing of the constituents, the capacity of the mixer was restricted to 2 cubic ft. Since the majority of castings were much greater than this figure in total, two or more mixes were required. These separate mixes were intermixed by hand on metal trays (The trays were brushed with a damp broom before receiving the mix to prevent loss of moisture to the tray.) for 2 minutes after all the concrete had been mechanically mixed. The tests for workability were conducted 10 minutes after the last batch had been removed from the mixer.

Cement Paste

For the cement pastes, a different technique had to be adopted as these mixes, when the water/cement ratio exceeds 0.35, bleed excessively when cast. Firstly, the cement and the water were mixed in the pan mixer for a 5 minute period. The pan was then removed from the mixer, covered with damp hessian and polythene to prevent evaporation, and stored for a four hour period. After four hours, the covering was removed and the constituents were hand mixed. The pan was then recovered and the mix was allowed to stand for a further half hour. The cover was then removed and it was observed whether or not separation of the constituents had started in the interim. The paste was remixed by hand, covered, and left for a further 10 minutes. When it was apparent that appreciable separation of the constituents had ceased, indicated by the lack of an accumulation of free water on the surface of the paste, the specimens were cast. The delay between the adding of water to the cement and the final casting of the paste specimens was never more than 8 hours.

Care was taken that the moulds were covered with wet hessian and polythene immediately after casting as plastic shrinkage cracks were observed in the first batch of cement paste specimens due to moisture loss with the result that the specimens had to be discarded.

Using the above procedure, it was possible to test relatively homogeneous specimens of cement paste with water/cement ratios as high as 0.60 and which approximated more closely the conditions of the cement paste in the concrete and mortars.[73] However, it is appreciated that local bleeding in cement paste pockets may occur in concrete mixes with water/cement ratios greater than 0.55.

The temperature of the laboratory in the casting area, the water and materials, and where the aggregates were stored, was maintained at 70°F. ± 3°, which is within the temperature range designated by A.S.T.M.[71] and 4° higher than the temperature specified by the British Standard Institution.[74]

6.3.3 Moulds

Standard steel moulds complying to the British Standard Specification 1881:1952 were used.

For the direct tension specimen, the moulds were flexural beam moulds as above, but with a small modification. In order to obtain the required reduced section (see chapter 4) two accurately machined plates were bolted firmly to either side of the mould.

The direct tension, indirect tension and flexural tension beams, and the compression prisms were all cast placing successive layers of concrete perpendicular to the eventual direction of the applied load.

6.3.4 Workability Tests

The workability tests used in this investigation were the compacting factor test and the slump test.

For both tests, the equipment and the method were in accordance with B.S.1881:1952. The required fully compacted weight of the concrete in the compacting factor cylinder was calculated from the specific gravity of the constituents assuming no air voids, which was contrary to the Standard(75). This was employed for simplicity with the knowledge that the specific gravity of the constituent element was well known (Table 6.10).

The tests were started 10 minutes after the last batch of material had been discharged from the mixer.

The workability was only measured during the casting of the Series 2 mixes.

Table 6.11 - Compacting Factor and Slump for Series 2 Mixes

Aggregate Type:	A		B		C		D		E	
mix prefix number	C.F.	S	C.F.	S	C.F.	S	C.F.	S	C.F.	S
1	0.84	$\frac{1}{4}$"	0.76	0	0.87	$\frac{1}{2}$"	0.91	$\frac{1}{4}$"	✓	✓
2	0.96	6"*	0.93	$1\frac{1}{2}$"	0.97	6"*	0.91	$\frac{1}{4}$"	✓	$9\frac{3}{4}$"
3	0.82	0"	0.79	0"	0.88	$\frac{3}{4}$"	0.84	0"	0.88	$7\frac{1}{4}$"
4	0.93	$2\frac{1}{2}$"	0.91	$1\frac{1}{2}$"	0.94	$5\frac{3}{4}$*	0.90	1"	✓	✓
5	0.85	0"	0.80	0"	0.88	$\frac{1}{4}$"	0.80	0"	0.96	8"
6	0.89	$\frac{1}{2}$"	0.85	0"	0.91	$\frac{1}{4}$"	0.83	0"	✓	$9\frac{1}{2}$"

* - collapse slump ✓ - excessively wet mix

It is apparent from the results tabulated above that for the range of mixes considered in the investigation, the slump test was an unsatisfactory method for measuring workability.

For each mix prefix number, the mix proportions for all concretes are identical by volume. As expected, the rounded Thames Valley River Gravel produced the more workable mixes compared with the mixes containing the crushed aggregates, as measured by the compacting factor test.

Neither of the two methods proved suitable for detecting changes in workability in the lightweight aggregate mixes. A penetration method (such as the Wigmore consistometer) had recently been reported by Elvery[76] as being more suitable for measuring the workability of concretes containing lightweight aggregates than either of the two methods used in this work.

6.3.5. Compaction

The moulds were held firmly on a standard vibrating table (ALLAM - 776) and filled in three layers; each layer was vibrated until full compaction was obtained, as indicated by removal of a majority of entrapped air, with neither segregation nor excessive laitance.

After removing the moulds from the vibrating table to a level surface, the top of each mould was levelled and finished with a steel trowel.

Care had to be taken when floating the surface of the direct tension specimens as the grips would not accomodate specimens whose height in the mould was greater than $4\frac{1}{32}$"

6.3.6 Curing

After the specimens had been trowelled, the moulds were bridged with a steel framework over which saturated hessian was draped. The hessian was covered with polythene sheeting and the moulds were left undisturbed under these conditions for 24 hours.

The day after casting, the specimens were removed from the moulds and immediately placed in the curing tanks; the flexural and the direct tension beams were stored vertically in the tanks to prevent warping.

The curing tanks were maintained at a temperature of 68°F. ± 1°. All specimens were kept in the curing tanks until the day of testing with one exception: three 4 x 4 x 20 ins. beams in Series 1 and 3 were used at 7 days for non-destructive tests, then returned to the curing tank to be used again at 28 days.

In Series 1 and 2, all the specimens during a given test day were removed from the curing tanks, placed on a trolley, and wrapped in wet hessian until required.

Series 3 involved the fixing of extensometers and consequently rather lengthy exposures to laboratory air before the commencement of testing. An attempt was made to maintain the specimens in a saturated condition by painting each specimen with three coats of 'Ritecure', membrane curing compound(supplied by S.B.Dickens Ltd., Boreham Wood, Herts.)after removing the specimen from the curing tank.

A pilot test on at least two specimens was carried out to determine the effectiveness of the membrane curing compound.

The weight loss of the specimens over a twelve hour period after the final coat was applied, was recorded. From the results, it was observed that some moisture was removed by evaporation during this period, of the order of 0.1 per cent. However, it was thought that evaporation from the larger pores near the surface of the specimen was responsible for the majority of the moisture loss and that all the test specimens treated in this manner were, as far as this investigation was concerned, saturated during the tests.

Further to the above, a visual check of the two fracture surfaces of the direct tensile specimen immediately after testing, revealed that there was no apparent drying out at the edges of the specimen due to moisture loss through the surface of the curing compound, as the fracture surfaces were evenly saturated throughout.

6.4 DESTRUCTIVE TESTING

6.4.1 Loading Rates

In all the destructive testing, the loading rates were contrary to the British Standard Specifications 1881:1952.

For the testing of the Series 1 and 2 specimens, it was decided to adopt a loading rate in compression and tension which would make the time required to reach the ultimate load in each case relative and compatible.

Previous investigations have shown that the direct tensile strength of concrete is, in general, one tenth of the compression strength. This ratio was used to establish the loading rates. The British Standard[74] states that the loading rate for compression specimens should be 2000 psi/min., which, using the above logic, gives a value in tension of 200 psi/min. This was considered to be too high . Therefore, it was decided to adopt the loading rates of 1500 psi/min. for the compression test and 150 psi/min. for the tension test. It will be noted that both of these values are within the limits of A.S.T.M. Specifications[37,77], and that the direct tensile figure is larger than the British Standard[74] for testing flexural beams.

For test Series 3, which involved the simultaneous reading of two extensometers during loading, a slower loading rate was indicated. Maintaining the ratio of 1 to 10 between the tension and compression loading rates, the values chosen were: tension - 100 psi/min.; compression - 1000 psi/min.

Wright[16] used 100 psi/min. in an investigation involving three different tests and this value is also the lower limit for the new tenative A.S.T.M. Specification[37] for the indirect tension test.

6.4.2 Compression

(a) General:

Three types of compression specimens were used throughout this investigation:

1. 6" cube
2. 4" cube (and equivalent cube)
3. 4"x 4"x 12" prism

The cubes were included for control and for purposes of comparison. As shown by Lachance[78], the prism is in fact a more true uniaxial compression test, and as such, was treated as the standard compression test throughout this investigation.

In all compression tests, packing plates were used between the specimen and the bottom and top bearing plates. This technique had the effect of raising the centre of curvature of the machine spherical seating a distance equal to the thickness of the packing. This is contrary to B.S. 1881:1952, sect.58. However, tests conducted by Sigvaldason[79] have shown that there is no detectable effect on the ultimate load of specimens tested in this manner compared with specimens tested conforming to the standard. The advantage of the technique was that a replacement procedure could be adopted with a spare set of packing pieces and the required surface tolerances laid down by the specification for bearing platens could easily be maintained.

It should be emphasized that the compression machine was in continual use and the time required for removal of the spherical seating and the ram of the machine for grinding was unacceptable

(b) Testing Machines and Procedure:

i. All the compression specimens for test Series 1 and 2 were tested on a 200 tons-force* Denison compression testing machine which complies with British Standard specification 1610, grade B. The compression tests were performed in accordance with B.S.1881:1952 except for the general exceptions as noted in 6.4.1 and 6.4.2(a), above.

ii. The frame of the 100 tons-force Compression Testing Machine was designed structurally as a 100 tons-force frame, but, due to the use of a relatively small ball seating and the general lack of longitudinal, lateral, and torsional rigidity, this machine was entirely unsuitable for destructive testing, especially where the expected ultimate load exceeded 50 tons-force[79].

The function of the machine in the present work was to load the 4 x 4 x 12 ins. prisms during Series 3 when strain measurements were taken up to a load where a significant divergence from a linear stress/strain relationship was

* - for definition of tons-force, see 6.5.2

observed. The limitations of this machine had no appreciable effect on the tests performed as the ultimate load was never sought and furthermore, the maximum applied load never exceeded 50 tons-force. The machine was particularly suited to this purpose as the pumping unit was isolated from the frame of the machine and did not convey any vibrations to the specimen under test and hence, the roller extensometers, which were extremely sensitive to such movement.

A complete description of the procedure for the above tests is given in 6.7.

6.4.3 Tension

(a) General:

Three different methods were employed to develop a tensile stress in test specimens:

(i) Direct Tension Test - where the whole section of the specimen is subjected to a uniform stress or strain and failure is initiated at any point in a plane perpendicular to the applied load.

(ii) Flexural Test - where the extreme fibres of a beam in pure bending are in tension due to the loading system, and failure is initiated in these fibres.

(iii) Indirect Tension Test - where a compressive force applied to opposite generators of a cylindrical specimen causes a tension failure along the diametral plane containing those generators. Failure can be initiated at any point in this plane.

(b) Testing Machines and Procedure:

(i) Direct Tension:

A detailed description of the direct tension testing machine and an outline of the testing procedure is given in Chapter 4.

(ii) Flexural Tension:

The laboratory is equipped with a standard flexural testing machine in general accordance with B.S.1881:1952.

The flexural test was performed according to the British Standard Specifications with two exceptions; firstly, the loading rate as discussed in 6.4.1 ; secondly, the specimen was placed in the machine with the floated face always towards the operator, i.e. the load was applied perpendicular to the uppermost surface as cast in the mould.

The reason for the selection of a different loading rate has been given. The loading of the specimen parallel to the direction of casting means that the test result is less affected by non-homogenity of the specimen due to segregation during casting. Better alignment was obtained using this method and more consistent results had been recorded. This method is actually specified in the corresponding A.S.T.M. Specification;C78-57.

(iii) Indirect Tension:

The 200 ton Denison compression machine with a detatchable base plate designed to accomodate a loading strip (1/2 x 1/8 x 10 ins. hardboard) and a 6 x 9 ins. cylindrical specimen, was used on the 0 - 40 tons-force range, Plate 6.1. This test was performed as part of Series 2.

Since there is, as yet, no British Standard Specification for this test, a laboratory procedure was developed and adhered to for all the indirect tension tests.

The specimen was placed in the testing machine and with the use of wedges and diametral lines inscribed on the ends of the specimen, it was located and held in such a manner that the diametral lines were parallel to the direction of the applied load, and the axial plane which contained the diametral lines was centrally located in all directions on the bottom loading strip.

A second bearing strip was now located on the top of the specimen and the upper platen of the machine was lowered until the specimen was held firmly. The wedges were now removed from under the specimen and loading commenced at the rate of 150 psi/min.

The procedure was similar to that used by Wright[16] and conforms to the new tentative A.S.T.M. Specification[37].

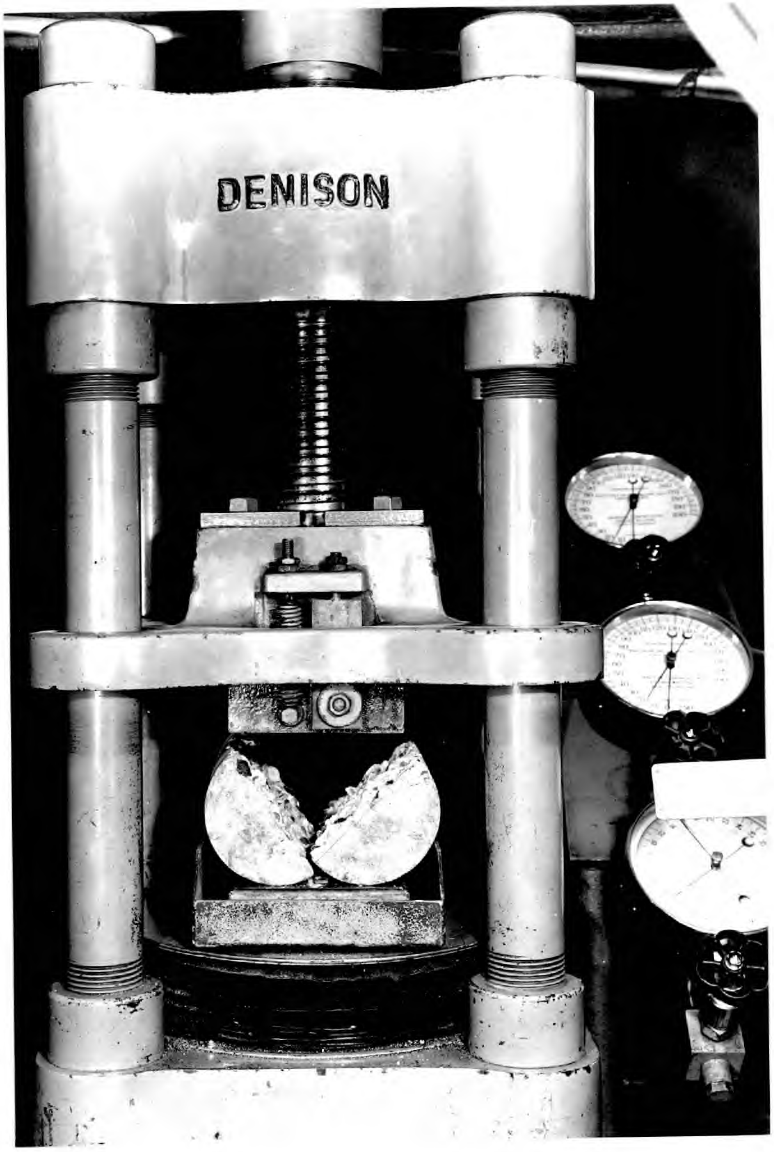


Plate 6.1 Indirect tension specimen failed ~~mounted~~ in Denison 200 T. compression machine.

6.5 CALIBRATION OF THE TESTING MACHINES

6.5.1 General

In this investigation, failing loads ranged from 1 ton-force for the lowest flexural tests, to over 160 tons-force on 6 ins. cubes in compression.

The problem arose of obtaining significant relationships between the results for such a wide range. The solution was provided by the National Physical Laboratory of Teddington which kindly supplied: firstly, a 150 tons-force load column; secondly, a 50 tons-force load cell; and thirdly, a calibration of a 5 tons-force tension-compression proving ring. With these devices, it was possible to provide a range of calibration which encompassed all the loads measured on all the machines, and in which force could be measured to an absolute accuracy of better than $\pm$ 0.2 per cent.

It has been stated elsewhere[80] that for a significant calibration of any piece of apparatus, the calibration device should have an order of accuracy five times better than the equipment for which the calibration is being sought. Since the accuracy and the repeatability of the various testing machines used in this investigation was of the order of $\pm$ 1.0 per cent, the above relationship was attained.

With one exception, the 200 tons-force Denison testing machine, all the testing machines used in this investigation were equipped with hydraulic loading systems and the measurement of load was actually a measurement of pressure; the Denison machine is hydraulically operated but the gauges are graduated in tons-force instead ofp.s.i.g. This distinction is significant in view of the British Standard Specification No.1610, which is concerned with the calibration of machines equipped with load measuring devices and not pressure measuring devices. The Denison does comply with the British Standard Specification No.1610 and is, in fact, a Grade B machine.

For the remainder, the machines were calibrated to give a true load/p.s.i.g. vs. p.s.i.g. relationship and the accuracy of measurement of the true load was stated. In all cases, the load on the specimen could be measured to better than $\pm$ 1.5 per cent over the entire working range of the machine.

6.5.2 Units of Measurement

All the testing machines were calibrated in terms of standard gravitational units of force based on the pound mass. The ton mass is equal to 2240 pound mass.

The pound force is equal to that force which acting on a one pound mass will give it an acceleration of 980.665 cm./sec^2. The ton-force is 2240 pounds-force.

The proving devices, and therefore the calibrated loads on the testing machines, were in terms of the tons-force.

6.5.3 Calibration Procedure

(a)General:

Where possible, the technique used was as outlined in B.S.1610: Part 1. Any exceptions are noted.

All testing machines were calibrated before and after each test series.

(b) Testing machine calibration:

i. 200 tons-force Denison:

This machine is equipped with three pressure gauges graduated in tons-force. The middle and top gauges are identical and cover the range 0 - 200 tons-force and the bottom gauge covers a range from 0 - 40 tons-force.

All load readings were taken on either the bottom gauge or the middle gauge, with the middle gauge being used for all loads over 25 tons-force.

Calibration of the middle gauge was affected using the 150 tons-force N.P.L. column and the 50 tons-force N.P.L. load cell. Both calibration devices met the Grade 1 Specification of

B.S. 1610, Part 1 on the upper four fifths of their working range and therefore an overlap was obtained and the apparatus could be calibrated effectively from 10 tons-force to 150 tons-force.

The 50 tons-force load cell was sufficient to calibrate the bottom 0 - 40 tons-force gauge over the range of failing loads measured, 12 tons-force to 25 tons-force.

The 200 tons-force Denison testing machine complies with B.S. 1610, Part 1 and is a Grade B machine.

True load/indicated load vs. indicated load graphs were drawn after the calibration and were used to determine the true failing loads of the specimens.

ii. 100 tons-force compression machine:

The load measuring devices on this machine were two pressure gauges, a 10,000 p.s.i.g. and a 2000 p.s.i.g. This particular machine was not used for ultimate load testing but was employed as the standard compression frame for the Series 3 testing program.

As the purpose of the Series 3 program was to investigate the stress-strain relationship of specimens from zero load to failure, the desired range of load calibration was extended towards the lower end of the scale as far as was practicable. Calibration was effected from 300 p.s.i.g. to 5000 p.s.i.g. or, 3 tons-force to 50 tons-force respectively.

The 50 tons-force N.P.L. load cell was used to calibrate this machine and the accuracy of load measurement was $\pm$ 1.5 per cent over the working range.

iii. Direct tension machine:

Two pressure gauges with maximum pressures of 300 p.s.i.g and 1500 p.s.i.g. were used in the hydraulic system of the testing machine to measure the applied load.

The maximum failing pressure recorded by the system was 1200 p.s.i.g. and since this apparatus was also used to determine the stress-strain relationship of tensile specimens from zero load to failure, the lower limit was extended to 50 p.s.i.g.

The 5 tons-force tension-compression proving ring was used for the calibration. The ring was connected to each grip of the testing machine through a universal joint and a rigid mounting cast in a 4 in. cube, the cube being held by the grip. Since the proving device and fixings were approximately the same weight as a test specimen and were held in a similar manner by the grips of the machine an effective calibration was obtained.

Loads could be measured by the machine to $\pm$ 1.5 per cent over its working range.

iv. Flexural testing machine:

The load measuring device on this apparatus was a 2000 p.s.i.g. pressure gauge. The failing pressure range of the specimens tested in this machine varied from 800 p.s.i.g. to 1800 p.s.i.g. or 0.8 tons-force to 1.8 tons-force.

The 5 tons-force tension-compression ring was used to calibrate this machine.

The loads were measured to an accuracy of $\pm$ 1.5 per cent over the working range of the machine.

6.6 NON-DESTRUCTIVE TESTING

6.6.1 General

The non-destructive tests performed throughout this investigation were:

i. Determination of density

ii. Measurement of Ultrasonic Pulse Velocity

iii. Measurement of Resonant Frequency and Band width.

In all the testing programs the non-destructive tests were carried out before the destructive tests. The specimen used for all these tests was the 4 x 4 x 20 ins. flexural beam. The standardized procedure for the transportation and handling of the specimens, as outlined in 6.3.5, was strictly adhered to.

All electrical equipment associated with the non-destructive testing program was switched on and allowed to warm up prior to its use,for at least two hours.

<u>6.6.2 Density Measurement</u>

(a) Apparatus:

The laboratory is equipped with a 30 Kg. Avery balance, and a set of standard gram weights correct to $\pm$ 1/2 gm. Using a trial and error procedure,sufficient weights to cover the expected range in this work were selected and used exclusively by this investigation. With care, it was possible to weigh to an accuracy of $\pm$ 1 gm. over the range of weights measured.

(b) Procedure:

The 4 x 4 x 20 ins. beam was properly cleaned and surface dried before weighing to obtain its surface dry weight. Approximately 10,500 gms. of water was measured into a 6 x 6 x 24 ins. trough, the exact weight of the trough and water was recorded and the trough was left on the balance. The surface dry beam was lowered into the pan on a suspension system operating from the ceiling, care being taken that firstly, neither the beam nor the suspension system touched any part of the trough, and secondly, there was sufficient water to completely cover the beam. The weight of the new system was recorded. The increase in weight recorded by the balance in gms. when the beam was immersed in the water was the volume of the beam in millilitres. Since the surface dry weight of the beam was known, the specific gravity and hence the density was easily calculated.

Three beams were tested in this manner. Each beam was individually marked at the time of the density test to allow for correlation with the results obtained from the other non-destructive tests.

The overall accuracy of the density measurement of specimens using the above technique was $\pm$ 0.1 per cent.

6.6.3 Ultrasonic Pulse Velocity

(a) Apparatus:

The standard laboratory ultrasonic pulse velocity apparatus is a Cawkell U.C.T.2 which is equipped with 100 Kc/sec. barium titanate transducers.

A special transducer holder, Plate 6.2, was made in order to facilitate and improve the accuracy of readings. The use of this holder enabled the measurements to be carried out with little or no assistance. In addition, it was apparent from pilot tests that more repeatable results were obtained due to the constant pressure with which the transducers were held onto the specimen. Finally, the holder was designed to accomodate 20 ins. beams, 4 in. and 6 in. cubes, as it was intended to try and use the U.P.V. tester for crack detection in specimens under load, as first used by Jones[81].

(b) Procedure:

The design and operation of the U.C.T. 2 equipment to obtain the time interval for a pulse to travel through a specimen, from which the U.P.V. can be calculated, has been dealt with adequately by Jones[82].

In accordance with the conclusions of Kaplan[93] and Evans[84], the ultrasonic pulse velocity was measured longitudinally on the 4 x 4 x 20 ins. flexural beam specimen. i.e. with a 20 in. path length. Three readings were taken with the centres of the transducers held at the centre of the beam and then at 1/2 in. above, and then 1/2 in. below the centreline of the beam. The beams were always tested with the floated face uppermost and generally speaking, the U.P.V. was lower in the top of the beam than in the bottom, indicating slight segregation.

In all tests, a quantity of mould grease was maintained between the transducer elements and the specimen, ensuring good contact. This gave a condition which required the minimum amount of gain for the incoming pulse to show full scale on the oscilloscope of the machine. This is most desirable as a high gain introduces distortion into the incoming signal and makes accurate determination of the onset of the pulse very difficult.

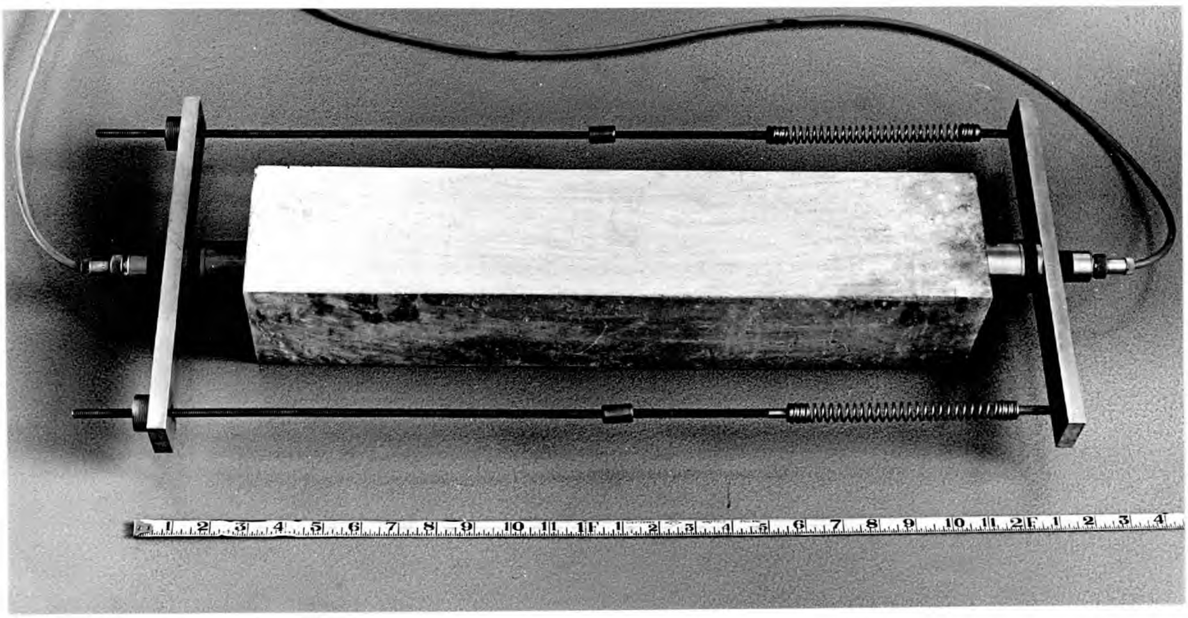


Plate 6.5 Uni-axial compression specimen with Lamb's longitudinal roller extensometer and Lachance lateral gauge attached. (test series 3).

(c) Accuracy of measurement:

Generally, the onset of the pulse could be estimated, and the results repeated, to $\pm$ 0.3 micro secs. This means that pulse velocity could be measured to within $\pm$ 0.5 per cent over a 20 in. path length.

6.6.4 Resonant Frequency

(a) Apparatus:

The resonant frequency apparatus consisted of:

1. Cawkell resonant frequency test bench fitted with a Goodman's vibrator as the driving unit and a piezoelectric pickup unit.
2. Advance variable frequency signal generator, Type J, Model Z.
3. Advance transistorised measuring counter, Type T.C.1.
4. Advance A.C. valve voltmeter, Type 77.

A schematic diagram of the apparatus is given in Fig.6.1.

The legs of the table supporting the apparatus were supported on rubber shims to help eliminate spurious vibrations and resonances. Further, all the equipment was placed on a 1/2 in. thick rubber sheet on the top of the table.

The apparatus was new and had not been used for research purposes prior to this investigation. Consequently, it was decided to undertake a series of pilot tests to observe what effect the following set up errors had on its operation and the results obtained from the equipment.

Error parameters:

Four basic variables can be considered:

1. If the beam is placed askew on the support.
2. If the beam is not balanced properly.
3. If the direct line between the transducers does not pass through the centre line of the specimen.
4. The effect of various contact pressures of the driver and pickup units.

As this was basically a problem of isolating the effect of each of the parameters, any secondary effects due to the nature of the specimens should be eliminated. To this end, a machined 4 x 4 x 20in.

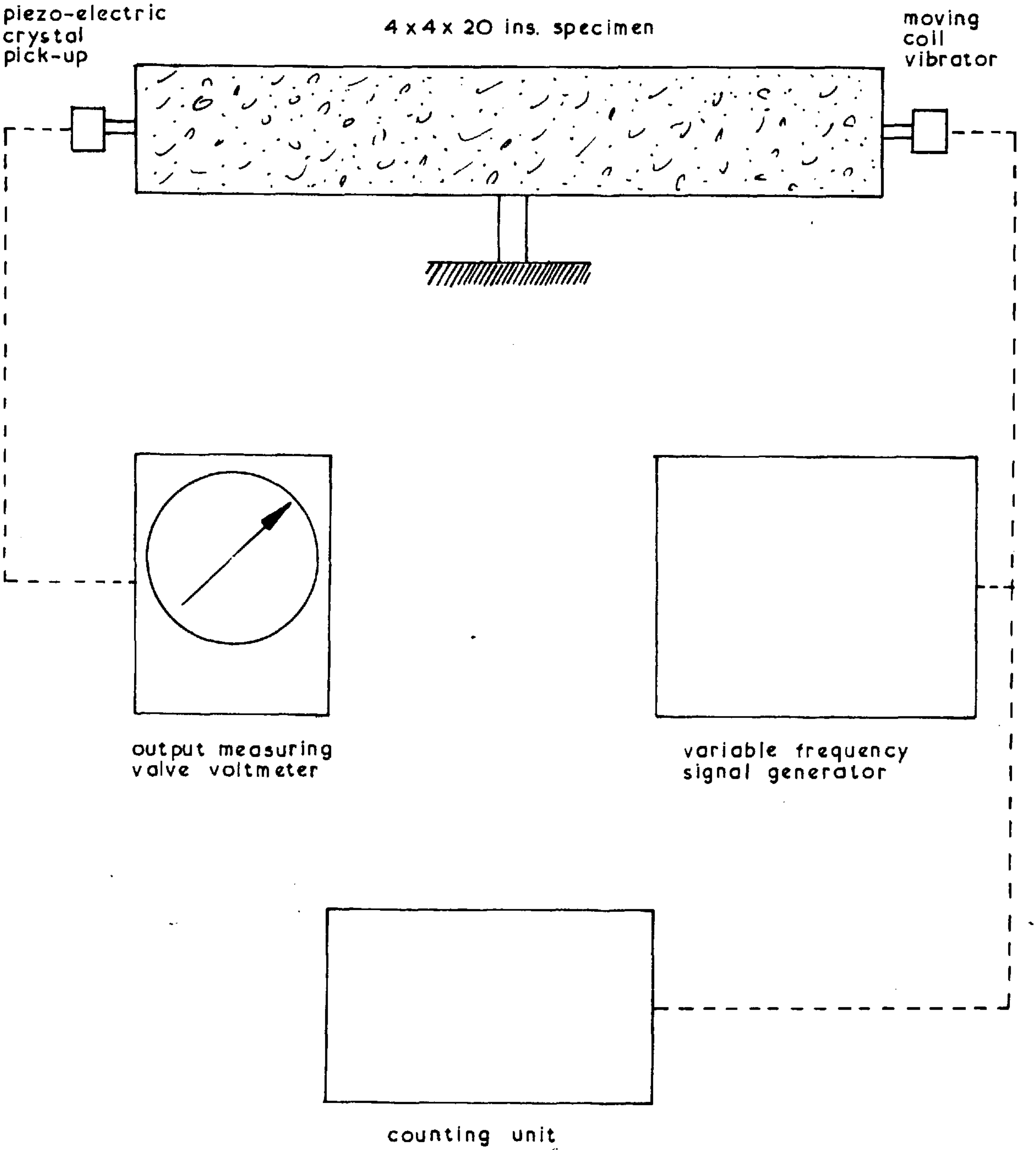


Fig. 6.1 SCHEMATIC DIAGRAM OF THE APPARATUS USED TO MEASURE RESONANT FREQUENCY.

aluminium beam was used as the test specimen. Each condition was investigated and the following conclusions were drawn:

1. A small amount of imbalance could be tolerated. The centreline of the beam could be not more than 1/4 ins. from the centre line of the support.
2. Non alignment of the centreline of the beam by as much as 1/2 in. with the direct line between the transducers should be avoided as the reproducibility of the results was affected.
3. If the specimen was slightly askew, up to 2°, there appeared to be no noticable effect on the results.
4. Varying the contact pressure of the transducers did change the observed resonant frequency by 0.1 per cent.

(b) Procedure:

As a result of the initial pilot experiments with the aluminum beam, a standard laboratory procedure was established for setting up the specimens. The procedure, to be outlined, ensured that reproducable and accurate results were obtained from the equipment.

Firstly, the specimen was balanced on the support with equal overhangs, care being taken that the driving and pickup transducers were in line with the centreline of the specimen. The driving transducer was then brought into contact with the beam and a nominal amount of excitation was provided. The most sensitive scale was selected on the valve voltmeter and the pickup transducer was brought towards the beam until contact was registered by an output from the valve voltmeter. The feed screw on the unit was then given 3/4 of a turn.

A similar procedure was adopted for the driving transducer which was now removed from contact with the beam and then just brought up to the point of contact as with the pickup transducer. The feed screw on the driver transducer was given one full turn.

The values used above for the transducer pressures were obtained from the pilot test which showed that contact pressure was the most significant part of any error. This standard set-up procedure was rigidly adhered to for all specimens tested on the resonant frequency bench.

During a test, the range of the valve voltmeter was selected so that the amount of energy required to give full scale deflection of the meter was a minimum. There was a lower limit to this procedure which was the minimum applied voltage to trigger the counting unit. The frequencies were measured using a count over a ten second period with the counting unit.

Resonance and band width measurements were repeated at least four times, the band width being taken as the difference between frequencies measured at 0.707 maximum amplitude on either side of the resonant frequency.[84,85] The dynamic modulus of elasticity was calculated as in B.S. 1881:1952 using the mean values of resonance obtained from the tests.

(c) Accuracy of measurement of the longitudinal resonant frequency and band width:

The within test range of the measurements taken of resonance was 2.0 cycles per second or, expressed as a per cent of the lowest recorded resonant frequency, $\pm$ 0.05 per cent.

Since the band widths could be measured to a within test range of 3 cycles per second and the lowest band width measured was 35 cycles, the percentage accuracy of the measurement was always within $\pm$ 5 per cent.

6.6.5 Accuracy of Measurement of the Dynamic Modulus of Elasticity, Damping Factor and the Dynamic Poisson's Ratio.

(a) Dynamic Modulus of Elasticity was calculated from[74]:

$$E_D = 6.0 \times 10^{-6} f_L^2 \ell^2 \rho$$

where, f_L = longitudinal resonant frequency (cycles/second)

ℓ = length of specimen (inches)

ρ = density of specimen (lbs/ft^3)

Summarising the accuracy of measurement of the independent variables in the above equation,

error in f_L $= \pm$ 0.05 per cent

ℓ $= \pm$ 0.3 per cent

ρ $= \pm$ 0.4 per cent

Therefore the accuracy with which the dynamic modulus could be measured was:

$$= \pm\left[2(0.05) + 2(0.3) + 0.40\right] \text{ per cent}$$

$$= 1.1 \text{ per cent}$$

(b) Accuracy of measurement of the damping constant:

The constant was calculated using the formula[84,85]:

$$Q = \frac{n_1 + n_2}{2(n_2 - n_1)}$$

where n_1 and n_2 are the upper and lower frequency values of the band width at 0.707 full amplitude - cycles per second.

Since the band width could be measured to better than $\pm$ 5 per cent and the frequency to $\pm$ 0.05 per cent, the accuracy of the Damping Constant was $\pm$ 5 per cent.

The large errors in the measurement of the band width were due to the method of measurement which involved a relatively small difference of two large numbers. This uncertainty was reflected in the accuracy of the Damping Constant.

(c) Dynamic Poisson's Ratio:

The value of the dynamic Poisson's Ratio was obtained from the specially enlarged nomograph of King and Lee[86] supplied by the Cement and Concrete Association.

Density, E_D, and U.P.V. were used on the nomograph and the probable error of using these values was always within $\pm$ 4 per cent on all specimens, and probably better than this with the normal concretes.

6.7 STATIC TESTING

6.7.1 General

In test Series 3, stress-strain relationships to failure in compression and tension were obtained on specimens where the load was applied continuously to failure. The longitudinal and lateral incremental strain readings were recorded orally on a commercial tape recorder which enabled instantaneous reading and recording of the two strain measuring instruments at any load increment; the tape was played back immediately after testing and the strain readings were permanently recorded.

A small load, approximately 15 per cent of the ultimate failing load, was applied to all specimens before the actual tests were performed to ensure that the gauges were functioning properly. All specimens were tested in a saturated condition, having been coated with a membrane curing compound immediately after removal from the curing tank (see 6.3.6).

The longitudinal strains in both the tension and compression tests were measured with the same 4 in. gauge length Lamb's longitudinal roller extensometer. A 4 in. gauge length Lamb's lateral roller extensometer was used to measure the lateral strains up to failure in the tension tests. However, as the failure in the compression test is in the form of a violent lateral bursting, it was decided to use a less expensive and somewhat expendable detachable gauge to measure the compression lateral strains to failure.

Because considerable care was taken to obtain significant repeatable results, the complete series of tests sometimes occupied a fourteen hour period. Since the two specimens tested in either compression or tension were tested within 5 hours of each other at the most, no time corrections were made to any of the results.

6.7.2 Instruments for Strain Measurement:

(a) Lamb's 4 in. longitudinal and lateral roller extensometer:

The operation of these instruments has been adequately described elsewhere[87,88] and it will suffice to state that these instruments are optical, mechanical, and detachable strain measuring instruments of high accuracy and very sturdy construction.

At the start of each test day, both instruments were dismantled and cleaned to remove any dust from the surface of the roller bearing surfaces which might have hindered the smooth operation of the gauges. This was particularly important when the longitudinal gauge was used for tension tests as the required contact pressure to hold the knife edges of the longitudinal gauge firmly against the specimen was only supplied by rubber bands. Therefore, there had to be as little restriction to movement as possible in the gauge itself to ensure that a continuous and factual reading was obtained from the gauge during the test without the gauge slipping. The above procedure was adopted with knowledge of the results of a calibration of a similar 8 in. longitudinal extensometer which showed that these instruments could be dismantled, cleaned, and then reassembled with an insignificant loss in repeatability and accuracy[89].

The significance of the results obtained from these gauges are related to the following:

<u>i. Sensitivity</u>:

4 in. lateral extensometer : 0.58 micro-strain

4 in. longitudinal extensometer : 1.03 micro-strain

These values are the strains represented by a one mm. scale division, the smallest whole division on the sighting scales and also the accuracy to which the scale could be read, operating with a scale distance of 30 feet. This sensitivity could have been improved by increasing the scale distance, but it was not possible to discriminate to better than 1 mm. at the greater distance. The limiting factor in these tests was the width of the crosshair of the telescope which appeared to obliterate one half of the 1 mm. division of the scale with a 30 foot scale distance. Furthermore, considerable eye strain was experienced even with a 30 foot scale distance due to distortion of the reflected image of the scale. Therefore, it was concluded that any further increase in scale distance with the equipment available was not warranted.

ii. Accuracy:

The measurements upon which the accuracy of performance of the instrument depends are the following:

1. Diameter of the rollers.
2. Gauge length of the instrument.
3. The scale distance.
4. The distance between the mirrors.
5. The reading on the scale.

The diameter of the rollers is known to 0.005 per cent and the gauge distance is well within 0.1 per cent. These are the permanent errors of the instrument.

The scale distance was maintained to within 0.15 per cent for all readings using a curved scale, the radius of curvature of which was equal to the scale distance. The error incurred in measuring the distance between the extensometer mirrors was within 0.1 per cent.

The scale proper could be read to 0.5 mm. so that the error introduced from this source could be as high a 1 per cent when the total reading was of the order of 50 mm; i.e. tension testing. The combination of the errors enumerated cannot lead to an error greater than 1.2 per cent overall or 1/2 microstrain, whichever is the greater.

It has been observed elsewhere[87] that an ambient temperature change of 1°F. could effect an error of 4 microstrain in the readings taken with either of the two extensometers. Other than reconstructing the longitudinal extensometer, this has to be accepted with this instrument. However, with the lateral extensometer, the connecting arms which hold the gauge in position and in which the temperature strain is developed, were reconstructed from Invar steel, which effectively minimized the error from this source. As a further precaution, a thermometer was located in the vicinity of the test specimen and the temperature was recorded before and after each test. During the course of any test, no noticable change in the temperature readings was ever detected. Furthermore, each test specimen with all gauging attached was left for about one half hour before the commencement of the test proper to ensure temperature compatibility between the specimen, gauges, and the ambient conditions.

(b)Lachance 4 in. lateral gauge:

Lachance[78], while working at Imperial College, developed a detachable lateral gauge consisting of two perspex strips, with electrical resistance strain gauges affixed, and two steel bars; the pieces were connected together in such a way that any relative displacement of the steel bars, which are located against opposite sides of the specimen, is registered in the perspex connecting strips.(see Plate 6.3)

A Peekel (Model 540 DNH) strain measuring unit was used with this lateral gauge. This instrument can be employed using the deflection method or the manually operated zero method of measurement (null method). The main feature of the Peekel box is its ability to provide a direct reading in microstrain (deflection method). This is achieved after an initial balance is obtained in the Wheatstone bridge containing the electrical resistance strain gauges of the lateral gauge by means of adjustable fixed resistances and the slide wire of the Peekel unit. The advantage of this method is that once an initial balance has been obtained, the system does not have to be disturbed. Furthermore, using the null method with the Peekel, it is only possible to discriminate to 2 microstrain, while it is possible to read to one half of a microstrain using the deflection method, using the most sensitive range giving a full scale deflection of 30 microstrain.

A disadvantage with this measuring unit is that the applied voltage to the bridge is 5V. A.C.; this is high for strain gauges mounted on perspex as it does affect the long term stability of the gauges. However, pilot tests did show that negligible drift was obtained if the system was turned on for an hour before use and

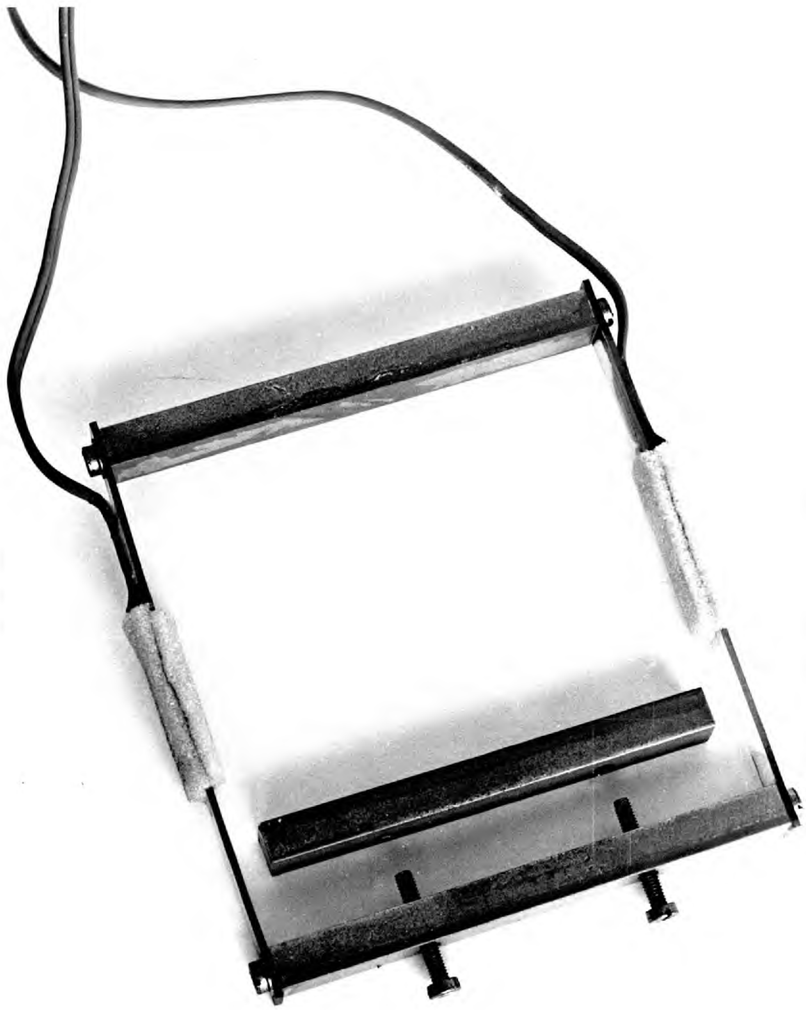


Plate 6.3 The Lachance lateral gauge.

if the tests were of short duration. Both of these conditions were satisfied throughout these tests.

The gauge factor of the electrical resistance strain gauge is known to 1 per cent. Since the recording instrument could be read to 1 microstrain easily, the error is assumed to be less than 2 microstrain or 1 per cent of the reading.

(c) Electrical Resistance Strain Gauges

Electrical resistance strain gauges were used to measure strain on a variety of concrete, mortar, and aluminium tensile test specimens. The object of their use was to measure ~~wether~~ the eccentricity of a specimen under test or for the determination of the stress-strain relationship right up to failure, using the continuous scan facility of the Solartron Data Logger, a very fast (10 readings per second) pulse excitation strain recorder.

Technigraph 1 in. linear foil gauges of 80 ohm resistance and a gauge factor of 2.01 were used throughout. In all cases identical dummy gauges were mounted on material of the same nature as the test specimens.

Two epoxy adhesives were used for fixing the strain gauges. Araldite MY573 was used to fix the strain gauges to the aluminium specimens and some of the dryed out mortars. Hottinger Messtechnik X - 60, a fast setting epoxy adhesive, was used to fix gauges on the saturated concrete and mortar specimens.

The procedure adopted for fixing the strain gauges to the specimens was as follows. Firstly, the surface was rubbed down with a suitable abrasive and then washed with Acetone. Guide lines for locating the gauge were drawn. A small amount of adhesive was spread across the marked position and a strain gauge, which had been rinsed in Acetone, was placed into position on the adhesive and firmly pressed down. Care was taken that all air bubbles were removed from under the gauge. After the adhesive had hardened and the leads had been carefully soldered to the gauge, it was protected by either covering with a coat of the original adhesive or coating with a layer of low viscosity wax.

The Peekel 540 DNH, using the deflection method described in (b) above, was used in some of the pilot tests where specimen eccentricity was measured.

The gauge factor of the electrical resistance gauges used in this work were given to 1 per cent. Since the measuring unit could be read to 1 microstrain, it is assumed that the accuracy of the strain measured with the electrical resistance gauges was of the order of 1 per cent or 1 microstrain, whichever is the greater.

6.7.3 Tension Testing Procedure

Except for some pilot tests where one or two specimens were used to investigate a particular parameter, e.g. rate of straining, different curing conditions, etc., the measurement of strain in tension was confined to test Series 3. However, the description of the method of test does, in more general terms, outline the standard procedure for all the strain measurement tests in tension. Individual exceptions will be noted in the results.

The stress-strain relationship for concrete in uniaxial tension was obtained from two specimens at each age. The test specimens were brought into the vicinity of the testing machine as soon as possible after being removed from the curing tank and coated with a membrane curing compound (see 6.3.6). Together with the strain measuring instruments which were to be used in the test, the specimens remained in the test area for a minimum of three hours before the testing preliminaries were started.

After the stabilization period, the first specimen was placed in the testing frame in accordance with the established procedure. (see Chapter 4).

Small strips of the curing compound were scraped from the specimen where the bearing surface of the lateral extensometer were expected to make contact with the specimen sides. Immediately after scraping, the uncovered area was smeared with a waterproof grease to prevent moisture loss. This procedure was demanded because pilot tests had shown that the contact pressure on the nibs of the lateral extensometer was sufficient to slowly force the nibs through the soft curing compound with the result that continual drifting, sometimes of three hours duration, was registered by the gauge. The grease provided no resistance at all in this respect however, and a stable condition was always obtained with the above procedure.

The lateral extensometer was now fitted on the specimen and suspended by rubber bands from two magnetic base blocks located on the upper grips (Plate 6.4). The extensometer, as mounted, measured the overall lateral strain in the test specimen on a plane about one inch above the mid-height of the specimen. As the contact pressure of this instrument is maintained by a small displacement of a relatively long spring, the contact pressures from test to test were quite similar.

The 4 in. longitudinal extensometer was now mounted using two stiff rubber bands on each side of the specimen to hold the instrument firmly in place. The same contact pressure was maintained from test to test as the same strong rubber bands were employed throughout. This instrument is equipped with sharp knife edges which cut through the curing compound and therefore the longitudinal gauge did not suffer from the same fault as the lateral extensometer. Furthermore, the curing compound directly affects the readings obtained from the lateral extensometer whereas the longitudinal extensometer readings can be affected only by slipping of the knife edges.

Both instruments were now prepared for testing with regard to the alignment of the mirrors to provide a reflected image of the scales in the telescope, etc. All ancillary equipment, such as the tape recorder, stop watch, thermometer, etc., were placed in their respective positions and the completed set-up was left undisturbed for about one half hour. After this period, the author took up a position behind the telescopes and an initial reading of both the extensometers

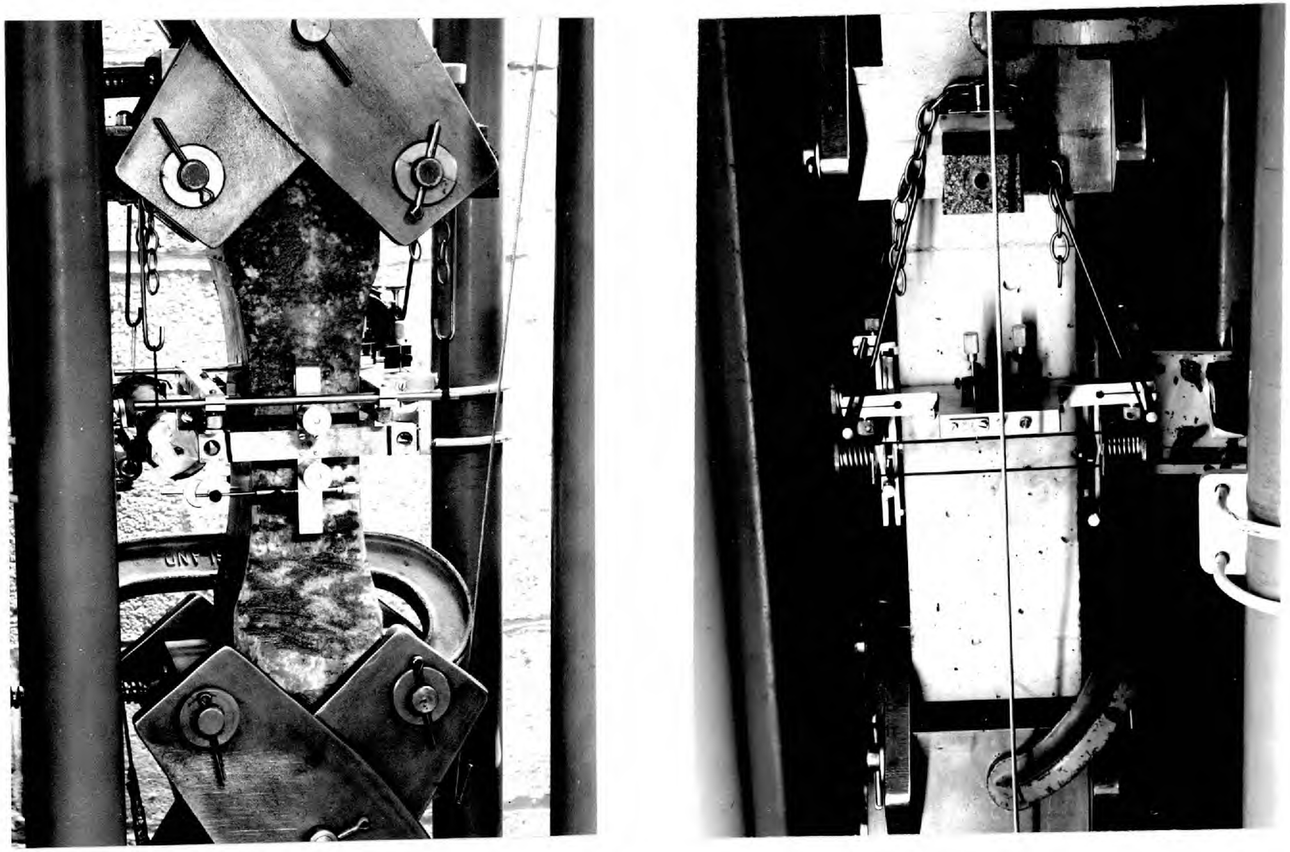


Plate 6.4 Uni-axial tension specimen with Lamb's longitudinal and lateral extensometers attached. (Test Series 3).

and the thermometer were recorded. After five minutes, the readings were repeated. If there had not been a change of more than 1 microstrain in any of the readings, the test was started immediately. If a change was observed (usually in the lateral extensometer readings since this was the most sensitive instrument) the start of the test was postponed until a stable condition was obtained.

The test was started by applying a small load to the specimen to ensure that both extensometers were functioning. If this test gave satisfactory results, the specimen was loaded to failure at the rate of 100 pounds per square in. per minute, both extensometers being read at load intervals of 30 pounds per square in., the readings being given orally into the tape recorder which was now running continuously.

After failure, the location of the break was recorded with the temperature and the total time taken for the test. The fractured surface was examined for loss of moisture through the curing compound, but, in all cases, the surface appeared to be evenly saturated. A similar procedure was used for the second specimen which was tested immediately after the first.

6.7.4 Compression Testing Procedure

The measurement of longitudinal and lateral strains in compression to failure was confined to test Series 3. The 4 x 4 x 12 ins. prism was used exclusively. An attempt was made to follow the recommendations of Lachance[78] explicitly by measuring the longitudinal and lateral strains in the central 4 x 4 x 4 section of the prism. However, although it was possible to mount the lateral gauge in this region, it was physically impossible to mount the 4 in. longitudinal gauge there as well. Anson[87] showed that the same average longitudinal strains are obtained at distances up to 2 ins. away from the central portion of the prism. The longitudinal gauge was mounted with the lower knife edge just clearing the Lachance detachable lateral gauge by 1/4 in.

The general pre-test procedure adopted for the testing of the two compression specimens was very similar to the method used in the tension testing described in (a) above. The main difference between the two methods was the introduction of the detachable Lachance gauge to replace the Lamb's lateral extensometer so that lateral straining right up to failure could be measured. The Peekel measuring unit associated with this gauge was always turned on two hours before being required, to achieve gauge stability (6.7.2(b)). During the setting up of the gauge, it was allowed to rest on packing pieces which held it in a position $4\frac{3}{4}$ ins. from the lower end of the specimen as placed in the testing machine. The adjustment screws were slackened, the Peekel was balanced, then the adjustment screws were turned in alternately, until an extension of 500 microstrain was registered on the Peekel meter. This corresponds to a lateral stress in the prism of 5 psi only. As this procedure was used each time, the contact pressure of the gauge was identical for each test.

The 4 in. longitudinal extensometer was mounted so that the lower knife edges were just clearing the top of the Lachance gauge. Tie rods were used to clamp the extensometer onto the specimen, the same number of turns being given to the fixing nuts during each set up to obtain the same contact pressure from test to test. The gauge, when mounted, required approximately the same amount of force to 'slide' it on the specimen as in the tension set-up where rubber bands were used to hold the extensometer. Plate 6.5 shows the set-up of the system for the compression tests.

The compression specimens were loaded continuously to failure at the rate of 1000 pounds per square inch per minute. Loading was stopped when the lateral strain exceeded 600 microstrain and failure was imminent.

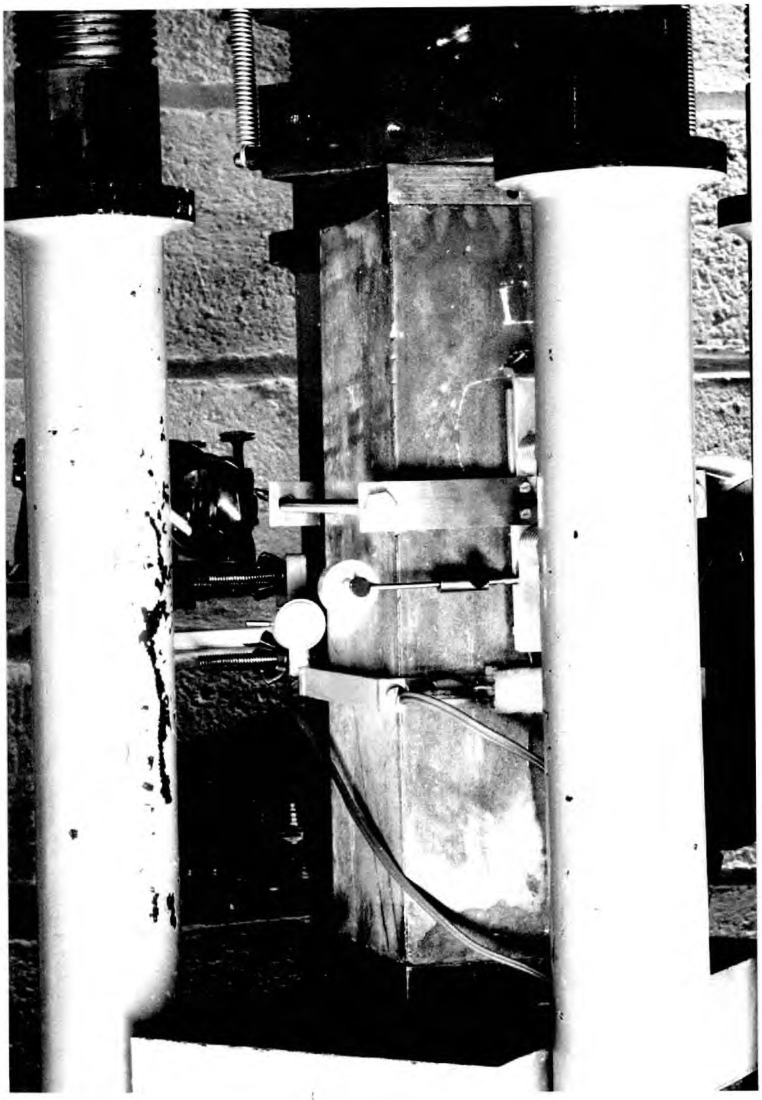


Plate 6.5 Uni-axial compression specimen with Lamb's longitudinal roller extensometer and Lachance lateral gauge attached. (test series 3).

CHAPTER 7

THE VARIANCE OF THE UNI-AXIAL TENSION TEST COMPARED TO OTHER DESTRUCTIVE TESTS - TEST SERIES 1

7.1 INTRODUCTION

The first series of tests were carried out to establish the variance of the strength results obtained from the new testing technique. In such a series, it was important that variations within the concrete under test should be reduced to a minimum. For this reason, a mortar mix with sand/cement ratio 2.0 and water/cement ratio 0.45 was used. This mix produced suitably homogeneous and uniform specimens, with which to 'calibrate' the machine. Thirteen mixes were made, nine mixes being tested at 28 days and the remainder at 7 days.

In this chapter, the results of this series are presented and analysed and comparisons made between variations of the ultimate strengths obtained from the various tests used.

7.2 GENERAL

The testing methods used in test series 1 conformed to relevant British Standards[74] with the exceptions already noted in Chapter 6.

The primary object of this test series was to compare the relative distributions of failing stresses obtained from a number of destructive testing methods, with those obtained from the new uni-axial tension test. This is important since Wernimont[95] states that these relative distributions define the performance of a given test. The definitions of performance of a testing method according to Wernimont are:

(1) precision: a measure of the variability of a test method when used to make repeated tests under carefully controlled conditions.

(2) repeatability: the precision of results obtained by a single operator using a single piece of test equipment.

(3) reproducibility: the precision of results obtained by different sets of operators using different sets of test equipment.

(4) accuracy: an abstract term for mechanical type testing which depends in general on the testing method.

Both (1) and (2) can be examined by test series 1. Since only one operator and one machine was used to perform all of the tests, the reproducibility of the testing method could not be determined. Furthermore, it is not possible to estimate the accuracy of the test method, since this definition depends on the general acceptance of a given testing method. However, it is considered that the uni-axial tension test gives 'true' tensile strengths of concrete materials compared with results obtained from either the indirect tension or flexural tests.

The second purpose of this test series was to investigate whether the parts of the specimen which are held in the grips of the uni-axial testing machine were damaged significantly during the test. Equivalent cube tests[74,96] and indirect tension tests, using the method proposed by Nilsson[41], were carried out on the residual ends of the broken direct tension and flexural specimens. If a significant difference was observed in comparative results, it could be assumed that some structural damage had probably occured in the end of the direct tension specimen ends. In this case, it would not be possible to use the ends in the same way as the residual ends of the broken flexural specimens which are covered by British[74] and American[96] Standards for use as an equivalent cube test.

7.3 PRESENTATION OF RESULTS

Column 4, Table 7.1, contains the average values for the failing strengths obtained from the various destructive tests carried out in test series 1. Column 5 gives the coefficient of variation (V) for all specimens tested for each test method at each age. Column 6 gives the average range (R) obtained from each group of test specimens tested at one time. The estimate of the within-batch coefficients of variation (V_1),which have been estimated from the values of the within-batch ranges given in column 6 using a method suggested by A.S.T.M.[93], are

Table 7.1 Series 1, Summary of destructive testing results.

Specimen and Test	Total Number	Age (days)	Average Failing Strength (psi)	V	R	V_1	Per cent 28 day Strength	Per cent (A)		Per cent (B)		Per cent (C)	
(A) 4 ins. cube -	12	28	7230	3.6	400	3.3			100		107		
compression	13	7	5185	6.7	172	2.0	74	100		108			
(B) 4x4x12 ins.prism-	27	28	6727	4.3	266	2.3			94		100		
compression	12	7	4814	4.8	212	2.6	72	93		100			
(C) 4x4x20 ins.beam -	27	28	890	6.8	62	4.1			12.3		13.2		100
flexure	12	7	700	4.0	55	4.6	79	13.5		14.5		100	
(D) reduced flexural	12	28	564	3.5	23	2.4			7.8		8.4		63
beam -uni-axial tension	12	7	469	5.6	34	4.2	83	9.1		9.8		67	
6Øx9 ins.cylinder -	27	28	563	6.2	34	3.6			7.8		8.4		63
indirect tension	12	7	408	8.9	25	3.6	73	7.9		8.5		58	
Direct tension ends-	27	28	6914	4.2	202	1.8			96		103		
equiv.cube comp.test	12	7	5094	5.4	192	2.2	73	98		106			
Flexural beam ends-	27	28	7116	4.6	273	2.3			98		106		
equiv.cube comp.test	12	7	5094	5.3	225	2.6	71	98		106			
4 ins. cube -	9	28	657	5.5	41	3.7			9.1		9.8		74
indirect tension	9	7	562	8.4	45	4.7	85	10.8		11.7		80	
Direct tension ends-	27	28	674	8.0	50	4.4			9.3		10.0		76
indirect tension	12	7	560	5.2	45	4.7	83	10.8		11.6		80	
Flexural beam ends -	27	28	703	7.3	47	4.1			9.7		10.4		79
indirect tension	12	7	560	3.2	25	2.8	83	10.8		11.6		80	
Column No. 1	2	3	4	5	6	7	8	9		10		11	

given in column 7. The 7 day strengths , expressed as a percentage of the 28 day strengths are given in column 8. For the purpose of comparison, various percentage relationships between all the tests performed at 7 and 28 days were established with respect to certain datums as shown below:

Column No.	Datum (100 per cent)
9	4 ins. cube - compression
10	4 x 4 x 12 ins. prism - compression
11	4 x 4 x 20 ins. flexural beam - flexure
~~12~~	~~reduced flexural beam - direct tension~~

It was decided to express the variability of each test using the coefficient of variation since this factor is relatively independent of the order of stress, and an effective comparison could be carried out between the various tension and compression tests. The standard deviation alone would not have been satisfactory for this purpose since for similar variances the standard deviation is dependent on the order of stress and a factor of 10 was involved between failing stresses in tension and compression. Furthermore, it was necessary to establish a coefficient of variation of each destructive test to estimate the relative precision of each test method (see 7.2). A complete discussion is given in Appendix D of the statistical methods employed in this chapter.

Table 7.2 gives the summary of the results of the non-destructive tests used and the relevant statistical analysis of variance.

Table 7.2 Series 1 , Non-destructive testing results.

Non-destructive test	Age	No. of specimens	Average value	Coefficient of variation (per cent)
Dynamic Modulus of Elasticity	7	12	4.25	1.5
	28	27	4.88	1.1
Damping Constant	7	12	75	16.9
	28	27	84	13.4

7.4 DISCUSSION OF RESULTS

The following discussion is based on the summary of results and statistical analyses given in Table 7.1.

As expected[16,52,57], the standard flexural beam test gave the highest failing strengths (column 11) of all the tension tests. The cylinder indirect tension and the uni-axial tension test gave similar results which were approximately 60 per cent of the flexural values. The flexural beams gave a higher 28 day and lower 7 day between-batch coefficient of variation (V) (column 5) than the cylinder indirect tension or uni-axial tension specimens. There was little difference in the between-batch coefficients of variation (column 5) of the uni-axial tension test and the cylinder indirect tension test, although there was an indication that the former gave a lower variance. However, it was observed that no significant difference was obtained between the within-batch (column 7) coefficients of variation (V_1) for these three tension tests.

The indirect tension tests using the 4 ins. cube or the equivalent cube gave higher results than either the cylinder indirect tension or the uni-axial tension tests (column 11). During testing, it was observed that vertical cracks had formed in the cubical indirect tension specimens at approximately 90 per cent of the ultimate load. Chapman[31] has measured strains on relatively large circular disc specimens loading in a similar manner to the indirect tension tests performed here. He observed cracking between the loading points well below the ultimate load of the specimen, in one instance at 60 per cent. It is suggested that the high results obtained with the cubical form of the indirect tension test is basically a statistical size effect[45,46], and possibly a restraint effect due to the low height of the specimen. These indirect tension tests gave the highest values for V of all the destructive tests performed. Since it has been that the variance of a test increases for a decrease in specimen size, a relatively higher variation of test results was expected[16,35,36].

The residual ends of the broken flexural and uni-axial tension beams tested as equivalent compression cubes gave essentially the same results. Both of these tests gave results which were only 2 per cent lower and 6 per cent higher than the plain cube and prism respectively (columns 9 and 10). The within-batch and between-batch coefficients of variation for all of the compression tests were in general sufficiently lower than any of the tension tests, which is in accordance with the results of Wright(16) and Rüsch(98). It is apparant from columns 8, 9 or 10 that the mortar tested has a greater increase in compressive strength with age in comparison to the increase in tensile strength with age. This phenomena has been observed by many investigators(19,25,51,57) and is the reason for the curvilinear relationship observed between tensile and compression strengths. The tension/compression strength ratio decreases with an increase in the overall mix strength. This point will be discussed more fully in Chapter 10.

In Table 7.2, the results of the non-destructive tests are given. It is seen that the 7 day dynamic modulus of elasticily was 87 per cent of the 28 day modulus. This compares favourably with results obtained by Evans(94). The damping constant (Q) increased with age which corresponds to a decrease in the logarithmic decrement, agreeing with work published by Baker and Kesler(97) on the damping properties of mortars.

Finally, it should be noted that all of the destructive and non-destructive testing in this series was carried out by one person, the author. This also was chronologically the first test series. The variability obtained using any test method must be a function of the experience of the operator and this should be borne in mind when the variability obtained from any of the tests performed is considered. This point is important since it might have been argued that the new uni-axial tension test required a high degree of skill. It is therefore significant that a similar variance was obtained with this new test in comparison to other destructive tests which are in general easier to perform by an inexperienced operator.

It is appreciated that in some cases there was meager data for an accurate statistical analysis. The coefficients of variation at both ages calculated for the uni-axial tension test and for all of the other destructive tests at 7 days are only on the basis of twelve individual results obtained from four batches. The calculated values of V are, to a certain degree, a cummulative indication of the within-batch and between-batch coefficients of variation. Nevertheless, it is reasonable to conclude that in general the variability of the various destructive tests are of a similar order.

The values of V obtained from the test results were compared with the A.C.I. 'Control Standards'[93] in order to assess the overall control at the laboratory level for batching, mixing and testing in this series. For the majority of the tests, 'excellent' control was obtained, i.e. V and V_1 were less than 5 per cent and 3 per cent respectively. The remainder of the tests conformed with 'good' control as defined in the Standards.

7.5 CONCLUSIONS

(1) It is possible to obtain results with the uni-axial tension test which compare favourably in terms of variability as measured by the coefficient of variation with other forms of destructive tests. It is therefore concluded that the uni-axial tension test has similar precision[95] (see 7.2) to other forms of destructive testing.

(2) It is feasible that one equivalent cube compression strength result and one indirect tension strength result could be obtained from the residual portions of the uni-axial tension specimen, as well as from the flexural beam specimen.

(3) There is a decrease in the ratio of ultimate tensile strength to compression strength with an increase in both the age and strength of the mortar mix used. This trend was observed when the results of any compression and any tension test were considered.

(4) The new uni-axial tension test was able to conform to the highest A.C.I. Standard rating for testing uniformity and overall control based on the within-batch and between-batch coefficients of variation.

CHAPTER 8

THE EFFECT OF VARYING MIX DESIGN PARAMETERS AND COARSE AGGREGATE TYPES ON THE FAILURE OF CONCRETE - TEST SERIES 2

8.1 INTRODUCTION

This chapter is concerned with the presentation and analysis of the results from test series 2. This test series was undertaken to obtain quantitative information on the relationships existing between the strengths of various concretes obtained from the uni-axial tension test and other compression and tension destructive tests. Four different coarse aggregates were chosen for this study.

A detailed description has been given in Chapter 6 of the test methods employed throughout test series 2. Furthermore, discussion is given in 6.6 of the method of calculation and the accuracy of the non-destructive testing results. Chapter 5 gives an outline of the experimental work and includes a list of the specimens used.

The discussion to follow (8.3) will be related to the evidence from test series 2 presented in this chapter. Other discussion will be left in some instances to parts of Chapter 9 and 10 where the additional information obtained from the measurement of strain on test specimens from similar mixes is required to explain some aspect of the work.

Reference is made in the discussion to the 'normal' concretes. In this text, normal concretes are those containing normal coarse aggregates; mix A - quartzite, mix B - granite, mix C - Thames Valley River gravel. The purpose is to differentiate from the lightweight aggregate concretes, mix D, and the mortars, mix E.

The amplification of any statistical methods used in this chapter is given in Appendix D.

8.2 PRESENTATION OF RESULTS

A histogram has been drawn in Fig.8.1 of the location of planes of failure for the direct tension specimens tested in series 2. The intention was to examine the distribution of breaks in the direct tension specimen to see if there was a tendancy for the loading system or the specimen shape to influence the location of failure. The histogram is the most satisfactory method to use for this analysis.

In Fig.8.2 to 8.6 the failing results from the three tension tests, the flexural, direct and indirect tension tests, have been plotted against the water/cement ratio by weight for each aggregate type. The different aggregate/cement ratios by weight have been indicated. In a similar manner the compression results from the 4 ins. and 6 ins. cubes and the 4 x 4 x 12 ins. prism have been presented in Fig.8.7 to 8.11. All the points on the above graphs represent the average of three specimens from the same mix.

The relationship between the direct tension test and the indirect tension test has been shown in Fig.8.12, that between the direct tension test and the flexural test in Fig.8.13. Each point on the graphs represents the average of three uni-axial tension specimens plotted against the average of three specimens from the same mix tested in the comparative tension test.

To examine the ratio of uni-axial tension to uni-axial compression, the ratios of all mixes have been plotted against increasing compression strength in Fig.8.14. Each ratio is calculated from the average obtained from three specimens tested from the same mix in uni-axial compression and tension. The results from Chapter 7 for the standard mortar have also been shown. Fig.8.15 is obtained from Mitchell(39). The Writer's results have been superimposed on Mitchell's figure which shows the results of several experimenters who have studied concrete in direct and indirect tension and compression. The ratio of cylinder splitting tension to 6 ins. cube strength for lightweight coarse aggregate mixes similar to the mixes used in this investigation has also been investigated by W.C. French Ltd.(104). Fig.8.16 shows the results of their investigation, each point being the plot of the average of three specimens. The results obtained from the Writer's work have been superimposed on this figure.

Fig.8.17 and 8.18 show the dynamic modulus of elasticity (E_D) plotted against the uni-axial tension and uni-axial compression strength respectively.

Table 8.1 is a summary of the averaged ratios of the various ultimate strengths obtained from test series 2. An estimate of the coefficient of variation at each ratio has been given using the methods described in Appendix D. The strength results of series 2 have been given in full in Table B, Appendix B.

8.3 DISCUSSION OF RESULTS

In the calculation of the ultimate failing stresses in the destructive tests, the applied failing load was assumed to act over the cross-sectional area represented by the full mould size, e.g. in the uni-axial tension test the cross-sectional area of failure was assumed to be $4 \times 2\frac{7}{8}$ ins.^{2} = 11.5 ins.^{2} for all the tests performed. Several measurements taken on specimens during early pilot tests showed that the cross-sectional area was in general that of the mould cross-section due to the care taken in the finishing operations during casting. The error introduced by this assumption is not believed to be greater than $\pm$ 1 per cent. It is estimated that the failing loads recorded could be measured to better than $\pm$ 1.5 per cent (see 6.5). Considering this and the comments on cross-sectional area, it is believed that the overall failing stresses from any test are of the order of accuracy of $\pm$ 2.5 per cent.

Test series 2 was the first complete testing program where the uni-axial tension machine was used to test a large number of specimens. A critical assessment of the test would be to see if there was a random distribution of failure sections along the reduced mid-section length of the test specimen. This would indicate that the test result was not in any way influenced by the testing method. Fig.8.1, which is a histogram of the location of such failure planes along the specimen length, shows that such a random distribution does occur. It will be noted that all the specimens failed in the reduced section and in the transition portion of the specimen up to the section where there is a 4 per cent increase in cross-sectional area, i.e. no specimens failed in the vicinity of the grips. It is not considered that any significance can be attached to the ten specimens which failed outside the central portion of the test specimen with the uniform cross-sectional area. Failure will occur away

HISTOGRAM OF LOCATION OF PLANES OF FAILURE IN DIRECT TENSILE SPECIMEN. TEST SERIES

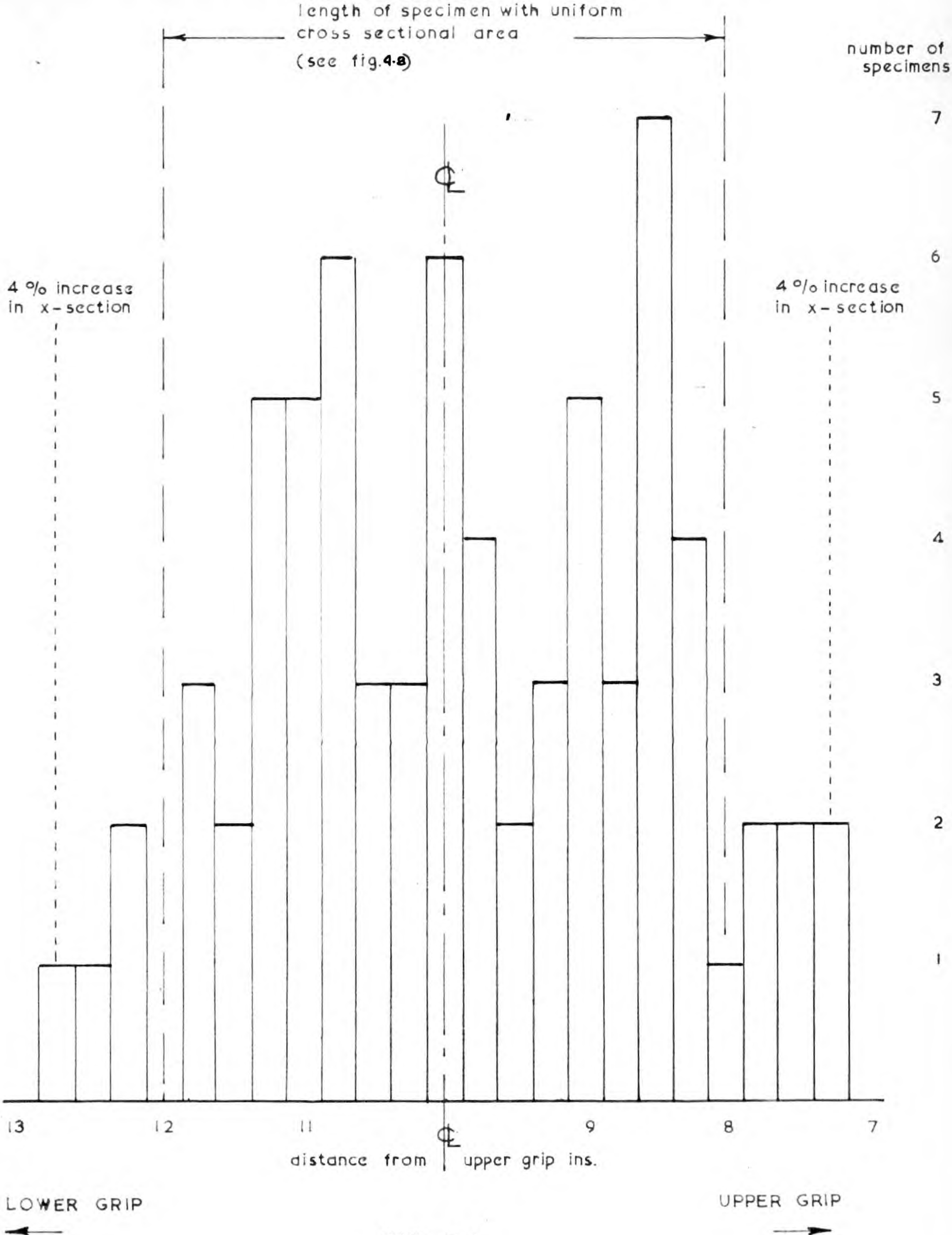


FIG. 8.1

from this central section only when the decrease in strength at a plane outside the section is not offset by the increased cross-sectional area at the plane. It is suggested that the number of specimens failing outside the uniform central section could be reduced if the length of the central section is increased. This would reduce the possibility that the weakest section is outside the reduced portion.

8.3.1 Tension Tests

In Fig.8.2 to 8.6, the results obtained from all the tension tests are plotted. For any concrete mix, the flexural test always gave higher results than either the direct or indirect tension test.

i. Relation between indirect and direct tension tests and flexure tests:

In the summary of the results in Table 8.1, it is seen that in general, the direct and indirect tension tests give results which are 74 per cent of the flexural strength for the normal aggregate concretes (see 8.1).

Blakey and Beresford(101) have indicated that in a flexure test, the **stress** distribution in the compression zone may be assumed to be linear, whereas the stress distribution in the tensile zone may be a second degree or cubic parabola. Taking the variation of stress in the tensile zone as a cubic parabola, it has been shown by Blakey and Beresford that the 'true' tensile strength is about 0.735 times the flexural tension strength of a beam calculated from elastic theory. This is in excellent agreement with the results presented here. Evans(12) has shown the results of calculations assuming a curvilinear tension stress distribution in a flexural beam at failure of up to a fifth degree parabola. He concluded that it was possible to balance the internal forces in a flexural beam at failure as long as the direct tension strength was not less than 59 per cent of the flexural strength in comparable tests. The Writer's work shows that in general, this condition is satisfied.

The ratio of direct and indirect tension tests to the flexural test results is approximately 5 per cent higher (80 per cent) for the mortar and Leca aggregate mixes than for normal concretes. It will be observed in Chapter 9 that considerable redistribution of stress is obtained in the mortar tests in direct tension relative to concretes. It is suggested that this phenomena is related to the larger number of

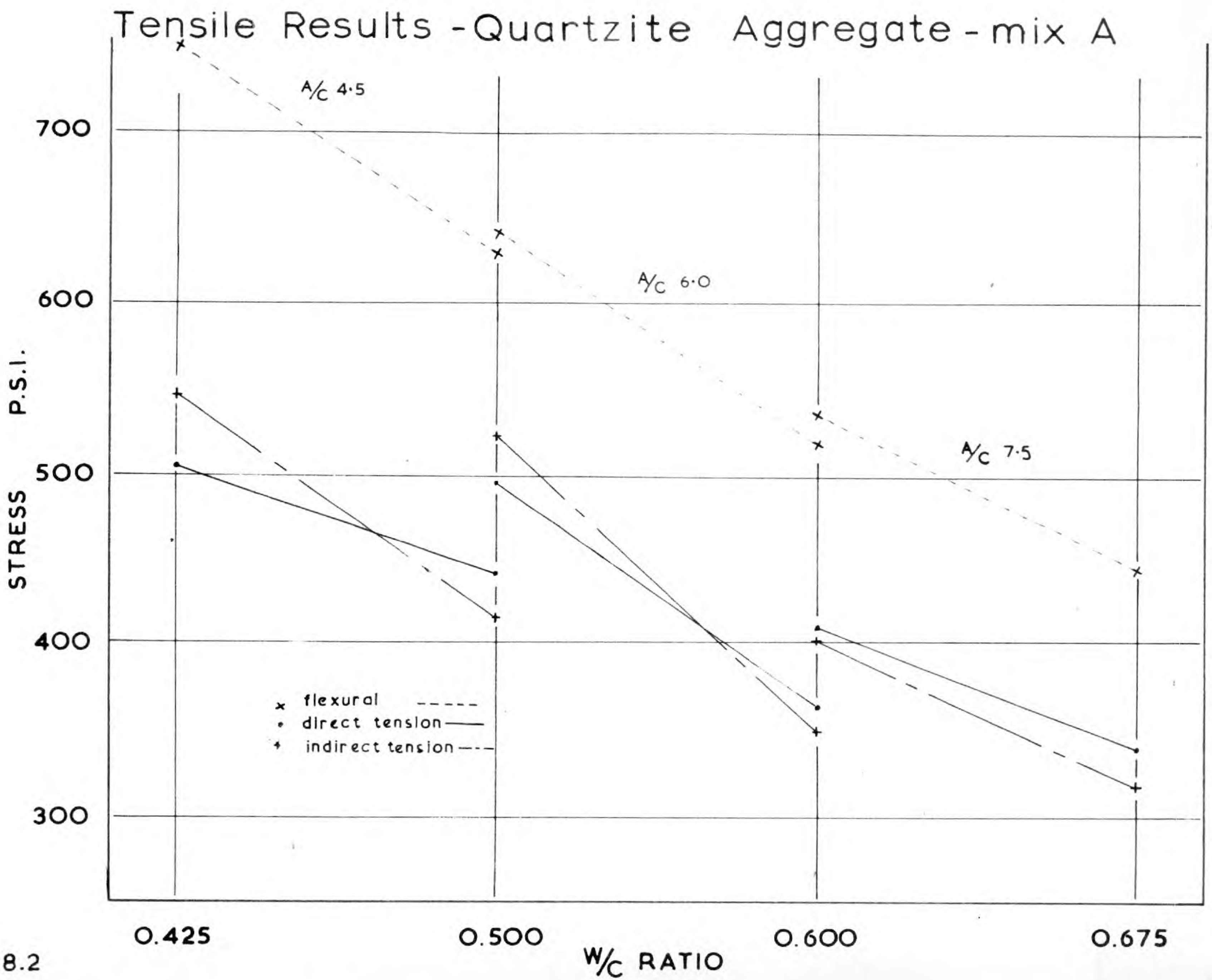
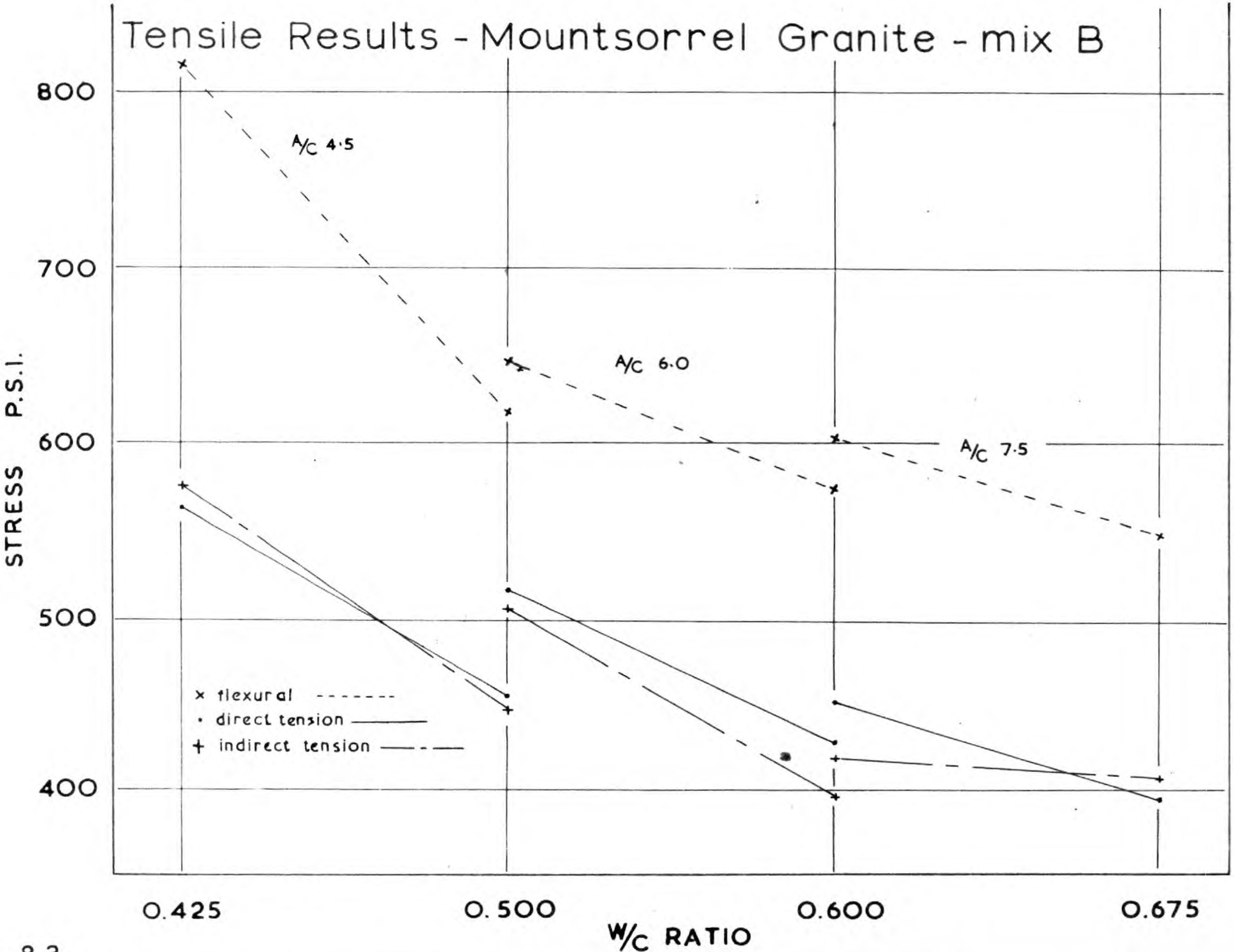
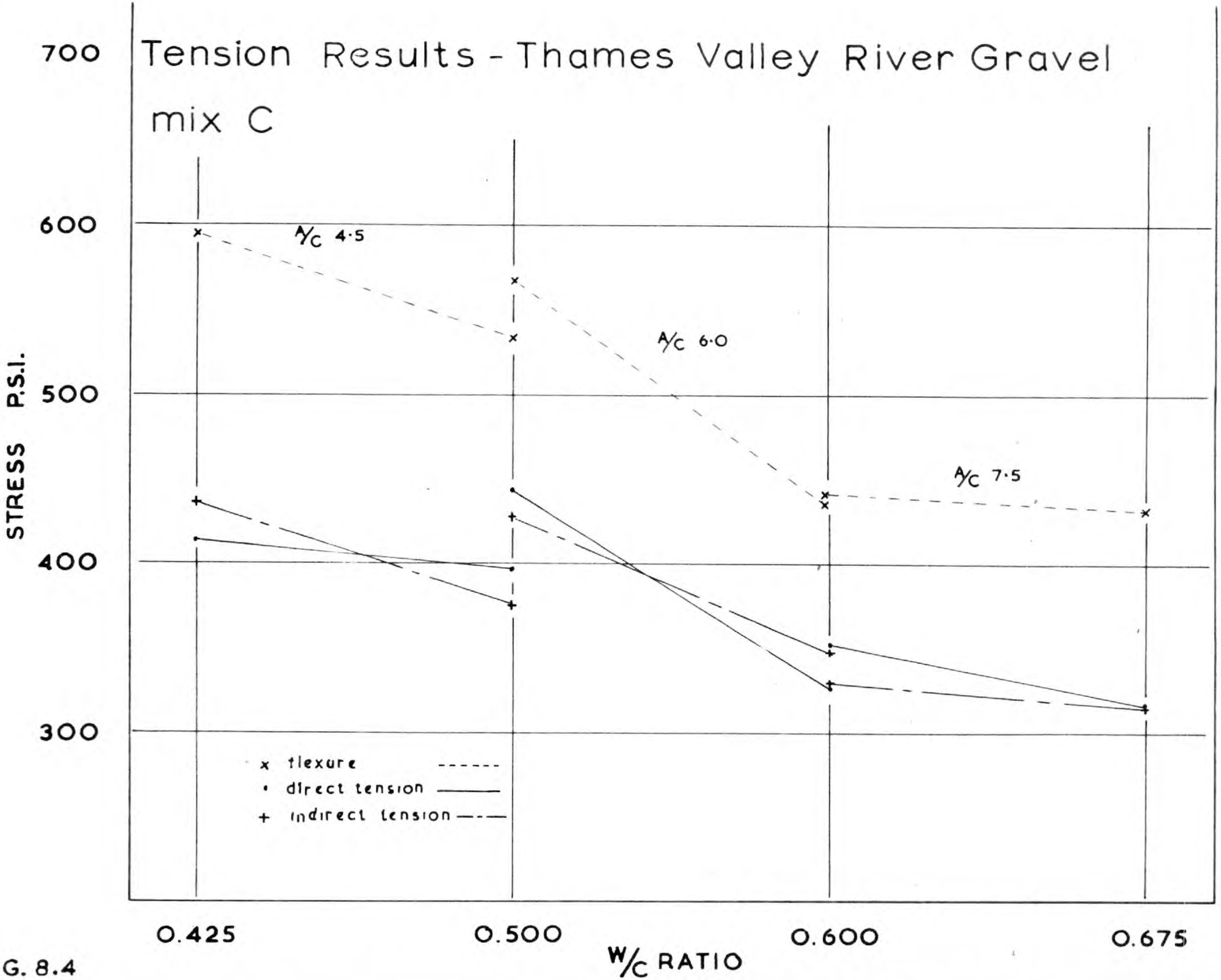

Tensile Results - Quartzite Aggregate - mix A
A/C 4·5
A/C 6·0
A/C 7·5
700
600
500
400
300
STRESS P.S.I.
flexural
direct tension
indirect tension
0.425
0.500
0.600
0.675
W/C RATIO

FIG. 8.2



Tensile Results - Mountsorrel Granite - mix B
800
700
600
500
400
STRESS P.S.I.
A/C 4.5
A/C 6.0
A/C 7.5
× flexural
· direct tension
+ indirect tension
0.425
0.500
0.600
0.675
W/C RATIO

FIG. 8.3



Tension Results - Thames Valley River Gravel
mix C
700
600
500
400
300
STRESS P.S.I.
A/C 4·5
A/C 6·0
A/C 7·5
x flexure
• direct tension
+ indirect tension
0.425
0.500
0.600
0.675
W/C RATIO

FIG. 8.4

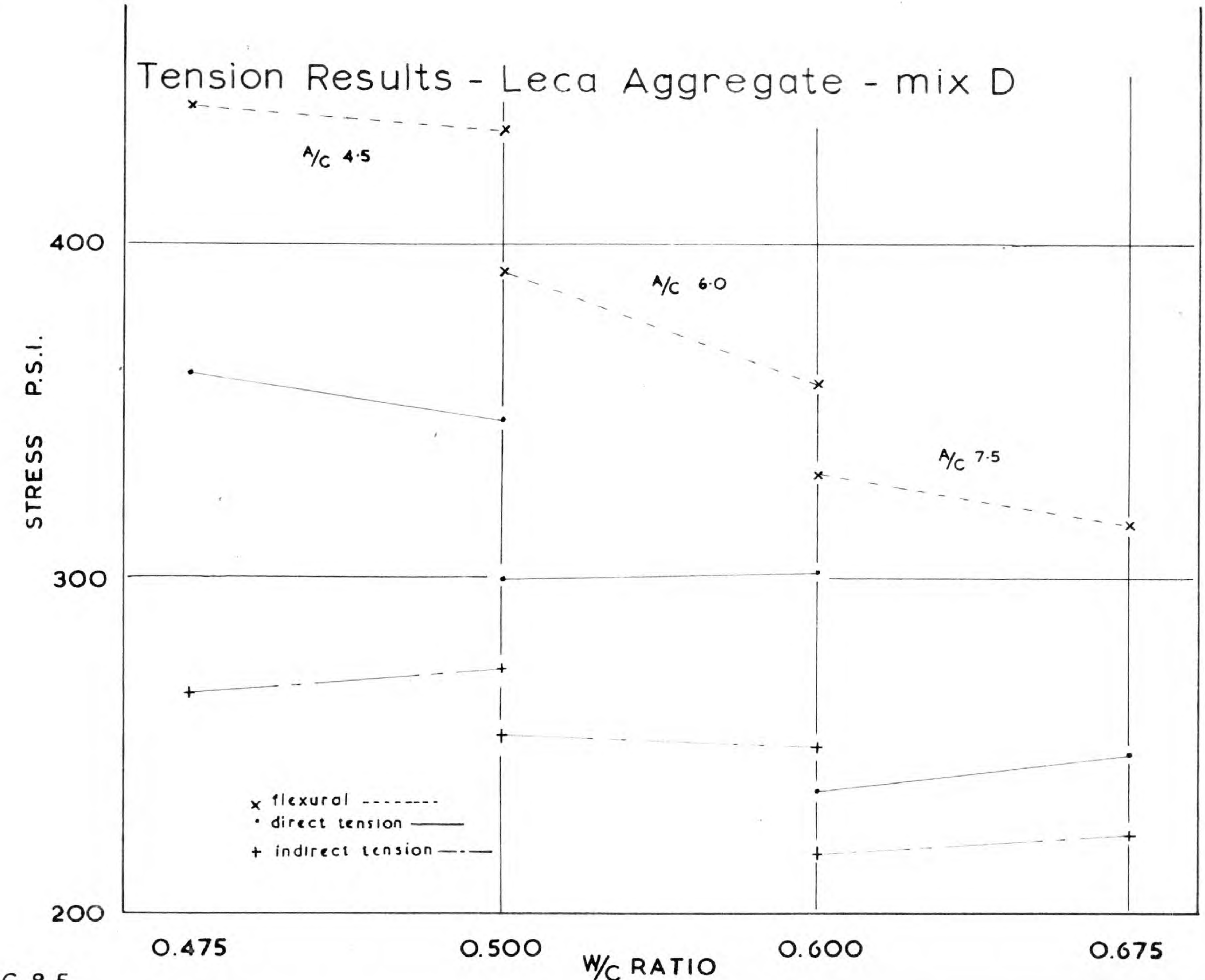

Tension Results - Leca Aggregate - mix D
A/C 4·5
A/C 6·0
A/C 7·5
400
300
200
STRESS P.S.I.
× flexural
• direct tension
+ indirect tension
0.475
0.500
0.600
0.675
W/C RATIO

FIG. 8.5

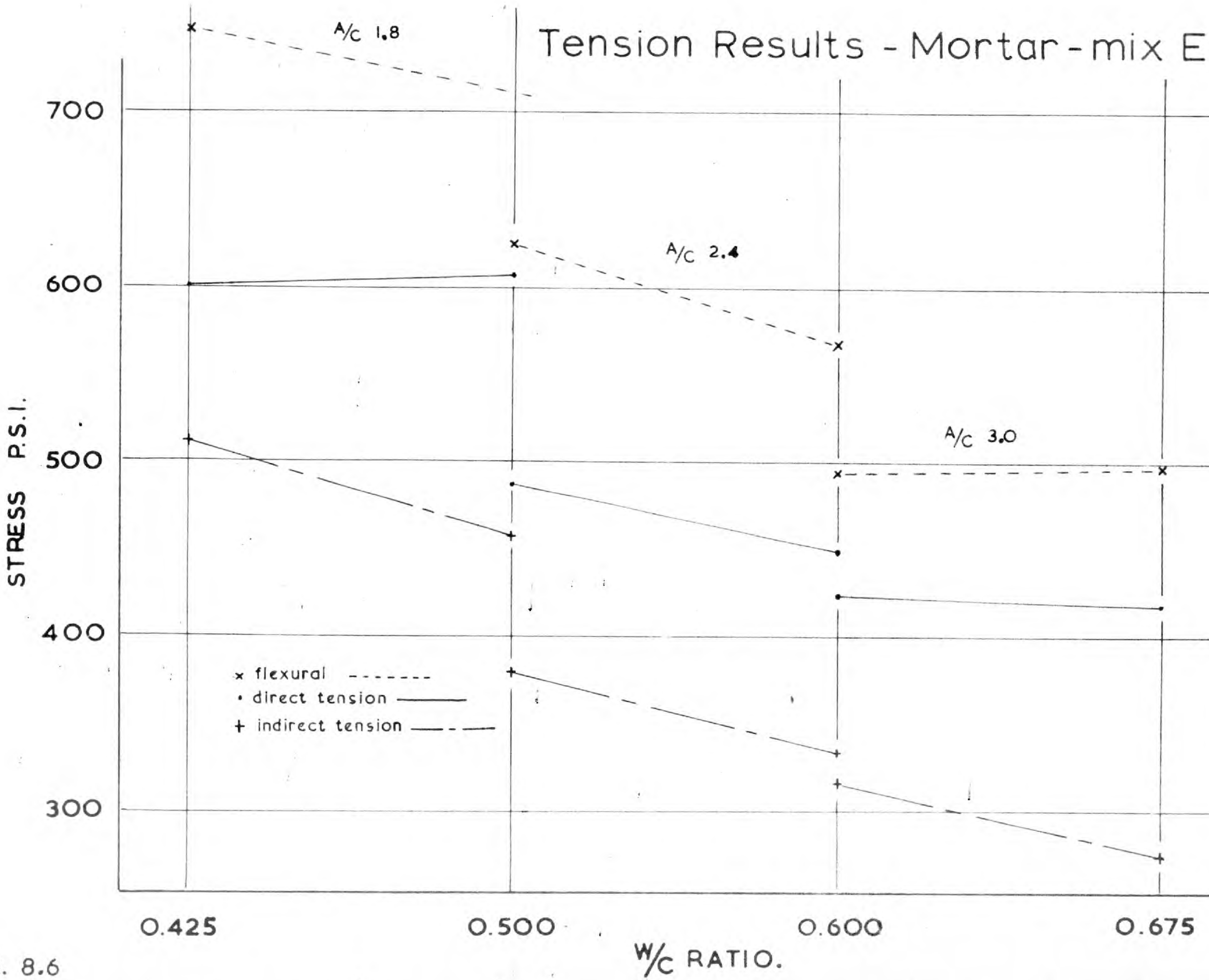

Tension Results - Mortar - mix E
A/C 1.8
A/C 2.4
A/C 3.0
700
600
500
400
300
STRESS P.S.I.
× flexural
• direct tension
+ indirect tension
0.425
0.500
0.600
0.675
W/C RATIO.

FIG. 8.6

possible crack paths available in the mortar due to the high particle number/volume ratio as compared to the concrete. In the Leca concrete, a similar situation might exist since the weak aggregate particles are themselves possible fracture points. It is further suggested that this increased ability of these two materials to undergo prolonged stress redistribution might have more effect on the ultimate strengths obtained from uni-axial tests compared to flexural tests. It is interesting to note that this increase in the ratio of indirect tension to flexural tension has also been reported recently by Grieb and Werner[57].

ii. Effect on tension strength of mix parameters:

For all normal concretes, Fig.8.2 to 8.4, it is seen that an increase in aggregate/cement ratio or a decrease in water/cement ratio results in an increase in tensile strength measured in flexure, indirect or direct tension. The Leca concretes and the mortars gave lower results for an increase in aggregate/cement ratio. For the Leca concretes, it is observed that there is little change in tensile strength for a change in water/cement ratio at a given aggregate/cement ratio. It appears therefore that the major factor in determining the tensile properties of concretes manufactured using this material is the tensile strength of the aggregate. Thus, when the quantity of Leca aggregate is increased in a mix, the tensile strength decreases. The discussion of the properties of the mortars in tension will be deferred to Chapter 10.

iii. Relation between direct and indirect tension for normal concretes:

There is an indication that the direct tension test gives slightly higher results than the indirect tension test for normal concretes. In Table 8.1, it is seen that in general the direct tension test is 2 per cent higher. Referring to Fig.8.2 and 8.4, it is seen that in 12 of 18 values reported, the direct tension test gave higher failing results. It should be realised that the indirect tension specimen used in this test series is a 6 ins.diameter and 9 ins. long cylinder. The standard specimen used by the majority of investigators is the 6 ins. diameter 12 ins. long cylinder. Although no experiments were carried out on size effects in this work, it is possible to infer what the effect would be of increasing the specimen size by considering the statistical argument for brittle materials of Weibull[45]. Weibull considered that flaws are randomly distributed through the volume of a specimen. Failure is initiated when the applied stress reaches a critical value at a flaw which is in the necessary position and orientation to initiate a crack. It is considered that the chances of a critical flaw occurring are greater for specimens with larger volumes at any given stress level, i.e. large specimens would be expected to exhibit lower mean strengths than small specimens. This is generally referred to as the 'weakest link' theory. Wright and others[6,60] have confirmed this predicted theoretical result in experiments with indirect tension specimens. It is considered that the larger cylinder in general could be expected to give lower mean strengths. The conclusion therefore is that the normal form of indirect tension test would give slightly, but significantly, lower results than the uni-axial tension test used here. Although this is an interesting conclusion, it is appreciated that it is of only empirical significance.

iv. Relation between direct and indirect tension for mortars and Leca concretes:

Of particular interest are Fig.8.5 and 8.6, and column 4 in Table 8.1. It is seen that, for the Leca concretes and the mortars, the uni-axial tension test has given ultimate strength results 20 and 30 per cent higher than the indirect tension test respectively. It is suggested that both of the above results were obtained because of the reliance of the indirect tension test on the method of applying the load to the specimen. To amplify, it was observed that at failure the mortar indirect tension cylinders broke into the normal half cylinders together with two 'wedge-like' pieces. The wedge pieces were formed under the loading strips along the whole length of the specimen and in some instances were up to 1.5 ins. deep. It is believed that the failure always initiated at these end points. The material used for loading strips in this test series was hardboard in strips $10 \times \frac{1}{2} \times \frac{1}{8}$ ins. In the first test series, where plywood of similar dimensions was used, the 'wedge' phenomena was not observed. It is thought that in mortars or any materials which have high relative ratios of tension to compression, the possibility exists that the 'wedge' compression effect may influence the result. The Leca mix is made with a coarse aggregate of extremely low compressive strength. The argument suggested by the exponents of the indirect tension test that the material under the loading strips is under a state of tri-axial compression and will therefore never fail before splitting takes place is, in this instance, invalid. The aggregate can fail at very low compressive stresses and local crushing can occur. Furthermore, it was observed that the loading strip used in this investigation actually embedded itself into the specimen. This occurred at loads 90 per cent of the ultimate and would definitely affect the ultimate strengths obtained, since failure was probably initiated from the ends. Recently, Rudnick et al[108] have shown, by a photoelastic analysis, how the characteristics of the packing medium affect the failure strengths obtained from the indirect tension test. They conclude that a tensile failure can be assured in this form of test only by the proper choice of material type and dimensions of the loading strip. Because of the anomolies obtained with this test for mortars and Leca concretes, this Writer suggests that considerable attention be given to this parameter in the future.

v. Effect of aggregate type on tension strength:

In direct, indirect or flexural tension, the mortar mixes gave the highest results in general. For the concretes, it is seen in Fig.8.2 - 8.5 that the mixes containing the granite coarse aggregate gave approximately the same ultimate tensile strengths as comparable quartzite mixes, while both were, in general, 1.3 times the ultimate tensile strengths obtained from comparable mixes containing the Thames Valley river gravel. All of the normal concretes gave ultimate tensile strengths significantly higher than results obtained from mixes containing the Leca lightweight aggregate. A discussion of the above statements is deferred to Chapter 10.

8.3.2 Compression Tests

i. Effect of mix proportions on compressive strength:

The compression results from the 4 x 4 x 12 ins. prism and the 4 ins. and 6 ins. cubes have been plotted in Fig.8.7 - 8.11. It is seen that for changes in water/cement ratio and aggregate/cement ratio the same trends are obtained as with the tension tests. That is, a decrease in the water/cement ratio and an increase in the aggregate/cement ratio results in an increase in strength as measured by any compression test.

ii. Effect of size and shape of specimen on compression strength of normal-concretes:

In Fig.8.7 - 8.9 and in column 7 and 8 of Table 8.1, it is observed that the prism specimen gives the lowest result for the compression strength of any one mix. In general, for the normal concretes, the 4 ins. prism and 6 ins. cube give results which are 77 and 96 per cent of the 4 ins. cube respectively. The ratio of 6 ins. cube to 4 ins. cube is in general agreement with Neville[102]. The discrepancy between the cube strengths can be explained by the statistical distribution of possible flaws with increasing size[45,46], i.e. the 'weakest link' theory (see 8.4.1). This is reasonable for the interpretation of the cube results which have a height to width ratio of 1. However, for the prism with a height to width ratio of 3, the effect of lateral restraint at the interface between the loading platen and the specimen has to be considered. For both of the cube sizes, it is argued that the restraint effects are similar. However, in the prism, it has been shown by Lachance that there is no effect from the lateral restraint on the failing strengths obtained from these specimens. The 4 ins. central portion of the 4 x 4 x 12 ins. prism can be considered to be under a state of uni-axial compression[78]. It can be concluded from the above that the uni-axial compression strength of the normal concretes is in general 75 per cent of the 4 ins. or 6 ins. cube strengths. It is appreciated that size effects, i.e. the 'weakest link' theory, do influence the ratio of the 'true' uni-axial compression strength to the

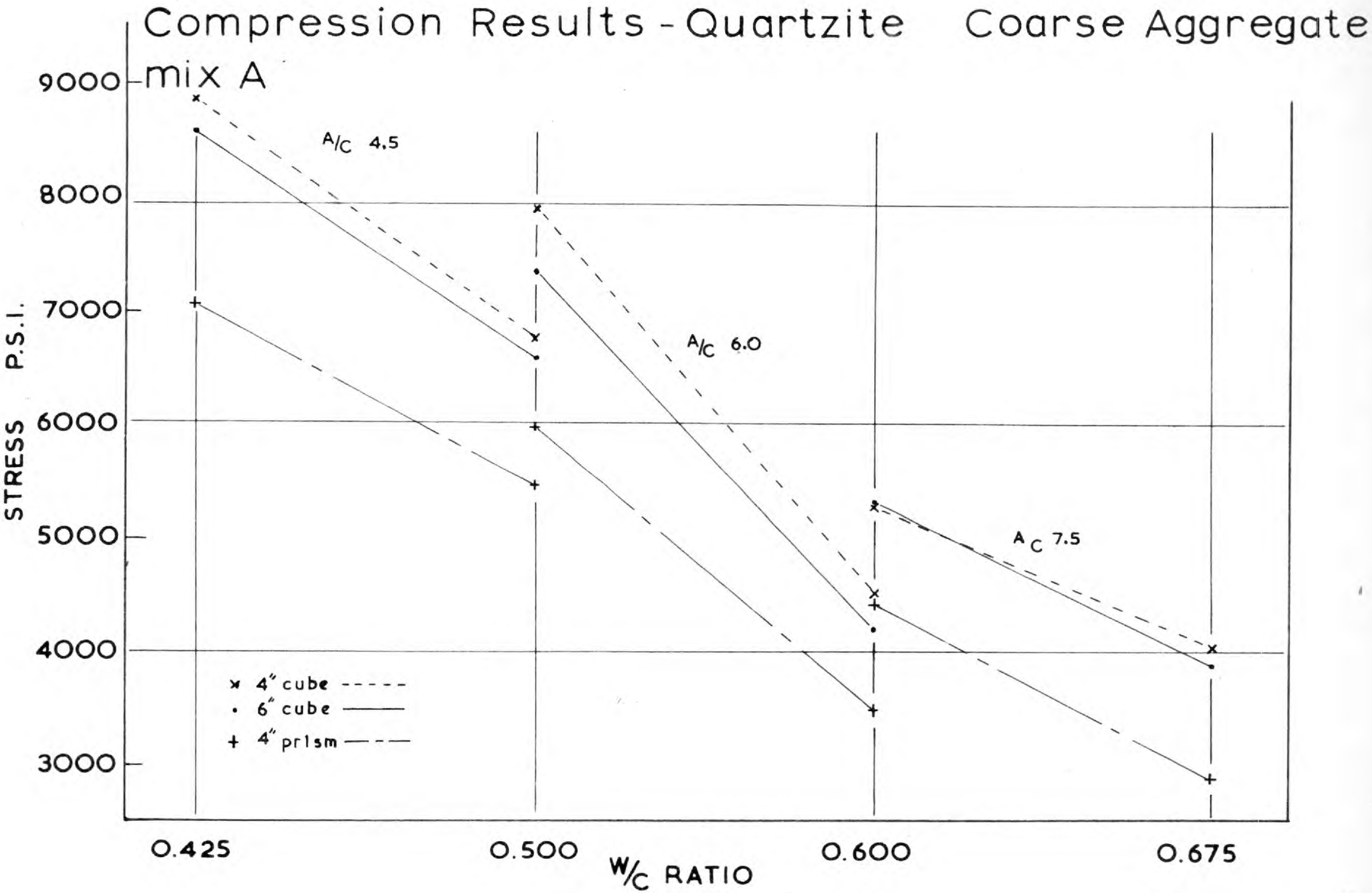

Compression Results - Quartzite Coarse Aggregate
mix A
9000
8000
7000
6000
5000
4000
3000
STRESS P.S.I.
A/C 4.5
A/C 6.0
A C 7.5
4" cube
6" cube
4" prism
0.425
0.500
0.600
0.675
W/C RATIO

FIG. 8.7

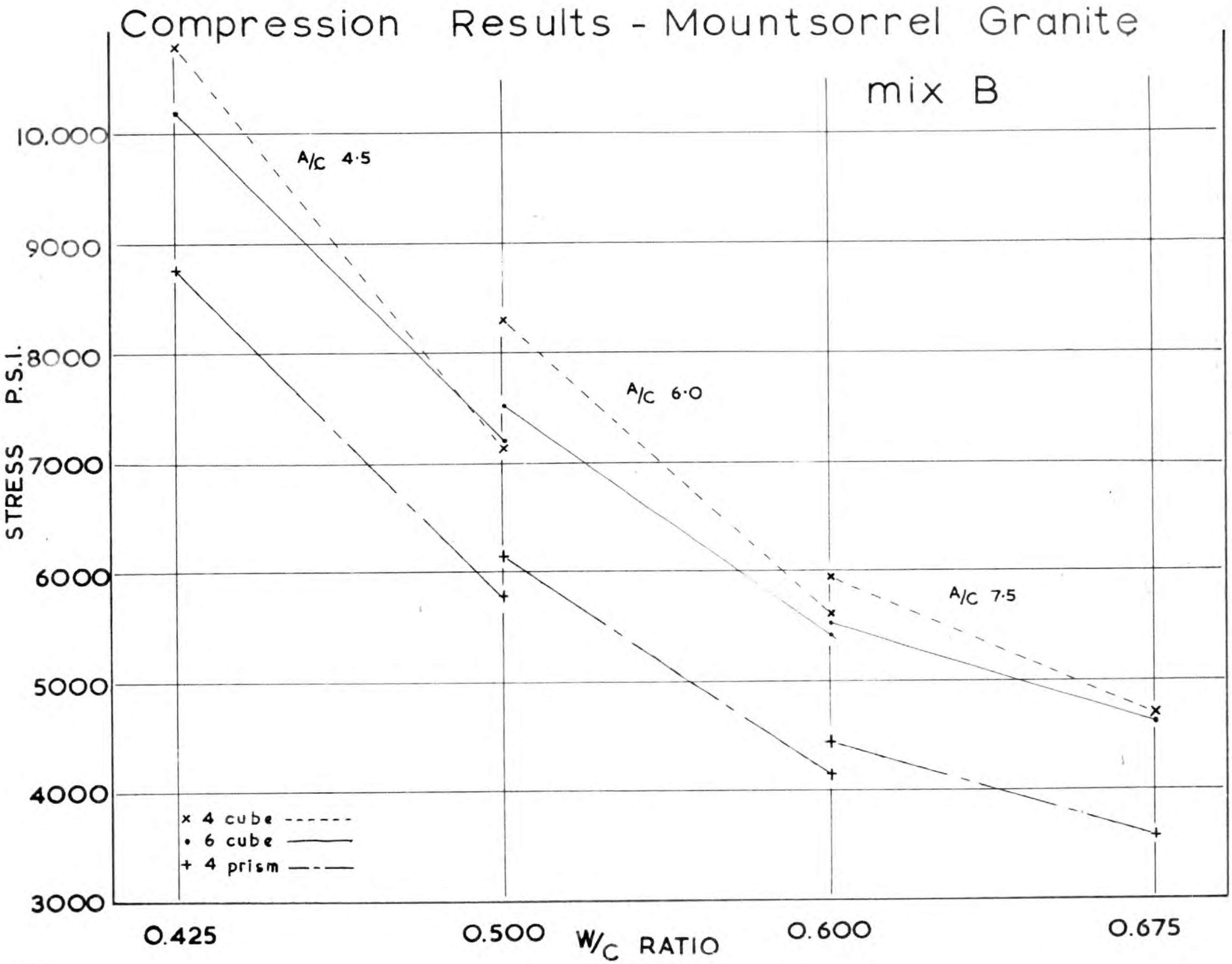
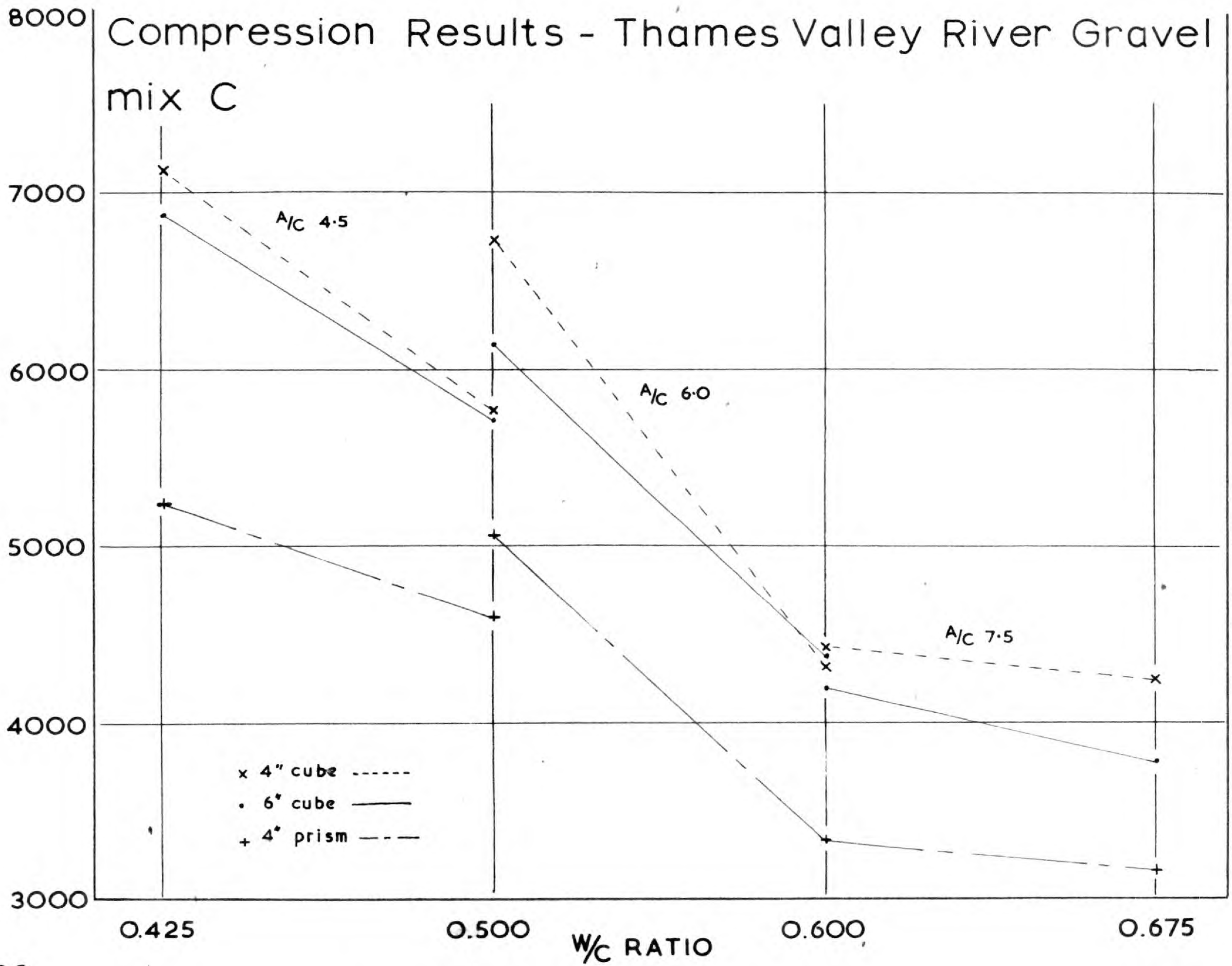

Compression Results - Mountsorrel Granite
mix B
A/C 4·5
A/C 6·0
A/C 7·5
10.000
9000
8000
7000
6000
5000
4000
3000
STRESS P.S.I.
0.425
0.500
0.600
0.675
W/C RATIO
× 4 cube
• 6 cube
+ 4 prism

FIG. 8.8



Compression Results - Thames Valley River Gravel
mix C
8000
7000
6000
5000
4000
3000
A/C 4·5
A/C 6·0
A/C 7·5
× 4" cube ------
• 6" cube ———
+ 4" prism — - —
0.425
0.500
0.600
0.675
W/C RATIO

FIG. 8.9

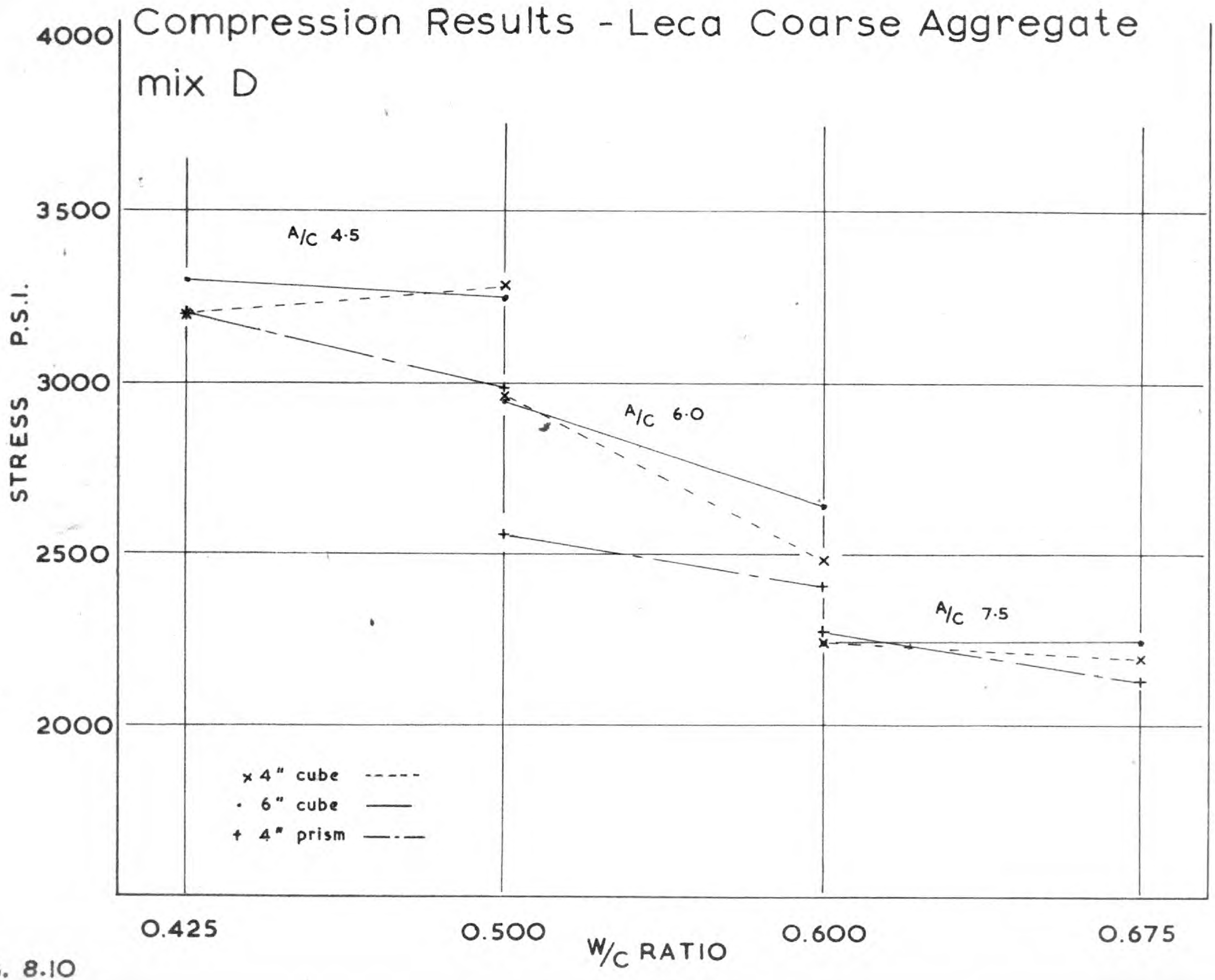
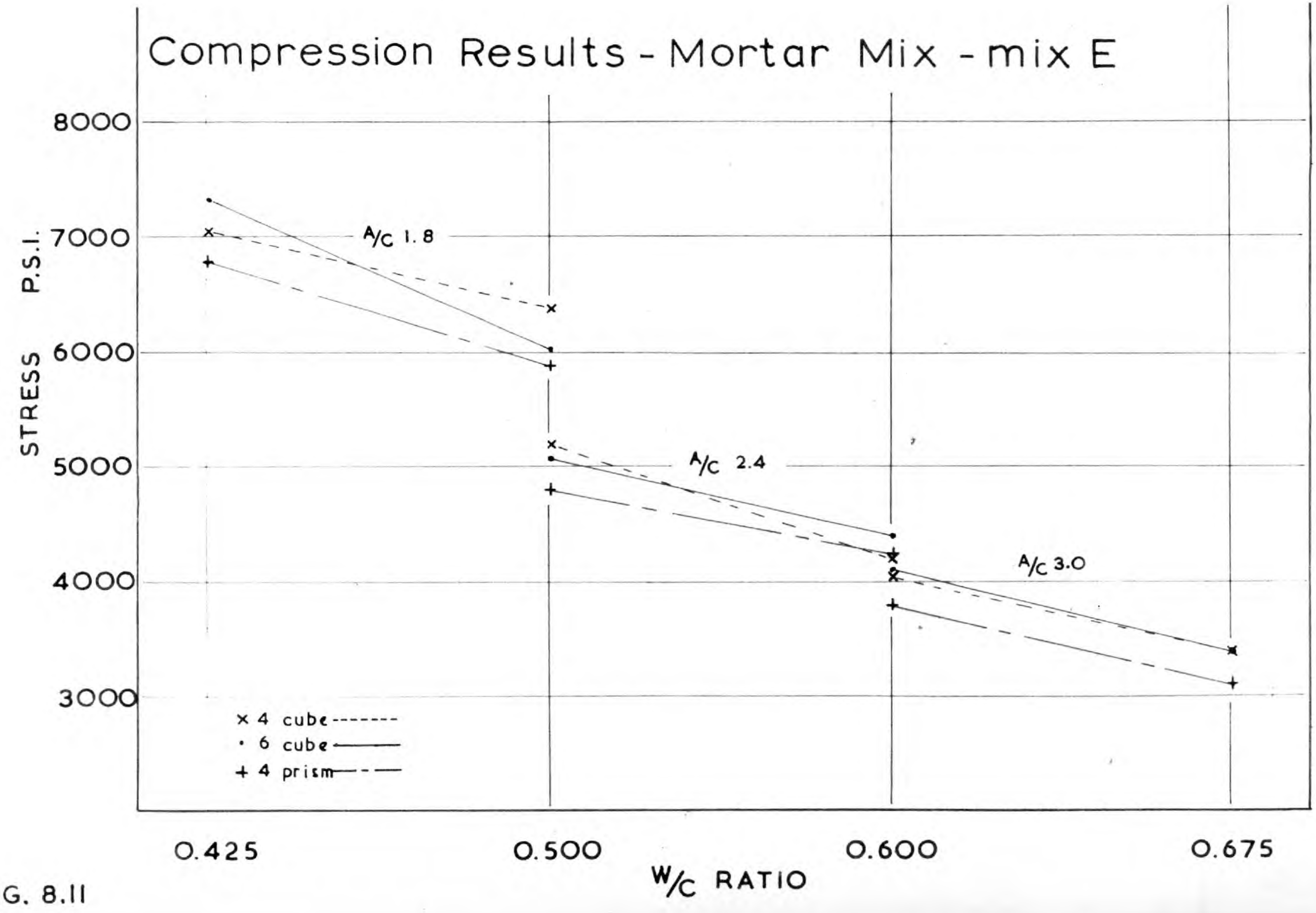

Compression Results - Leca Coarse Aggregate
mix D
4000
3500
3000
2500
2000
STRESS P.S.I.
A/C 4·5
A/C 6·0
A/C 7·5
× 4" cube
· 6" cube
+ 4" prism
0.425
0.500
0.600
0.675
W/C RATIO

FIG. 8.10



Compression Results - Mortar Mix - mix E
STRESS P.S.I.
8000
7000
6000
5000
4000
3000
A/C 1.8
A/C 2.4
A/C 3.0
× 4 cube
• 6 cube
+ 4 prism
0.425
0.500
0.600
0.675
W/C RATIO

FIG. 8.11

cube strength. For instance, it has been stated by several investigators[50,51] that the concrete for the strength range used in this work should give a cylinder to cube ratio (6 ins.diameter and 12 ins.long cylinder - 6 ins. cube) of from 0.75 - 0.80. From the previous argument this would mean that the cylinder is a measure of the 'true' uni-axial compressive strength of an mix. However, the height to diameter ratio is not large enough with the cylinder in order to obtain a state of uni-axial compression in the specimen[78]. The reason for the low ratio, which appears to agree with that obtained with the 4 ins. prism, is attributed to the larger volume of concrete in the cylinder coupled with a reduction in the lateral restraint effect due to the height to diameter ratio of 2.It is obvious that extreme care must be taken in interpreting ratios of apparent concrete strengths. Such ratios must be obtained from specimens of the same nominal dimensions under similar uniform states of stress.

iii. Effect of size and shape of specimen for martars and Leca concretes:

In Fig.8.10 and 8.11 and in columns 7 and 8 of Table 8.1, the compression results of the Leca concretes and the mortars are presented. The obvious difference from the results obtained with the normal concretes is the reduction in the differences between the various compression tests performed on any one mix. There appears to be no significant difference between the 6 ins. and 4 ins. cube strengths with either of these two materials. For the Leca concrete this is easily explained since the strength of these concretes is mainly a function of the strength of the aggregate particles. Thus, in general, little change in compression strength is obtained for a change in water/cement ratio, but significant reductions in strength are observed for an increase in aggregate/cement ratio. It is reasonable to assume that specimens of the same general order of size with 70 per cent of their volume consisting of material with very small compression strength would fail at similar overall ultimate stresses. The reason for the decrease in strength for an increase in aggregate/cement ratio observed in Fig.8.9 for the mortars is not so obvious. The discussion will be deferred to Chapter 10.

In column 8 of Table 8.1, it is seen that the ratios of the strengths obtained for mortars and Leca concretes from comparable 4 ins. prisms and cubes are significantly different compared to those for normal concretes. It is suggested that, for the mortar specimens, the particle size reduces the distance over which the end restraint is effective. Consequently, the cubes did not show such an appreciable strength advantage over the prisms as in the normal concretes. In fact, this has been proved indirectly by Lachance[78] who has shown that the effect of the lateral restraint on a mortar prism specimen diminishes more rapidly down the vertical height of the specimen compared with concrete prisms. From lateral measurements, Lachance observed that, in general, a concrete will exhibit restraint effects up to 2.5 ins. away from the ends of a 4 ins. prismatic specimen while for mortars, the restraint effect only extends for 1.5 ins. from the specimen ends.

iv. Effect of aggregate size on compression strength:

In work with Jones[49], Kaplan found that with the 4 ins. equivalent cube compression specimen used, the concretes, in general, gave higher values of compressive strength than comparable mortars. Recently, Kaplan[16] reported that the opposite result was obtained when using cylindrical compression specimens 6 ins.diameter and 12 ins. long. This latter specimen is less affected by end restraint than 4 ins. equivalent cubes. In the present work, similar results were obtained for the Thames Valley river gravel mixes, using the 4 ins. cube and prism. (see columns 10 and 12, Table B). It is suggested that the major contributing factor to this behaviour is the difference in the ratios of prism strengths to cube strengths for concretes and mortars, as discussed above. It will also be observed that the results of the high strength mortar mixes are in fact similar, or even higher than comparable Thames Valley river gravel concretes. It should be noted that the mortar sand was a Thames Valley natural river sand and therefore of similar shape and surface texture to the Thames Valley gravel coarse aggregate. It is thought that this can be attributed to the bond characteristics of comparable large and small size particles. Alexander[59] has shown that for an increase in particle size, a decrease in bond strength is obtained. In the lower water/cement ratio mixes, the bonding of paste to the Thames Valley river gravel coarse aggregate particles is the weakest part of the specimen. Mortars which consist of small particles with a relatively high bond strength would therefore be expected to produce strengths approaching more closely the stronger mixes containing coarse aggregate particles such as Thames Valley river gravel with lower paste-aggregate bond strengths. The outcome of the above discussion is that it is apparent that the mortar of a concrete has in general a higher 'true' compressive strength than the parent Thames Valley river gravel concrete. Where coarse aggregates are used which have better bond characteristics, the opposite result is found.

v. Effect of aggregate type on compression strength:

With all the compression tests, the mixes containing the granite aggregate gave the highest ultimate compressive strengths. Fig.8.7 - 8.11 show that the granite mixes give ultimate compressive strengths a) slightly higher than the results obtained from the quartzite mixes and the mortars, b) in general, 1.3 times the results from the Thames Valley river gravel mixes, and c) 2 - 3 times the strengths obtained from comparable lightweight aggregate mixes. This aspect will be discussed further in Chapter 10.

8.3.3 Relationship Between Uni-axial Tension and Indirect Tension

In Fig.8.12, the uni-axial tension ultimate strengths for all mixes (average of three specimens) have been plotted against the indirect tension ultimate strengths. The regression equations for the normal concrete for estimating D from C is

$$D = 0.858\ C + 63$$

$$r = 0.965$$

$$S_D = 17.6 \text{ psi.}$$

where: D = ultimate direct tensile strength

C = ultimate cylinder splitting strength

r = correlation coefficient (see Appendix D)

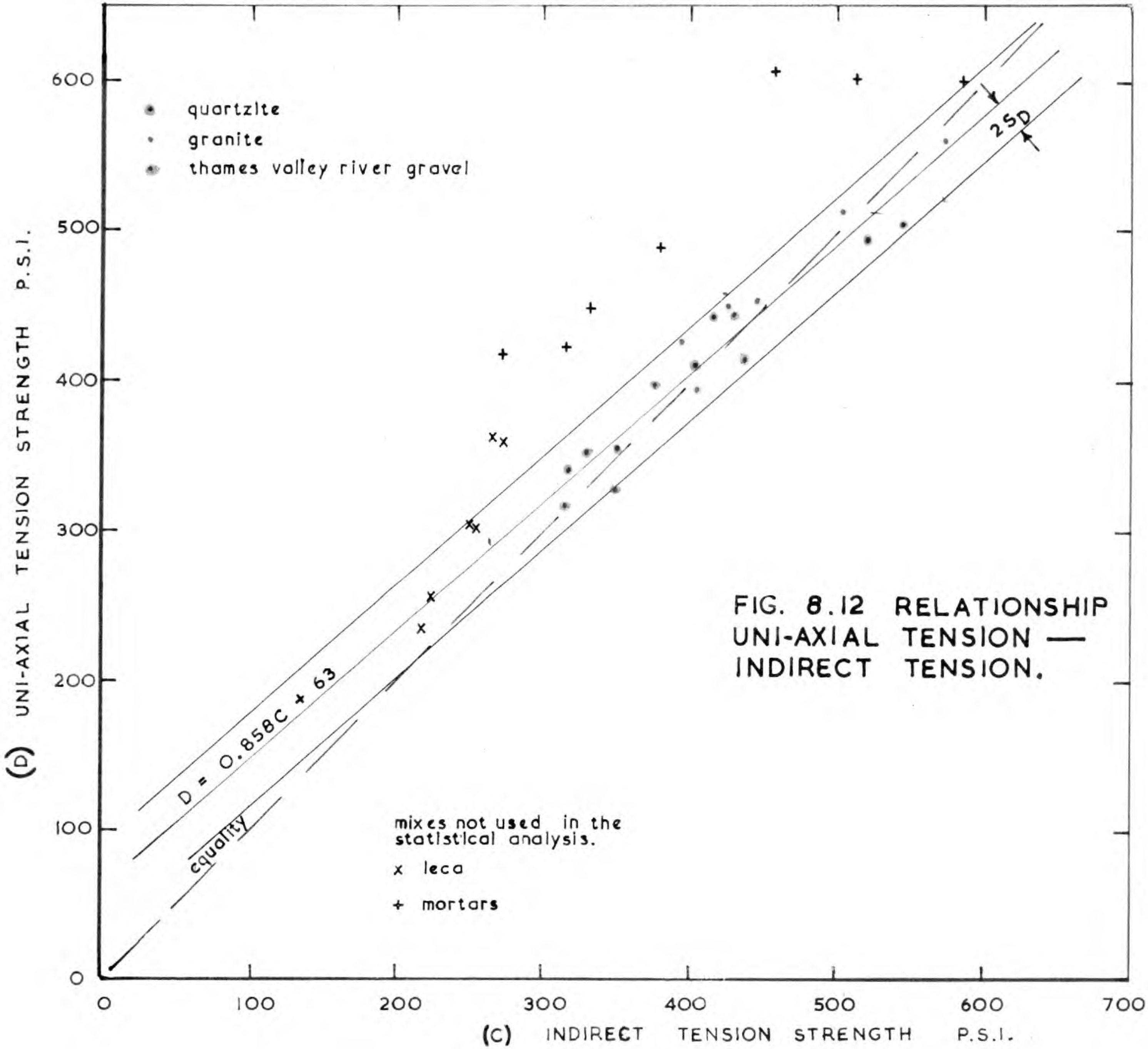


FIG. 8.12 RELATIONSHIP UNI-AXIAL TENSION — INDIRECT TENSION.

S_D = standard error of estimate of D. (i.e. in 95 per cent of the cases the actual values will lie within plus or minus two standard errors of the estimate values given by the regression equation.)

The regression equation for the normal concretes for estimating C from D is,

$$C = 1.085D - 40$$
$$r = 0.965$$
$$S_C = 19.8 \text{ psi.}$$

The high correlation coefficients, r, indicate that the derived relationships may be taken to be representative of the 'parent universe' with negligible risk of error. In fact, the correlation coefficients were checked using Student's't' distribution[109] and were found to be significant at the 0.1 per cent level. The standard errors of estimate S_D and S_C are low. The results show therefore that over a fairly wide range of strengths (300 - 600 psi.) the uni-axial strength of a mix may be predicted with confidence from indirect tension test results from the same mix for concretes containing the normal coarse aggregates used in this investigation. The importance of this conclusion will be readily apparent.

It was decided not to use the ratios where the indirect tension test results of the mortars and Lecas were involved since the indirect tension test did not give reliable values testing these materials. These values were plotted on Fig.8.12 and are indicated by crosses. There is a certain trend indicated by the figure that the ratio of indirect tension to uni-axial tension increases for higher strength mixes. It is suggested that the reason for this might be attributed to the position at which failure is initiated in the indirect tension test. Let us assume that, with the packing strips used in this investigation, the specimen began to fail under the loading strips in a compression-shear mode. The final failure is along the diametrical plane but at low overall strengths, the ratio of ultimate tension to compression strength is relatively high (see Fig.8.14). Thus it would be expected that the chances of failure being initiated under the loading strip would be considerably increased. Now if the loading strips were able to perform perfectly for the higher strength mixes where a significantly lower tension to compression ratio exists, it would be expected that failure would initiate anywhere on the diametral plane joining the loading strips and in general, slightly higher indirect tension strengths would be obtained. It is considered that this may provide also an explanation for the apparent decrease in the ratio of flexural to splitting ultimate strengths with increasing mix strength as reported by many investigators[61,103].

8.3.4 Relationship between Uni-axial Tension and Flexural Tension

Fig.8.13 shows the ultimate flexural strengths plotted against the ultimate uni-axial tension strengths (average of three specimens). The regression equation for all the concretes and mortars for estimating D from F is,

$$D = 0.631F + 61$$
$$r = 0.958$$
$$S_D = 23.5 \text{ psi.}$$

and for estimating F from D,

$$F = 1.45D - 45$$
$$r = 0.958$$
$$S_F = 35.5 \text{ psi.}$$

where F = the ultimate flexural strength

D = uni-axial tension strength

Both of the above regression equations are highly significant due to the high values of the correlation coefficient. The standard errors are small considerering that the calculation of the regression equations included the results of the normal and Leca concretes and the mortars. The intercepts of the regression equations are small particularly for the equation for estimating flexural strength from uni-axial tension results. This implies that there is almost a directly proportional

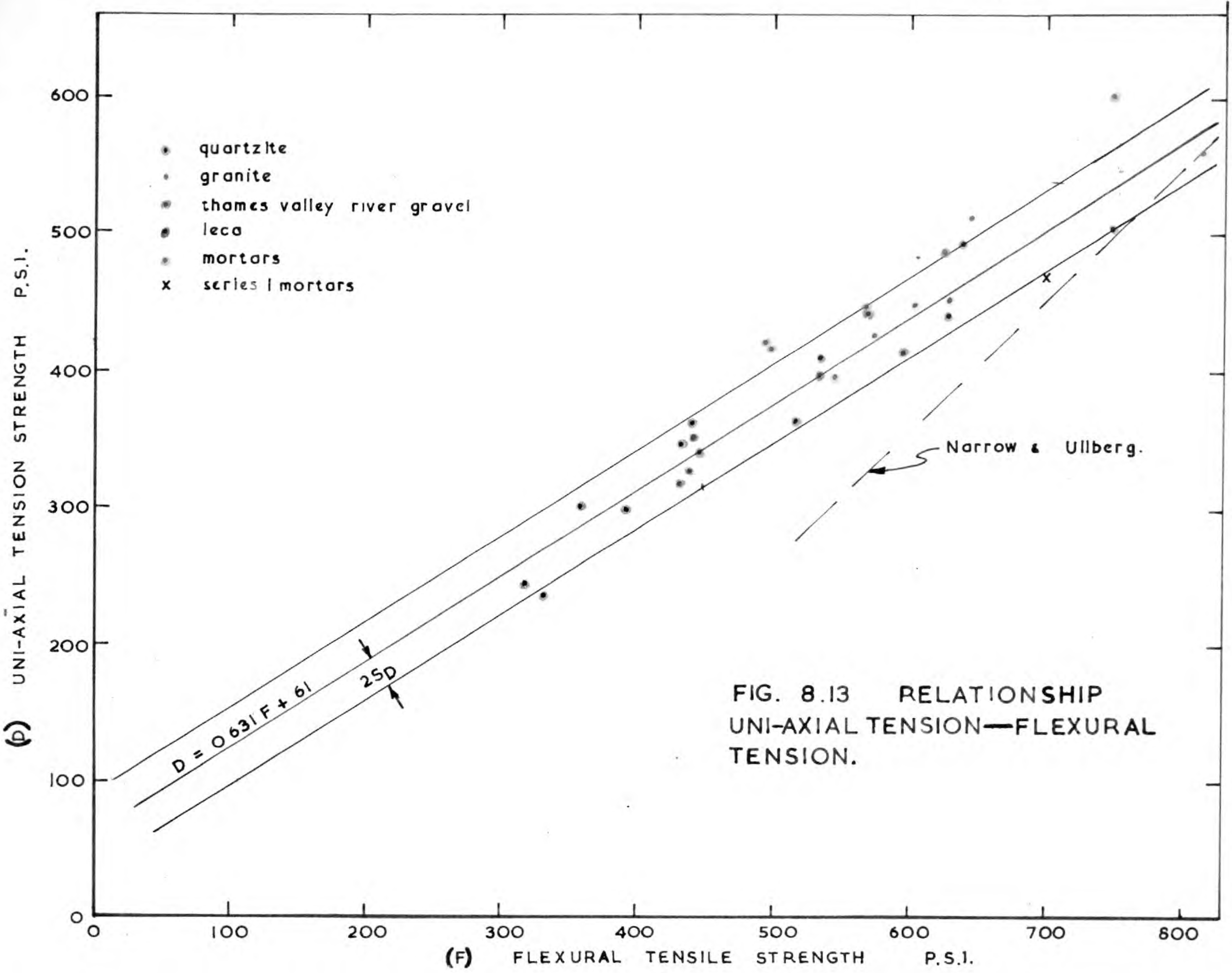


FIG. 8.13 RELATIONSHIP UNI-AXIAL TENSION—FLEXURAL TENSION.

relationship existing between flexural and uni-axial tension ultimate strengths.

It is interesting to note the results obtained by Narrow and Ullberg[61]. Their results are from indirect tension tests carried out on 6 ins. diameter and 12 ins. long cylindrical specimens, and flexural tests performed on 6 x 6 x 21ins. beams using a third point loading system. On Fig.8.13, the regression line for the results of Narrow and Ullberg[61] has been plotted,

$$F = 229 + 1.043C$$

$$r = 0.964$$

$$S_F = 20 \text{ psi.}$$

where the symbols have the same meaning as before. It is seen that the ratio of ultimate flexural strengths to ultimate indirect tension strengths decreases with increasing mix strength. This indicates that the incremental increase in indirect tension strength is relatively greater than the incremental increase in flexural strength for an increasing overall mix strength. Now it has been suggested in the discussion of Fig.8.12 that the ratio of uni-axial tension to indirect tension decreases for increasing mix strength. This would be in general agreement with the regression line of Narrow and Ullberg. It is thought that the slope and intercept of the Narrow and Ullberg results, which were highly significant statistically, could have resulted from two factors which would account for the greater increase in indirect tension strength to flexural strength with an increase in mix strength. Firstly, the strip effect outlined in the discussion of Fig.8.12 (their investigation used 'hard tempered masonite strips, 2 ins. wide and 1/8 ins. thick'). Secondly, the ultimate load behaviour of the indirect tension test at different mix strengths may be partially a function of the biaxial state of stress existing in the specimen instead of a wholly tensile failure as assumed.

8.3.5 Relationship between Uni-axial Tension and Uni-axial Compression

Fig.8.14 shows the relationship between uni-axial tension strengths as measured using the new testing method, and uni-axial compression strengths obtained from the Lachance 4 x 4 x 12 ins. prism. In general, a curvilinear relationship appears to exist between these two basic tests. The percentage of tension to compression ultimate strengths is highest for mortars at any mix strength, the highest value being 13.4 per cent at a uni-axial compression strength of 3100 psi. The lowest ratio percentage of 6.4 per cent was obtained from a mix containing granite coarse aggregate at a uni-axial compression strength of 8750 psi., corresponding to 4 ins. and 6 ins. cube strengths of 10,800 psi. and 10,000 psi. respectively. It is necessary to refer to the stress-strain relationships obtained from test series 3 to offer satisfactory explanations of the curvilinear relationship between uni-axial tension and compression. Therefore, discussion is deferred to Chapter 10.

Fig.8.15 has been obtained from Mitchell[39]. The general relationship curves obtained from a variety of investigators are shown. The curve of the Writer (obtained from Fig.8.14) has been superimposed onto the figure. It is seen that good agreement is obtained between the Writer's curve and other workers. The compressive strength in the diagram is from cylinders but it was shown theoretically earlier in the discussion that the results obtained from the uni-axial test in this work would be similar to cylinder results from the same mix. Accepting this, the location of the curve indicates that in general the results obtained from the uni-axial tension test are higher than those obtained from a variety of indirect and direct tesion tests used by other investigators.

Fig.8.16 was obtained from W.C.French Ltd.[104], the manufacturers of Leca lightweight aggregate. It is a figure showing the relationship between ultimate strength results obtained from 6 ins.diameter and 9 ins. long indirect tension specimens and 6 ins. compression cubes. Superimposed on the figure are the points obtained by the Writer from the indirect and direct tension tests and 6 ins. compression cubes. It will be remembered that this Writer used hardboard loading strips in the indirect tension test. The investigation carried out by French used strips of plywood of similar size. It is seen that the original and this Writer's indirect tension/6 ins. cube results are quite similar.

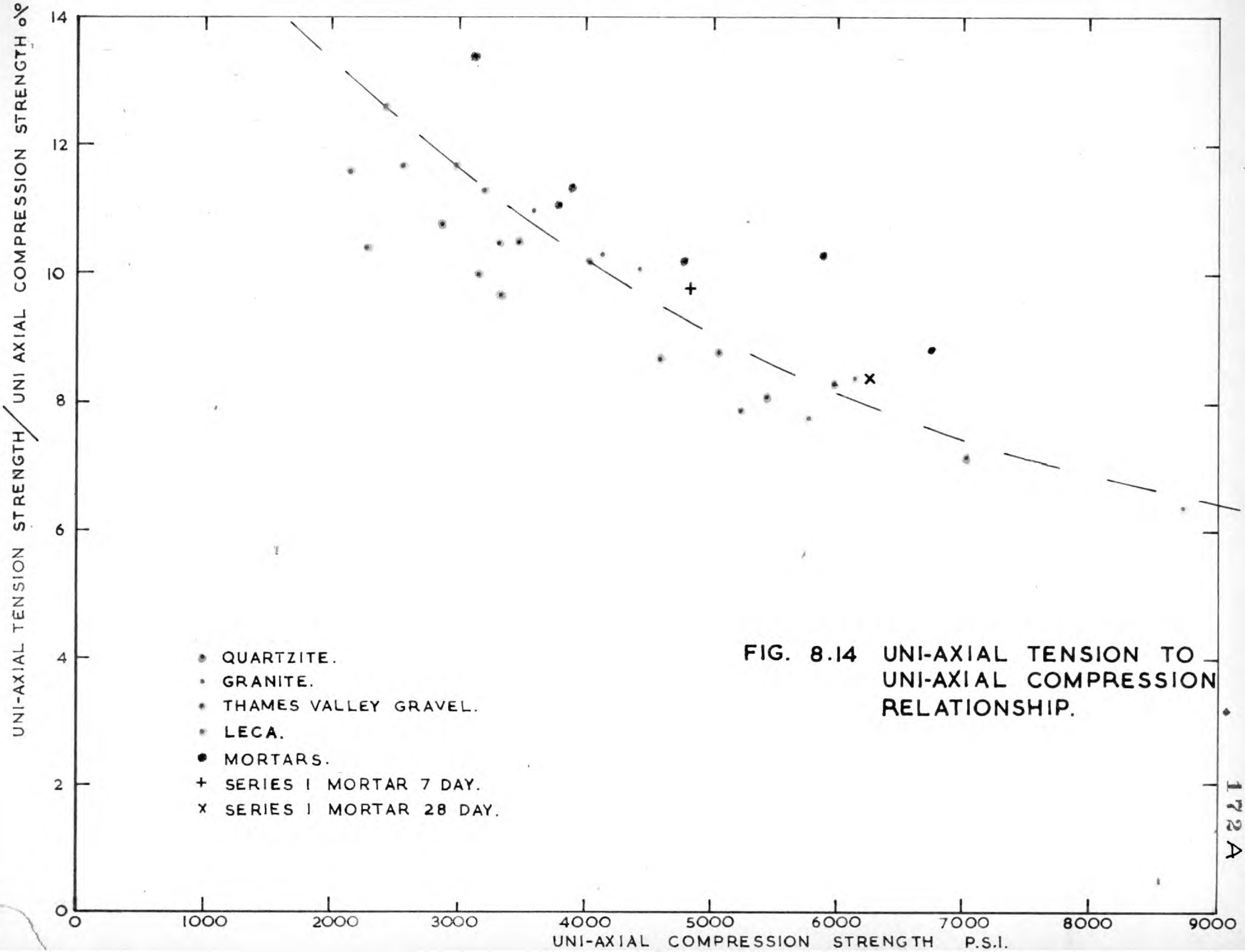


FIG. 8.14 UNI-AXIAL TENSION TO UNI-AXIAL COMPRESSION RELATIONSHIP.

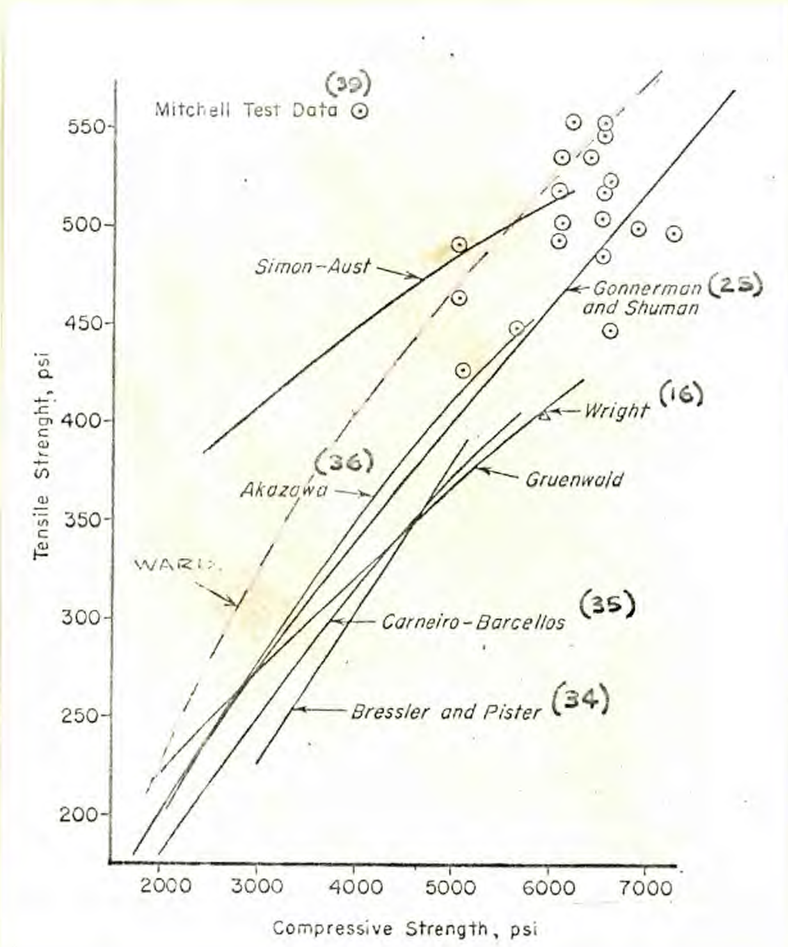


Fig. 8.15 Relation Compression to Tension (Mitchell[39])

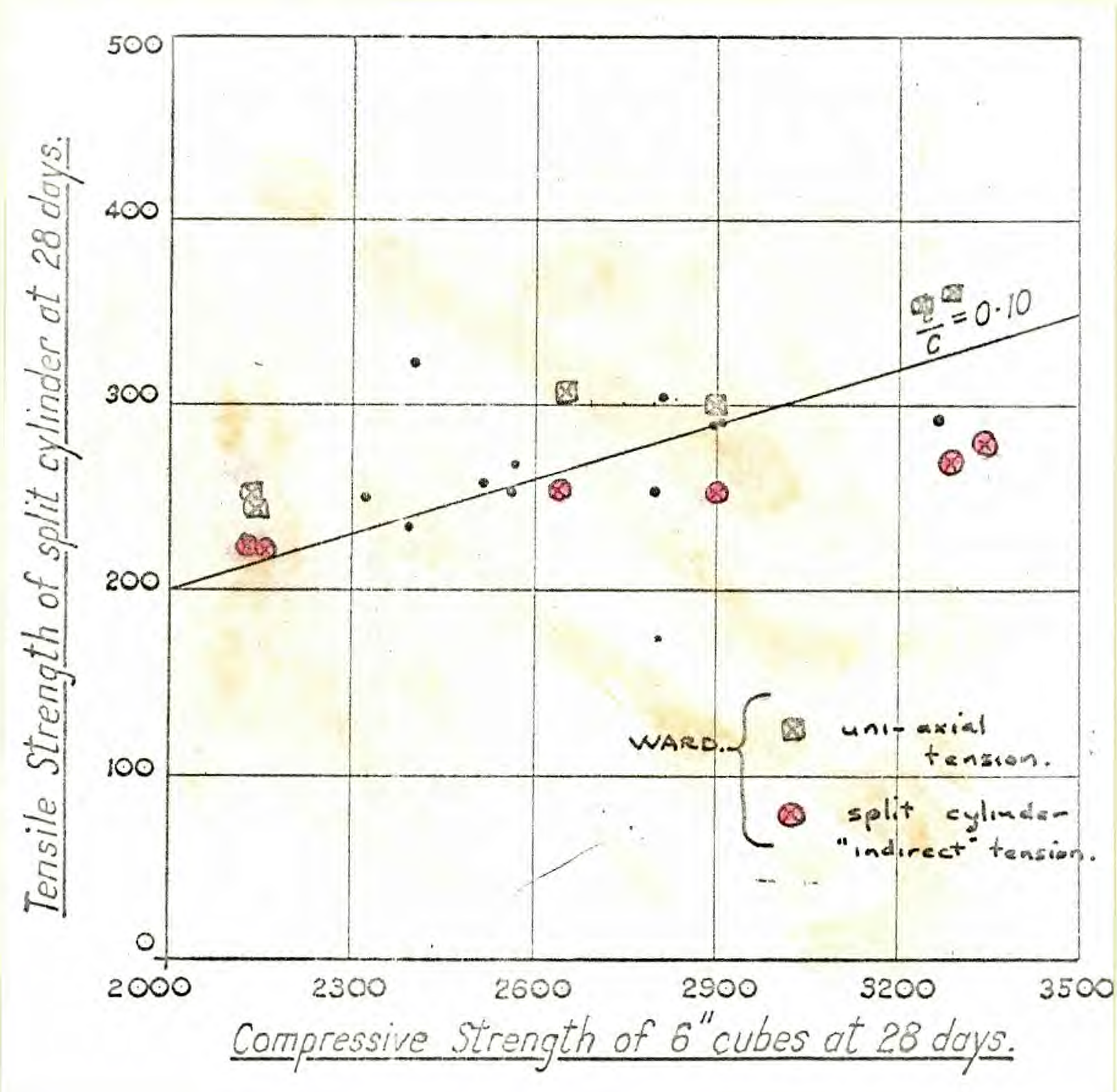


Fig. 8.16 Relation Compression to Tension
Leca aggregate concretes (W.C. French Ltd.)

In general, the uni-axial tension test gives results **greater than** either indirect tnsion test. It is thought that the initiation of failure in the tests of W.C. French may have taken place under the loading strips, as suggested for the Writer's tests. This would indicate that the indirect tension test as used here is not a suitable means for assessing the tensile strength of Leca concrete mixes.

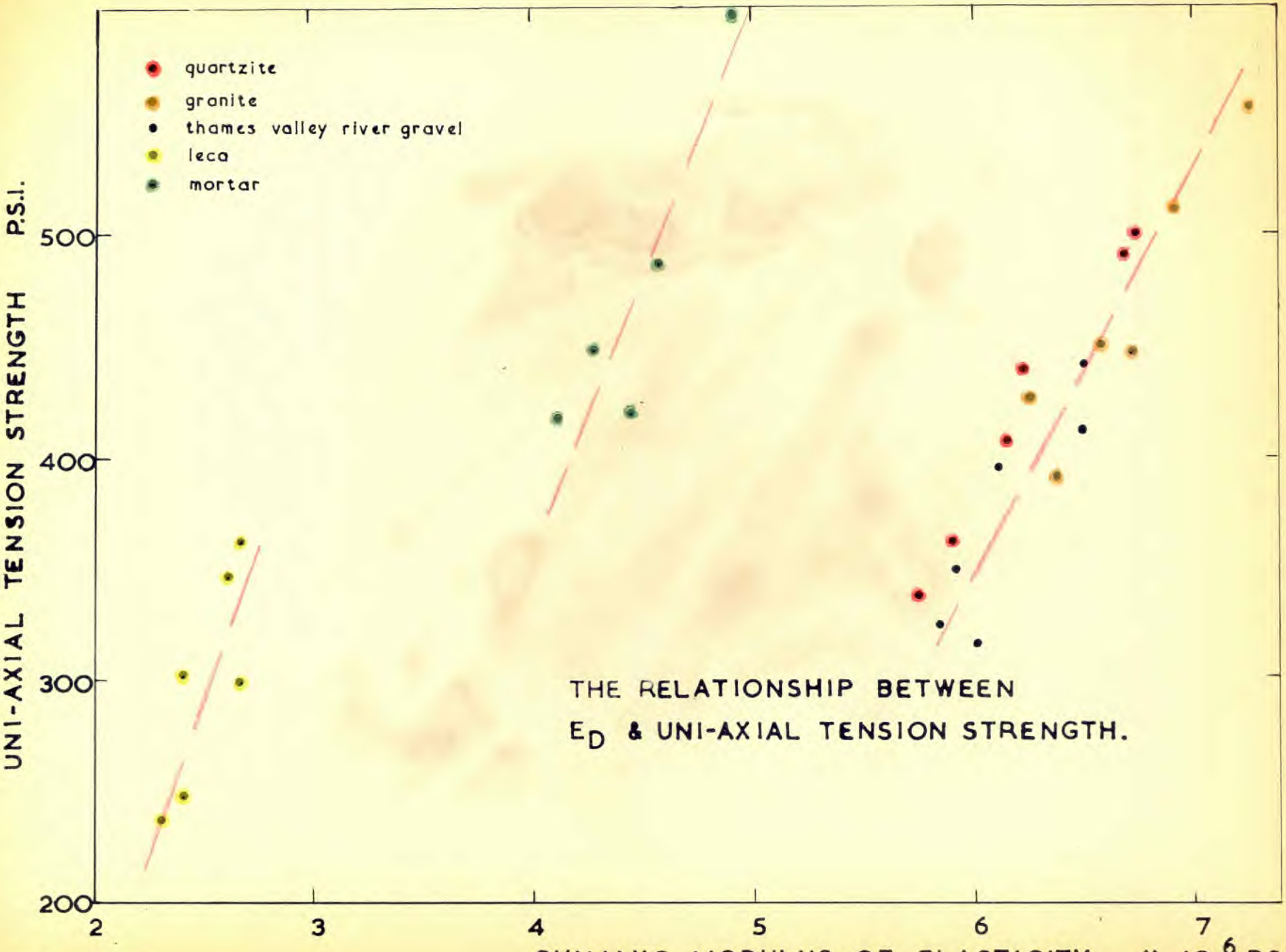
8.3.6 Non-destructive Testing

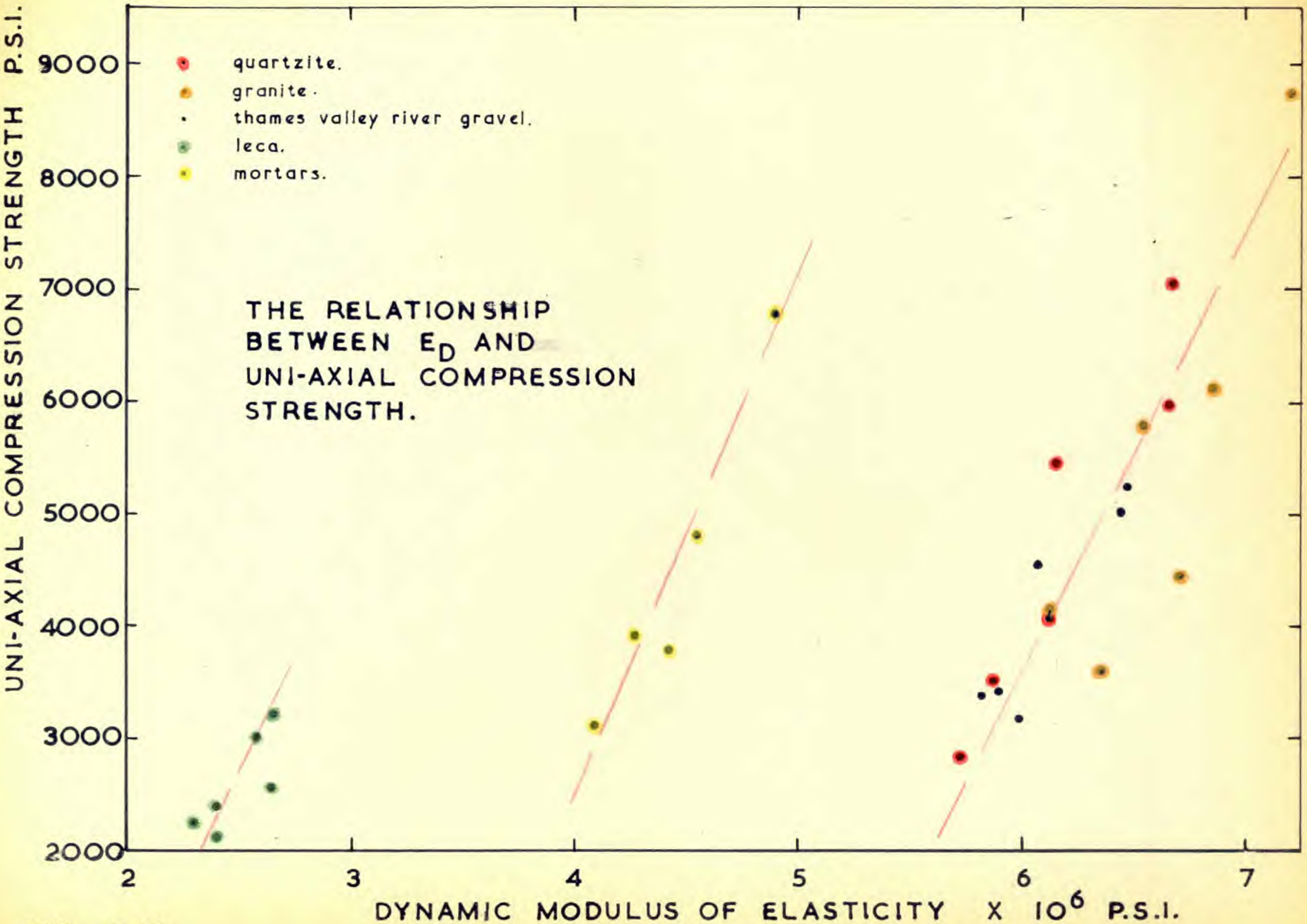
It is unfortunate that the Cawkell UCT2 tester which was employed to measure the ultrasonic pulse velocity through the flexural beam specimens, did not function properly during this test series. As a result, it is impossible to obtain meaningful information of the dynamic Poisson's ratio which is a function of U.P.V. and resonant frequency.

In Fig.8.17, the dynamic modulus of elasticity (E_D) has been plotted against the uni-axial tension strength. In general, a decrease in **dynamic modulus results in a reduction in the** measured ultimate uni-axial tension strengths for all normal concrete mixes. However, it is noted that over the range of normal concrete mixes tested, the range of dynamic modulus of elasticity was 5.75×10^6 to 7.25×10^6, an increase of 25 per cent, while the tensile strength varied from 315 psi. to 560 psi., an increase of 78 per cent. This indicates that dynamic modulus of elasticity is an insensitive indication of the tensile strength. It does suggest however, that some form of limiting strain criteria appears to exist for the normal concretes, the Leca concretes and the mortars separately.

Similar relationships and conclusions exist for Fig.8.18 where the dynamic modulus of elasticity has been plotted against ultimate uni-axial compression strengths. For the normal concrete mixes, the range of E_D is 5.75×10^6 to 7.25×10^6, an increase of 25 per cent, while the ultimate compression strength varied from 2800 psi. to 8750 psi., which is an increase of 210 per cent. It is apparent that E_D is even a more unreliable means of predicting compression strength than uni-axial tension strengths even within ranges of similar aggregate types.

The results above show that the range of ultimate tensile strengths is significantly less than the range of compression strengths. For the


quartzite
granite
thames valley river gravel
leca
mortar
UNI-AXIAL TENSION STRENGTH P.S.I.
500
400
300
200
2
3
4
5
6
7
DYNAMIC MODULUS OF ELASTICITY X 10 6 P.S.I.
THE RELATIONSHIP BETWEEN
ED & UNI-AXIAL TENSION STRENGTH.
FIG. 8.17



UNI-AXIAL COMPRESSION STRENGTH P.S.I.
9000
8000
7000
6000
5000
4000
3000
2000
quartzite.
granite.
thames valley river gravel.
leca.
mortars.
THE RELATIONSHIP BETWEEN E_D AND UNI-AXIAL COMPRESSION STRENGTH.
2
3
4
5
6
7
DYNAMIC MODULUS OF ELASTICITY $\times 10^6$ P.S.I.

FIG. 8.18

normal concretes, the range of tensile strengths showed that the highest was 1.78 times the lowest. The corresponding range for the compression strengths showed that the highest was 3.10 times the lowest. It is suggested that this is related to the mode of failure and does indicate that for complete disruption, different factors are important in determining the ultimate strengths. The major factor in tension is the limiting cohesion of the mix components which is affected by changes in aggregate type and water/cement ratio. It will be suggested in Chapter 9 that cohesive strength is related also to the lateral deformational behaviour of concrete in compression. However, in compression, changes in the stiffness and interparticle strength of the specimen resulting from decreases in the water/cement ratio etc., might bring about the wider fluctuations in compressive strength with changing mix parameters. This probably occurs because of the relatively higher internal stresses and strains which compression specimens are subjected to, as opposed to the tension specimens.

It is interesting to note that the ratio of the highest to the lowest ultimate strengths obtained in uni-axial compression and tension for the lightweight aggregate concretes as drawn in Fig.8.17 and 8.18 is the same, 1.5. This is attributed to the low compression and tensile strength of the Leca aggregate, which would thus appear to be the weakest component in all the lightweight mixes. For instance, it is seen in Fig.8.5 and 8.10 that the strength of the Leca mixes is more related to the aggregate cement ratio than a change in water/cement ratio.

In Table B, the damping constant (Q) measurements are given. It is seen that in general a decrease in strength is usually accompanied by a decrease in damping constant. This is similar to the results obtained by Evans[84]. Unfortunately, with the damping constant measurements, there was quite a large scatter of results. This is to be expected since part of the experimental determination involves obtaining a small difference from two large measured quantities. This work was carried out as an ancillary part of the programme and if a major study of the properties of concretes which affect the damping

Table 8.1 Summary of averaged results for all test for each aggregate type. (Complete results in Appendix B.

Coarse aggregate type (3/4 ins. max. size)	designation	Ultimate failing strength ratios											
		$\frac{\text{uni-ax.tens.}}{\text{uni-ax.comp.}}$		$\frac{\text{uni-ax.tens.}}{\text{cyl.ind.tens.}}$		$\frac{\text{uni-ax.tens.}}{\text{flexure}}$		$\frac{\text{cyl.ind.tens.}}{\text{flexure}}$		$\frac{\text{6 ins.cube}}{\text{4 ins.cube}}$		$\frac{\text{12 ins.prism}}{\text{4 ins.cube}}$	
		Ratio	V_1	Ratio	V_1	Ratio	V_1	Ratio	V_1	Ratio	V_1	Ratio	V_1
Quartzite	A	.093	2.0	1.01	5.1	.73	5.5	.73	4.9	.95	3.3	.77	5.2
Mountsorrel Granite	B	.090	2.0	1.02	3.9	.74	5.4	.72	5.0	.96	4.2	.77	3.6
Natural Thames Valley River Gravel	C	.094	1.1	1.02	5.1	.75	3.2	.74	4.8	.96	4.6	.76	3.2
Leca lightweight material	D	.116	0.8	1.20	9.0	.79	6.6	.66	6.0	1.01	3.2	.95	4.2
Mortar of above concretes	E	.109	1.7	1.33	11.0	.81	2.9	.61	8.4	1.00	4.0	.93	1.7
column No. (1)	(2)	(3)		(4)		(5)		(6)		(7)		(8)	

V_1 = the estimate of the coefficient of variation of the ratio using the method of Appendix D.

Each ratio given above has been obtained from 6 pairs of results, where each result is the average value obtained from three specimens.

constant were to be undertaken, the present apparatus should be modified. One major critism is the unsatisfactory nature of the supporting system on the sonic bench which caused a component of energy to be lost to the surroundings. It is of interest to note that the damping constant appeared to increase as the specimen dryed out, even though the specimen was drying over a period as little as ten minutes. This is one of the many problems associated with the measurement of Q.

8.4 SUMMARY

A summary of the main points of interest from test series 2 will now be given.

1. The location of the plane of failure of the uni-axial tension specimen has been shown to be restricted in general to the central reduced portion of the specimen having the reduced cross-sectional area.

2. For all the concretes, with the exception of those with Leca aggregates, an increase in aggregate/cement ratio results in an increase in the ultimate strength results obtained from all the destructive tests. With the Leca concretes and the mortars, the opposite result was obtained.

3. Using the same volumetric mix proportions, mixes containing a granite coarse aggregate gave higher ultimate compressive strengths than all other materials tested. In general, the mortars of the concretes gave the highest tensile strengths recorded. For the concrete, the granite coarse aggregate mixes gave the highest ultimate tensile strength results.

4. A curvilinear relationship exists between uni-axial compression and uni-axial tension failing strengths. The highest ratios between tension and compression strength were obtained from the lower strength mixes.

5. Good correlation has been obtained between uni-axial tension failing strengths and indirect tension and flexural tension failing strengths.

6. Good correlation was also obtained between the dynamic modulus of elasticity and the uni-axial tension and compression failing strengths, independently for the mortars, Leca concretes and normal concretes. For all materials, an increase in E_D is associated with an increase in ultimate tension or compression strength.

7. There is an indication from the results that the indirect tension test does not give acceptable results over the wide range of materials tested. In particular, anomolous low results were obtained with this test when used for Leca concretes and all mortars.

8. The general relationships between the various destructive tests are summarized in Table 8.1.

CHAPTER 9

THE STRESS-STRAIN BEHAVIOUR OF CONCRETES IN UNI-AXIAL COMPRESSION AND TENSION - TEST SERIES 3 AND 4

9.1 INTRODUCTION

Chapter 9 is concerned with the presentation and analysis of the results obtained from the third and fourth test series. Both of these test series were planned to investigate the deformational behaviour of concrete materials in uni-axial tension and compression. In test series 3, a concrete containing Thames Valley river gravel coarse aggregate, its mortar and the cement paste of the mortar were tested in uni-axial tension and compression. Strain readings were taken up to failure, longitudinally and laterally. Test series 4 was carried out on various concretes and mortars to provide information on the effect of the moisture condition and loading rate on the deformational behaviour and failure of these materials.

A detailed description is given in Chapter 6 of the testing methods employed throughout the two test series. Chapter 5 contains an outline of the experimental work and a list of the specimens used.

The discussion to follow in 9.3 will be related mainly to the evidence obtained from test series 3 and 4. Discussion is left in some instances to Chapter 10 where the interelation of the results obtained from these test series and test series 1 and 2, are covered.

The amplification of any statistical methods used in this chapter is given in Appendix D.

9.2 PRESENTATION OF RESULTS

Section 6.7.4 describes how a tape recorder was used in this test series to enable strain readings to be taken right up to the ultimate load of specimens tested in uni-axial compression and tension. The results obtained from the tensile tests are shown in Figs. 9.1 - 9.12 and from the compression tests in Figs. 9.13 - 9.24.

Dynamic modulus of elasticity values were obtained from two flexural beam specimens at each age in accordance with 6.6.5. The average of the results so obtained are plotted against (1) the tensile 'tangent' moduli in Fig.9.25, and (2) the compression 'tangent' moduli in Fig.9.26. The 'tangent' moduli are the apparent tangent moduli at zero stress which have been created by the preloading operation as described in 6.7.1. The 'tangent' moduli are therefore also the values of secant moduli corresponding to the stresses at approximately 15 per cent of the ultimate strengths of both the tension and compression specimens. In Fig.9.27, the compression tangent moduli are plotted against the tension tangent moduli. In the above graphs, all the results from tests on cement pastes, mortars and concretes have been plotted.

Using the tape recorder technique, it is considered that the strain was recorded until the uni-axial tension specimens were within approximately 2 per cent of their ultimate load. This conclusion is based on two factors: (1) in general, the strain was increasing very rapidly in the ultimate load region and there was no time to read the extensometer before failure occurred, and (2) extensometer readings were taken in general, at 15 psi. increments as the ultimate load approached. An attempt was made to assess the limiting tensile strain of the concretes, mortars and pastes from the tensile stress-strain graphs. It appears reasonable to assume from the arguments presented above that the strains recorded as the overall limiting tensile or cohesive strains were in fact those strains at about 98 per cent of the ultimate failing load of the specimen. These are given in Table 9.4. The average overall limiting tensile strain values observed in Figs.9.1 - 9.12 from the two tension specimens tested on any one day have been plotted in Fig.9.28 against the average lateral strains, measured at 60 and 70 per cent of the ultimate compressive strength, obtained in the corresponding uni-axial compression tests.

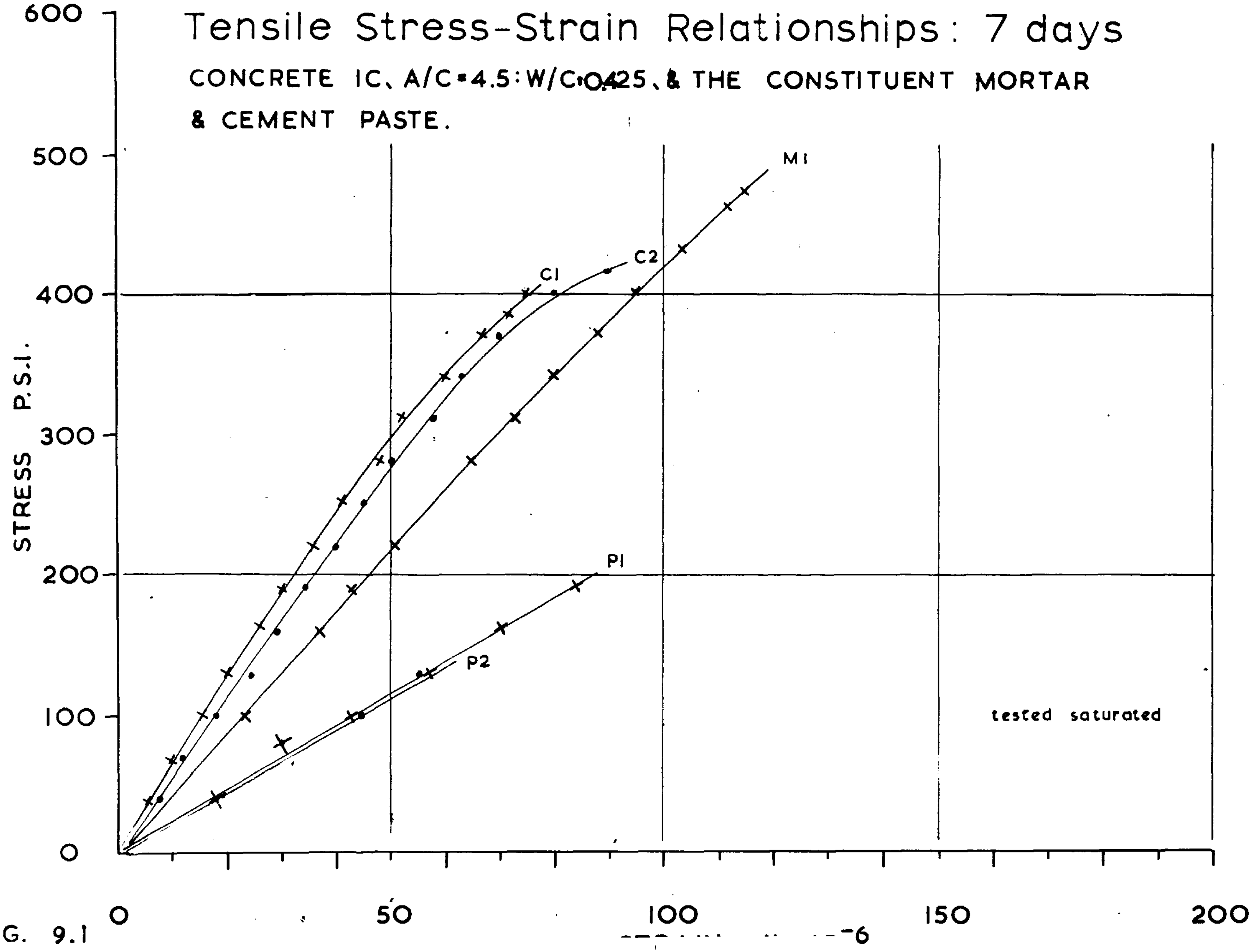
Unfortunately, for some unaccountable reason, the Lachance lateral gauges which were used in the compression tests could not be calibrated properly. The readings obtained from these gauges were consistent and of the right order which misled the Writer during the test series. It appears that the lateral gauge readings need to be multiplied by a factor of 0.74 approximately, to produce meaningful results. To overcome the difficulties imposed by the introduction of this fault, 'true' lateral readings were estimated by multiplying the longitudinal readings obtained from the Lamb's longitudinal roller extensometer by Poisson's ratios obtained by Anson. These values of Poisson's ratio were obtained in the Imperial College Laboratory with mixes containing similar sand and coarse aggregate, similar mix proportions, the same testing machine, at the same age and curing conditions, and using the same 4 ins. longitudinal extensometer as the Writer. However, Anson used a Lamb's lateral roller extensometer to measure the lateral strains instead of the Lachance gauge. In the light of the above and the fact that Anson's results showed that Poisson's ratio did not change significantly up to the load range of 60 - 70 per cent of the ultimate load, it is considered that the calculation technique adopted is justified. The adopted values of Poisson's ratio are given in Table 9.1.

Table 9.1 Poisson's ratios according to Anson[87].

Mix	A/C	Poisson's ratio
1C,MC,2C	4.5	0.152
3C,NC,4C	6.0	0.150
1E,ME,2E	1.8	0.190
3E,NE,4E	2.4	0.180

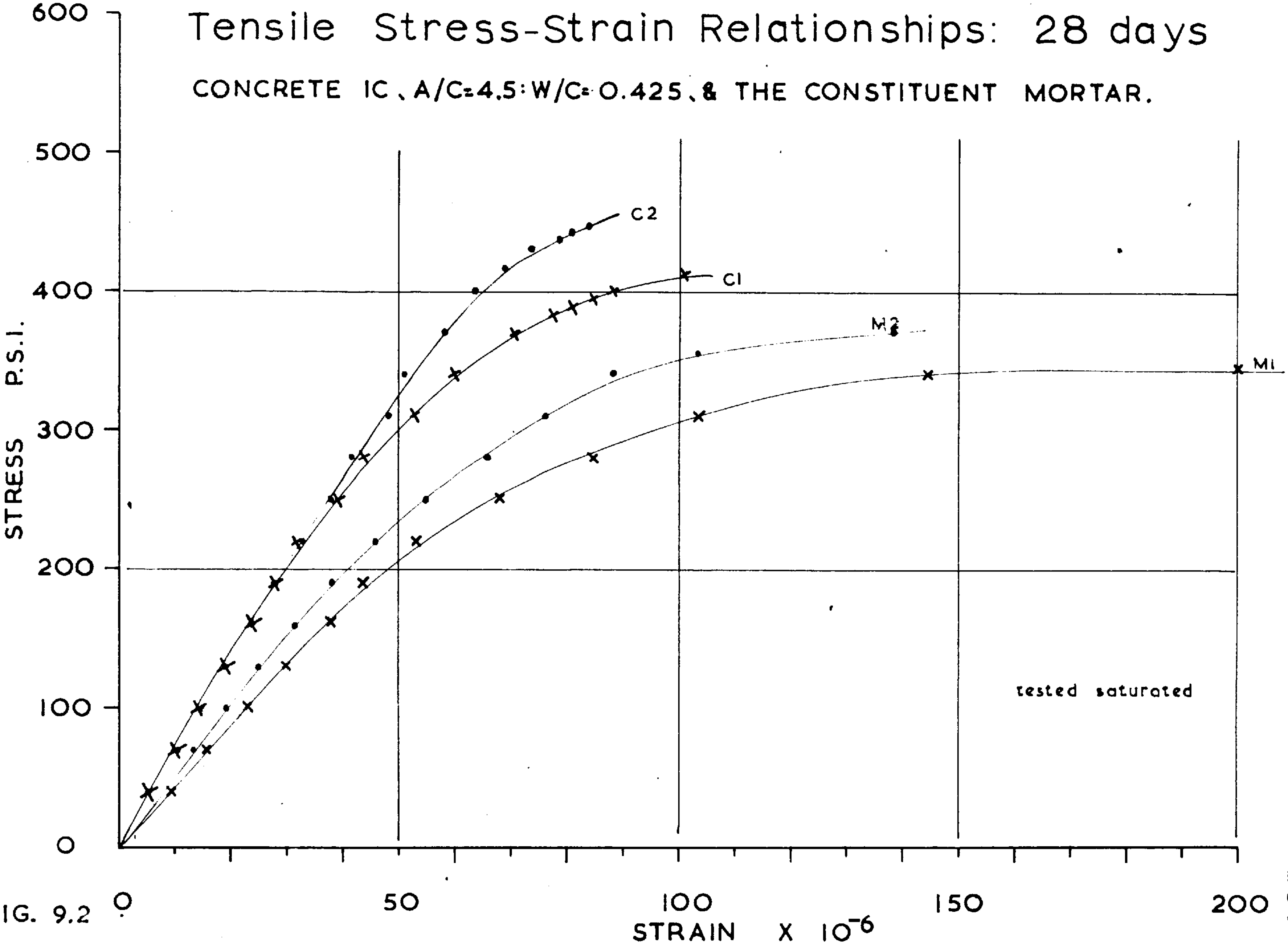
Fig.9.29 shows the pilot test results which were carried out to determine the effect of moisture loss on the tensile strength of concrete. The ultimate strength in uni-axial tension has been plotted against the number of days cured in air before testing.

The dynamic moduli of elasticity (E_D) obtained from resonant frequency measurements have been plotted in Fig.9.30 against the dynamic Poisson's ratios obtained from E_D and the ultrasonic pulse



Tensile Stress-Strain Relationships: 7 days
CONCRETE IC, A/C=4.5: W/C=0.425, & THE CONSTITUENT MORTAR & CEMENT PASTE.
STRESS P.S.I.
600
500
400
300
200
100
0
0
50
100
150
200
M1
C1
C2
P1
P2
tested saturated
-6

FIG. 9.1



Tensile Stress-Strain Relationships: 28 days
CONCRETE IC, A/C=4.5: W/C= 0.425, & THE CONSTITUENT MORTAR.
STRESS P.S.I.
600
500
400
300
200
100
0
C2
C1
M2
M1
tested saturated
0
50
100
150
200
STRAIN X 10^{-6}

FIG. 9.2

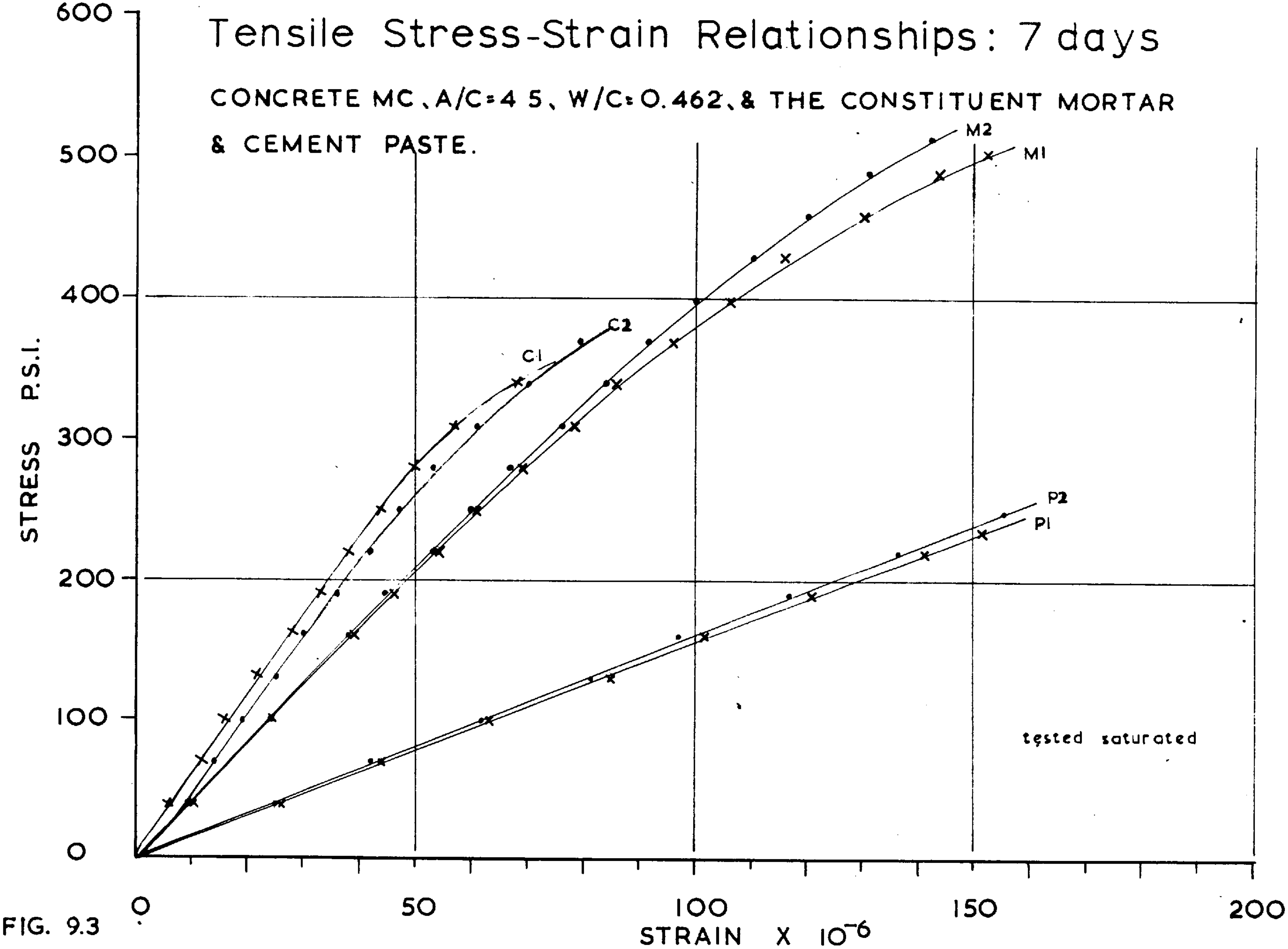
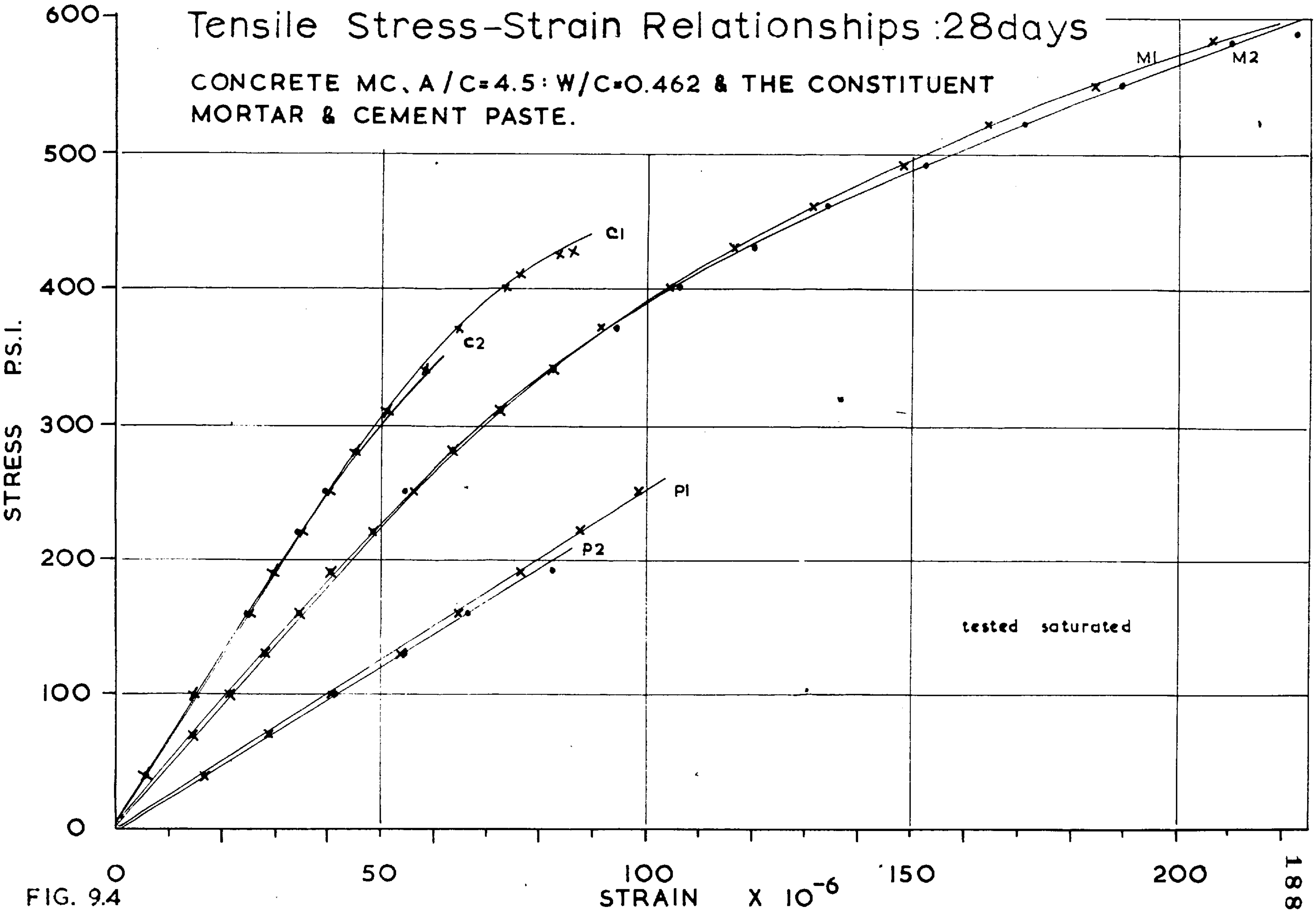
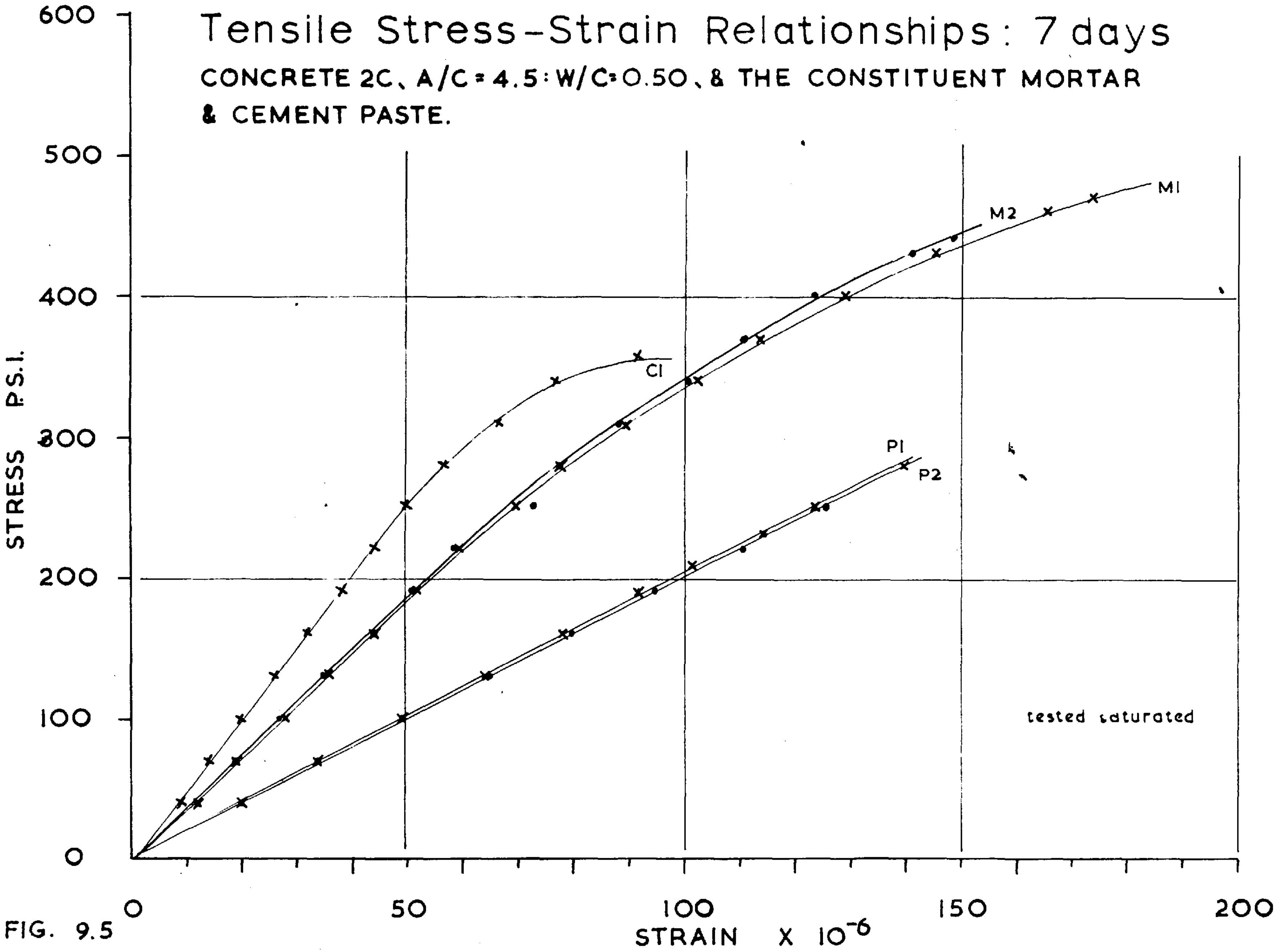

Tensile Stress-Strain Relationships: 7 days
CONCRETE MC, A/C=4 5, W/C=0.462, & THE CONSTITUENT MORTAR & CEMENT PASTE.
STRESS P.S.I.
600
500
400
300
200
100
0
0
50
100
150
200
STRAIN X 10^{-6}
C1
C2
M2
M1
P2
P1
tested saturated

FIG. 9.3



Tensile Stress-Strain Relationships :28days
CONCRETE MC, A/C=4.5 : W/C=0.462 & THE CONSTITUENT MORTAR & CEMENT PASTE.
STRESS P.S.I.
600
500
400
300
200
100
0
0
50
100
150
200
STRAIN X 10^{-6}
C1
C2
M1
M2
P1
P2
tested saturated

FIG. 9.4



Tensile Stress-Strain Relationships: 7 days
CONCRETE 2C, A/C = 4.5 : W/C = 0.50, & THE CONSTITUENT MORTAR & CEMENT PASTE.
STRESS P.S.I.
600
500
400
300
200
100
0
M1
M2
C1
P1
P2
tested saturated
0
50
100
150
200
STRAIN X 10^{-6}

FIG. 9.5

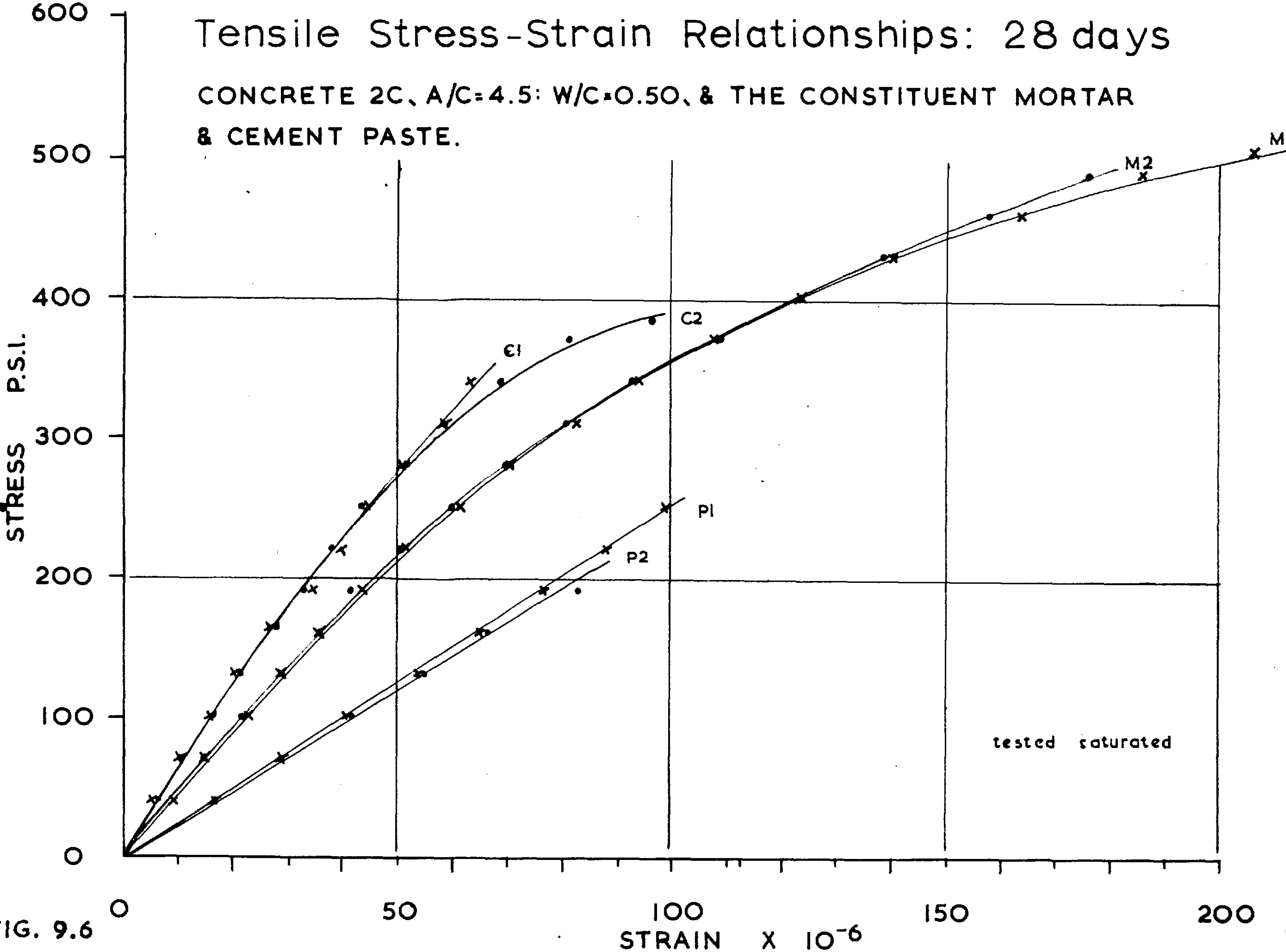

Tensile Stress-Strain Relationships: 28 days
CONCRETE 2C, A/C=4.5: W/C=0.50, & THE CONSTITUENT MORTAR
& CEMENT PASTE.
STRESS P.S.I.
600
500
400
300
200
100
0
STRAIN X 10^{-6}
0
50
100
150
200
C1
C2
M1
M2
P1
P2
tested saturated

FIG. 9.6

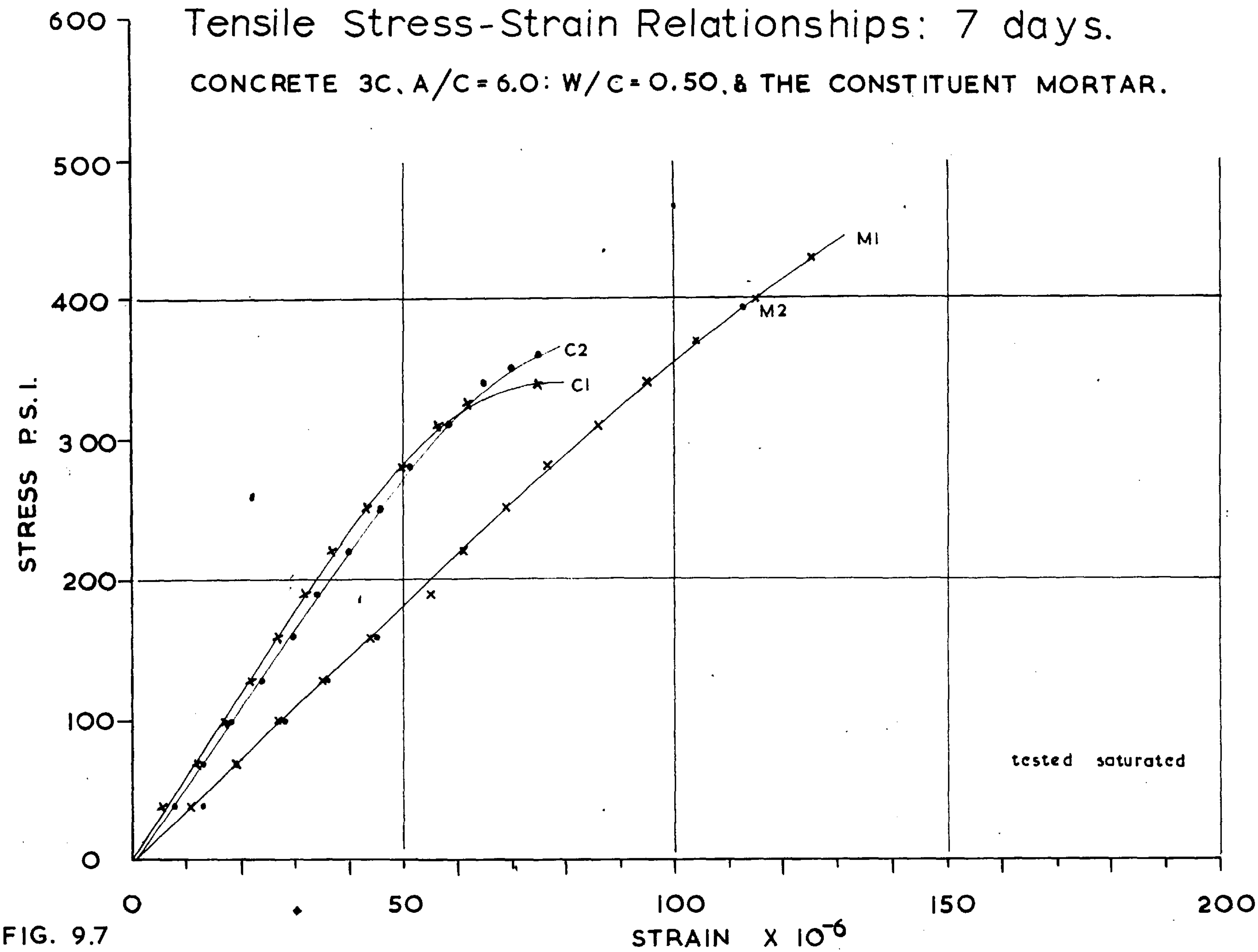

Tensile Stress-Strain Relationships: 7 days.
CONCRETE 3C, A/C = 6.0: W/C = 0.50, & THE CONSTITUENT MORTAR.
STRESS P.S.I.
600
500
400
300
200
100
0
STRAIN X 10^{-6}
0
50
100
150
200
M1
M2
C2
C1
tested saturated

FIG. 9.7

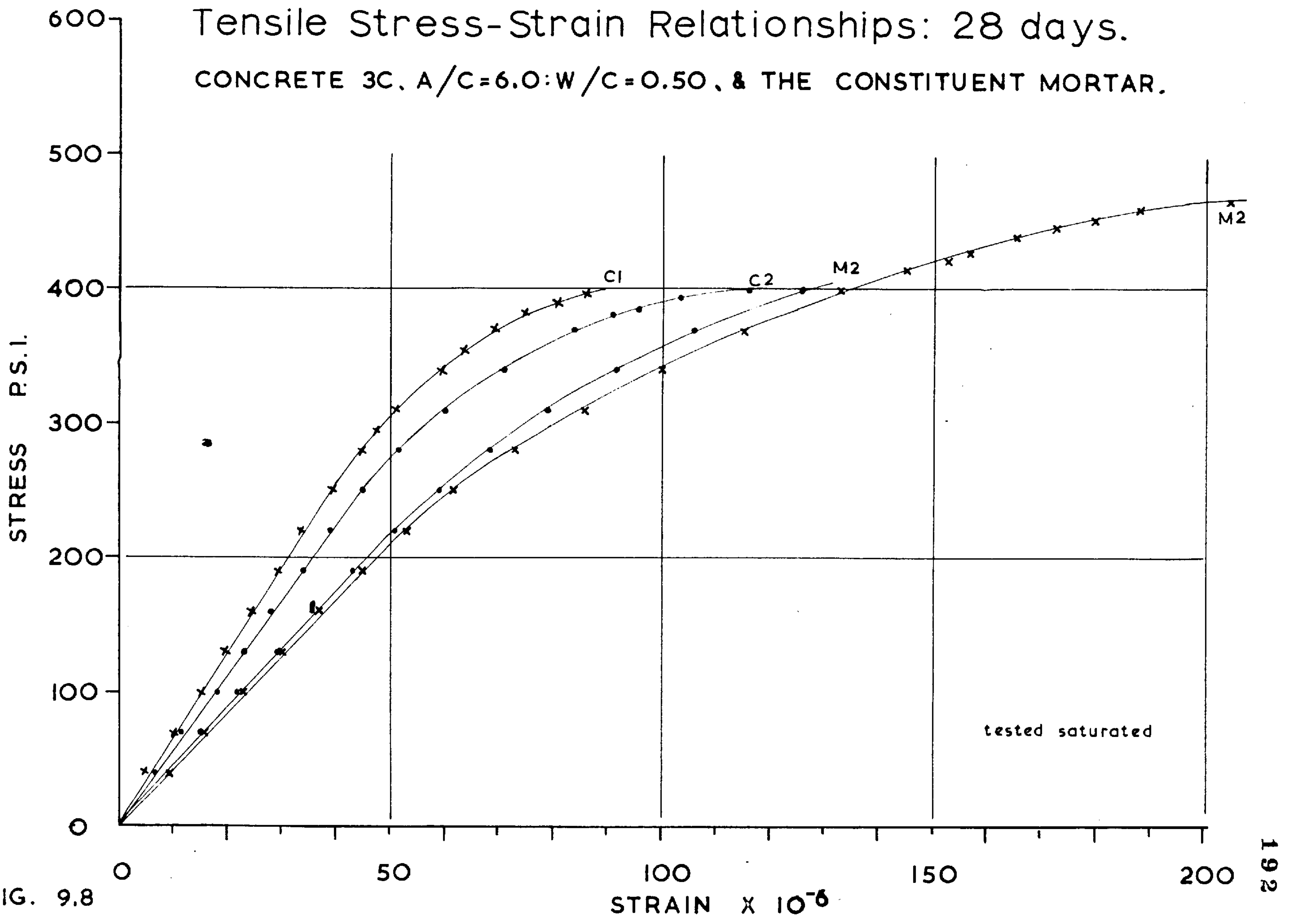
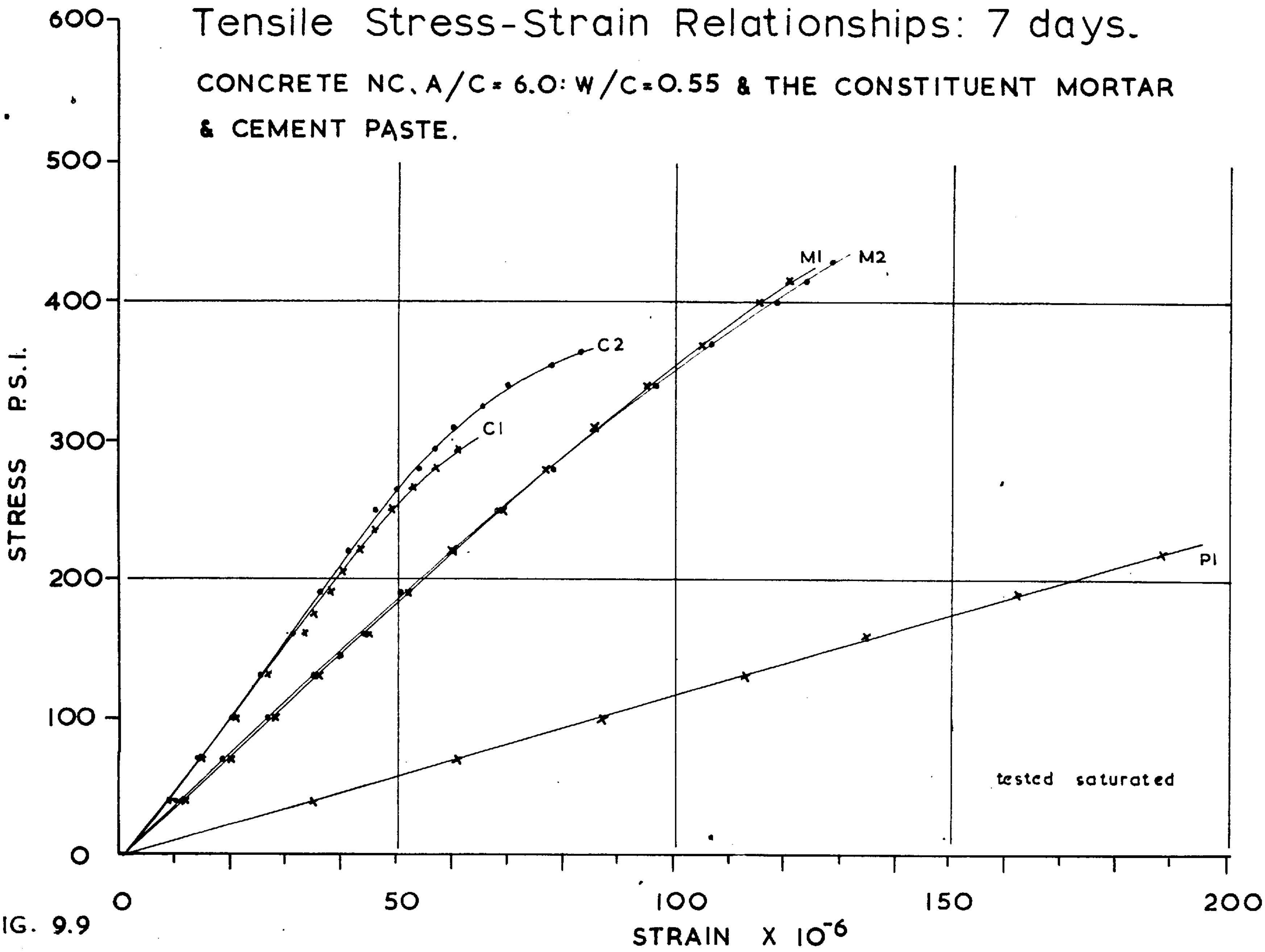
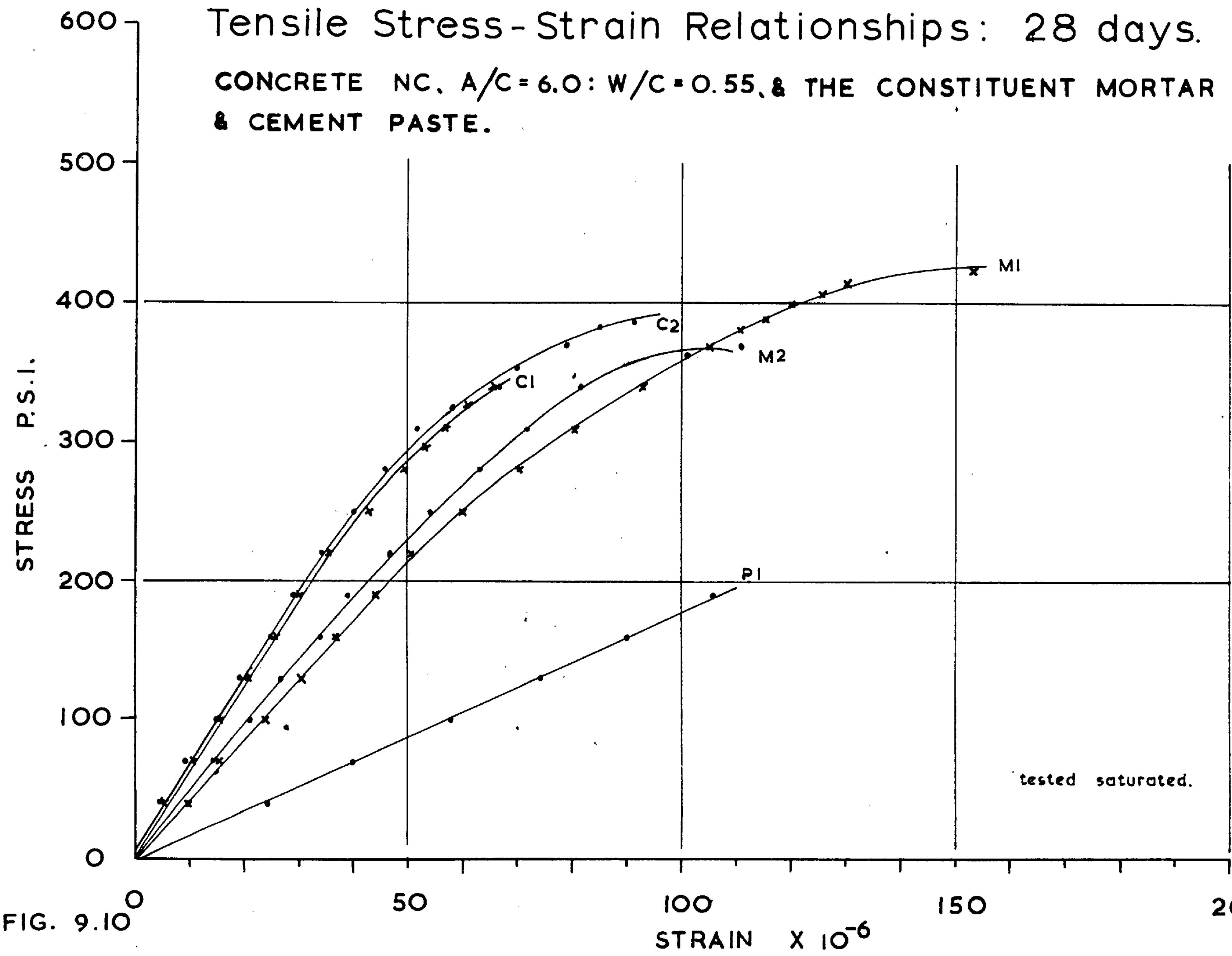

Tensile Stress-Strain Relationships: 28 days.
CONCRETE 3C. A/C=6.0:W/C=0.50. & THE CONSTITUENT MORTAR.
STRESS P.S.I.
600
500
400
300
200
100
0
C1
C2
M2
M2
tested saturated
0
50
100
150
200
STRAIN X 10^{-6}

FIG. 9.8



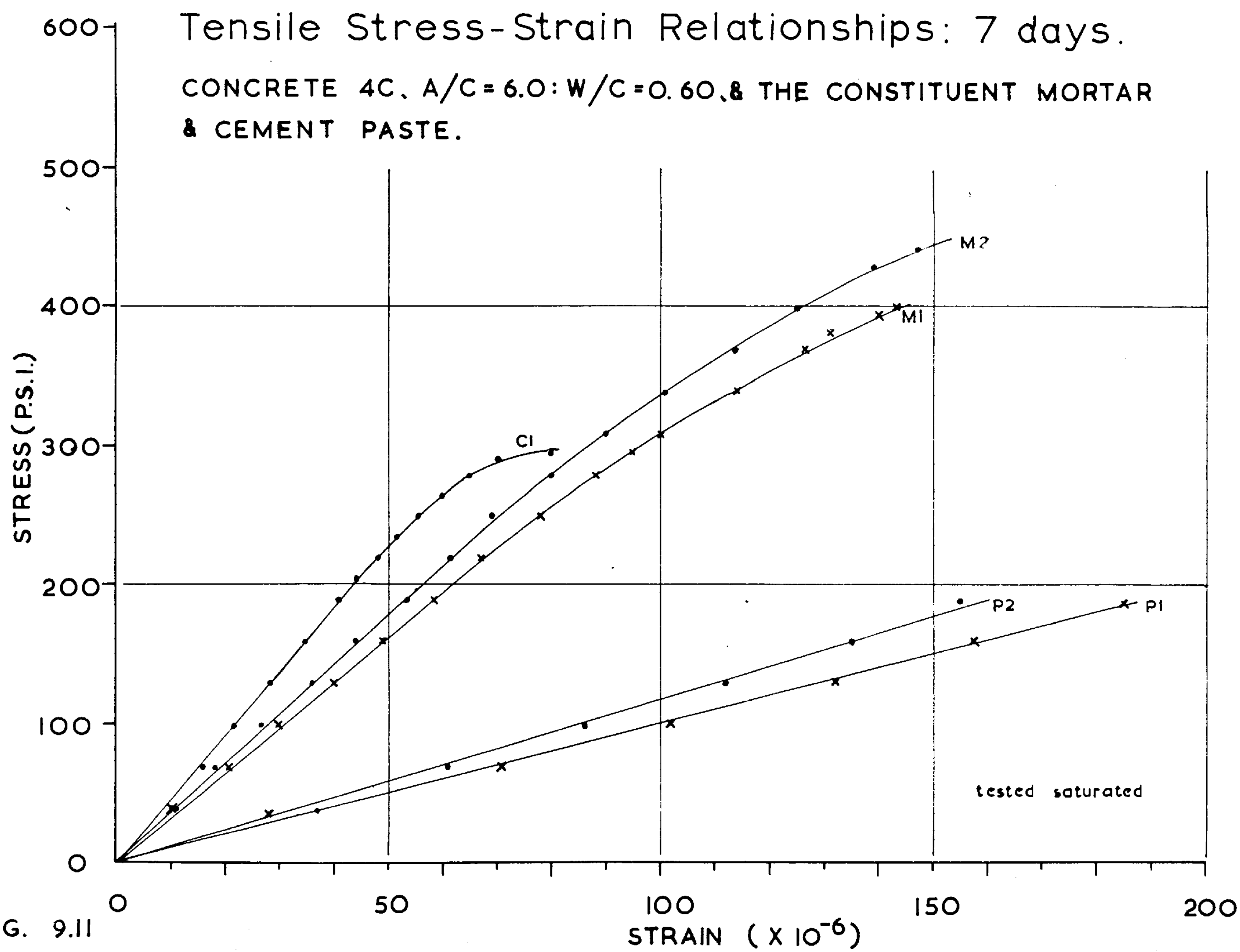
Tensile Stress-Strain Relationships: 7 days.
CONCRETE NC. A/C = 6.0: W/C = 0.55 & THE CONSTITUENT MORTAR
& CEMENT PASTE.
600
500
400
300
200
100
0
STRESS P.S.I.
C2
C1
M1
M2
P1
tested saturated
0
50
100
150
200
STRAIN X 10⁻⁶

FIG. 9.9



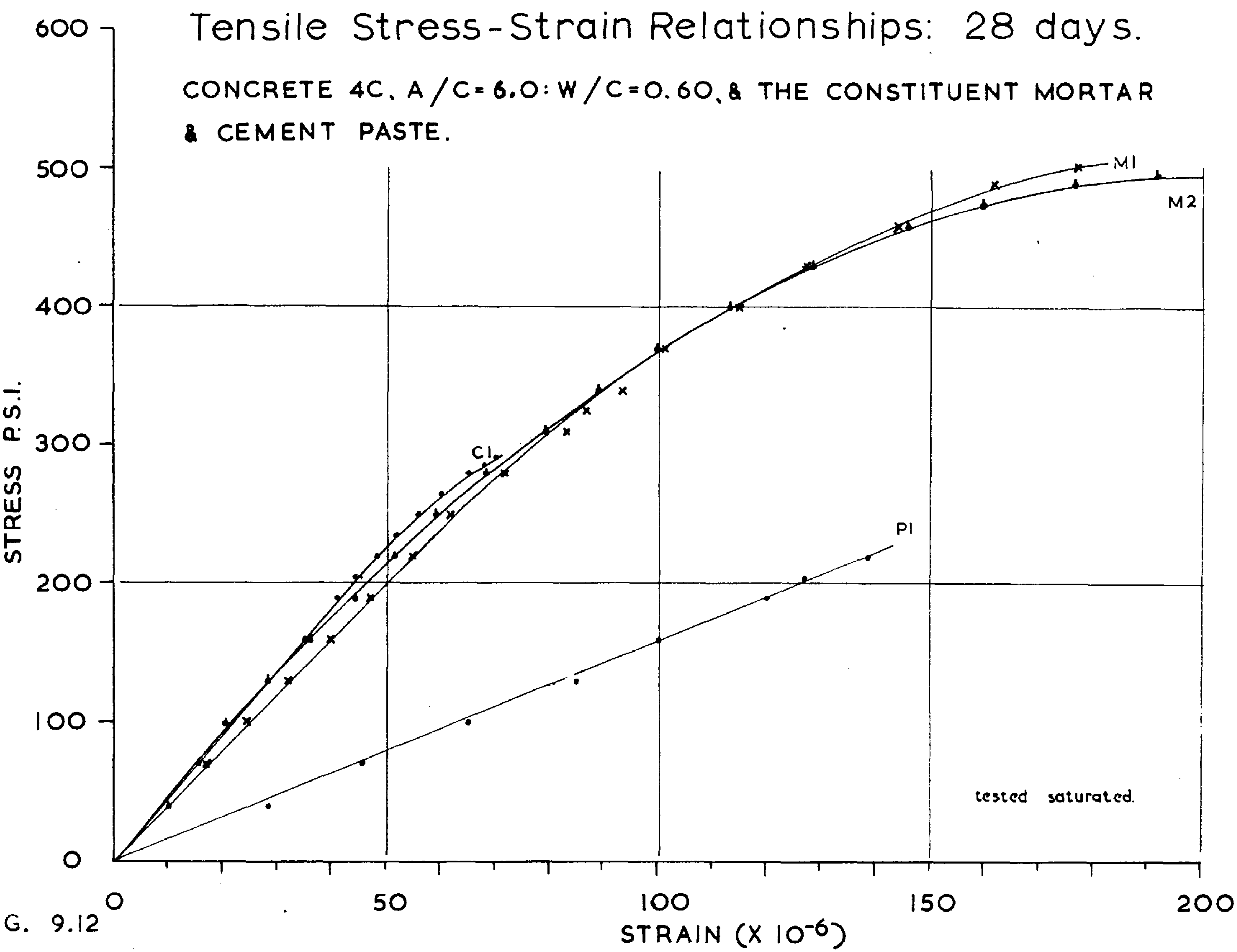
Tensile Stress-Strain Relationships: 28 days.
CONCRETE NC. A/C = 6.0 : W/C = 0.55 & THE CONSTITUENT MORTAR & CEMENT PASTE.
STRESS P.S.I.
STRAIN X 10^{-6}
600
500
400
300
200
100
0
0
50
100
150
200
C2
C1
M1
M2
P1
tested saturated.

FIG. 9.10



Tensile Stress-Strain Relationships: 7 days.
CONCRETE 4C. A/C = 6.0 : W/C = 0.60 & THE CONSTITUENT MORTAR & CEMENT PASTE.
STRESS (P.S.I.)
600
500
400
300
200
100
0
0
50
100
150
200
STRAIN (X 10⁻⁶)
C1
M2
M1
P2
P1
tested saturated

FIG. 9.11



Tensile Stress-Strain Relationships: 28 days.
CONCRETE 4C. A/C=6.0: W/C=0.60. & THE CONSTITUENT MORTAR & CEMENT PASTE.
STRESS P.S.I.
600
500
400
300
200
100
0
STRAIN (X 10^{-6})
0
50
100
150
200
M1
M2
C1
P1
tested saturated.

FIG. 9.12

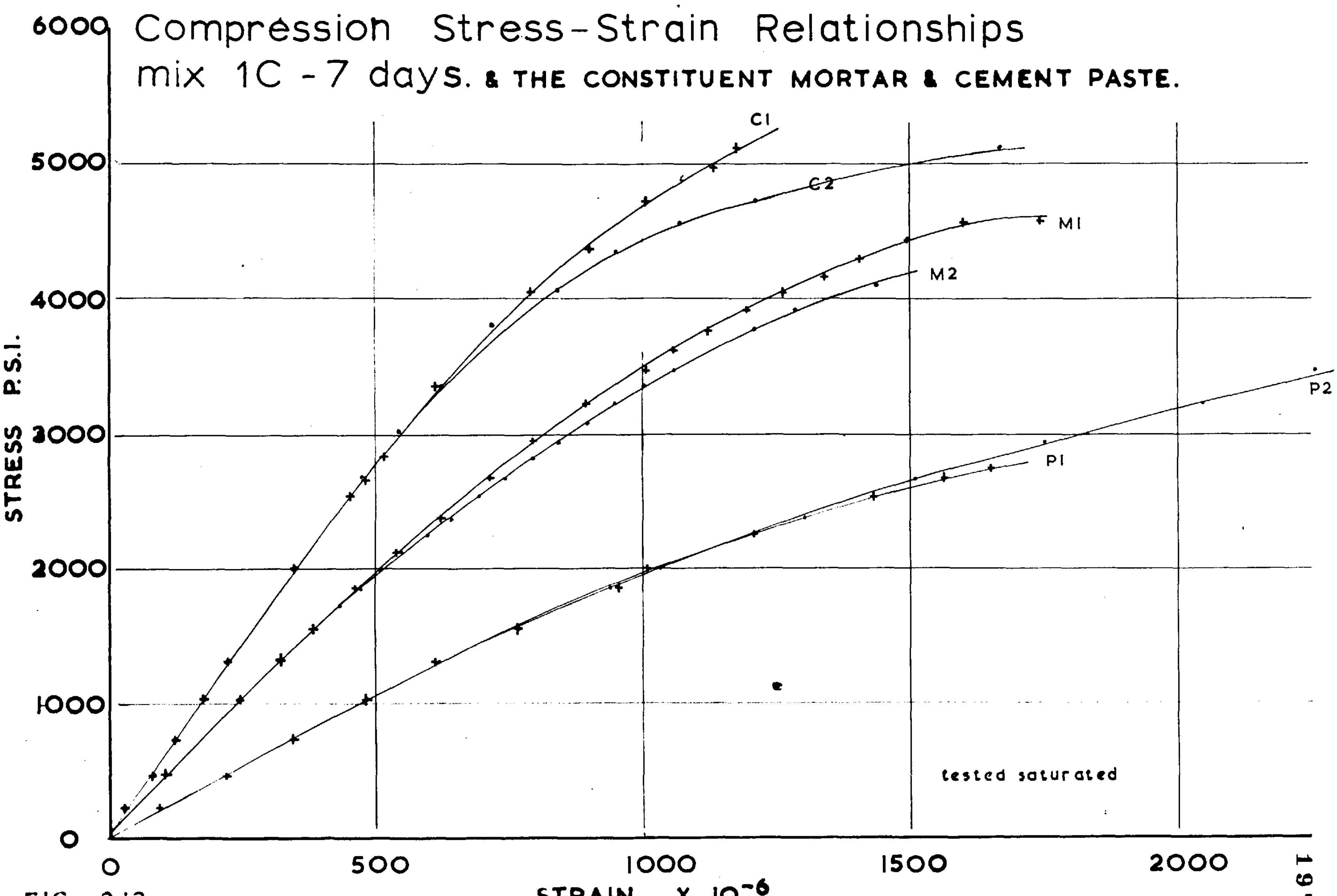

Compression Stress-Strain Relationships
mix 1C -7 days. & THE CONSTITUENT MORTAR & CEMENT PASTE.
STRESS P.S.I.
STRAIN X 10^{-6}
6000
5000
4000
3000
2000
1000
0
0
500
1000
1500
2000
C1
C2
M1
M2
P1
P2
tested saturated

FIG. 9.13

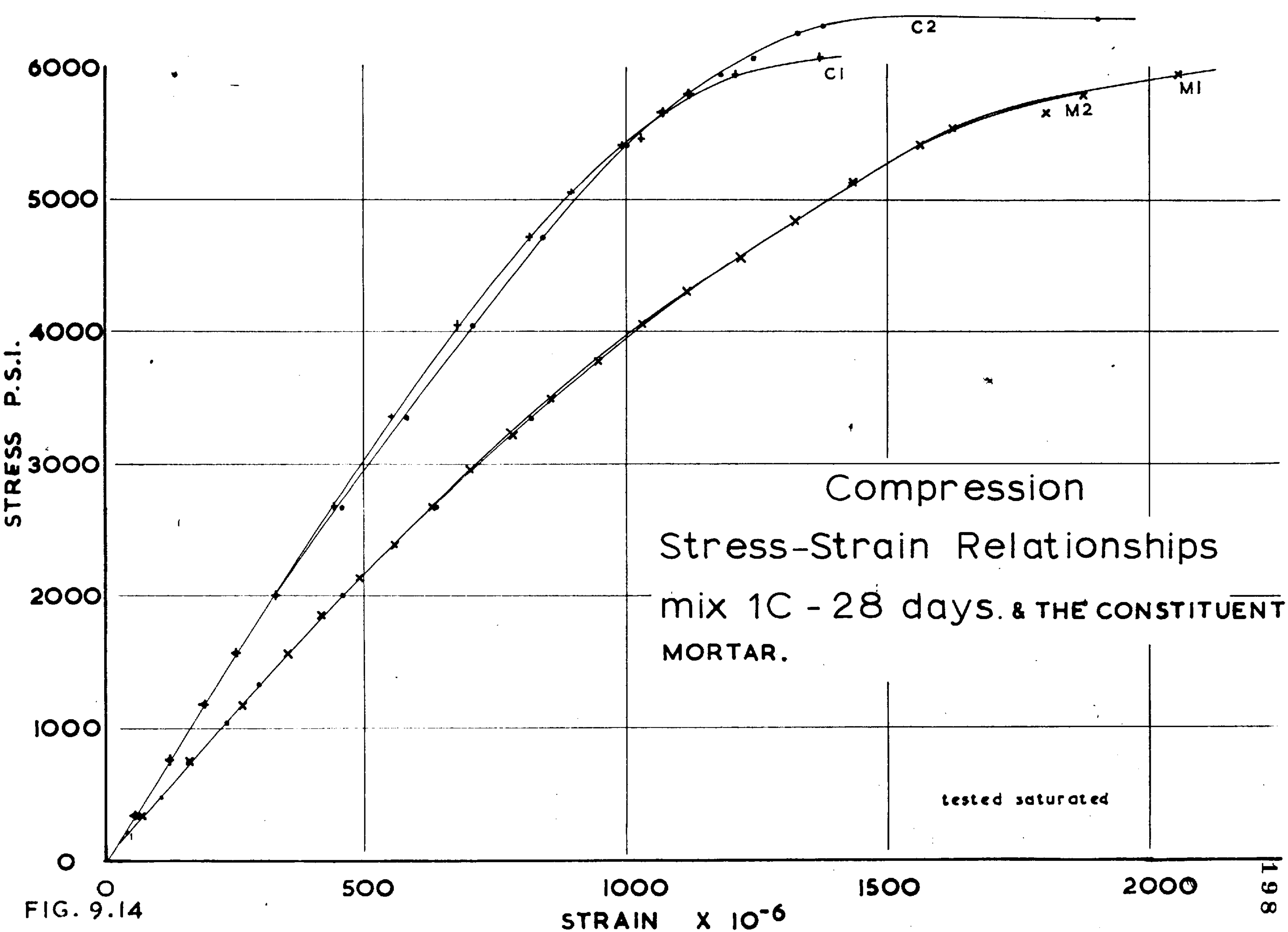
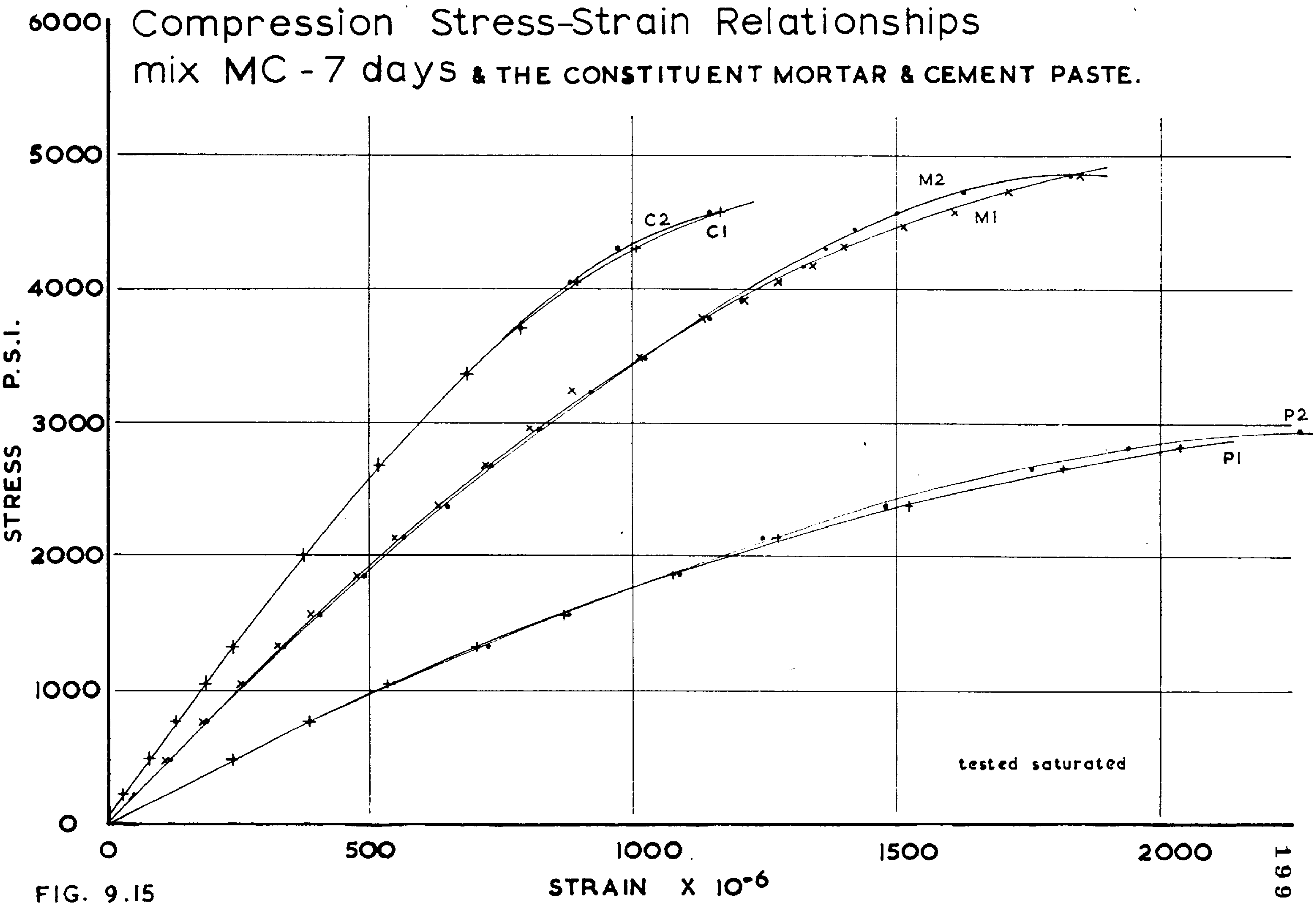
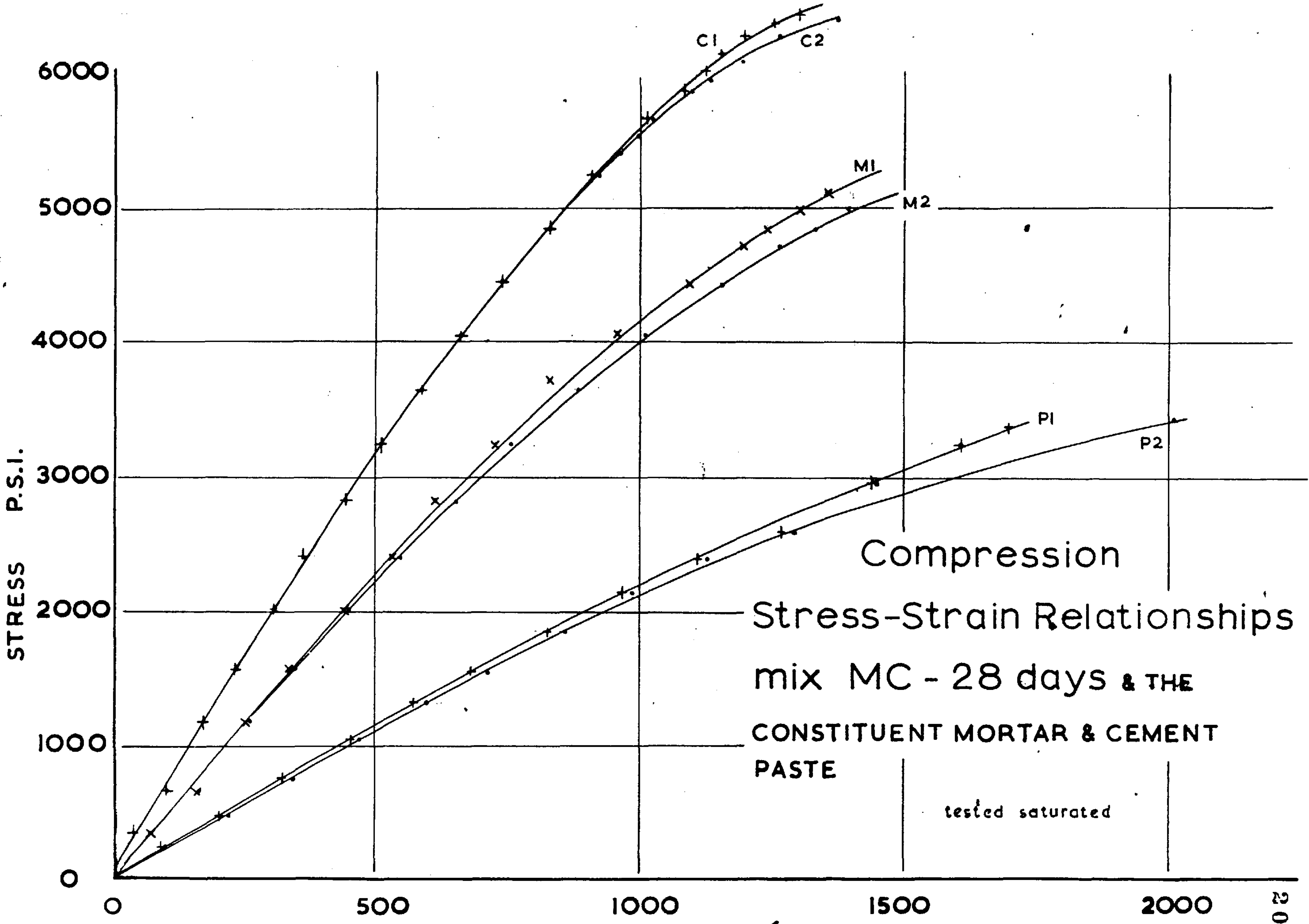

Compression
Stress-Strain Relationships
mix 1C - 28 days. & THE CONSTITUENT
MORTAR.
tested saturated
C2
C1
M1
M2
STRESS P.S.I.
6000
5000
4000
3000
2000
1000
0
0
500
1000
1500
2000
STRAIN X 10^{-6}

FIG. 9.14



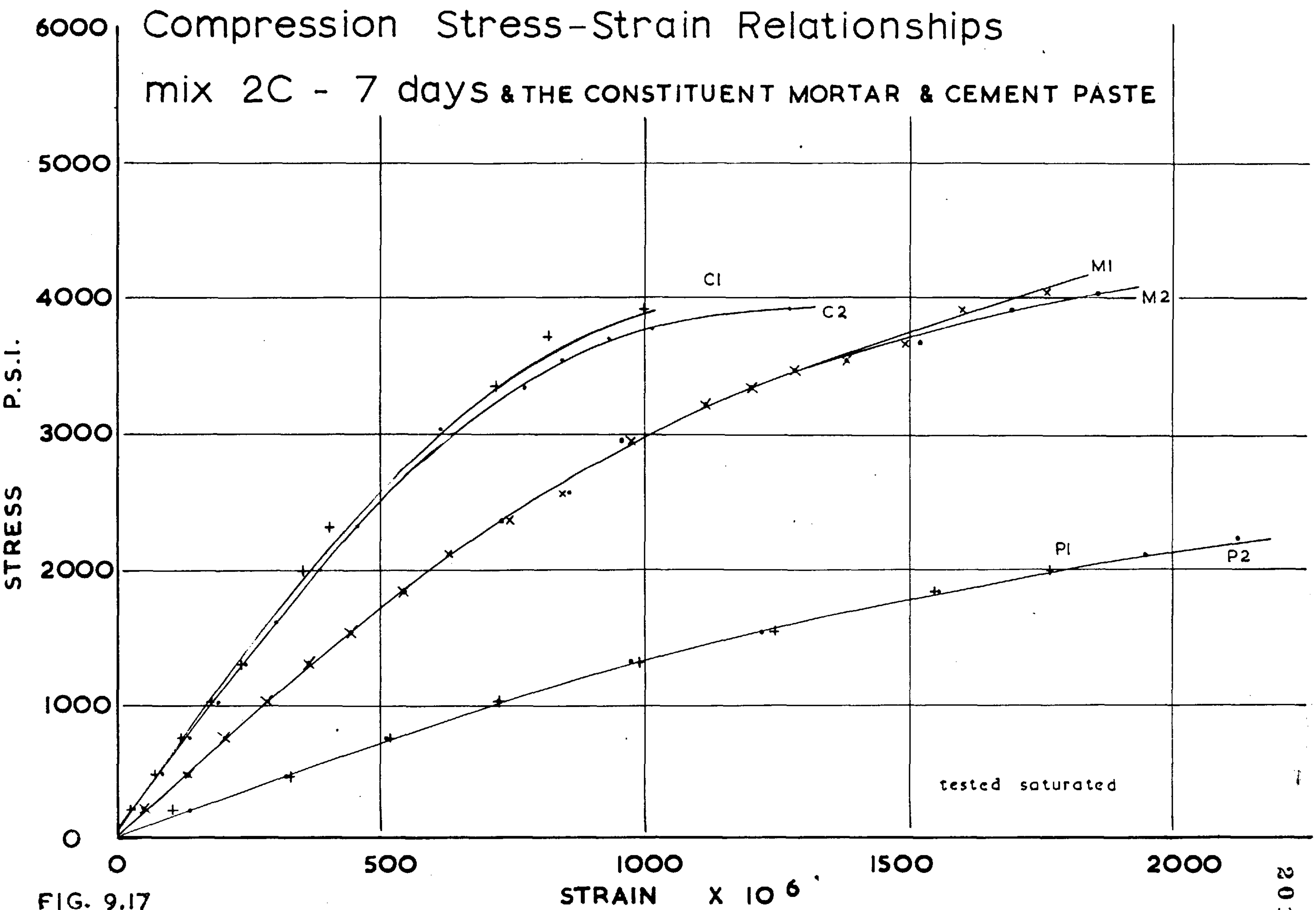
Compression Stress-Strain Relationships
mix MC - 7 days & THE CONSTITUENT MORTAR & CEMENT PASTE.
6000
5000
4000
3000
2000
1000
0
STRESS P.S.I.
0
500
1000
1500
2000
STRAIN X 10^{-6}
C2
C1
M2
M1
P2
P1
tested saturated

FIG. 9.15



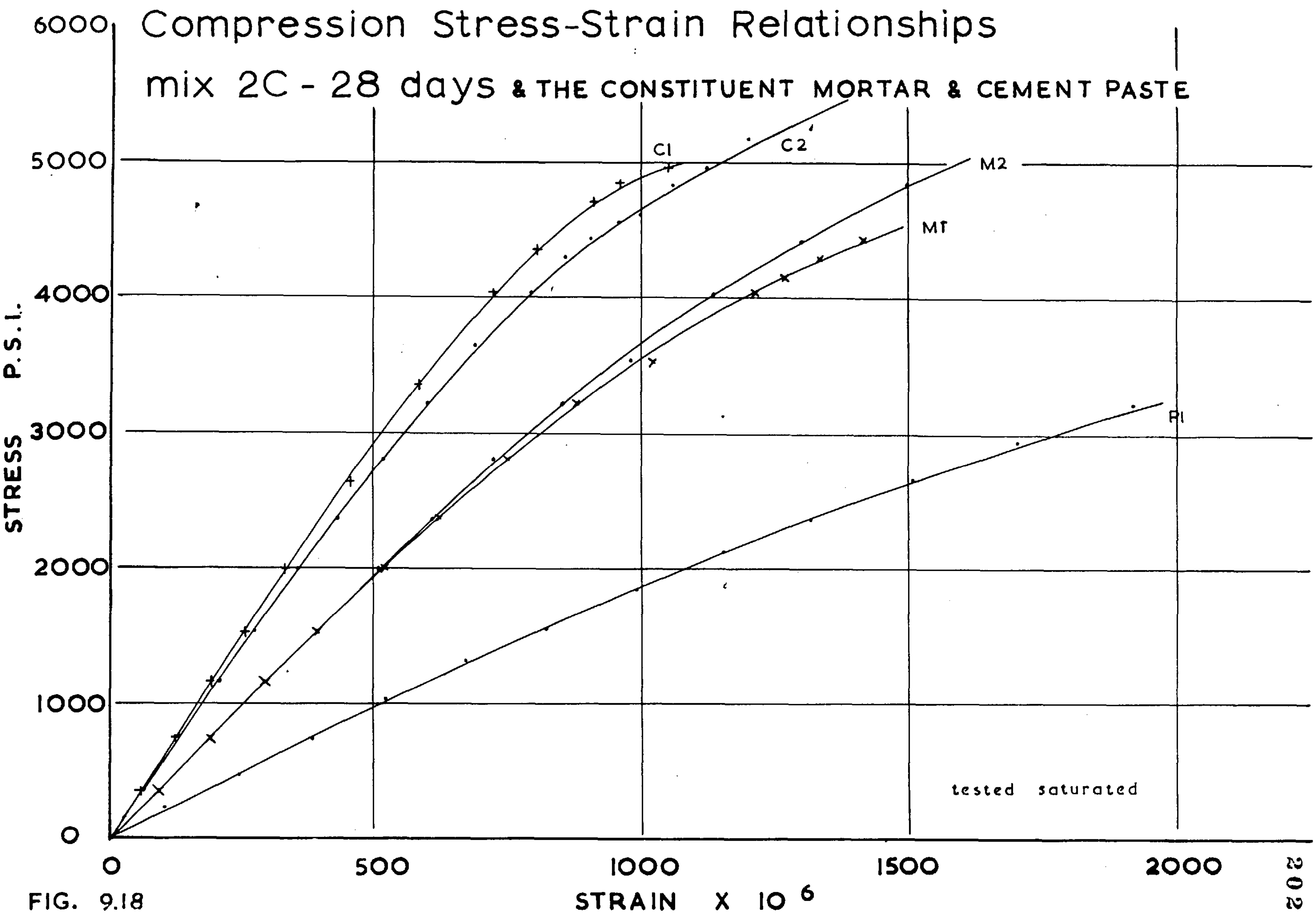
Compression
Stress-Strain Relationships
mix MC - 28 days & THE
CONSTITUENT MORTAR & CEMENT
PASTE
tested saturated
STRESS P.S.I.
STRAIN X 10 6
C1
C2
M1
M2
P1
P2
0
1000
2000
3000
4000
5000
6000
500
1500

FIG. 9.16



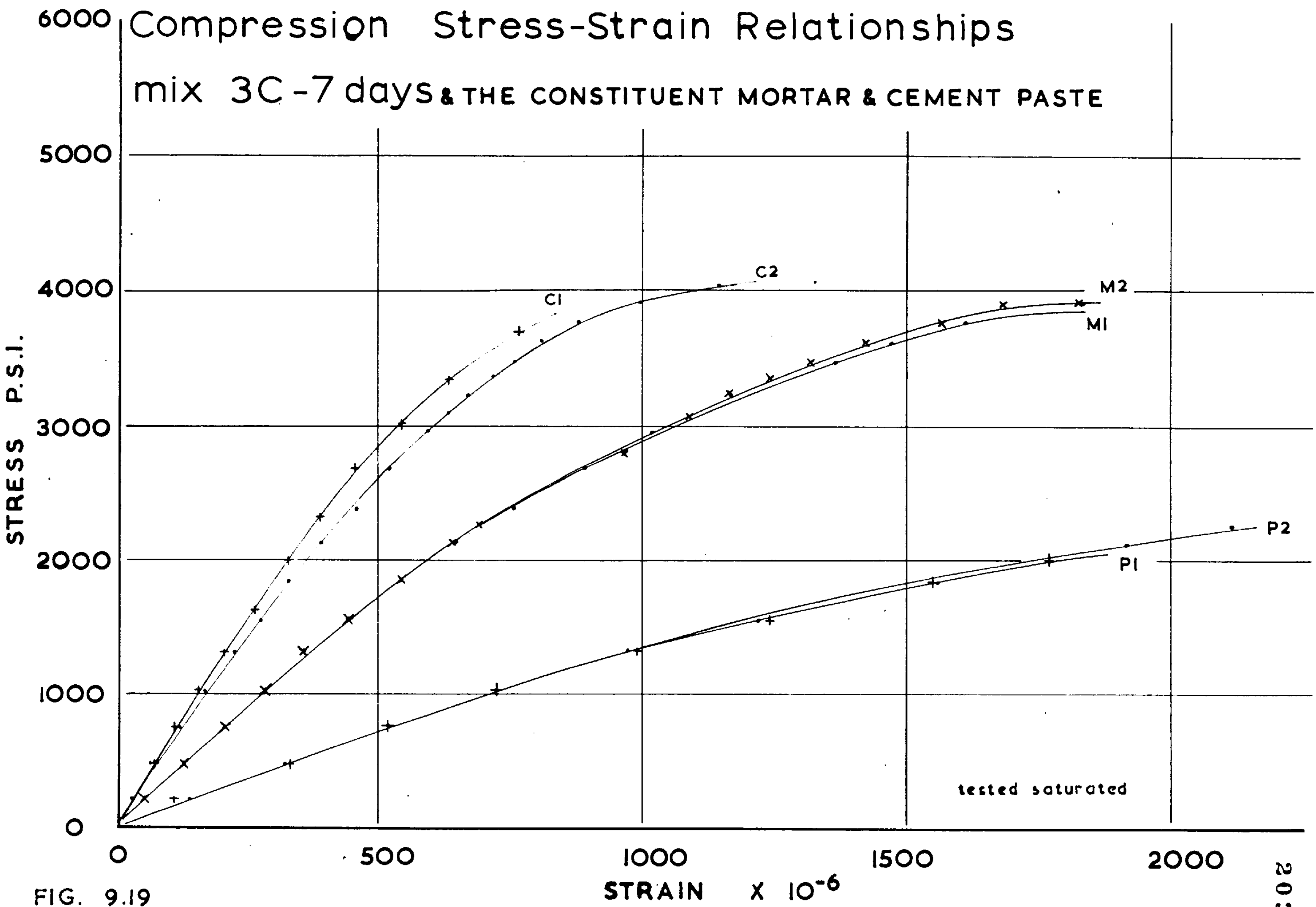
Compression Stress-Strain Relationships
mix 2C - 7 days & THE CONSTITUENT MORTAR & CEMENT PASTE
6000
5000
4000
3000
2000
1000
0
STRESS P.S.I.
C1
C2
M1
M2
P1
P2
tested saturated
0
500
1000
1500
2000
STRAIN X 10 6

FIG. 9.17



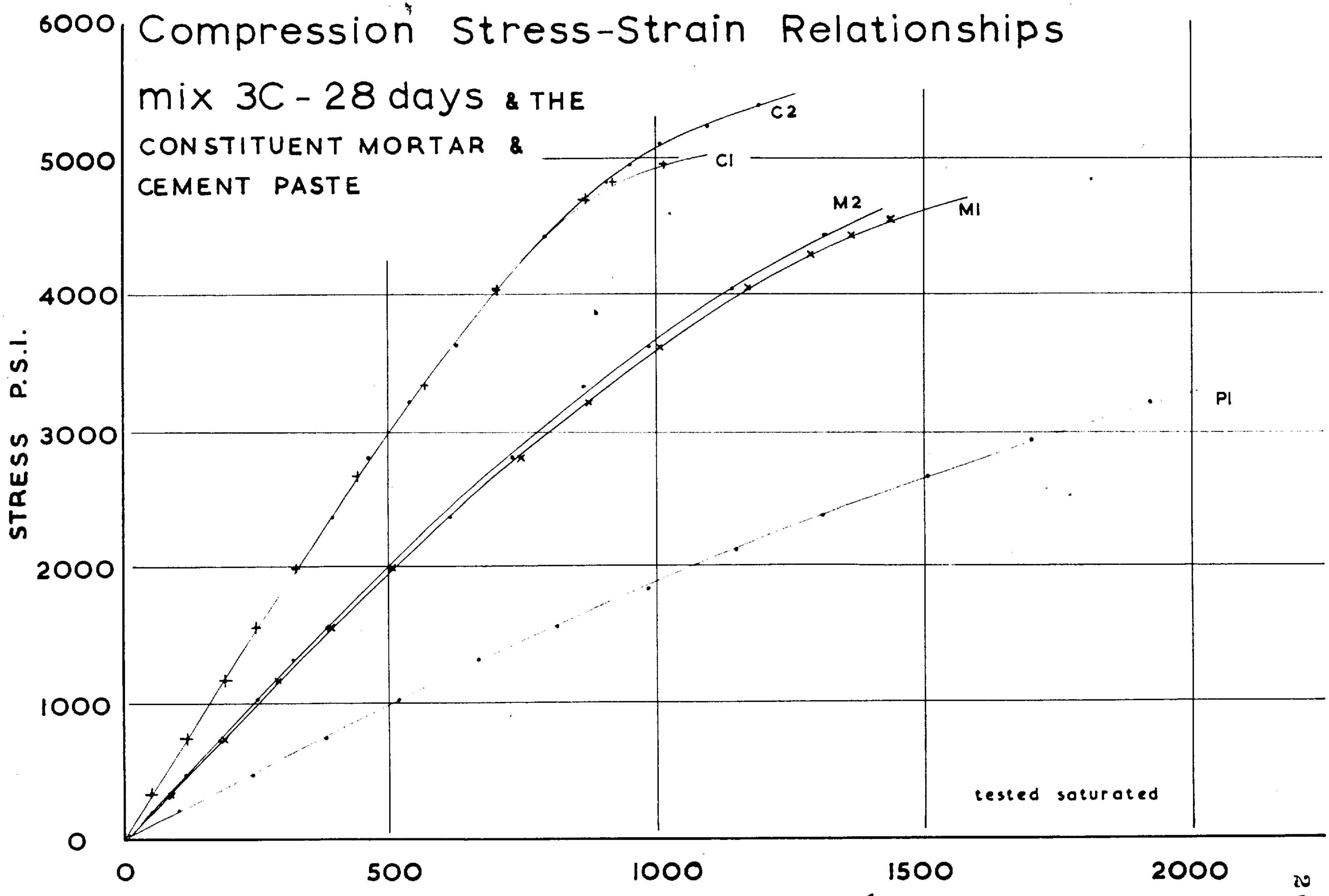
Compression Stress-Strain Relationships
mix 2C - 28 days & THE CONSTITUENT MORTAR & CEMENT PASTE
STRESS P.S.I.
6000
5000
4000
3000
2000
1000
0
C1
C2
M2
M1
P1
tested saturated
0
500
1000
1500
2000
STRAIN X 10 6

FIG. 9.18



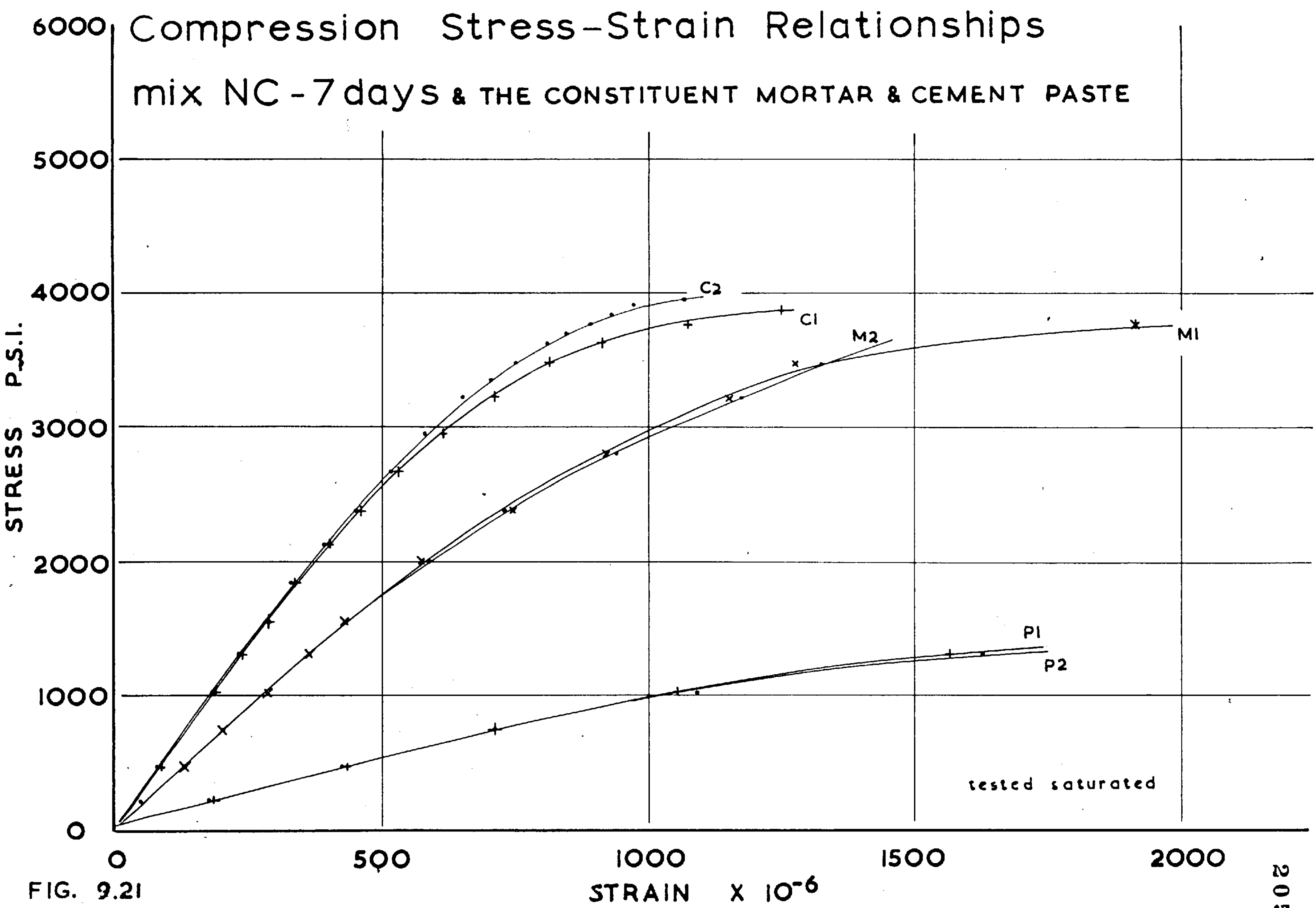
Compression Stress-Strain Relationships
mix 3C-7 days & THE CONSTITUENT MORTAR & CEMENT PASTE
6000
5000
4000
3000
2000
1000
0
STRESS P.S.I.
C1
C2
M2
M1
P2
P1
tested saturated
0
500
1000
1500
2000
STRAIN X 10^{-6}

FIG. 9.19



Compression Stress-Strain Relationships
mix 3C - 28 days & THE
CONSTITUENT MORTAR &
CEMENT PASTE
STRESS P.S.I.
6000
5000
4000
3000
2000
1000
0
C2
C1
M2
M1
P1
tested saturated
0
500
1000
1500
2000
STRAIN X 10⁻⁶

FIG. 9.20



Compression Stress-Strain Relationships
mix NC - 7 days & THE CONSTITUENT MORTAR & CEMENT PASTE
STRESS P.S.I.
STRAIN X 10^{-6}
6000
5000
4000
3000
2000
1000
0
500
1000
1500
2000
C2
C1
M2
M1
P1
P2
tested saturated

FIG. 9.21

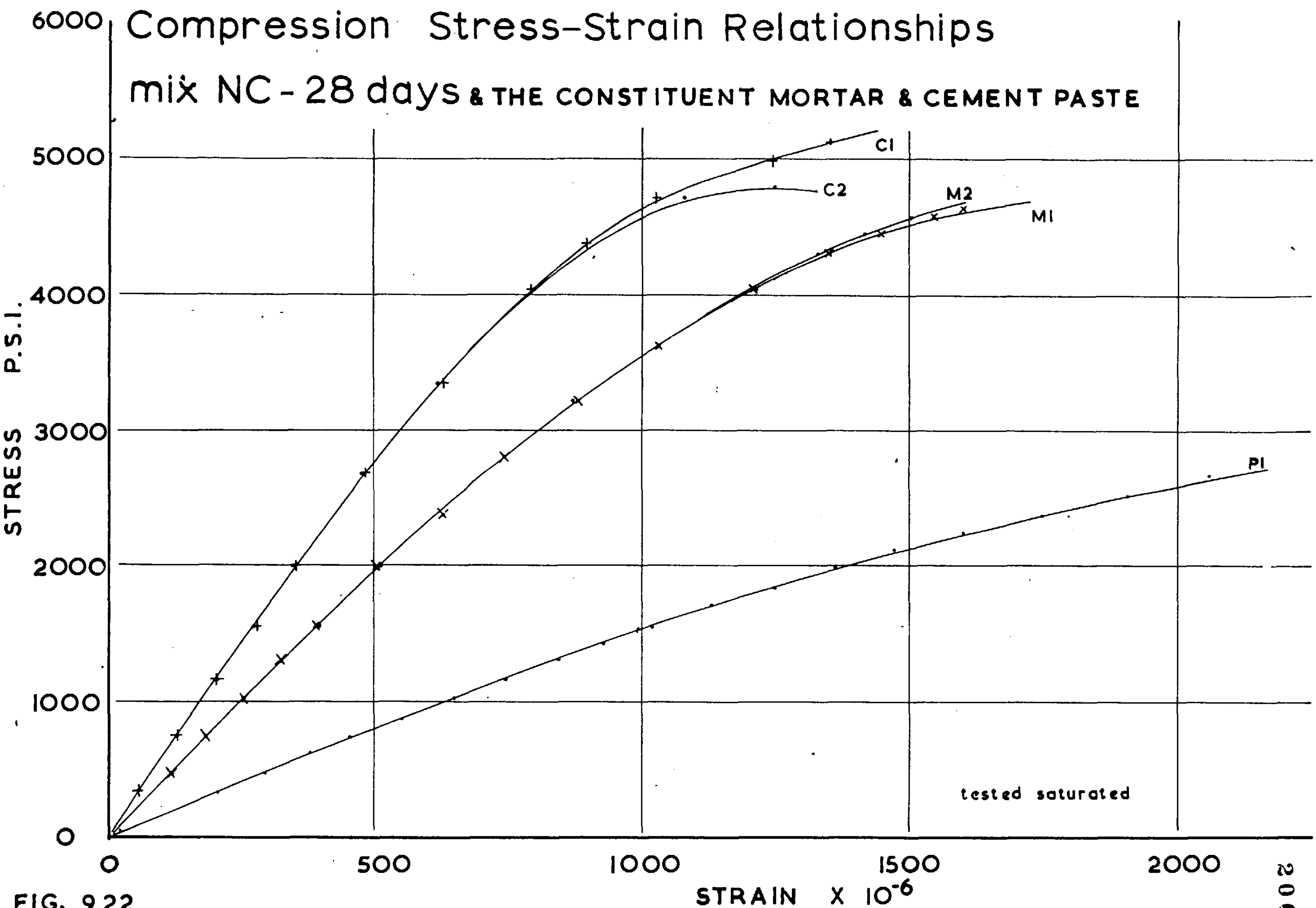
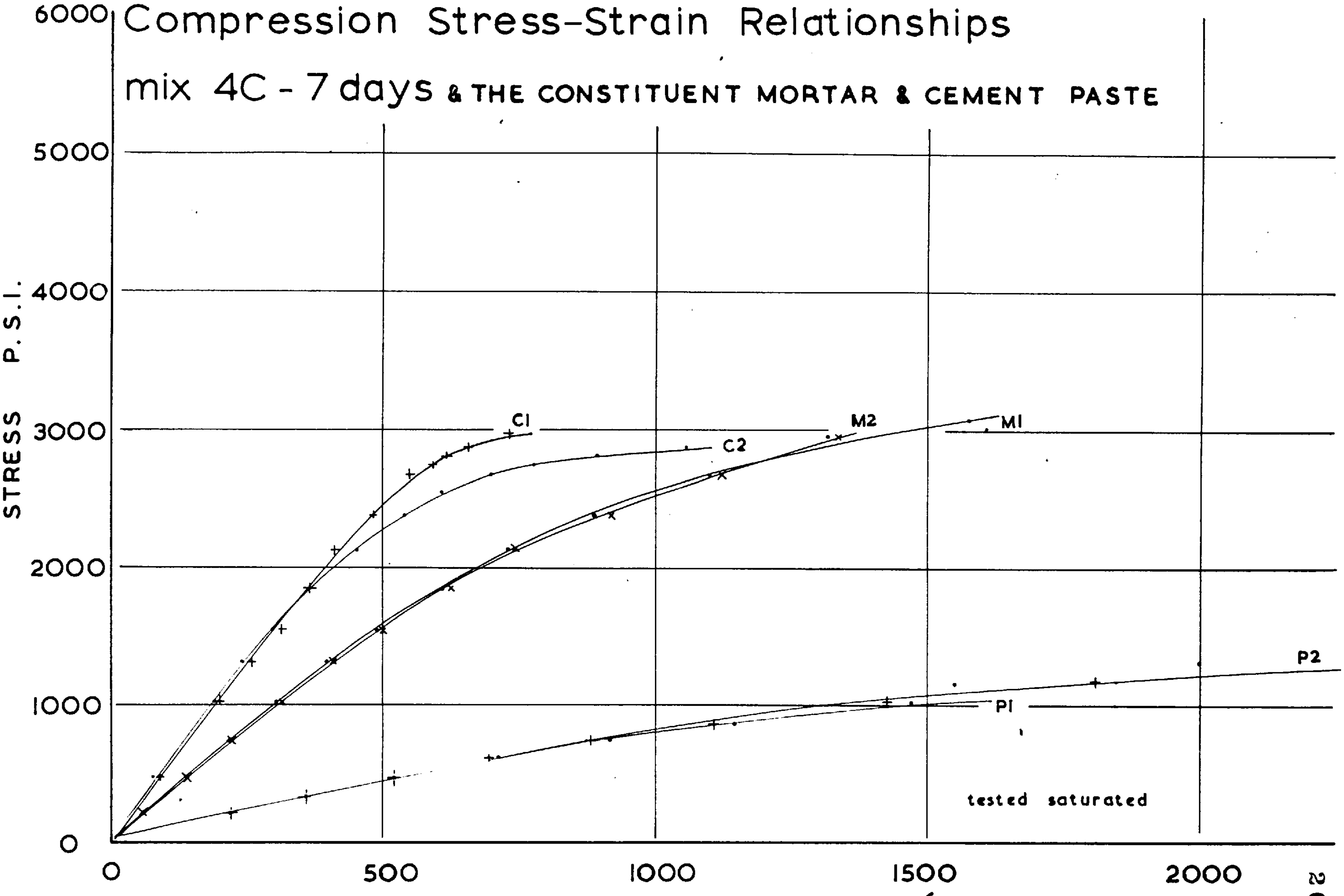

Compression Stress-Strain Relationships
mix NC - 28 days & THE CONSTITUENT MORTAR & CEMENT PASTE
STRESS P.S.I.
6000
5000
4000
3000
2000
1000
0
0
500
1000
1500
2000
STRAIN X 10-6
C1
C2
M2
M1
P1
tested saturated

FIG. 9.22



Compression Stress-Strain Relationships
mix 4C - 7 days & THE CONSTITUENT MORTAR & CEMENT PASTE
6000
5000
4000
3000
2000
1000
0
STRESS P.S.I.
C1
C2
M2
M1
P2
P1
tested saturated
0
500
1000
1500
2000
STRAIN X 10^{-6}

FIG. 9.23

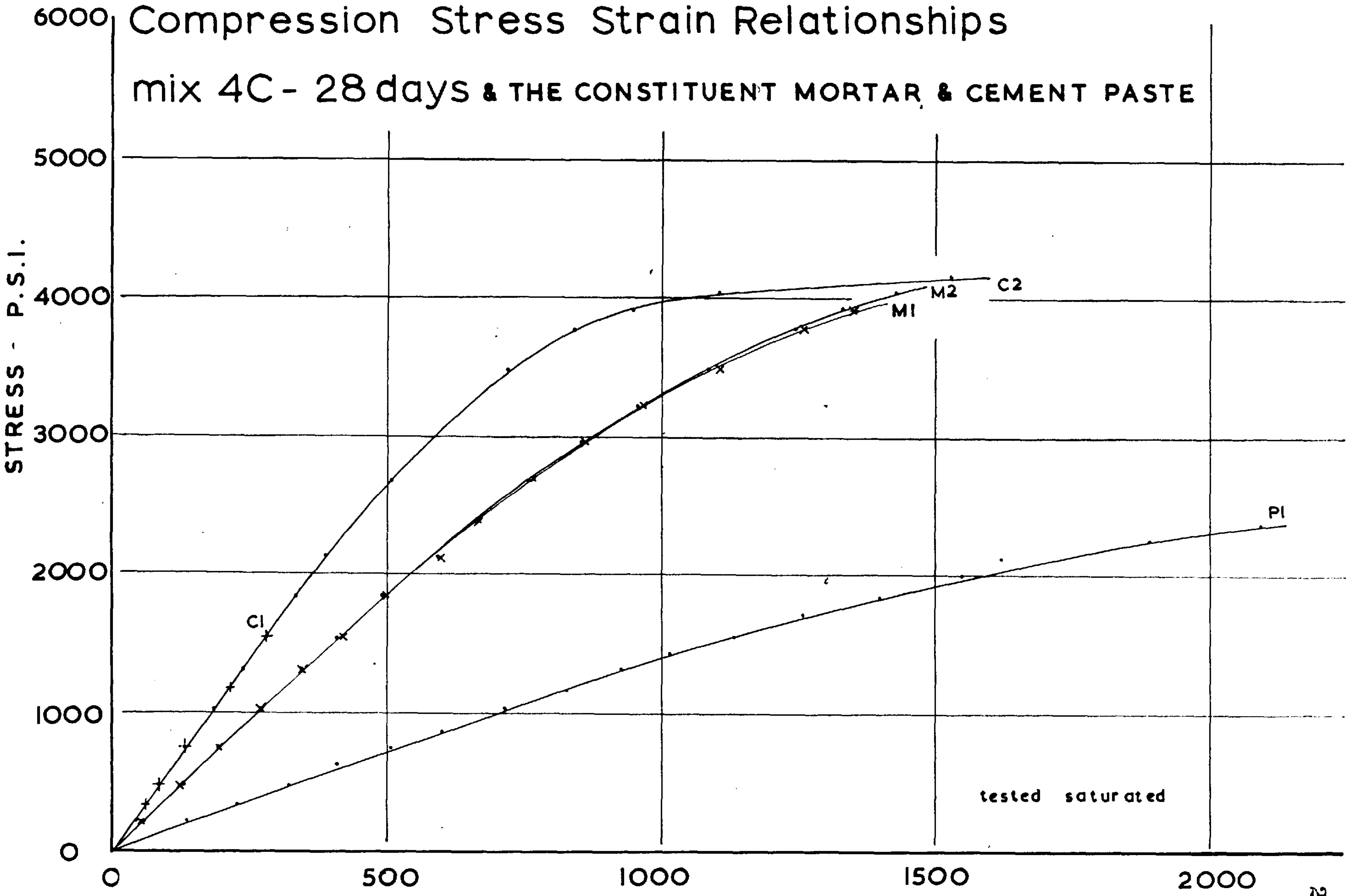

Compression Stress Strain Relationships
mix 4C - 28 days & THE CONSTITUENT MORTAR & CEMENT PASTE
STRESS - P.S.I.
6000
5000
4000
3000
2000
1000
0
0
500
1000
1500
2000
C1
C2
M1
M2
P1
tested saturated
STRAIN X 10^{-6}

FIG. 9.24

velocity measurements (see 6.6.5(c)).

Table 9.2 shows the mix proportions by volume of the mixes used in this third test series.

Table 9.3 presents a summary of the results of investigators who have tried to determine the starting point for internal disruption of a concrete under a uni-axial compression loading using ultrasonic pulse velocity measurements, listening devices, etc.

Table 9.4 gives details of all the non-destructive test measurements obtained from the third test series. The table contains the results from the measurement of density on the flexural beam specimens. The calculated values of the damping factor (Q), the dynamic modulus of elasticity and the dynamic Poisson's ratio obtained from the measurement of longitudinal resonant frequency and ultrasonic pulse velocity are also given. The methods used in the calculation of the above quantities have been described in detail in 6.6.

9.3 DISCUSSION

9.3.1 Limiting Tensile Strains

In the selection of the mix proportions for the second and third test series, it was decided to include changes of aggregate/cement ratio at constant water/cement ratio and vice-versa. In the third series, two such aggregate/cement ratios with five different water/cement ratios were chosen, Table 6.9. However, although the aggregate/cement ratios by weight were significantly different (4.5 and 6.0), Table 9.2 shows that the corresponding aggregate/paste ratio by volume varied very little for the concretes and mortars tested. It is seen that in general the concretes contained approximately 70 per cent aggregate and the mortars approximately 50 per cent.

Upon examination of the ultimate failing strains in uni-axial tension given in Table 9.5, it is observed that in general for the mixes tested the average failing strains (i.e. the strains at 98 per cent of ultimate; see 9.2) were:

concretes : 90 microstrain

mortars : 160 microstrain.

There is some indication that the failing strains of the mixes tested at 28 days gave slightly higher failing strains than those tested at 7 days. If as a first approximation, it is assumed that the aggregate is infinitely rigid in comparison with the paste phase, the limiting tensile or cohesive strain limit of the paste phase within the concretes and mortars can be calculated from the failing strains by assuming all the strain to be in the paste phase:

$$\text{concretes} : \frac{\text{ultimate failing strain}}{\text{volumetric ratio of paste phase}} = \frac{90}{.3} = 300 \text{ microstrain}$$

$$\text{mortars} : \quad '' \quad = \frac{160}{.5} = 320 \text{ microstrain}$$

The strain attributed to the paste phase of these materials is composed of two different parts: (1) true strain within the cement paste itself, and (2) deformation due to the formation of cracks due to bond failures at the paste/aggregate interfaces or cement paste failures.

It is interesting to note that the overall strains at 98 per cent of the ultimate load obtained in this investigation for the concretes and mortars in uni-axial tension, compare favourably with those obtained by Kaplan(17). In his work, Kaplan also observed that the amount of overall strain that a concrete or mortar can withstand is dependent on the amount of aggregate in the mix. The limiting tension strain decreases with an increase in coarse aggregate content.

It is suggested that such a limiting tensile or cohesive strain limit in the paste phase, might be the extensional limit to which concrete materials may be subjected before rapid crack propogation is initiated. It is possible that this extensional limit might define the point where a concrete material ceases to behave as a continuum when subjected to any state of stress.

Table 9.2 Mix proportions by volume - series 3

Mix	per cent paste	per cent aggregate	aggregate/paste ratio
1C	29.5	70.5	2.4
MC	30.5	69.5	2.3
2C	31.6	68.4	2.2
3C	25.6	74.4	2.8
NC	26.8	73.2	2.7
4C	27.9	72.1	2.6
1E	51.4	48.6	.94
ME	52.7	47.3	.90
2E	53.9	46.1	.86
3E	46.7	53.3	1.14
NE	48.1	51.9	1.08
4E	49.6	50.4	1.01

9.3.2 Comparisons between the static modulus of elasticity in uni-axial compression and tension, and the dynamic modulus of elasticity.

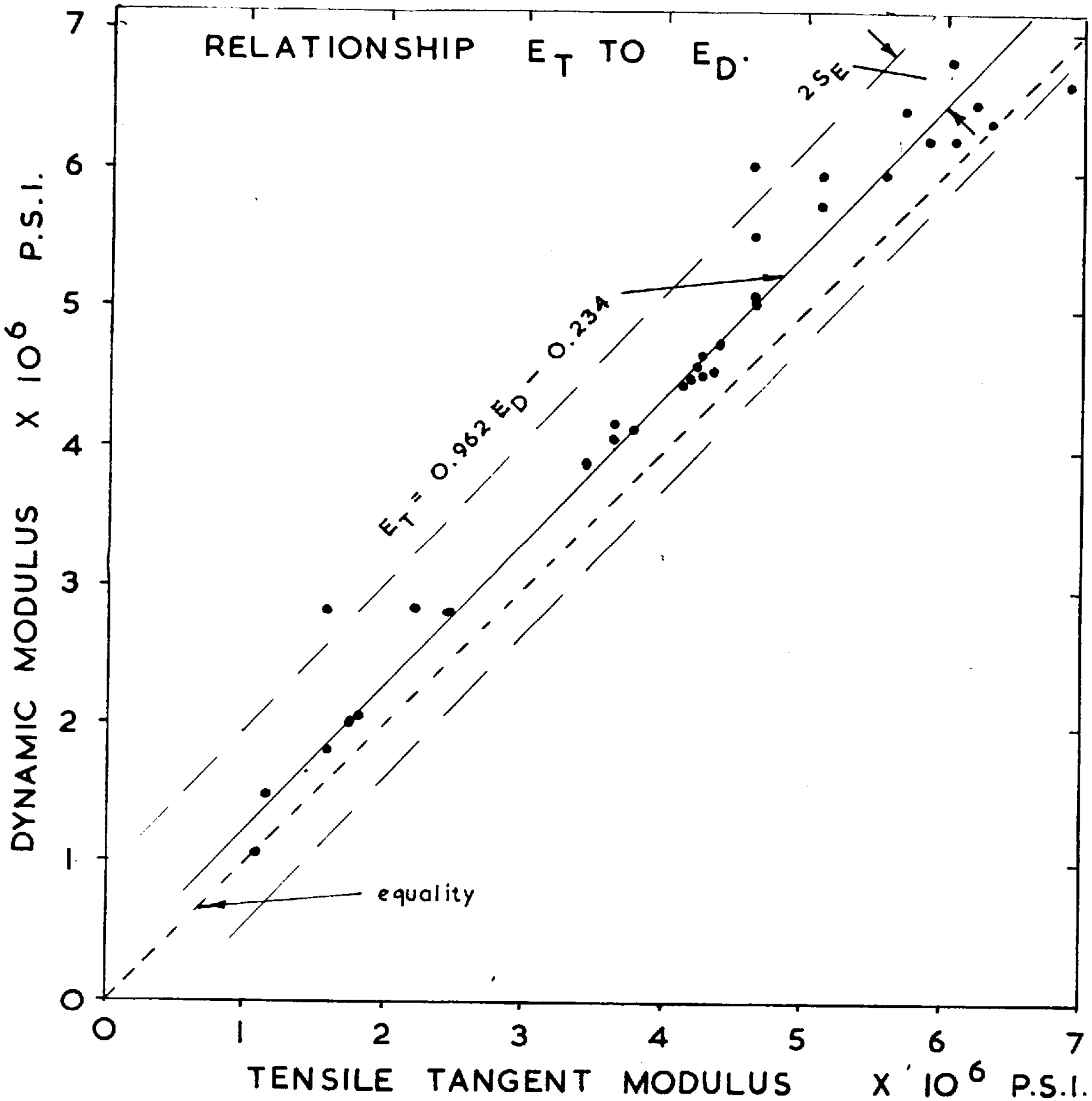
The statistical analysis used in this section is described in detail in Appendix D.

(a) The relation between dynamic modulus of elasticity (E_D) and the static tension modulus of elasticity (E_T).

In Fig.9.25, the dynamic moduli (E_D) obtained from the resonant frequency measurements taken on the flexural beams (see 6.6.4) are plotted against the tensile 'tangent' (see 9.2) moduli (E_T) obtained from the tension stress-strain relationships (Fig.9.1 - 9.12). The relevant regression equations have been calculated and are:

$$E_T = 0.962E_D - 0.234$$
$$r = 0.979$$
$$S_E = 0.328$$

$$E_D = 1.00E_T + 0.400$$
$$r = 0.979$$
$$S_E = 0.336$$



RELATIONSHIP E_T TO E_D.
2SE
$E_T = 0.962\,E_D - 0.234$
equality
DYNAMIC MODULUS X 10^6 P.S.I.
TENSILE TANGENT MODULUS X 10^6 P.S.I.
0
1
2
3
4
5
6
7

FIG. 9.25

The first regression equation is superimposed on Fig.9.25. Furthermore the control limits at 2 standard errors of estimate ($2S_E$) have been drawn on either side of the regression line. The significance of these limits is that about 95 per cent of the time, the actual values of the ratios will lie within plus or minus two standard errors of the estimate values given by the regression equation.

The correlation coefficient, r, indicates that the derived relationship may be taken to be representative of the parent universe with negligible risk of error.

(b) The relation between dynamic modulus of elasticity (E_D) and the static compression modulus of elasticity (E_C).

Fig.9.26 shows the relation of E_D to E_C. The compression 'tangent' moduli were obtained from the compression stress-strain relationships given in Figs.9.13 - 9.24. The relevant regression equations are:

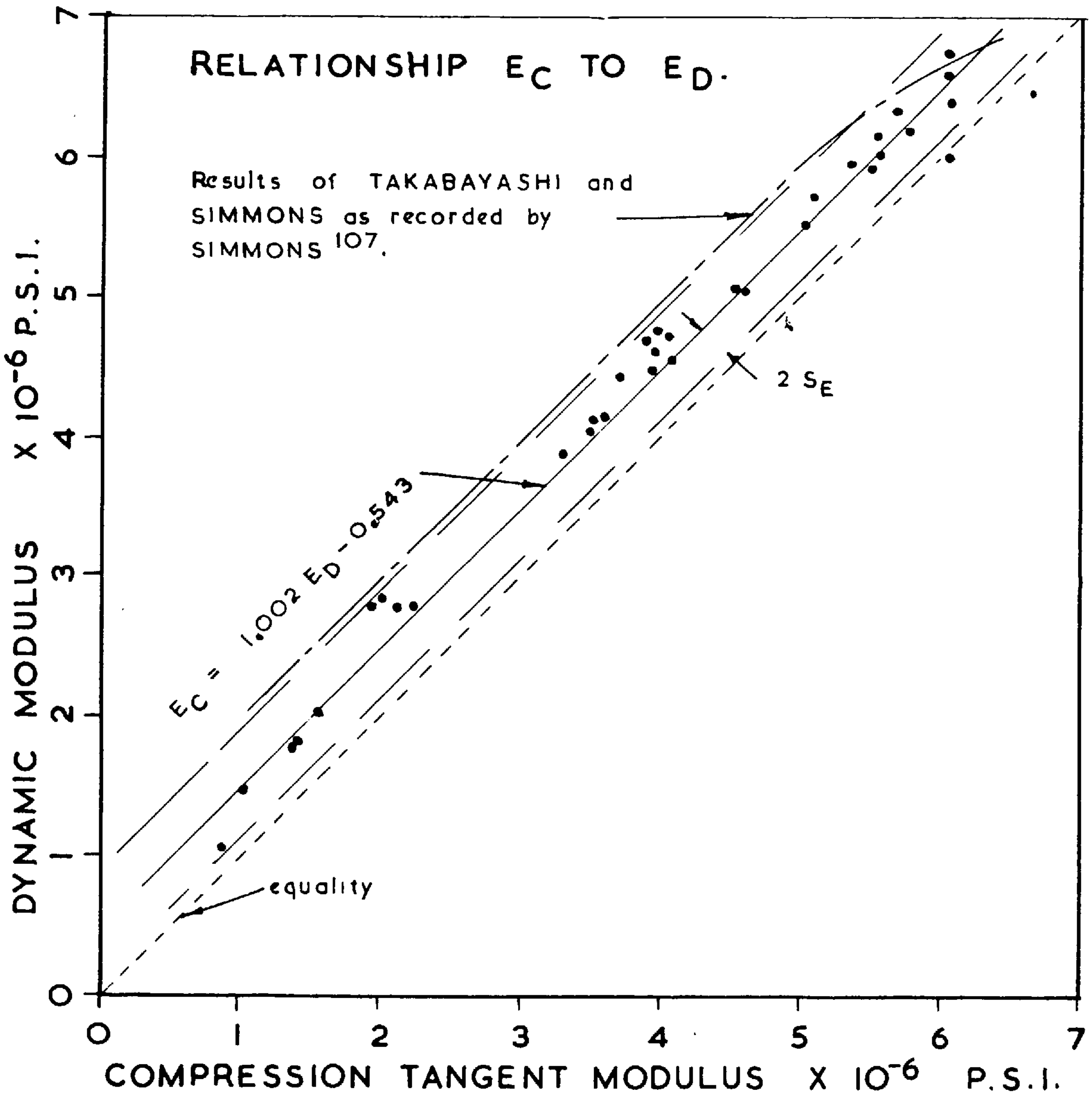
$$E_C = 1.002E_D - 0.543$$
$$r = 0.994$$
$$S_E = 0.175$$

$$E_D = 0.998E_C + 0.534$$
$$r = 0.994$$
$$S_E = 0.172$$

The first regression line is plotted on Fig.9.26 together with the control limits of two standard errors on either side of the regression line.

The correlation coefficients of the above regression lines are highly significant and with the low standard errors obtained, it is possible to predict either E_D or E_C with confidence.

The relationship obtained by Takabayashi and Simmons, as reported by Simmons[107], is plotted on Fig.9.26. It is observed that excellent agreement exists between the results obtained from the previous and present investigations, particularly with regard to slope. It is



RELATIONSHIP E_C TO E_D.
Results of TAKABAYASHI and SIMMONS as recorded by SIMMONS 107.
$E_C = 1.002\,E_D - 0.543$
$2\,S_E$
equality
DYNAMIC MODULUS X 10^{-6} P.S.I.
COMPRESSION TANGENT MODULUS X 10^{-6} P.S.I.
0
1
2
3
4
5
6
7

FIG. 9.26

suggested that the difference in intercepts between the two investigations is due to the method of obtaining the static moduli of elasticity. In the work of Simmons, the static moduli represented a secant value of a third of the ultimate load. In the present work, the 'tangent' moduli, which is equivalent to a secant value at approximately one sixth of the ultimate load, was used in plotting the relationship of Fig. 9.26.

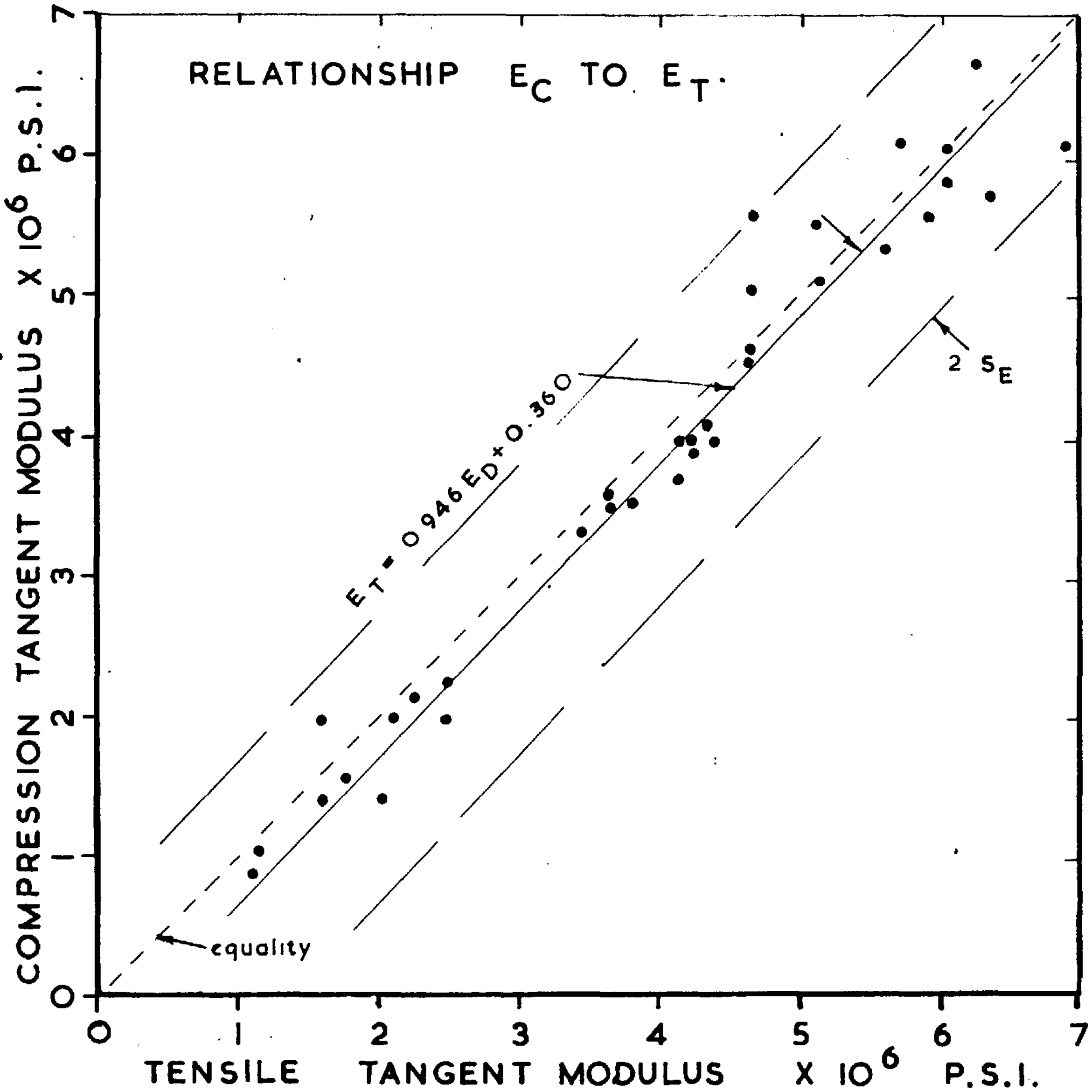
In general, the dynamic modulus is 0.5×10^6 psi. and 0.3×10^6 psi. greater than the corresponding static modulus in compression and tension respectively. It is apparent that the dynamic test is not measuring the same properties of concrete as the static test. It is suggested that the discrepancies in the measured moduli are due to the heterogeneous nature of concrete. It must be remembered that the calculations for dynamic modulus from the resonant frequency readings assume that the specimen is homogeneous, isotropic and continuous. Furthermore, whereas the dynamic tests do not impose a state of stress, the results from static E tests for concrete depend on the stress level used. However, the excellent correlation obtained does imply that the prediction of the compression static modulus from dynamic measurements is quite acceptable for a wide range of concretes, mortars and cement pastes under closely controlled conditions.

(c) The relation between static tension and compression moduli of elasticity.

Fig. 9.27 shows the relation between the compression 'tangent' moduli and the tension 'tangent' moduli. The relevant regression equations are:

$$E_T = 0.946E_C + 0.360$$
$$r = 0.952$$
$$S_E = 0.506$$

$$E_C = 0.96E_T + 0.010$$
$$r = 0.952$$
$$S_E = 0.510$$



RELATIONSHIP E_C TO E_T
$E_T = 0.946 E_D + 0.360$
$2 S_E$
equality
COMPRESSION TANGENT MODULUS X 10^6 P.S.I.
TENSILE TANGENT MODULUS X 10^6 P.S.I.
0 1 2 3 4 5 6 7

FIG. 9.27

The regression line for E_T is plotted on Fig.9.27 together with the control limits of 2 standard errors of estimate on either side of the regression line.

The regression equations do imply that E_T is greater than E_C. However, considering the standard error of estimate of the regression equation and the scatter of the data about the 'equality' line on Fig.9.27, this Writer is in agreement with Todd[18] and others that they are in fact equal.

It is possible to prove that E_T is less than E_C when using results from the simple flexural test of an unreinforced beam. Strain gauge readings taken at the top and bottom fibres of the beam can indicate that the values of E_T are less than E_C. It should be realised that for a given applied load, the value of E_T so obtained is a secant value at a stress level which is a significant percentage of the ultimate tensile strength. E_C obtained from the same test is a secant modulus of a stress level which is a very small percentage of the ultimate compression strength. Since both stress-strain relationships are curvilinear from zero stress, it is easy to see how such evidence can lead to an erroneous conclusion.

It is considered that the only sensible method of comparison is to compare the moduli of elasticity in terms of secant values at stress levels which are similar percentages of the ultimate load of a specimen in tension or compression but at stress levels below which fast crack extension begins. From the results presented in Fig.2.7 and the fact that the tension 'preloading' which defined the 'tangent' modulus obtained, was not quite such a high percentage of the ultimate as the compression 'preloading', it is considered that E_C must equal E_T for all practical purposes, at the same stress levels.

9.3.3 Relationship between the strains obtained from uni-axial tension and uni-axial compression tests.

Test series 3 was planned initially to obtain information on the behaviour of concretes, mortars and cement pastes in the so-called 'elastic' range and at failure. The object was to measure overall strains from zero stress to failure over the same gauge length and with the same strain instrument. Of particular interest was the inter-relation between the limiting overall strain in uni-axial tension and the lateral strain behaviour of a similar material subjected to uni-axial compression.

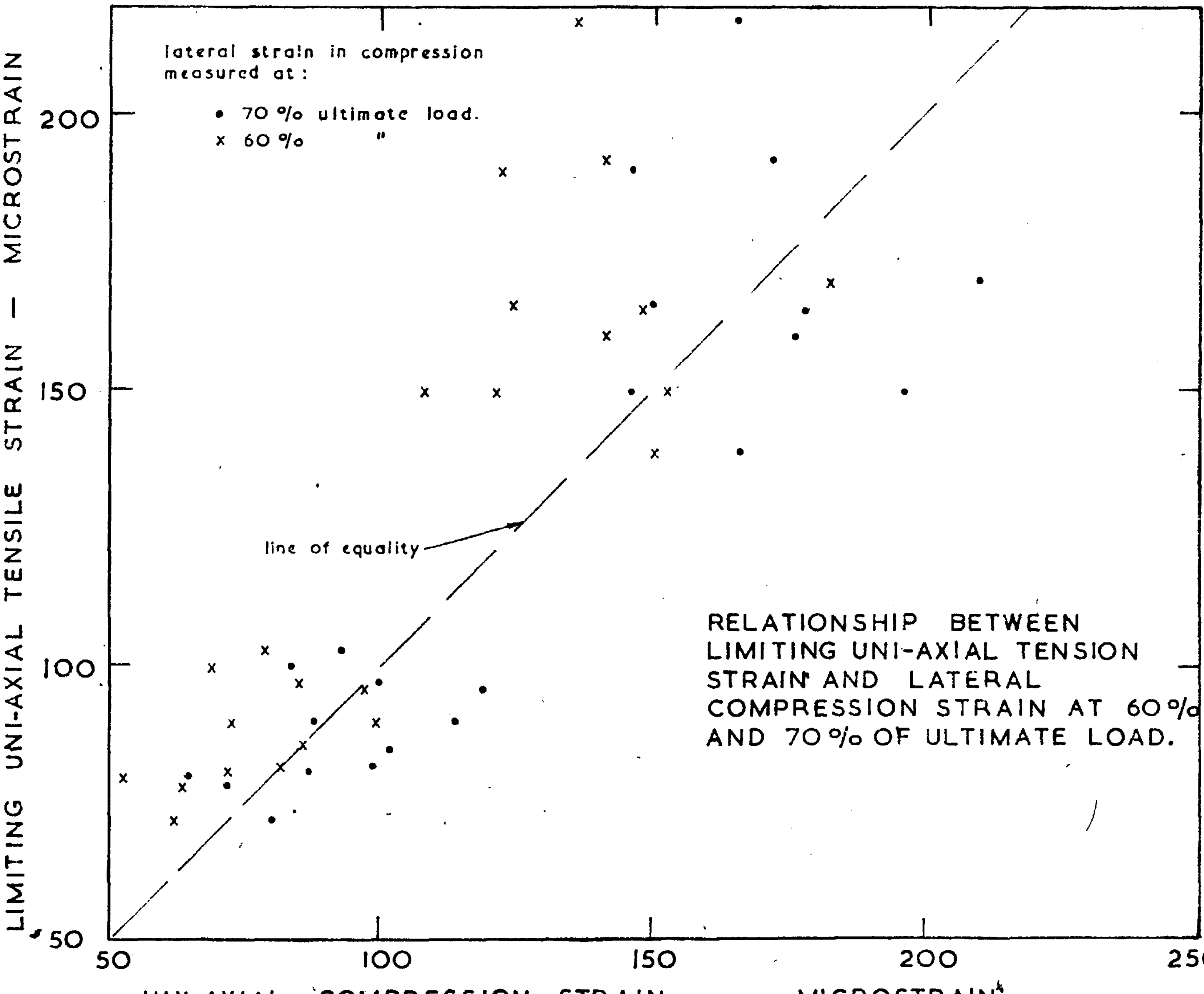
Fig.9.28 shows the relationship between overall limiting uni-axial tension strains at failure, and uni-axial compression lateral strains which are the average of two specimens in each case. For each tension failing strain two values of compression lateral strain have been plotted, the lateral strains at 60 and 70 per cent of the short term ultimate load of the specimen. Considering the distribution of the points about the equality line, it is suggested that the compression specimen has reached the limiting overall tensile or cohesive limit of the material in the region of 60 to 70 per cent of the ultimate load, probably nearer 60 per cent. In the uni-axial tension test, the overall limiting tensile or cohesive strain of the material corresponds to rapid crack propogation and complete disruption. However, in the compression test, although significant cracking occurs at 60 to 70 per cent of the ultimate load, the method of load distribution through the specimen allows the specimen to continue to support the load for a further considerable increase in stress.

In the compression test when the work of other investigators is considered, it is interesting to note that the stress level in this region corresponds to (1) a significant decrease in ultrasonic pulse velocity obtained by Jones[81] measured in the lateral direction on uni-axial compression specimens, (2) the rapid increase in Poisson's ratio observed by Lachance[78], Anson[87] and others, (3) the internal production of cracking noises obtained by L'Hermite[105] and Rüsch[53], and (4) the sustained load limit suggested by Rüsch[53]. These results

Table 9.3 Summary of the results of researchers who have investigated the point of onset of rapid curve propagation.

Investigator	Method	Strain (x 10^{-6})	Start of rapid curve propogation. (per cent ult. load) (static)	sustained load per cent of static loading.	Specimens
Jones[81]	decrease in ultrasonic pulse velocity	-	65	-	bobbins 3" ∅ 8" long central section
Berg[54]	observed micro-fissures with high powered microscope	100	55 - 60	55 - 60	8" x 8" x 30" prism.
L'Hermite[105]	sonic (listening) "cracking noise"	-	50 - 75	-	6" ∅ x 12" cylinders.
Rüsch[53]	sonic 'energy dissipation' method	-	60 - 75	75	prisms, $h/d < 3$.

Range of region at onset of rapid crack propogation = 55 - 75 per cent of ultimate static strength.



lateral strain in compression
measured at :
• 70 % ultimate load.
x 60 % "
line of equality
RELATIONSHIP BETWEEN
LIMITING UNI-AXIAL TENSION
STRAIN AND LATERAL
COMPRESSION STRAIN AT 60%
AND 70% OF ULTIMATE LOAD.
LIMITING UNI-AXIAL TENSILE STRAIN — MICROSTRAIN
200
150
100
50
50
100
150
200
250
UNI-AXIAL COMPRESSION STRAIN — MICROSTRAIN
IN THE LATERAL DIRECTION.

FIG. 9.28

are summarized in Table 9.3. It is suggested that at this zone, as defined by the limiting tensile or cohesive strain in uni-axial tension on specimens of similar proportions, the compression specimen ceases to behave as a continuum and becomes a highly redundant structure with no real Poisson's ratio or related behaviour. The specimen above this load is analogous to a group of closely spaced parallel columns loaded longitudinally, which suddenly lose lateral cohesion and start to buckle against and interact with each other. Although the structure can maintain load for a time, complete disruption is imminent either through crack propogation and creep or by an increase in the total load over a short period.

It is of interest to note that a number of workers who have performed tests in order to determine the fatigue limit of plain concretes under various stress conditions, have concluded that such a limit exists in the region of 60 per cent of the ultimate static strength[55].

9.3.4 Non-destructive Tests

The dynamic modulus (E_D) obtained from the longitudinal resonant frequency measurements has been compared with the compression and tension tangent moduli in 9.4.2. The dynamic Poisson's ratio (μ_D) measurements are given in Table 9.4. It is seen that in general, any factor which increases E_D decreases μ_D. This conclusion is in agreement with Simmons[107]. The values of μ_D are always greater than μ obtained from static measurements. The comparative average values for this investigation and for the work of Simmons are given in Table 9.5.

In Fig. 9.30, E_D is plotted against μ_D. It is seen that in general an independent linear relationship exists between μ_D and E_D for each group of materials tested: concretes, mortars and cement pastes. The results from Simmons, which were obtained from concretes only, have been superimposed on the graph and it is evident that his results were similar to those obtained by the Writer. The lack of points for the cement pastes is due to the uncertainty of the results for the resonant frequency which has been discussed in 6.6.5.

Table 9.4 Non-destructive testing results and a summary of the limiting tensile strains obtained from the uni-axial tension tests- Test series 3.

Spec. Desig.	Age	Density (lb/ft^3)	Damping Constant Q	E_D ($x\ 10^6$ psi)	μ_D	overall limiting tensil or cohesive strain. (uni-axial tension test) (microstrain)
1C	7	147.6	98	6.18	0.231	85
	28	147.9	116	6.60	.229	96
MC	7	146.4	109	5.96	.245	82*
	28	147.2	95	6.48	.230	90
2C	7	146.6	85	5.73	.248	90
	28	146.9	109	6.20	.236	97
3C	7	148.2	73	6.41	.233	78
	28	148.5	109	6.75	.216	103
NC	7	147.0	68	5.94	.245	72
	28	147.3	118	6.33	.230	81
4C	7	146.0	89	5.52	.254	80
	28	146.3	109	6.01	.242	90
1E	7	139.3	74	4.54	.249	150*
	28	140.1	94	5.07	.248	170
ME	7	139.8	75	4.49	.252	150*
	28	141.0	99	5.05	.242	217
2E	7	138.4	78	4.13	.256	165
	28	138.9	71	4.67	.247	192
3E	7	136.2	80	4.06	.250	150
	28	136.5	81	4.57	.248	166
NE	7	138.5	75	4.15	.254	160*
	28	138.8	77	4.74	.238	135
4E	7	138.2	70	3.89	.254	150
	28	138.5	91	4.44	.250	190
1P	7	124.0	66	2.79	.266	√
	28	124.3	√	√	√	√
2P	7	120.8	60	2.82	√	√
	28	124.4	59	2.89	.270	√
3P	7	118.5	59	1.82	.320	√
	28	119.4	65	2.85	√	√
4P	7	115.3	41	1.49	√	√
	28	116.3	70	2.01	.314	√
5P	7	112.3	51	1.08	.348	√
	28	113.5	62	1.80	√	√

√ no values available.

* estimated value by extrapolation.

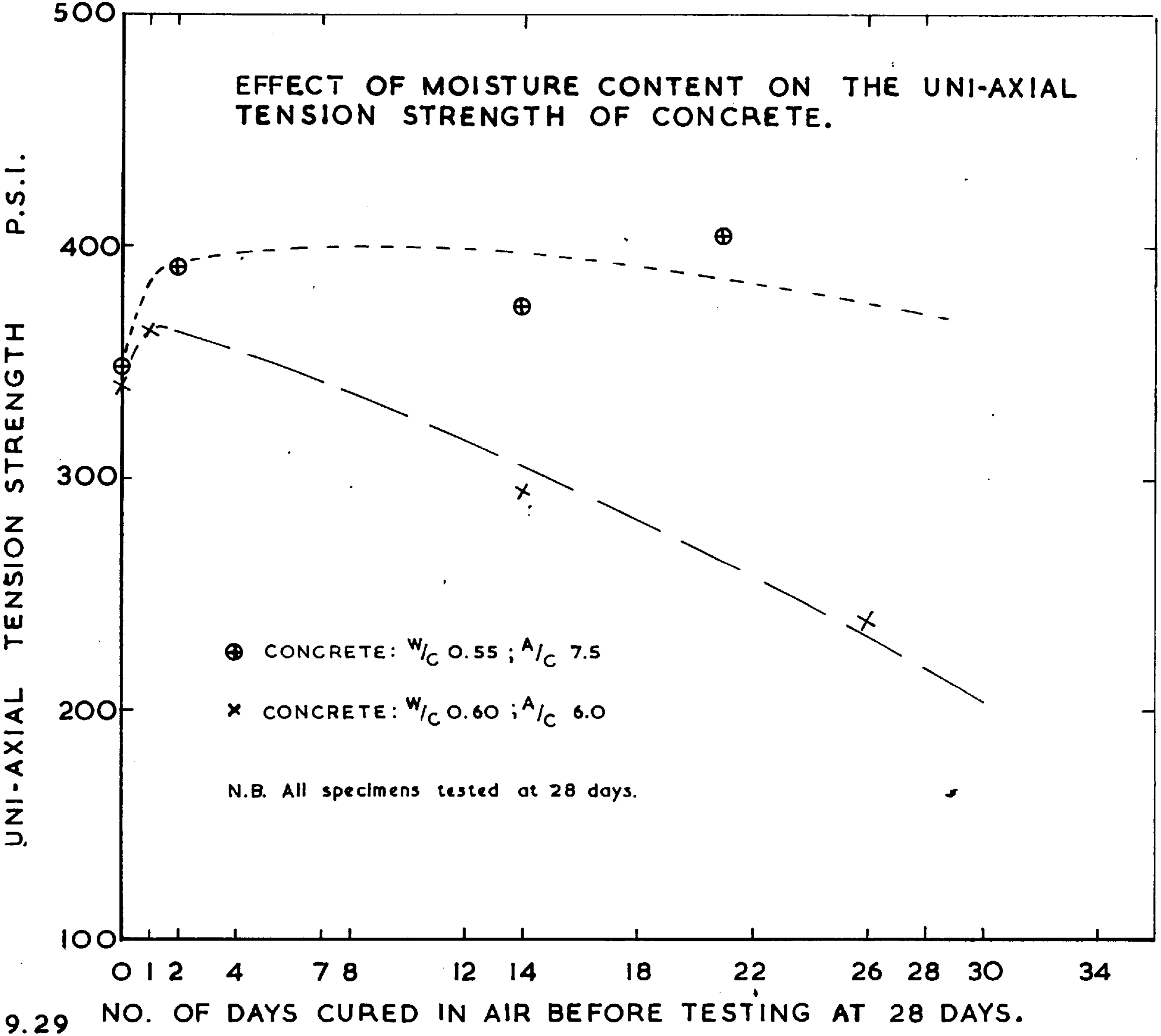

EFFECT OF MOISTURE CONTENT ON THE UNI-AXIAL TENSION STRENGTH OF CONCRETE.
UNI-AXIAL TENSION STRENGTH P.S.I.
500
400
300
200
100
0 1 2 4 7 8 12 14 18 22 26 28 30 34
CONCRETE: W/C 0.55 ; A/C 7.5
CONCRETE: W/C 0.60 ; A/C 6.0
N.B. All specimens tested at 28 days.
NO. OF DAYS CURED IN AIR BEFORE TESTING AT 28 DAYS.

FIG 9.29

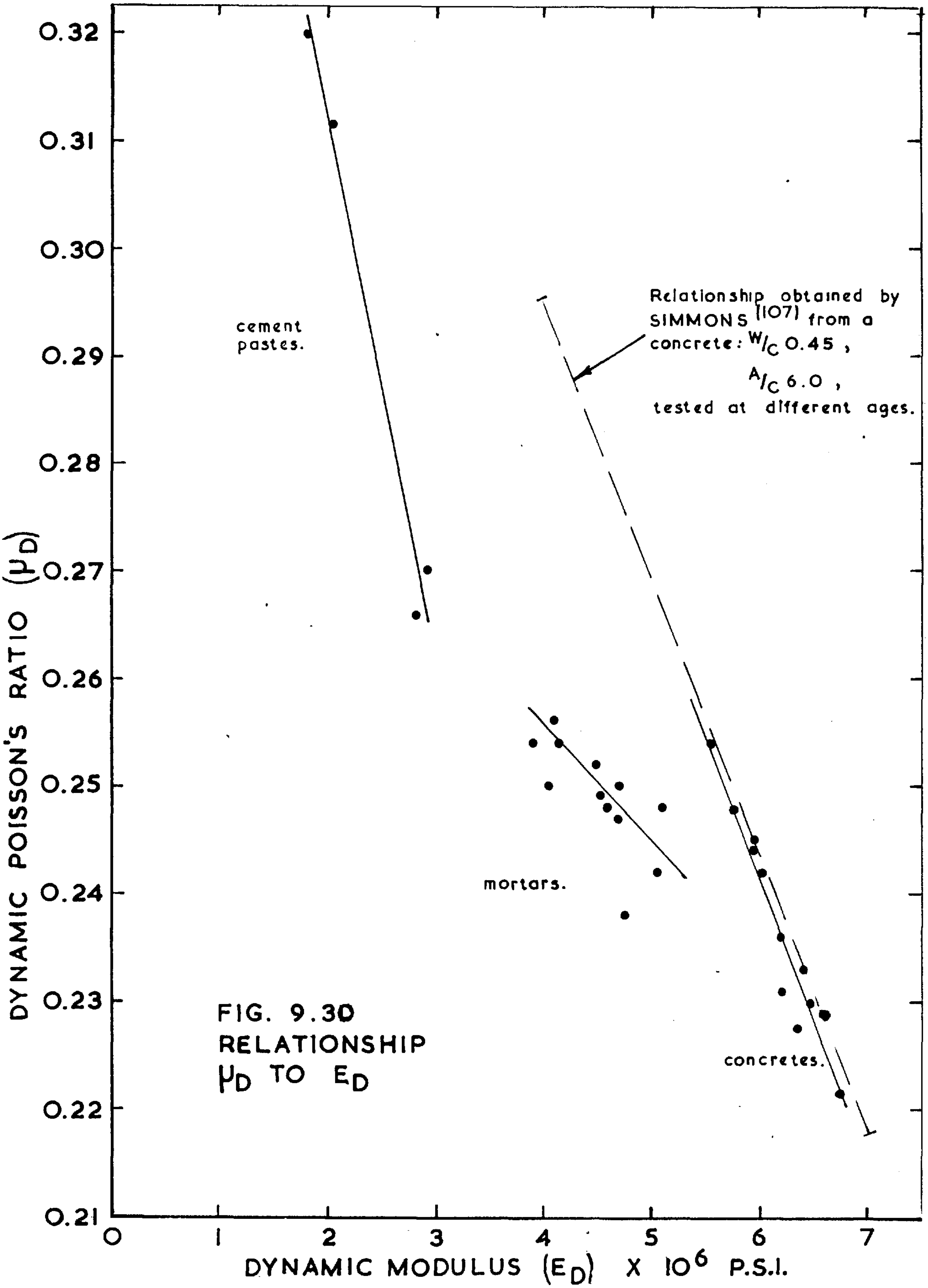


FIG. 9.30
RELATIONSHIP
μ_D TO E_D

In Table 9.4, the damping constants are given for all of the specimens tested. In general, there is an increase in damping constant (Q) with age which is in general agreement with Murdock and Kesler[55].

Table 9.5 Comparison of dynamic and static Poisson's ratio.

Source	Concrete A/C (by weight)	Concrete W/C (by weight)	Age	dynamic Poisson's ratio	static Poisson's ratio (1/3 ult.load) (see Table 9.4)
Writer	6.0	.55	7	0.254	0.152
			28	0.242	0.150
Simmons[107]	6.0		7	0.247	-
			28	0.235	0.157

9.3.5 Effect of Moisture Condition on Ultimate Tensile Strength.

To examine the effect of moisture loss on the tensile strength of concrete, two different concrete mixes were subjected to varying degrees of drying in laboratory air before testing. The stronger concrete had an aggregate/cement ratio of 7.5 and water/cement ratio of 0.55, and for the weaker mix, the aggregate/cement ratio was 6.0 and the water/cement ratio was 0.60.

Fig.9.29 shows the relationship between uni-axial tension strength and the number of days cured in air before testing at 28 days. Although the evidence is scanty, it is suggested that drying a concrete for one or two days does not adversely affect the tensile strength. For the weaker mix, it is evident that more than one or two days drying adversely affects the tensile strength. Such an effect is not so obvious for the stronger concrete, a result which is probably due to the greater difficulty of these specimens to dry out due to the denser more impermeable maxtrix formed from a cement paste with a lower water/cement ratio. When the fractured surfaces of the specimens were examined immediately after failure, the degree of drying was apparent from an outer zone of dry concrete, whilst an inner rectangular zone remained 'moist'. It was evident that the weaker mix lost more moisture for similar drying periods than the stronger mix. It should be mentioned that in all cases, except for the saturated specimens, the amount of drying was uneven. The thickness of the outer ring of dry material was always greater for the weaker mix.

The drier concretes had the higher moduli of elasticity in general but the evidence was not conclusive.

It is believed that the increase in strength observed with these mixes, when allowed to dry for one or two days before testing, is obtained from an increase in the bond between the mortar matrix and the aggregate due to the removal of free water. When sufficient moisture is removed to produce appreciable shrinkage stresses and cracks, a reduction in apparent tensile strength is obtained.

It is interesting to note the comments of Alexander[59] who reported that the exposure to laboratory air of small 1/2 ins. square mortar and cement paste flexural beams adversely affected the ultimate flexural tension strength later obtained, even when the period of drying was only in the order of minutes. It would appear that this was a surface phenomena and the effect of drying out was exaggerated by the small specimen size and the method of eventual testing.

9.3.6 Effect of loading rate on ultimate tensile strength.

Three specimens from a mortar mix with a sand/cement ratio of 2.0 and a water/cement ratio of 0.40 were tested in uni-axial tension where the total time required to load to failure was 25 seconds, 2 minutes and 4 minutes respectively. The results suggest that no appreciable difference in ultimate strength occurs for these loading rates, but the evidence is scanty. The modulus of elasticity obtained and the rate of crack propogation as indicated by the increase in curvature of the stress-strain relationship was of the same order.

It will be remembered that the loading rate in uni-axial tension for the destructive tests conducted in the second test series was 150 psi. per minute, compared to a loading rate of 100 psi. per minute in the third test series. Upon examination and comparison of the ultimate strength results obtained from the two test series, it is apparent that the specimens of test series 3 gave results which were about 5 per cent lower for comparable mixes. It is therefore concluded that the effect of loading rate on the tensile strength of concrete is similar to the effect obtained in compression, i.e. specimens tested at a lower loading rate give lower results.

It is considered that the effect of loading rate is related to the behaviour of concrete in the rapid crack propogation zone above 70 per cent of the ultimate load as obtained at conventional loading rates. The specimens tested at a faster loading rate have less time to 'break-down' than the specimens tested at the lower loading rate.

9.4 SUMMARY

A summary of the main points of interest from test series 3 and 4 will now be given.

1. A limiting tensile or cohesive strain of approximately 300 microstrain, in the paste phase, appears to exist for the concretes and mortars tested.

2. A linear relationship was obtained between the dynamic modulus of elasticity and both the static tension and compression moduli. In general, the dynamic modulus was 0.3×10^6 psi. and 0.5×10^6 psi. greater than the tension and compression moduli respectively.

3. In general, the static modulus of elasticity in compression was found to be equal to the static modulus of elasticity in tension, when both moduli are secant values measured at stress levels which are the same percentage of the relevant ultimate strengths of the material.

4. It has been shown that a relationship exists between overall longitudinal strains measured at 98 per cent of the ultimate stress in a uni-axial tension test, with the overall lateral strains measured at 60 to 70 per cent of the ultimate load in a uni-axial compression test.

4. A linear relationship exists between the dynamic modulus of elasticity and the dynamic Poisson's ratio of concrete materials, for each material type; concrete, mortar and cement paste. There is an increase of the Damping Constant (Q) for all materials with age.

6. Periods of drying up to approximately two days, do not appear to adversely affect the ultimate tensile strength of a normal concrete. Drying for longer periods appears to lower the ultimate tensile strength appreciably.

7. In uni-axial tension specimens tested over a longer period, i.e. at lower loading rates, give lower ultimate strengths.

CHAPTER 10

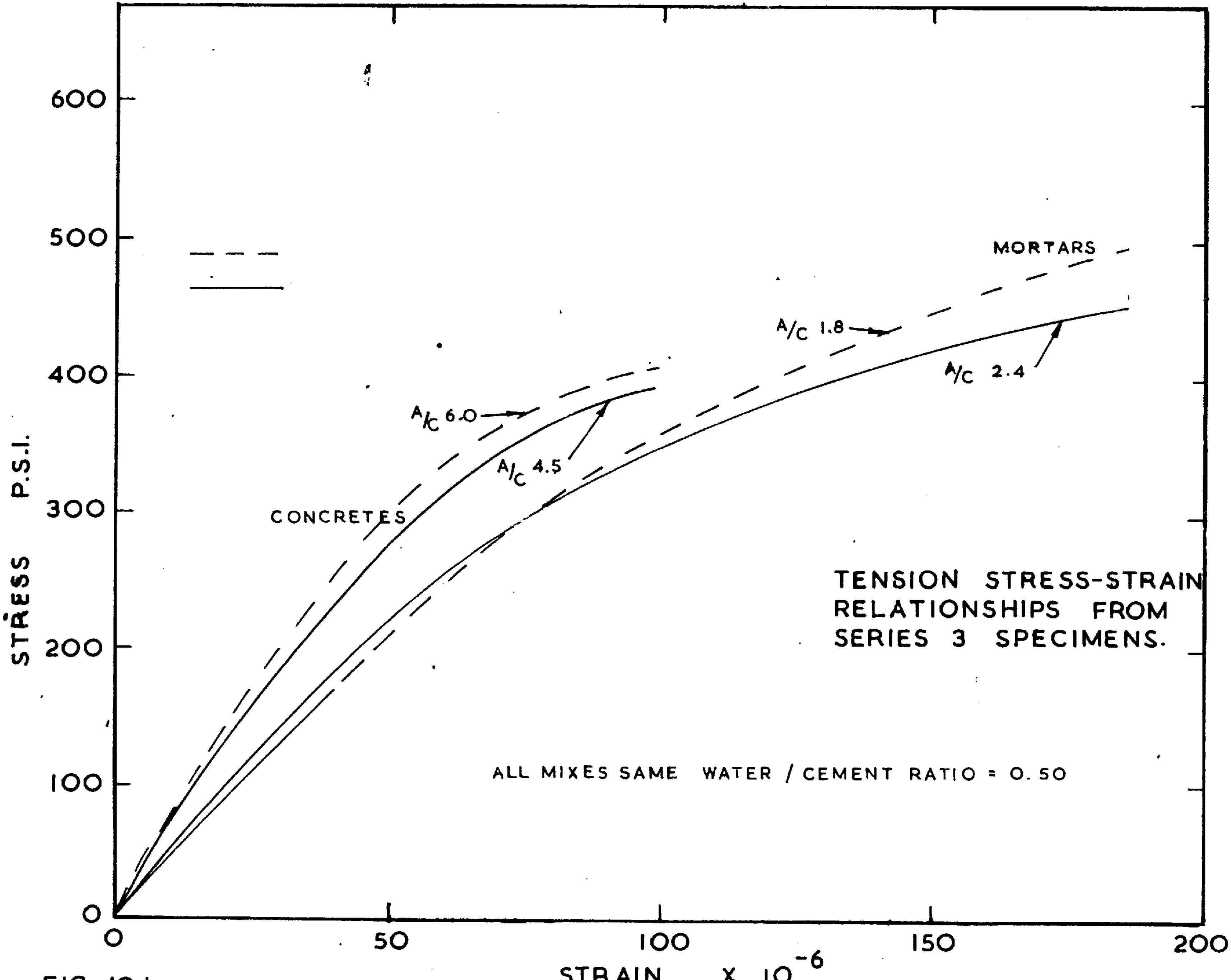
FURTHER DISCUSSION OF THE TEST RESULTS FROM TEST SERIES 1, 2, AND 3.

10.1 INTRODUCTION

Chapter 10 is a discussion of the deformation and failure behaviour of the concrete materials compiled from the information obtained in test series 1, 2 and 3, in the 'elastic' range and at failure. In particular, comments are made on the effect of the surface characteristics and shape of coarse aggregate particles on the behaviour of concretes. From information obtained from the stress-strain relationships of test series 3, it is possible to explain certain phenomena associated with the destructive testing results obtained from the second test series for a change in mix parameters.

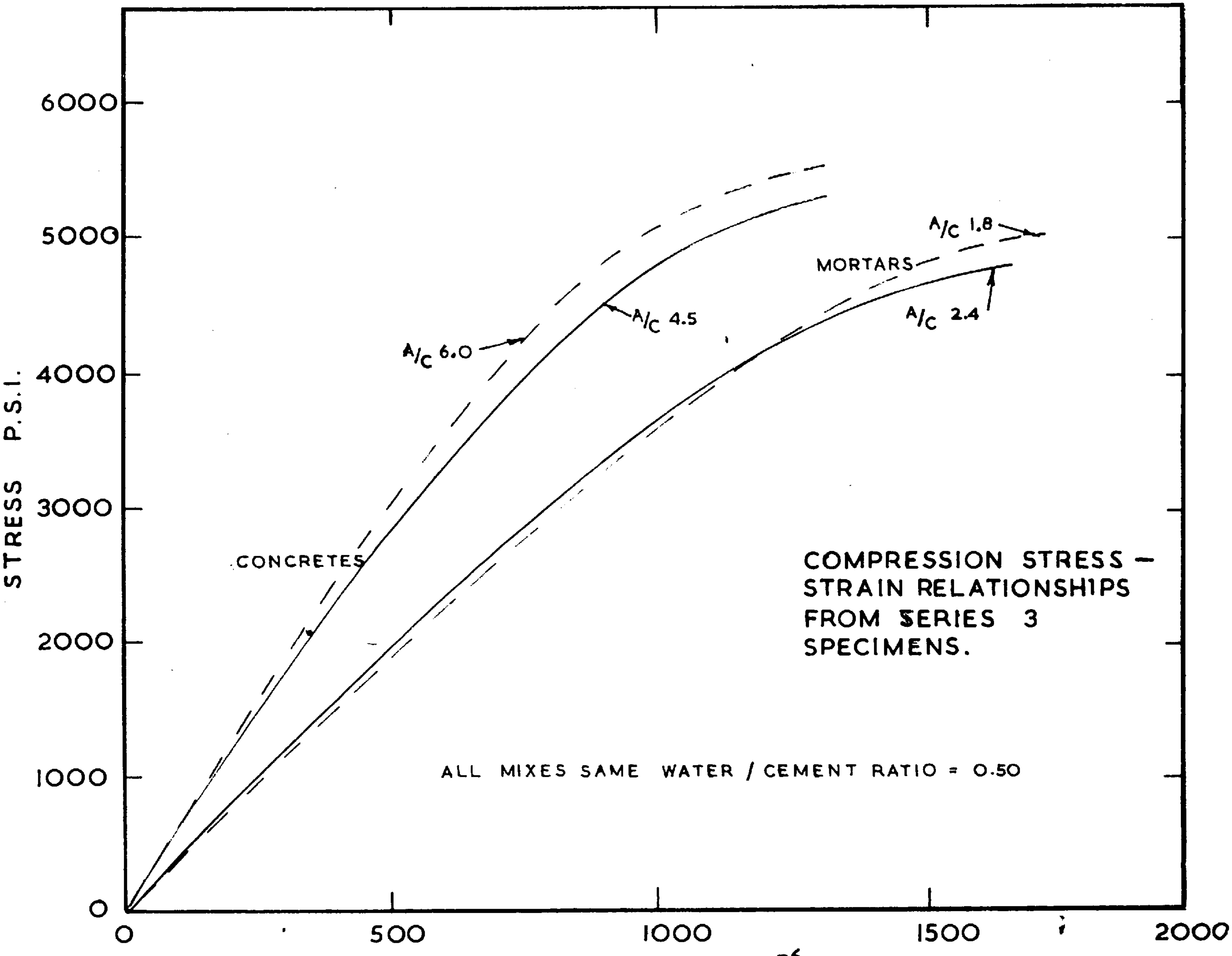
10.2 EFFECT OF AGGREGATE TYPE ON THE ULTIMATE STRENGTH OF CONCRETES

It will be remembered that the mixes used in series 2, although containing different aggregates, all have the same mix proportions by volume. Therefore, since the only difference between these mixes is the type of coarse aggregate used, the opportunity is presented of studying the effect of the type of coarse aggregate on the recorded ultimate strengths in uni-axial tension and compression. In an attempt to understand the mode of failure in uni-axial tension, photographs were taken and enlargements made of each of the two failure surfaces obtained from each test specimen. Although no quantitative information can be obtained from the photographs, the appearance of the failure surfaces does support the theory which has been developed to explain the differences between the strengths of the various mixes. Figs. 10.3 (A) and 10.4 (A) represent diagrammatically the local structure of concretes made from coarse aggregate with rounded shape and smooth surface texture, and Figs. 10.3 (B) and 10.4 (B), the structure of concretes made with angular coarse aggregate with rough surface texture. Consider the behaviour of the concretes first in compression and then in tension:



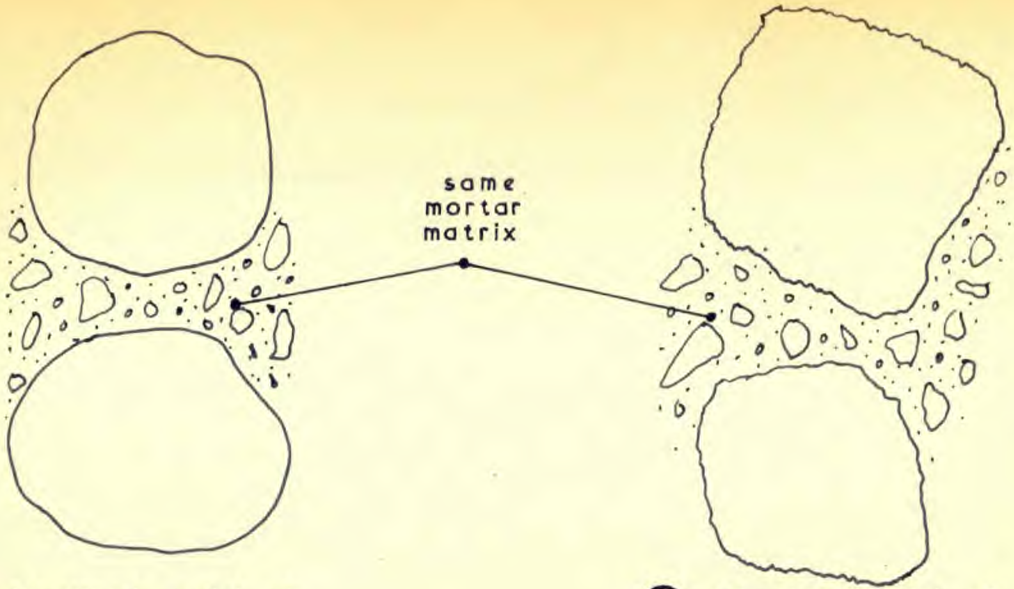
600
500
400
300
200
100
0
0
50
100
150
200
STRESS P.S.I.
STRAIN X 10^{-6}
MORTARS
CONCRETES
A/C 1.8
A/C 2.4
A/C 6.0
A/C 4.5
TENSION STRESS-STRAIN RELATIONSHIPS FROM SERIES 3 SPECIMENS.
ALL MIXES SAME WATER / CEMENT RATIO = 0.50

FIG. 10.1



6000
5000
4000
3000
2000
1000
0
STRESS P.S.I.
0
500
1000
1500
2000
STRAIN X 10^{-6}
A/C 6.0
A/C 4.5
A/C 1.8
A/C 2.4
MORTARS
CONCRETES
COMPRESSION STRESS — STRAIN RELATIONSHIPS FROM SERIES 3 SPECIMENS.
ALL MIXES SAME WATER / CEMENT RATIO = 0.50

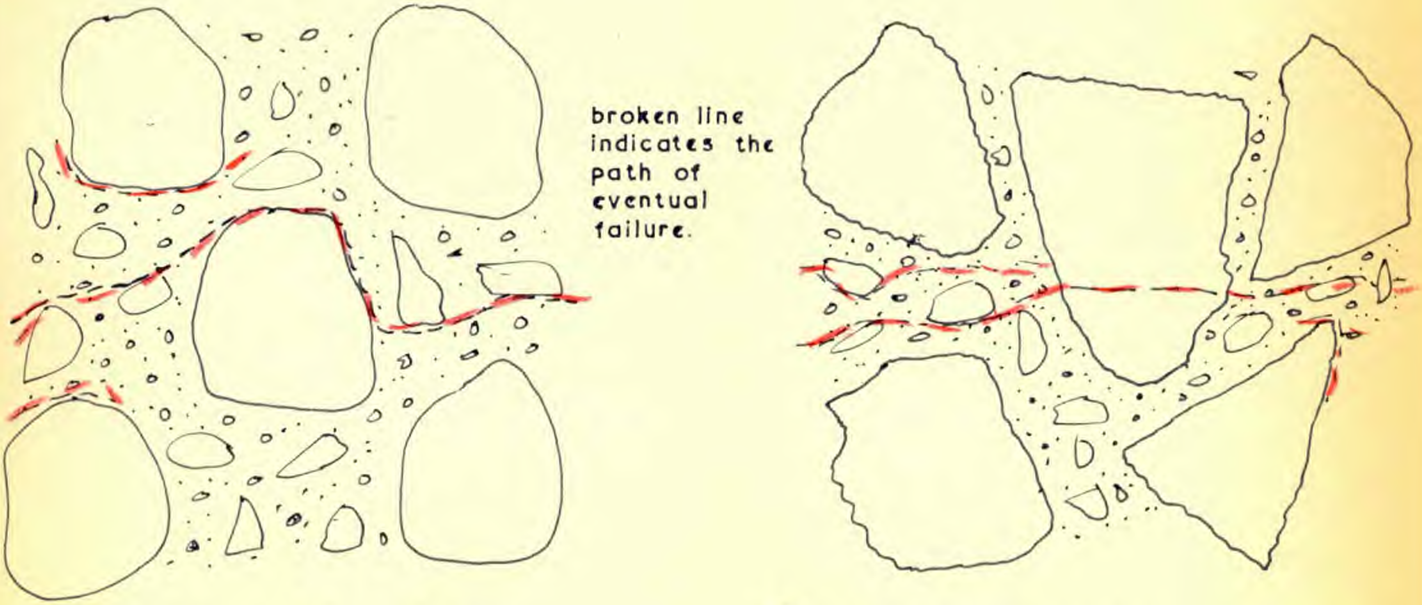
FIG. 10.2



Ⓐ Idealized model of rounded gravel coarse aggregate with weak bonding characteristics.

Ⓑ Idealized model of angular crushed aggregate with good bonding characteristics.

FIG. 10.3



Ⓐ Imminent failure for rounded gravel coarse aggregate.

Ⓑ Imminent failure for angular crushed aggregate.

FIG. 10.4

1. Compression:

Assume that the aggregate particles form a part of a concrete specimen subjected to a compressive loading. It is well known that concrete specimens, when subjected to compressive forces while being restrained from expanding laterally according to the Poisson's ratio of the material, can withstand much greater applied loads than in the unconfined compression test. This is shown by the higher strengths obtained from the 4 ins. cube test in compression with the 4 ins. prism used in this work. The rough texture of the angular coarse aggregate in Fig.10.3 (B) is likely to develop excellent bond with the surrounding mortar. Furthermore, the angular shape will provide 'mechanical interlocking' effects throughout the specimen. Both of these effects will tend to restrain the mortar between the pieces of aggregate. The rounded aggregate shown in Fig.10.3 (A), or any such aggregate, will not develop adequate tensile or shear bond strengths under high loads to provide the necessary restraint to the mortar phase. Thus, for such aggregates, the curvilinear stress-strain relationship and the eventual failure is the result of a series of aggregate-mortar bond failures followed near ultimate load by complete disruption of the mortar matrix. For the mixes containing the angular coarse aggregate (B), greater stresses may be developed in the matrix due to the increased restraint and 'mechanical interlocking' before failure occurs. This results in higher overall compression strengths than those obtained with the rounded aggregate (A). At failure, complete disruption is a combination of mortar crushing and aggregate failure.

If the compression strength of mortar is taken as the datum strength, it is seen from Table B that in general, the introduction of coarse aggregate particles into the mortar increase its compressive strength. It is known also that the introduction of coarse aggregate particles increases the order of stress concentrations within the specimen(110). It seems logical to assume therefore, that the existence of such restrained pockets of mortar could account for the higher ultimate strengths obtained. Anson(87) has shown that the existence of such a restraint effect would explain the low values of Poisson's ratio obtained for concretes (0.15 approximately) which are otherwise unaccountably lower than that of the constituent materials.

It is suggested that the ultimate compression strength of a specimen is a function of:

(1) the ability of the coarse aggregates to restrain the mortar pockets.

(2) the ultimate strength of the mortar layer in a restrained or unrestrained condition.

(3) the compression strength of the aggregate.

The above factors are listed in order of importance.

2. Tension:

Now assume that the aggregate particles in Figs.10.3 and 10.4 are part of a concrete specimen subjected to a tensile load system. It should be appreciated that the tensile strength of the mortar of a concrete has been found to be greater than the tensile strength of the concrete in the Writer's work and independently by Jones and Kaplan[49]. Therefore, the introduction of any coarse aggregate must reduce the ultimate tensile strength of a given mortar mix.

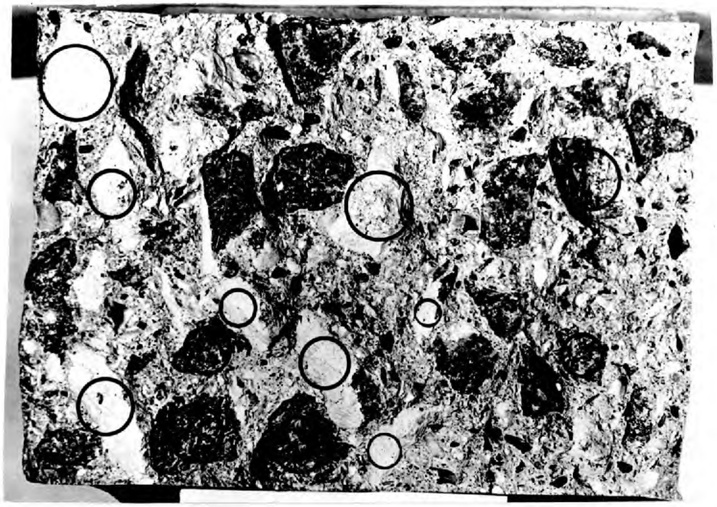
It has been established by Dantu[110] that local stress concentrations exist in concrete under load such that locally, the stress can be nearly three times the average stress across the section. It is suggested that the non-linear stress-strain relationship for concretes is related directly to such stress concentrations developed within a specimen under load.

Suppose that the model aggregates in Fig.10.3 (A) and 10.3 (B) represent the area in two concrete specimens under load where a stress concentration is developing. In (A), because of the weak bond properties of the aggregate, there will be a breakdown in aggregate-mortar bond. In (B), where a stronger aggregate-mortar bond exists, a higher stress level can be tolerated before failure occurs in the mortar phase.
It is considered that with aggregate (A), the non-linearity of the stress-strain relationship and ultimate failure is comprised of a

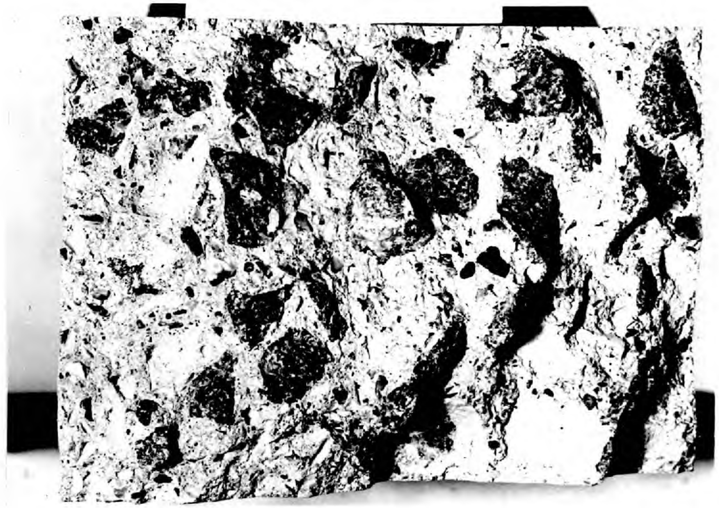
series of local bond breakdowns followed progressively by mortar failures as the load is increased. The situation at imminent failure for concretes containing smooth rounded aggregates is shown in Fig.10.4 (A). It is suggested that the failure plane will progress across the specimen section passing around the coarse aggregate particles. Thames Valley river gravel coarse aggregate has many of the characteristics of the idealised aggregate of type 10.3 (A) and 10.4 (A). A typical failure section of this aggregate tested in uni-axial tension is shown in Plate 10.1 (A). It is seen that the final failure followed the weak bond interfaces as described above.

For rough angular aggregate, continued loading is more likely to induce failure in the matrix until disruption is imminent. Fig.10.4 (B) shows such a concrete at this stage. At rupture, failure has progressed across a plane through the mortar matrix. Because of the high bond characteristics of the rough surface of the coarse aggregate restrained on each side of the 'failure plane', failure is more likely to occur through the aggregate. In such a case, the total bond forces and possible interlocking effects, if the aggregate is angular, are greater than the tensile strength of the aggregate. The Quartzite gravel and the Granite aggregates of test series 2 approach the characteristics of this model aggregate. The failure planes of a typical concrete containing these aggregates is shown in Plate 10.1(B),(C). It is observed that the failure plane has in general, passed through the coarse aggregate particles instead of around the aggregate-mortar interfaces as in (A) above.

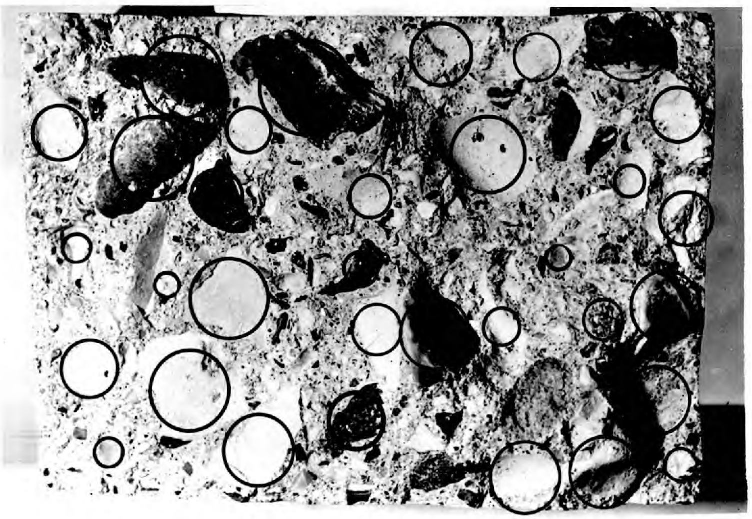
The Leca coarse aggregate concretes are a special case. In these concretes, the bonding ability of the porous surface of the coarse aggregate is high. However, the aggregate is weak and its stiffness is low. As a result, the ultimate strength behaviour of these materials is dictated almost entirely by the mortar matrix. If some portion of the specimen is subjected to a stress concentration, it is possible that the aggregate particles in the vicinity would fail. However, in the general case, it is considered that the mortar will eventually fail first and then the failure develops and propogates through the aggregate. A typical failure plane is shown in Plate 10.1 (D).



(B)



Circles indicate aggregate-mortar bond failures



(A)

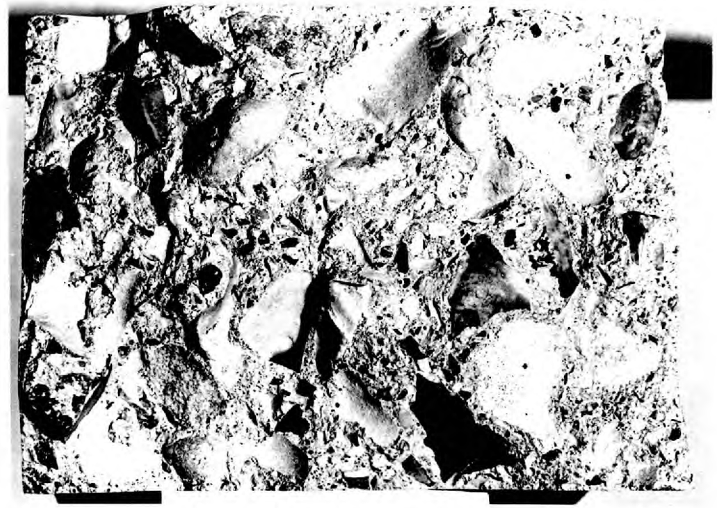
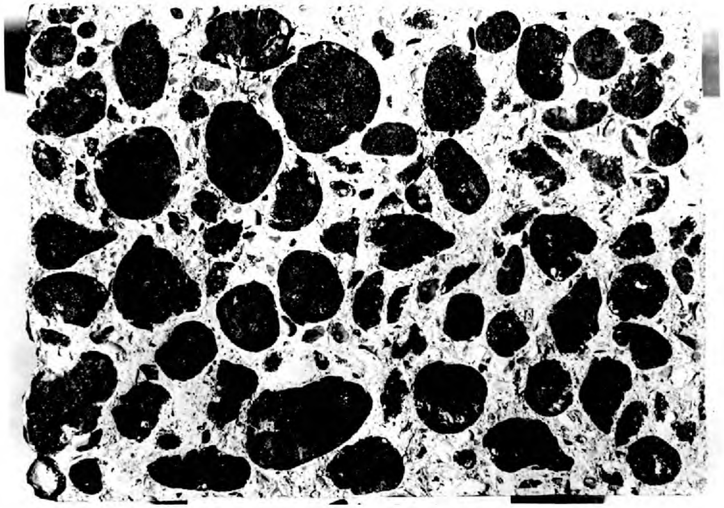
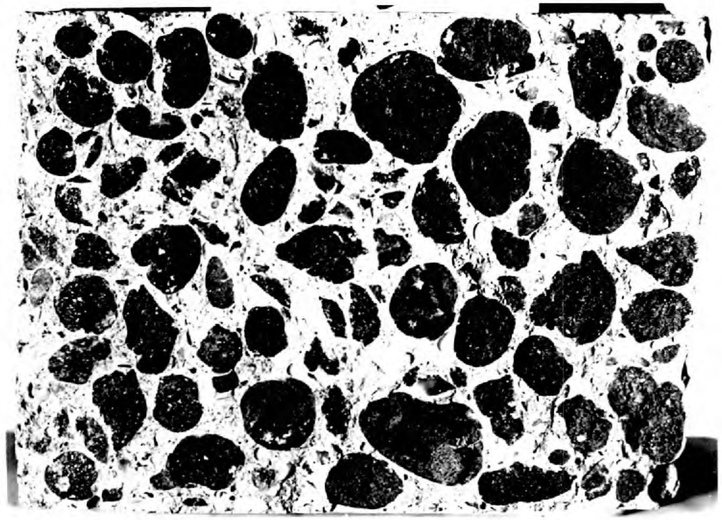


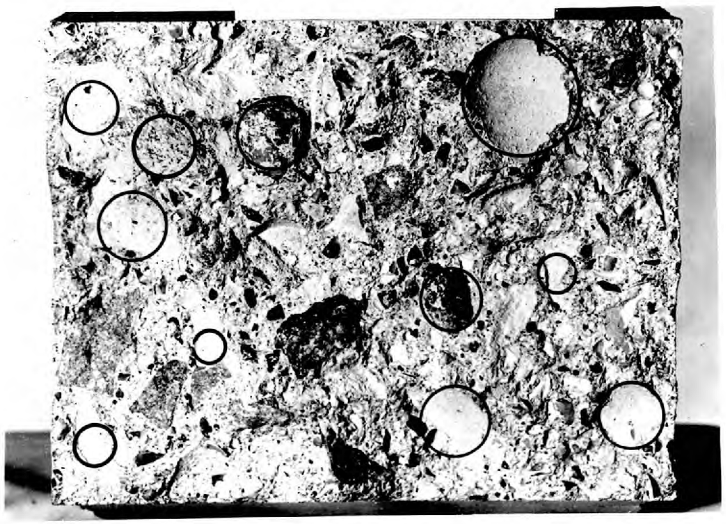
PLATE 10.1

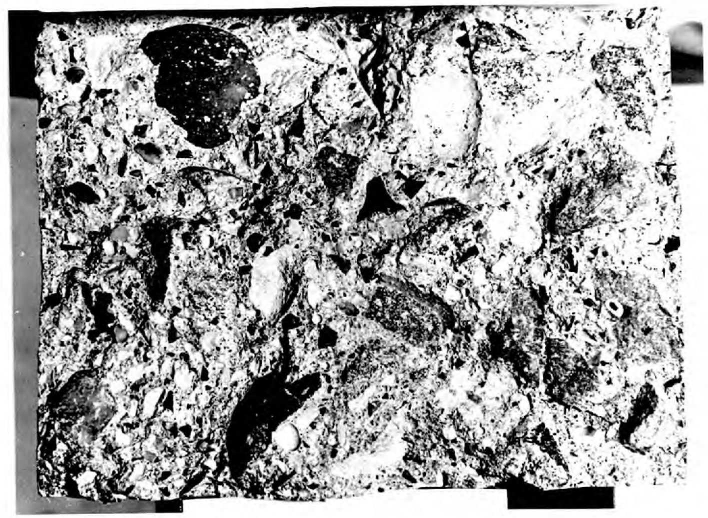




(D)

Circles indicate aggregate-mortar bond failures.





(C)

PLATE 10.1(cont.)

For mortars, because of the increased particle/volume ratio, it is suggested that the chances of a local stress concentration (which results in an early breakdown of the aggregate-paste bond or local paste failure) propogatiing to failure is lower because of the greater possibilities of stress redistribution available in such a material. Furthermore, mortars are more nearly 'homogeneous' than concretes and effective stress concentrations are probably smaller in magnitude and more local in effect. It has been shown by Alexander[59] that the bond strength of small aggregate is greater than that of comparable large aggregate which could also have an effect on the ultimate tension strengths obtained from mortars.

3. Summary:

The effect of type of coarse aggregate on the strength of concrete can be summarized as follows. If the strength of mortar is taken as the datum strength, its apparent compression strength can be increased by the addition of stronger coarse aggregate particles which are angular with rough surfaces textures. Such an aggregate will promote good tensile and shear bond strengths together with 'mechanical interlocking'. The tensile strength of the mortar cannot be increased by the addition of normal aggregates since the introduction of the coarse aggregates gives rise to local stress concentrations in the mortar which initiate failure at a lower strength than the parent material. In addition, some aggregates with weak aggregate-mortar bond strengths introduce possible incipient weaknesses into the specimen.

It is suggested that the limiting strain in tension would be of similar order for concretes containing either the rounded or the angular aggregates for similar mix proportions by volume. It appears that the overall limiting strain for mortar is similar for any mix strength. This is shown by the results of test series 3 in which the failing strains measured on the comparatively weak mortars compare favourably with those failing strains obtained from the strong 'calibration' mortars of Table 4.4. It is therefore logical to suggest that in concretes, regardless of the mode of failure, i.e. whether failure initiates at the aggregate-mortar interface or through the mortar matrix, the overall limiting strains for concretes containing either aggregate type should be of the same order of magnitude.

In the foregoing discussion, it was suggested that the surface texture and shape of the coarse aggregate particles are the major contributing factors in determining the ultimate strength behaviour of concrete in tension and compression. It is realised that Kaplan[111] has shown that the major factors in determining the flexural and compressive strength of concrete are shape, surface texture and modulus of elasticity of the coarse aggregate. However, it is believed by the Writer that the effect of shape and surface texture of the coarse aggregate on the strength of concrete in compression and tension is more significant than the effect of aggregate modulus of elasticity. A small increase in aggregate stiffness would result in an increase in restraint effects. Nevertheless, with the normal concrete aggregates, the effect of this increase in restraint is considered to be much smaller than that resulting from a change in aggregate shape and/or surface texture.

It is noted that Hsu et al[112] have suggested a similar argument to the Writer's for the mode of failure of concretes in compression. Their contribution is based on evidence from an experimental programme where slices of concrete, which had been loaded to a predetermined strain, were examined for cracks. Their evidence and their resulting argument could only be related to the failure of concrete containing weaker rounded aggregate concretes (A) since 6 ins. diameter cylinders tested at 197 days gave average results of only 3800 psi.

10.3 THE BEHAVIOUR OF MORTARS AND CONCRETES FOR INCREASING AGGREGATE/CEMENT RATIO

In Fig. 10.1 and 10.2, compression and tension stress-strain relationships obtained from the test series 3 results have been drawn for the concretes with aggregate/cement ratios of 4.5 and 6.0 and the mortars of the concretes with aggregate/cement ratios of 1.8 and 2.4. It will be remembered that both of the above pairs of mixes had a constant water/cement ratio of 0.5.

Examination of the figures shows that for the concretes, the mix with the higher aggregate/cement ratio gives the higher ultimate strength in both tension and compression. This result was also obtained throughout test series 2. The rate of crack propogation, as indicated by the rate of decrease in slope of the apparent instantaneous tangent modulus at any strain level, appears to be similar for both mixes and the increase in strength appears to be related to an overall increase in stiffness for the material.

Considering the stress-strain relationship shown for the mortars, it is observed that the increase in aggregate/cement ratio has brought about the reverse effect in tension and compression, an observation which was also reported recently by Ishai[113]. The lower aggregate/cement ratio has given the higher strength. As before, this is a result obtained throughout test series 2. It is seen that the specimen with the higher aggregate/cement ratio is stiffer initially, as would be expected, but at about 85 per cent of the ultimate loads shown, the stress-strain curves for the specimens cross over. It is obvious that the rate of crack propogation of the higher aggregate/cement ratio mortar is greater after 60 per cent of the ultimate loads have been exceeded.

The result for the concrete is easily explained. When the aggregate/cement ratio of a concrete is increased, the thickness of the cement paste film in the mortar of the concrete will be reduced (this film thickness has been calculated to be of the order of 0.005 ins. thick in normal concretes[79]). This results in an added restraint effect so increasing the ultimate compressive strength of a given concrete. A small increase in aggregate/cement ratio involves the introduction of an extra quantity of aggregate with a given surface area. As the paste-aggregate bond relationship is in general the weakest part of a concrete mix, it could be argued that the increase in surface area and a reduction in the total cement paste volume in the concrete (see Table 9.2) would cause lower strengths. Because this does not occur, it may be assumed that the increase in restraint effect resulting from the thinning of the paste coating throughout the specimen has more than offset the effect of increased surface area in bond. Therefore, higher compressive strengths are the result.

When the concrete is in tension, it could be argued that since restraint at a tension aggregate-mortar bond interface tends to induce a state of triaxial tension in the mortar, the strength of the bond might decrease when the restraint effect is increased. However, shear bonds and 'mechanical interlocking' effects can only increase due to increased restraint effects and it is probable that these effects more than offset the tension bond effects, resulting in an increase in strength.

For the mortars, although the change in aggregate/total paste ratio by volume is the same as for the concrete, the material added to raise the aggregate/cement ratio will increase the surface bond area considerably due to the high specific surface (i.e. sq.cms/gm.) of the smaller size materials. This added aggregate does increase the stiffness of the specimen at low stresses. However, it is suggested that the crack propogation at loads near the ultimate is in the form of local bond failures. Therefore, it is logical to assume that the increased rate of crack propogation of the higher aggregate/cement ratio mortar is due to this significant increase in the total bond area. It would seem that this effect has not been offset by the marginal increase in stiffness evident at low loads. Therefore, lower ultimate tensile and compressive strengths are the result.

The above argument does provide a satisfactory explanation of the behaviour due to a change in aggregate/cement ratio for concretes and mortars in either tension or compression for the range of mix parameters used in this work.

10.4 THE RELATIONSHIP EXISTING BETWEEN UNI-AXIAL COMPRESSION AND UNI-AXIAL TENSION STRENGTHS

In Chapter 8, Fig.8.14 indicates that a non-linear relationship exists between ultimate uni-axial tension and uni-axial compression strengths.

It has been shown in 10.2 that the limiting criteria for the strength of concrete in tension is the tensile strength of the constituent mortar. However, in compression, the introduction of coarse aggregate can produce a restraint effect which causes an increase in the effective mortar compressive strength. With the exception of aggregate failure, the ultimate strength of concrete in compression is related therefore to the ability of the mortar pockets to withstand high stresses. This depends on such parameters as the water/cement ratio, age, etc. In tension, the limiting criteria of the strength of a concrete mix is the local tensile strength of the constituent mortar and the aggregate-mortar bond.

It can be argued that the reason for the greater percentage increase obtained in the compression strength of a mix compared with the percentage increase in the tensile strength of the same mix, can be found in those factors which determine the compression and tension ultimate strengths.

Consider the effect of an increase in the tensile and shear bond strength of the coarse aggregate-mortar interface resulting from an increase in the age of a concrete specimen. In tension, an increase in either bond strength will raise the stress level to which the interface may be subjected without failure. If, in the younger concrete, failure was taking place through the mortar because of adequate surface roughness of the aggregate, etc., the increase in bond as the concrete matures would probably increase the strength of the mortar matrix in a similar way. In compression, it is argued that an increase in the aggregate-mortar bond has a much greater effect on the strength of the restrained mortar pockets, i.e. small increases in the amount of lateral restraint in a local compressive state of stress can increase the apparent compression strength considerably.

As a result of the above argument, any factor which increases the tensile strength of a concrete mix would have a correspondingly greater effect on the compressive strength of concrete. This would explain the apparent non-linear relationship between the ultimate strengths obtained from concretes tested in uni-axial compression and uni-axial tension.

10.5 SUMMARY

It is of interest to review the conclusions reached in this Chapter.

1. A theory of failure has been proposed to explain the different strengths obtained from concretes with the same volumetric mix proportions, but with different types of coarse aggregates. It is suggested that any strength differences are related in general, to the surface texture and shape of the coarse aggregate particles. Smooth round aggregates provide poor aggregate-mortar bond and therefore, lower tension and compression strengths, whereas rough textured angular aggregate produce good aggregate-mortar bond and higher tension and compression strengths.

2. The relative behaviour of concretes and mortars for increasing aggregate/cement ratios is explained is terms of (1) above and a discussion of the rate of crack propogation as observed from the appropriate stress-strain relationships in test series 3. It is suggested that increased internal stiffness accounts for the increase in ultimate compression and tension strengths observed when more aggregate is added to a concrete. However, the faster rate of crack propogation at high loads because of greatly increased area of bond surfaces appears to be the cause of the apparent decrease in ultimate strength in tension and compression for mortar mixes for small increase in aggregate content.

3. From consideration of the general theory of failure in (1) above, a rational argument is developed to explain the apparent non-linear relationship existing between uni-axial tension and compression. It is concluded that any factor such as, in particular, the internal restraint effect, which might increase the tensile strength of a mix, will have an even greater effect on the compressive strength.

CHAPTER 11

CONCLUSIONS AND SUGGESTIONS FOR FUTURE RESEARCH

11.1 SUMMARY OF MAIN CONCLUSIONS

The main conclusions of this work can be effectively divided into three parts:

(1) The uni-axial tension testing machine. (Chapters 3, 4 and 7).

(2) Ultimate strength properties of concrete materials. (Chapters 8 and 10).

(3) Deformational properties of concrete materials. (Chapters 9 and 10).

(1) The uni-axial tension testing machine:

i. A uni-axial tension machine has been developed which satisfies the criteria,

$$\sigma_x = \sigma_y = 0$$
$$\sigma_z \neq 0$$

ii. It has been shown theoretically and experimentally that (1) the testing machine and the test specimen are capable of performing a uni-axial tension test on normal concrete materials such that eccentricity due to non-alignment of the applied load with the test specimen cross-section is negligible, (2) the specimen shape does not introduce stress concentrations, and (3) the testing machine is capable of loading by constant increase of stress but not constant increase of strain.

iii. As a result of the first test series, it is concluded that the new tension test has similar precision and reproducibility as other tension and compression tests, defined as a function of the variability of test results obtained with the machine.

(2) Ultimate strength properties of concrete materials:

i. A non-linear relationship exists between the uni-axial tension and uni-axial compression strength of concretes and mortars. For increasing overall mix strength the ratio of tension to compression decreases. In general the tensile strength is 7 - 14 per cent of the compression strength.

ii. A linear relationship has been established between uni-axial tension and indirect tension (cylinder splitting) ultimate strengths for normal concretes. In general, the uni-axial tension results are 2 per cent higher than indirect tension results for normal concretes. It has been shown that for Leca concretes and mortars the indirect tension test does not give a good indication of the uni-axial tensile strength of these materials.

iii. For the concretes tested, those containing the granite coarse aggregate gave the highest compressive strengths. In tension, the mortar of the concretes gave the highest tensile strengths.

iv. An increase in aggregate/cement ratio results in an increase in the observed compression and tension strength results, with any test method, for all concretes with the exception of Leca concretes. In mortars and the Leca concretes, an increase in aggregate/cement ratio resulted in a decrease in observed strengths, with any test method, in contradiction to the foregoing conclusion.

v. A theory of the mode of failure for concrete materials containing different coarse aggregate particles for tension and compression is proposed. Failure is related to the surface characteristics, the shape and the strength of the coarse aggregate particles. It is concluded that the strength of concrete materials depends on a local internal restraint effect and the aggregate-mortar bond strength, both of which are a function of the shape and surface characteristics of the coarse aggregate particles.

vi. An explanation is given to account for the behaviour of concretes and mortars when the amount of aggregate is increased. This is based on conclusion (v.) above

vii. The uni-axial tension strength of a concrete specimen is increased when the specimen is allowed to dry for a short period before testing. A decrease in tensile strength is obtained when long periods of drying are allowed before testing. It would appear that this is due to the development of micro-cracks resulting from appreciable shrinkage stresses within the specimen when the moisture loss reaches certain limits.

viii. It appears that the effect of loading rate on apparent uni-axial tension strengths is similar to that observed for compression and flexure tests. Thus, slower loading rates yield lower apparent tension strengths.

(3) Deformational properties of concrete materials:

i. A limiting tensile or cohesive strain limit exists for the paste phase of concrete materials loaded to failure in uni-axial tension.

ii. Linear relationships exist between the dynamic modulus of elasticity (E_D) and the static secant modulus of elasticity in tension (E_T) and compression (E_C), and between E_C and E_T. E_D is always greater than E_C or E_T. In general, the secant E_T is equal to the secant E_C at comparable stress levels.

iii. The limiting tensile strain of concretes and mortars from a uni-axial tension test is related to the lateral extensional strain at approximately 60 per cent of the ultimate load of compression specimens.

11.2 SUGGESTIONS FOR FUTURE RESEARCH

In the light of this work, the following topics for future research and investigation are suggested:

(1) It would be of interest to extend the experimental programme of work to include a more fundamental approach to establish the effects of the following parameters on the deformational and failure behaviour of concrete materials:

i. modulus of elasticity of aggregate.

ii. surface texture and properties of aggregate.

iii. shape of aggregate.

It is suggested that such an experimental programme could include model aggregates, prelocated in the test specimen, such that the above parameters could be changed by using different surface coatings, etc.

(2) Another avenue for future research would be the preloading to different stress levels of the central reduced portion of the uni-axial tension specimen by uni-axial compression. This could be accomplished using two cubes of the same material as platens, loading in a direction perpendicular to the eventual tensile loading axis of the specimen. It is considered that valuable information on the internal breakdown of concretes could be obtained using this technique.

(3) A study of the properties of cement pastes in uni-axial tension would be useful in determining the part which the cement paste plays in the deformation and failure of concrete materials.

(4) A new method is proposed here by the Writer for testing concrete materials in uni-axial tension. This method, which is based on the lazy tong system, would be an improvement on the Writer's machine, since the loading system can be used in a uni-axial compression machine (see Fig.11.1). It is suggested that the design and construction of this device is a logical development of this work.

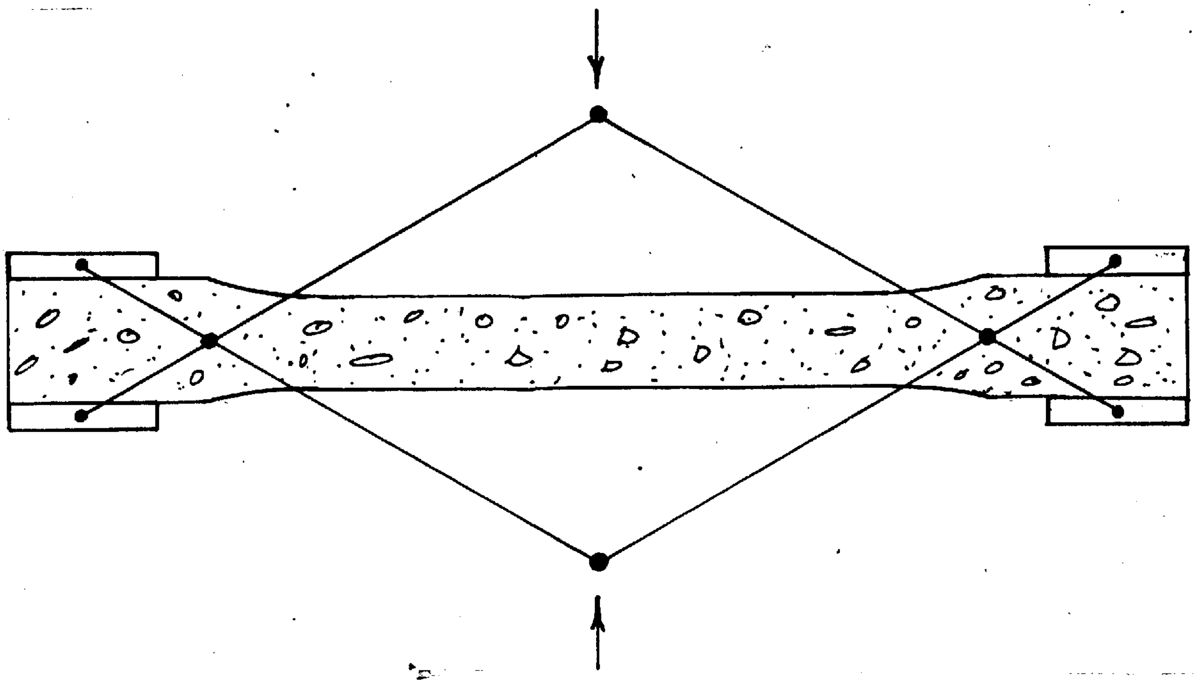


Fig.11.1 Schematic drawing of proposed lazy tong uni-axial tension device for use in a standard compression machine.

(5) The present work was restricted to the testing of concrete materials continuously to failure. It is suggested that a better understanding of the cracking of concretes would result from an investigation in which strains were measured while subjecting the materials to repeated loading and unloading cycles. It is only through such tests that a true understanding of the 'elastic' portion of the stress-strain curve up to the onset of rapid crack propogation may be gained.

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APPENDIX A

MACHINE SHOP DRAWINGS FOR THE FRAME COMPONENTS OF THE UNI-AXIAL TENSION TESTING MACHINE

TOP PLATE

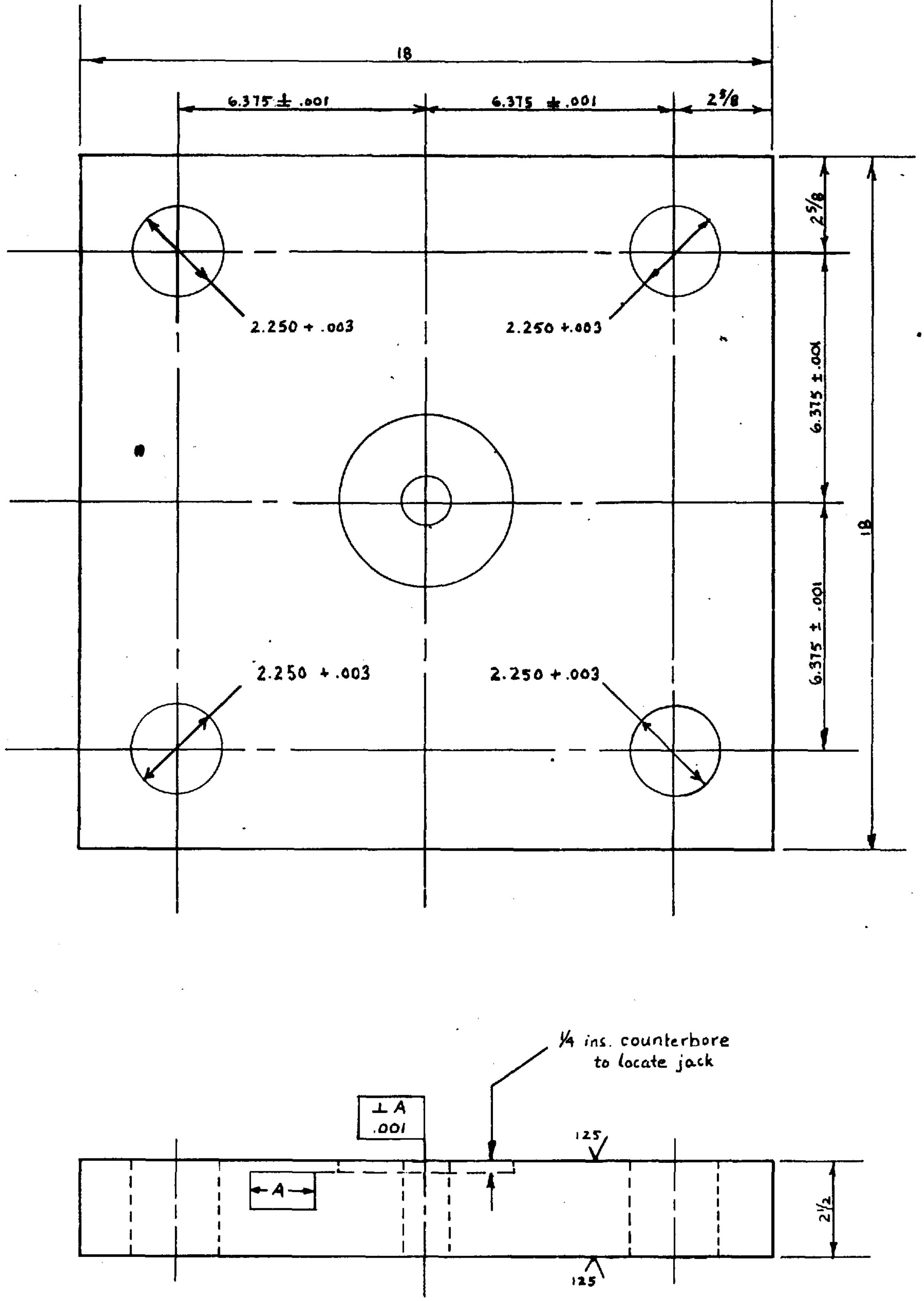


FIG. A1 quarter scale

JACK LINK

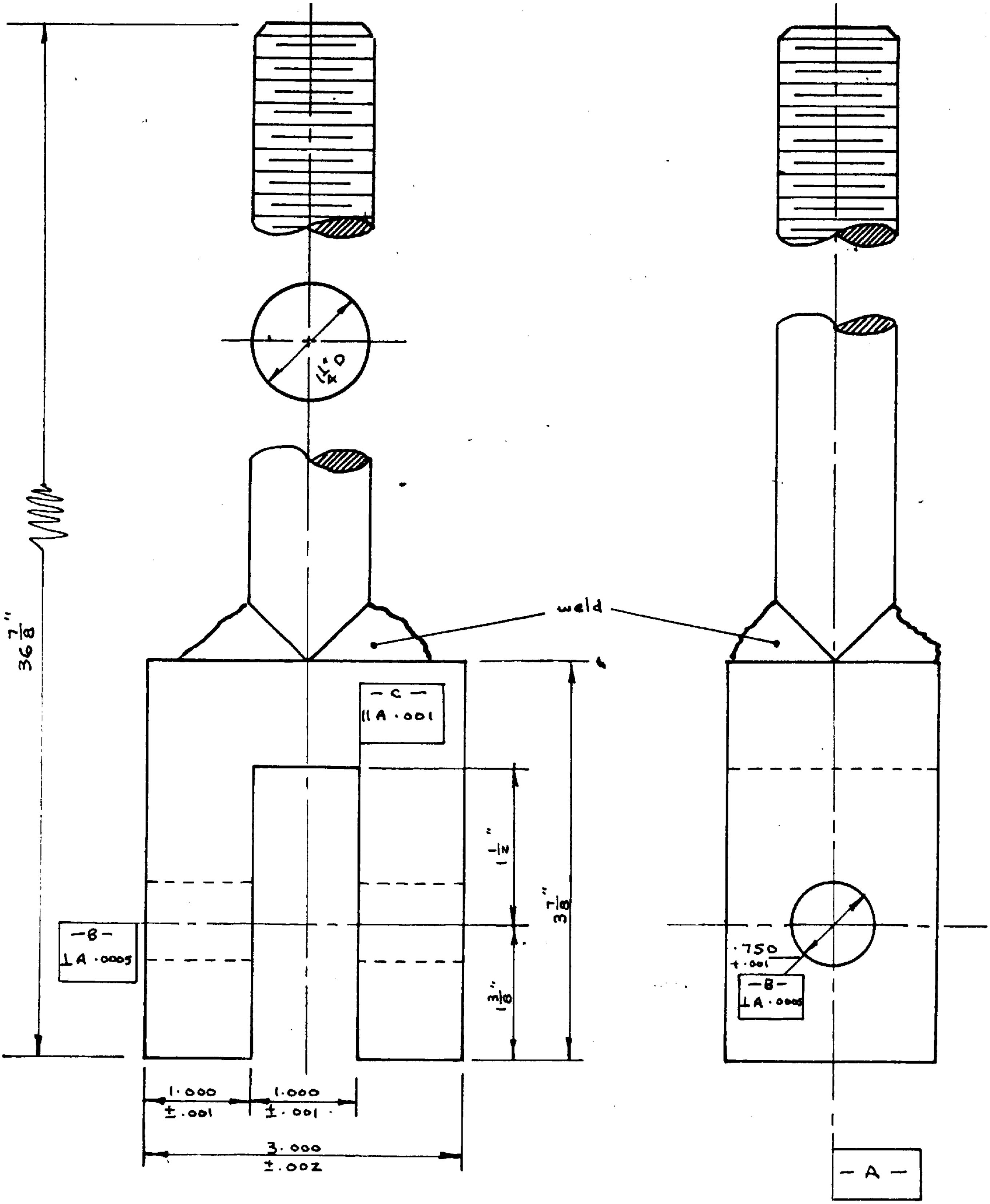


FIG. A2

scale: $\frac{3}{8}'' = 1''$

MIDDLE LINK

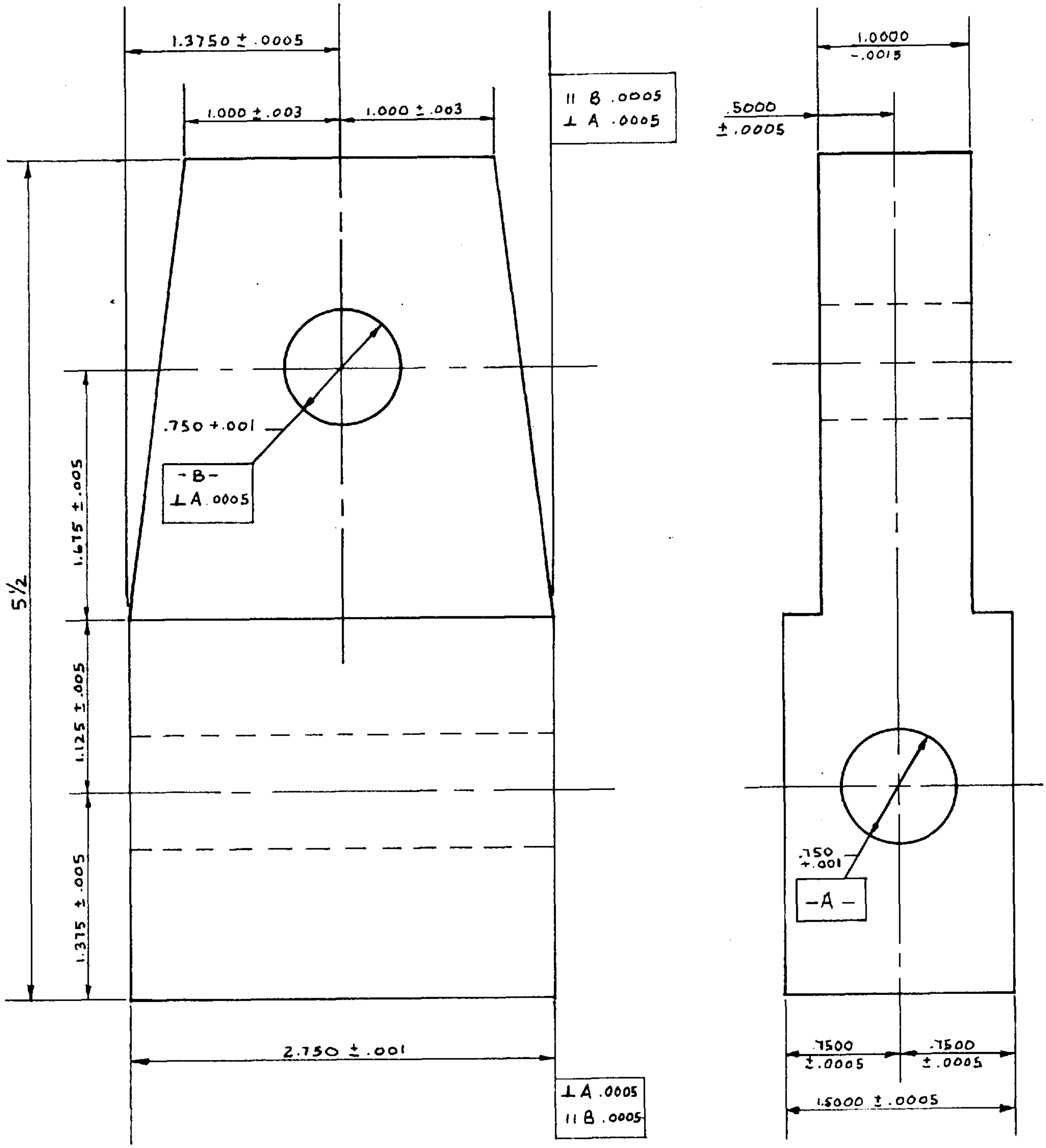


FIG. A3 full scale

BOTTOM LINK

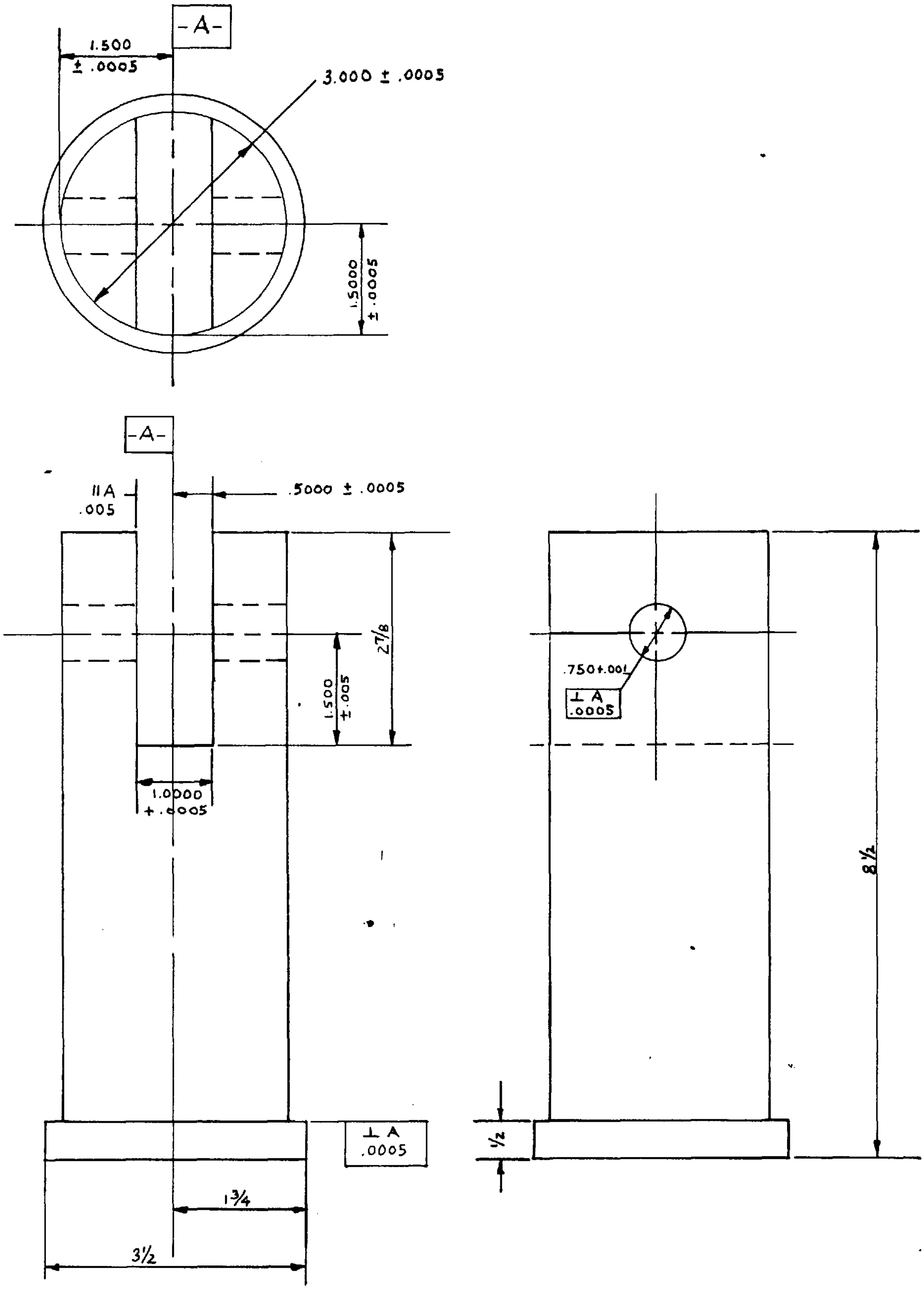
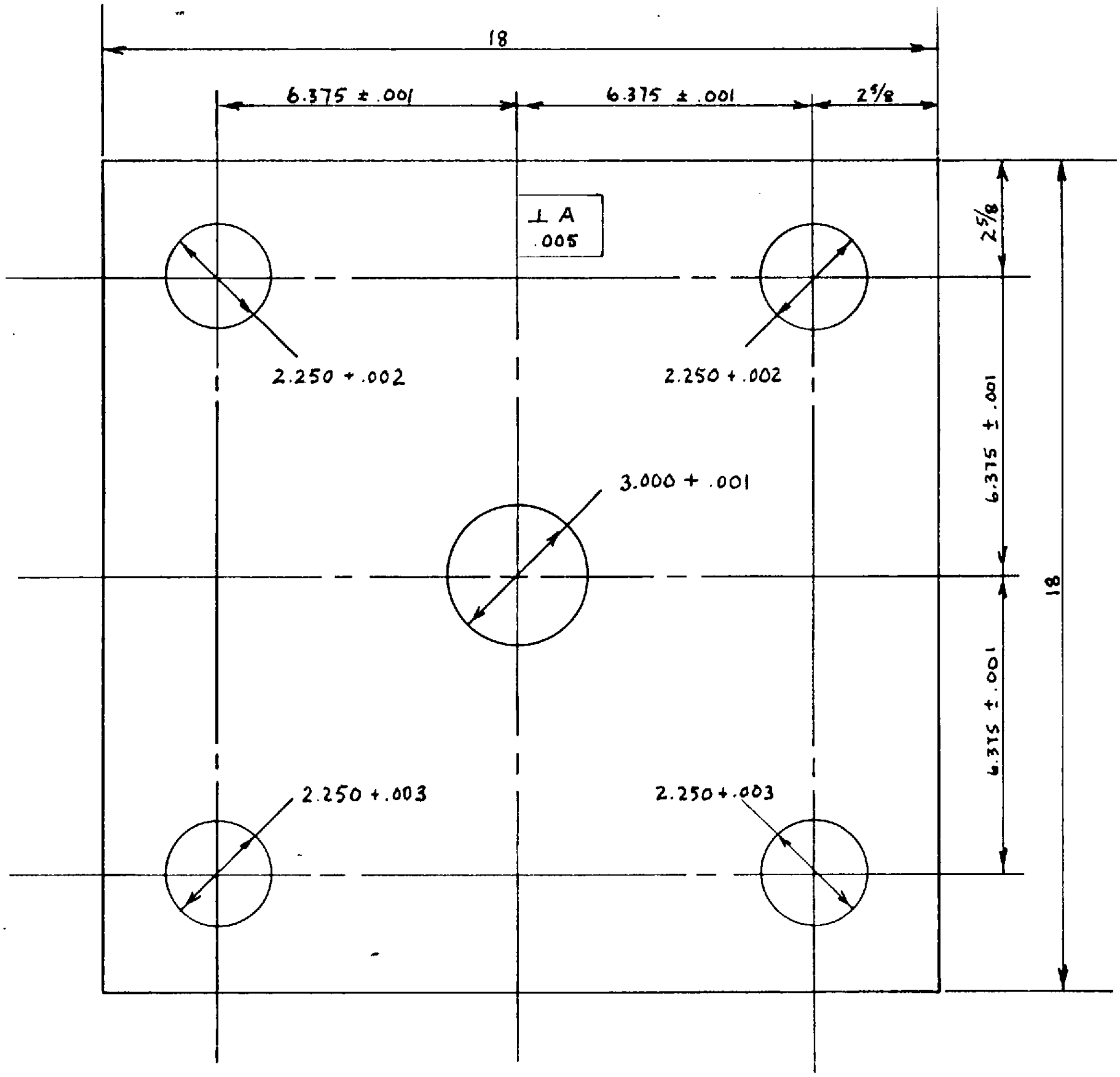


FIG. A4

half scale

BOTTOM PLATE



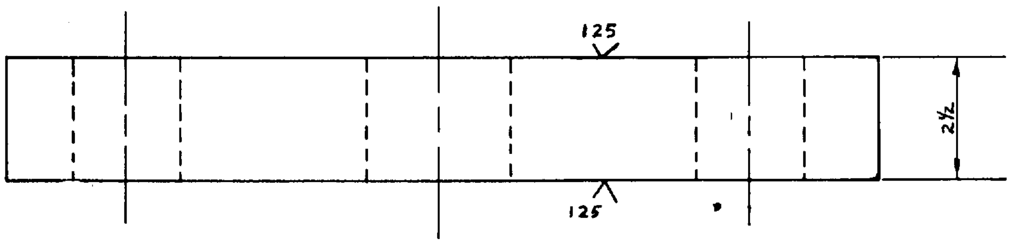


FIG. A5 quarter scale

COLUMN NUTS

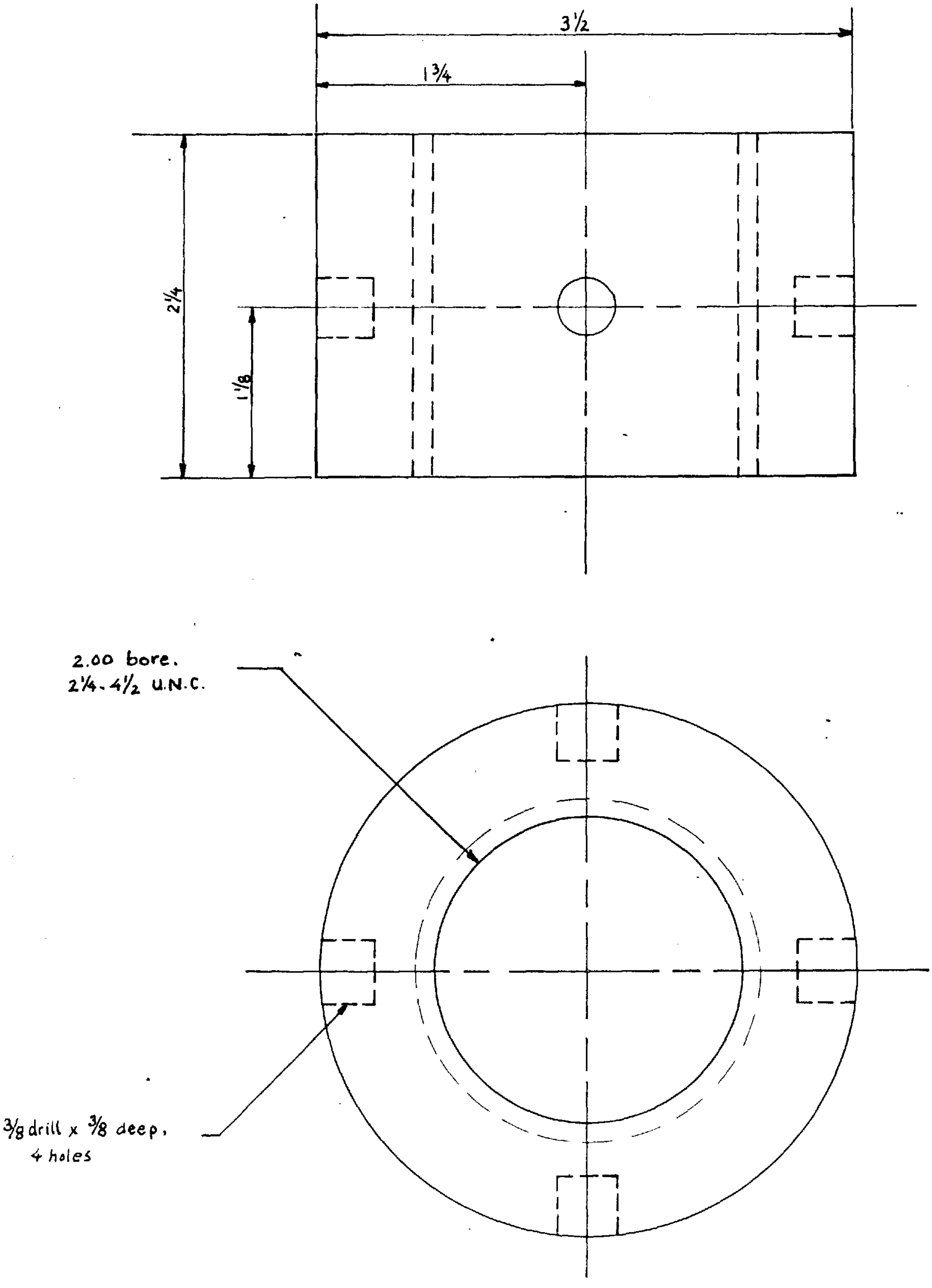


FIG. A6 full scale

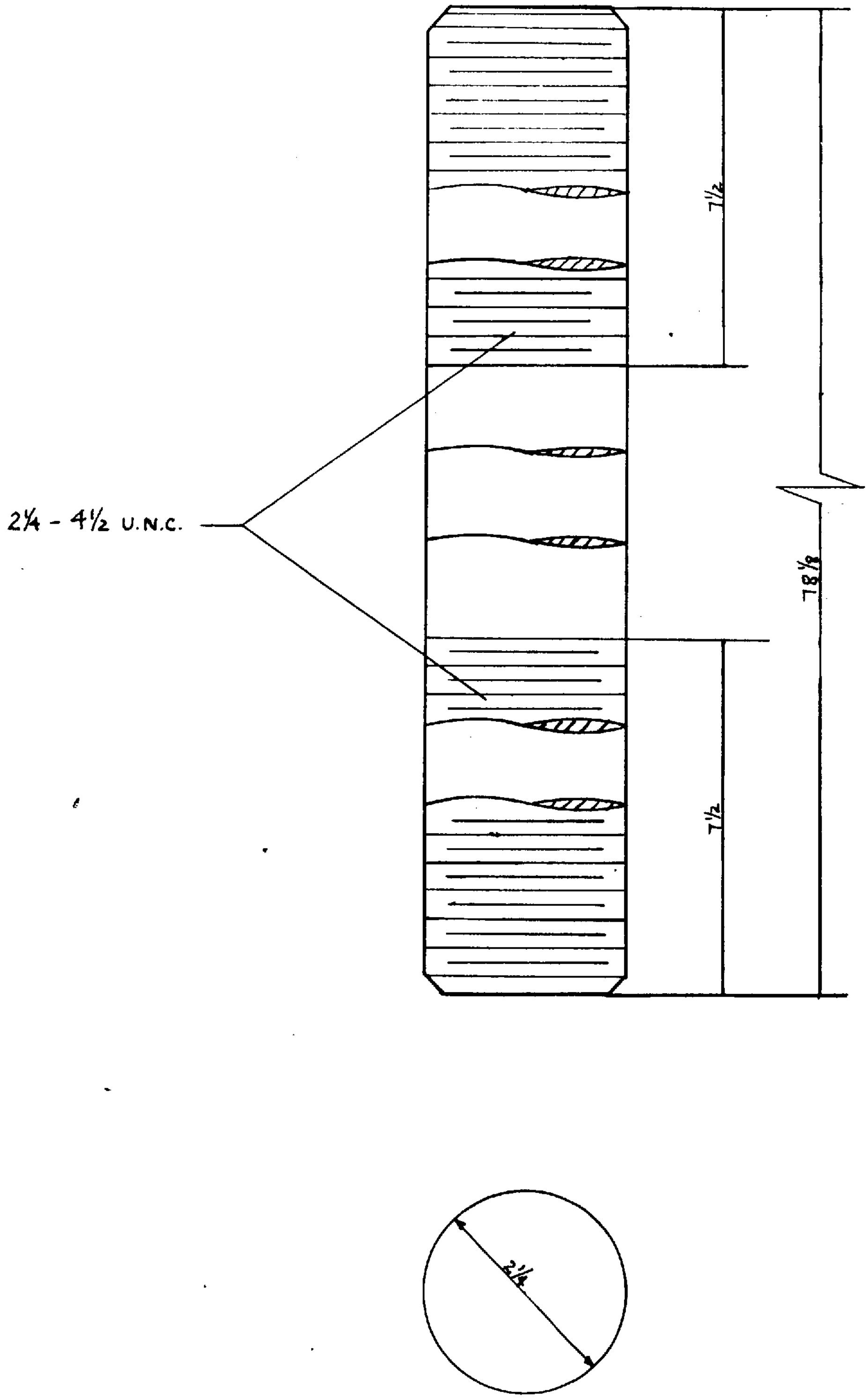


FIG. A7

half scale

APPENDIX B

RESULTS OF TEST SERIES 1 AND 2

Table B. Averaged results for test series 2 for all destructive and non-destructive tests.

Mix (see 6.2.5(b))	Density (lbs./ft.^{3})	E_o (x10^6 lb/in.^{2})	Q	U.P.V. (ft./sec.)	Poisson's ratio	Tension tests: flexure	Tension tests: indirect tension	Tension tests: direct tension	Compression tests: 4 ins. cube	Compression tests: 6 ins. cube	Compression tests: 4 ins. prism
1A	150.8	6.68	67	-	-	749	546	506	8850	8593	7060
2A	149.0	6.15	55	-	-	627	417	442	6753	6573	5463
3A	149.8	6.64	65	15,620	0.243	639	522	495	7883	7347	5987
4A	149.0	5.88	52	14,923	0.258	518	350	365	4517	4187	3487
5A	148.9	6.12	59	-	-	534	403	411	5270	5304	4040
6A	148.7	5.72	58	14,403	0.217	444	319	340	4007	3893	2870
1B	152.6	7.21	109	-	-	815	575	562	10,800	10,183	8763
2B	150.5	6.54	94	-	-	628	446	453	7143	7210	5793
3B	151.5	6.85	93	16,037	0.259	645	503	515	8303	7523	6153
4B	149.7	6.23	88	15,503	0.267	574	396	429	5627	5427	4163
5B	151.4	6.69	87	-	-	602	428	450	5967	5540	4450
6B	150.1	6.35	79	-	-	546	404	394	4707	4653	3587
1C	147.7	6.47	88	15,643	0.246	595	437	415	7133	6897	5253
2C	146.5	6.07	98	-	-	533	376	399	5793	5833	4600
3C	147.3	6.44	112	-	-	569	429	445	6743	6150	5083
4C	146.4	5.83	88	-	-	437	348	327	4330	4387	3357
5C	146.3	5.90	56	-	-	441	329	352	4430	4200	3343
6C	146.6	5.98	83	-	-	431	316	318	4263	3797	3177
1D	105.2	2.66	65	11,155	0.230	440	266	362	3203	3300	3200
2D	104.4	2.58	68	-	-	433	273	348	3290	3253	2993
3D	107.2	2.66	46	11,960	0.255	392	253	300	2970	2900	2563
4D	105.2	2.39	42	11,726	0.286	358	250	302	2493	2653	2413
5D	100.8	2.30	62	-	-	332	218	337	2250	2257	2283
6D	101.2	2.41	60	-	-	317	223	249	2203	2253	2143
1E	137.6	4.87	89	13,773	0.224	748	512	601	7043	7327	6797
2E	137.7	-	-	13,416	-	-	457	609	6380	6010	5887
3E	136.8	4.56	78	13,400	0.236	625	379	489	5197	5067	4863
4E	137.0	4.27	78	13,576	0.271	569	332	449	4187	4387	3930
5E	136.5	4.42	56	13,323	0.242	495	315	422	4040	4073	3790
6E	136.3	4.09	71	-	-	498	271	419	3390	3373	3117
Col. 1	2	3	4	5	6	7	8	9	10	11	12

Notes: Each of the results tabulated in Table B is the average value obtained from three specimens.

The missing ultrasonic pulse velocity and Poisson's ratio values in column 5 and 6 respectively, are due to an inoperational period of the Cawkell U.C.T.2 tester (see 6.6.3).

mix designation	Compression				Tension					
	4 x 4 x 12" prism	4 ins. cube	equiv. cube		direct tension	flexural tension	indirect tension (6" cylinder)	indirect tension (4" cube)	equiv. ind. tension	
			direct tension	flexural tension					direct tension	flexural tension
Test series 1 - 28 days:										
A	7160	x	7020	6940	x	966	606	x	x	x
	6940	x	6820	6830	x	953	616	x	x	x
	6750	x	7670	7240	x	947	628	x	x	x
B	7230	x	6360	6800	x	817	525	x	670	727
	7160	x	6520	6660	x	812	567	x	683	778
	6725	x	6520	6370	x	796	607	x	687	710
C	6800	x	6820	6900	x	873	601	x	703	710
	6500	x	6800	6940	x	702	570	x	678	702
	6610	x	6600	6800	x	861	622	x	663	675
D	6940	x	7310	7550	x	922	567	x	766	787
	6720	x	6950	7380	x	940	567	x	783	745
	6950	x	7310	7400	x	884	567	x	695	735
E	6870	x	7380	7450	x	878	606	x	765	812
	7380	x	7090	7820	x	1005	580	x	687	773
	7000	x	7240	7230	x	912	552	x	747	x
F	6440	7025	6800	7300	552	884	539	651	662	699
	6440	6800	6800	7380	570	928	539	574	674	703
	6440	7680	6800	7450	537	934	562	x	666	686
G	6570	7390	6870	7450	552	890	512	x	608	645
	6500	7460	7090	7300	546	900	525	x	612	686
	6650	7160	6800	7160	564	895	525	x	591	715
H	6430	7080	6800	6940	564	890	525	686	620	670
	6280	7240	6660	7080	552	878	546	677	628	662
	6570	7080	6800	7080	562	895	552	698	653	666
J	6570	7380	6950	7080	606	925	518	638	690	678
	6500	7120	6950	6800	576	916	552	664	666	637
	6500	7340	6870	6800	588	834	525	668	575	583
Test series 1 - 7 days										
I	4460	4750	4680	4820	456	680	354	530	520	575
	4320	4680	4680	4750	438	691	381	508	614	553
	4390	4680	4680	4530	423	707	369	477	507	575
K	4820	5480	5190	5480	450	718	388	570	570	558
	4900	5190	5190	5050	446	707	416	565	575	572
	4750	5410	5340	5190	498	646	402	603	579	579
L	4970	5630	5340	5330	498	717	457	593	557	566
	4930	5480	5260	5260	486	702	450	584	537	532
	5050	5480	5340	5190	471	674	471	627	571	550
M	5340	5230	5340	5190	504	706	416	x	578	566
	4900	5410	4900	5150	480	767	416	x	545	558
	4940	5230	5190	5190	477	690	381	x	572	540

x - results not available.

Test series 2 - Results:

mix desig.	Tension			Compression		
	uni-axial tension	indirect tension	flexural tension	4 ins. cube	6 ins. cube	4 x 4 x 12" prism
1A	522	536	705	8740	8340	6920
	487	576	627	9010	8760	7200
	508	525	716	8800	8680	7060
2A	445	391	627	6730	6500	5340
	428	422	638	6710	6750	5560
	453	438	617	6820	6470	5490
3A	487	549	650	8100	7400	5900
	493	506	623	7560	7300	6050
	505	512	644	7990	7340	6010
4A	365	335	494	4590	4210	3570
	352	346	527	4820	4120	3390
	378	370	533	4140	4230	3500
5A	414	404	510	5070	5100	3950
	408	401	572	5370	5410	3990
	412	404	520	5370	5410	4180
6A	358	333	460	4070	3940	2860
	343	306	455	3960	3930	2820
	318	319	417	4000	3810	2930
1B	578	574	750	11,200	10,280	8820
	530	565	855	11,100	10,250	8720
	578	586	840	10,100	10,020	8750
2B	428	449	623	7280	7390	5800
	445	451	600	6980	7220	5720
	488	438	660	7170	7020	5860
3B	512	499	610	8430	7720	6150
	487	486	664	8580	7680	6240
	545	523	660	7900	7120	6070
4B	450	409	594	5700	5330	4080
	412	386	617	5630	5470	4350
	425	393	510	5550	5480	4060
5B	456	446	588	5800	5490	4520
	445	433	617	5980	5650	4410
	450	404	600	6120	5480	4420
6B	373	351	572	4500	4540	3580
	385	327	527	4840	4570	3660
	425	335	538	4780	4850	3520
1C	432	438	577	6980	6920	5290
	418	428	604	7140	6910	5220
	395	446	604	7280	6860	5250
2C	385	401	472	5840	5860	4520
	418	364	554	5840	5840	4670
	393	362	572	5700	5800	4610
3C	450	420	572	6720	6140	5080
	430	438	567	6710	6050	5120
	455	428	567	6800	6260	5050

continued on next page.............

4C	325	356	467	4450	4210	3140
	318	364	400	4230	4660	3360
	337	325	444	4310	4290	3570
5C	360	325	395	4450	4210	3280
	352	333	467	4320	4230	3360
	345	328	462	4520	4160	3390
6C	318	306	423	4220	3680	3070
	325	317	444	4270	3890	3320
	313	325	425	4300	3820	3140
1D	365	244	410	3290	3210	3180
	363	283	467	3100	3300	3180
	357	272	444	3220	3390	3240
2D	365	248	460	3420	3220	2690
	327	288	422	3160	3260	3150
	343	282	417	3290	3270	3140
3D	293	254	397	3150	2920	2640
	293	248	410	2900	2830	2410
	315	258	368	2860	2950	2640
4D	295	252	347	2490	2670	2350
	313	248	360	2640	2690	2590
	297	250	367	2350	2600	2300
5D	212	210	328	2220	2200	2340
	230	222	350	2350	2340	2350
	270	222	317	2180	2230	2160
6D	348	235	287	2170	2200	2160
	248	206	317	2220	2340	2160
	252	229	328	2220	2220	2110
1E	549	523	805	6980	7500	6870
	640	512	764	7100	7350	6850
	615	502	674	7050	7130	6640
2E	610	473	-	6430	6160	5960
	646	444	-	6270	5900	5820
	573	454	-	6440	5970	5880
3E	490	370	622	5260	4920	4890
	505	372	622	5040	5030	4770
	472	394	632	5290	5250	4750
4E	442	346	567	4160	4300	4000
	450	343	572	4220	4230	4060
	455	306	567	4180	4630	3720
5E	418	309	483	4080	3940	3940
	415	319	467	4060	4160	3570
	433	317	535	3980	4120	3860
6E	408	269	517	3390	3400	3070
	420	272	488	3420	3440	3210
	428	272	488	3360	3280	3070
Col.1	2	3	4	5	6	7

APPENDIX C

RESULTS OF TEST SERIES 3

Cumulative Strain Readings — 1C - 7 day

Uni-axial Compression

Specimen 1			Specimen 2		
Stress (p.s.i.)	Strain (microstrain) long.	lat.	Stress	Strain (microstrain) long.	lat.
0	0	0	0	0	0
213	30	9	213	30	8
491	79	20	491	77	15
756	127	27	756	125	23
1030	177	38	1030	175	32
1310	224	46	1310	222	39
2000	349	73	2000	346	62
2540	459	94	2680	480	89
2640	483	104	3020	547	102
2810	521	112	3360	625	122
3360	618	116	3700	720	145
4040	790	218	4040	840	178
4370	898	293	4370	950	218
4710	1010	366	4570	1070	269
4980	1132	503	4710	1212	384
5110	1175	795	5110	1670	524

Uni-axial Tension

Stress	long.	Stress	long.
0	0	0.	0
7	2	7	3
38	6	38	8
69	10	69	12
100	16	100	18
131	20	131	24
163	26	163	29
190	30	190	34
220	36	220	40
250	42	250	45
280	48	280	50
310	52	310	58
340	60	340	63
370	67	355	67
385	72	370	70
400	75	385	74
-	-	400	80
-	-	415	90

The lateral gauge was inoperative for both the above specimens.

Cumulative Strain Readings   1C - 28 day.

Specimen 1			Specimen 2		
Stress (p.s.i.)	Strain (microstrain) long.	lat.	Stress (p.s.i.)	Strain (microstrain) long.	lat.
Uni-axial Compression:					
0	0	0	0	0	0
352	58	15	352	61	12
756	125	30	756	127	25
1180	193	44	1180	199	37
1560	255	55	1560	263	49
2000	328	64	2000	335	61
2680	438	90	2680	455	84
3360	552	116	3360	576	112
4040	676	148	4040	705	143
4710	817	186	4710	840	186
5050	887	-	5400	999	254
5400	986	292	5650	1070	300
5450	1028	346	5790	1123	366
5650	1070	384	5920	1180	410
5790	1120	437	6080	1240	490
5920	1210	505	6260	1325	637
6000	1368	795	6300	1375	795
			6350	1890	1850
Uni-axial Tension:					
0	0	0	0	0	0
7	2	.5	7	2	1
38	6	1.5	38	6	1.5
69	10	2.5	69	11	2
100	14	3	100	15	3
131	19	4	131	19	4
163	24	4.5	163	24	4.5
190	28	5.5	190	28	5
220	32	6	220	33	6
250	39	6.5	250	38	6.5
280	44	7.5	280	42	7.5
310	53	8	310	48	8
325	56	-	325	49	-
340	60	9	340	51	9.5
370	71	10	355	54	-
382	77	-	370	58	10
388	81	-	385	62	-
394	85	-	400	64	11.5
400	88	-	412	68	12
412	101	-	418	69	-
			421	72	-
			424	73	-
			430	74	13
			436	79	-
			442	81	-
			448	84	-

Cumulative Strain Readings MC - 7 day.

Specimen 1			Specimen 2		
Stress (p.s.i.)	Strain (microstrain)		Stress (p.s.i.)	Strain (microstrain)	
	long.	lat.		long.	lat.
Uni-axial Compression:					
0	0	0	0	0	0
213	35	7	213	35	9
491	86	20	491	86	17
756	136	33	756	137	26
1030	190	42	1030	193	36
1310	244	65	1310	245	46
2000	376	80	2000	377	73
2680	526	116	2680	520	100
3360	687	160	3360	688	146
3700	788	200	4040	887	226
4040	897	254	4300	975	306
4300	1008	332	4570	1150	396
4440	1092	425	-	-	-
4570	1165	477	-	-	-
Uni-axial Tension:					
0	0	0	0	0	0
7	2	1.5	7	3	1
38	6	2.5	38	9	1.5
69	12	3.5	69	14	2.5
100	16	4.5	100	19	3
131	22	5	131	25	4
163	28	5.5	163	30	4.5
190	33	7	190	36	5.5
220	38	7.5	220	42	6
250	44	8.5	250	47	6.5
280	50	9	280	53	7
310	57	10	310	61	8
334	64	10.5	340	70	9
340	68	-	370	79	11.5
370	78	12.5	-	-	-

Cumulative Strain Readings MC - 28 days

Specimen 1			Specimen 2		
Stress (p.s.i.)	Strain (microstrain) long.	lat.	Stress (p.s.i.)	Strain (microstrain) long.	lat.
Uni-axial Compression:					
0	0	0	0	0	0
352	42	12	352	40	12
756	105	24	756	104	25
1180	172	34	1180	170	34
1560	232	46	1560	234	46
2000	305	59	2000	304	59
2390	383	75	2390	360	70
2810	440	89	2810	440	84
3220	511	107	3220	511	98
3620	583	126	3620	583	114
4040	660	145	4040	665	134
4440	736	168	4440	738	152
4850	826	199	4850	830	186
5250	910	240	5250	920	224
5650	1015	300	5400	958	248
5860	1078	348	5520	992	268
6000	1117	384	5650	1020	299
6130	1152	404	5860	1096	366
6260	1190	429	5920	1132	386
6340	1250	505	6080	1192	458
6410	1303	656	6260	1263	534
-	-	-	6340	1368	762
Uni-axial Tension:					
0	0	0	0	0	0
38	6	1	38	6	1
100	15	2	100	16	2.5
163	26	3	163	25	3.5
190	30	4	190	30	4
220	36	4.5	220	35	4.5
250	41	5	250	40	5.5
280	46	5.5	280	46	6
310	52	-	310	52	7
325	56	6	340	59	7.5
340	59	-	-	-	-
355	62	6.5			
364	64	6.5			
370	65	6.5			
376	67	6.5			
382	68	6.5			
388	70	6.5			
394	71	6.5			
400	74	6			
412	77	-			
424	84	-			
427	87	-			

Cumulative Strain Readings 2C - 7 day.

Specimen 1 Stress (p.s.i.)	Specimen 1 Strain (microstrain) long.	Specimen 1 Strain (microstrain) lat.	Specimen 2 Stress (p.s.i.)	Specimen 2 Strain (microstrain) long.	Specimen 2 Strain (microstrain) lat.
Uni-axial Compression:					
0	0	0	0	0	0
213	28	8	213	34	8
491	79	19	491	89	17
756	130	30	756	140	32
1030	184	40	1030	196	47
1310	236	51	1310	248	60
2000	360	82	1630	312	75
2320	603	140	2000	388	94
3360	718	183	2330	462	112
3700	822	259	2680	550	138
3910	997	448	3020	616	173
4040	1690	792	3360	773	226
			3560	840	284
			3700	932	348
			3780	1015	396
			3910	1282	528

Uni-axial Tension:

0	0
7	4
38	9
69	14
100	20
131	26
163	32
190	38
220	44
250	50
280	57
295	62
310	67
325	72
340	77
355	92

Cumulative Strain Readings

2C - 28 day.

Specimen 1			Specimen 2		
Stress (p.s.i.)	Strain (microstrain) long.	lat.	Stress (p.s.i.)	Strain (microstrain) long.	lat.
Uni-axial Compression:					
0	0	0	0	0	0
352	58	13	352	64	12
756	126	30	756	135	27
1180	196	44	1180	209	41
1560	260	56	1560	276	56
2000	332	72	2000	356	70
2680	454	102	2390	432	89
3360	577	136	2810	512	107
4040	717	191	3220	592	128
4370	802	226	3630	681	152
4710	910	277	4040	786	212
4850	961	332	4300	854	254
4980	1040	373	4440	898	274
			4570	950	316
			4710	995	350
			4850	1055	394
			4980	1118	432
			5120	1190	495
			5250	1312	635
Uni-axial Tension:					
0	0	0	0	0	0
7	1	-	7	2	0
38	6	1.5	38	7	1
69	11	2	69	11	1.5
100	16	3	100	17	2
133	21	4	131	22	3
163	27	4.5	163	28	3.5
190	35	5.5	190	33	4
220	40	6	220	38	4.5
250	45	7	250	44	5.5
280	51	8	280	52	6
310	59	9	295	56	-
340	63	-	310	59	6.5
			322	63	7.5
			328	65	-
			334	67	7.5
			340	69	-
			346	71	-
			352	74	-
			358	77	8.5
			364	78	-
			370	82	9
			376	88	-
			382	97	-

Cumulative Strain Readings 3C - 7 day.

Uni-axial Compression:

Specimen 1 Stress (p.s.i.)	Specimen 1 Strain (microstrain) long.	Specimen 1 Strain (microstrain) lat.	Specimen 2 Stress (p.s.i.)	Specimen 2 Strain (microstrain) long.	Specimen 2 Strain (microstrain) lat.
0	0	0	0	0	0
213	28	8	213	27	7
491	69	15	491	75	16
756	113	23	756	123	25
1030	160	31	1030	175	35
1310	206	40	1310	226	42
1630	267	44	1560	277	49
2000	328	63	1850	333	59
2330	390	73	2130	394	68
2680	462	86	2390	457	75
3020	538	104	2680	520	89
3360	636	128	2960	594	107
3700	764	160	3220	673	131
			3360	716	145
			3490	761	160
			3630	808	186
			3780	879	213
			3910	972	259
			4040	1140	366

Uni-axial Tension:

Specimen 1 Stress (p.s.i.)	Specimen 1 Strain (microstrain) long.	Specimen 1 Strain (microstrain) lat.	Specimen 2 Stress (p.s.i.)	Specimen 2 Strain (microstrain) long.	Specimen 2 Strain (microstrain) lat.
0	0	0	0	0	0
7	2	0	7	2	-
38	7	1	38	8	1.5
69	12	2	69	13	2
100	17	2.5	100	18	3
131	22	3	131	24	3.5
163	27	4	163	30	4.5
190	32	4.5	190	34	5.5
220	37	5.5	220	40	6
235	40	-	235	42	-
250	43	6	250	46	7
265	46	-	265	48	-
280	50	6.5	280	51	8
295	53	7	295	55	-
304	56	-	310	58	9
310	57	7.5	322	61	-
325	62	-	328	62	9.5
334	68	-	337	64	-
337	75	-	340	65	-
			349	68	-
			355	70	10
			364	74	11
			370	-	-

Cumulative Strain Readings 3C - 28 day.

Specimen 1 Stress (p.s.i.)	Specimen 1 Strain (microstrain) long.	Specimen 1 Strain (microstrain) lat.	Specimen 2 Stress (p.s.i.)	Specimen 2 Strain (microstrain) long.	Specimen 2 Strain (microstrain) lat.
Uni-axial Compression:					
0	0	0	0	0	0
352	55	12	352	54	11
756	120	25	756	118	22
1180	188	37	1180	187	34
1560	251	48	1560	249	45
2000	326	61	2000	322	59
2680	444	82	2390	390	71
3360	570	110	2810	463	87
4040	701	150	3220	540	105
4710	872	367	3630	627	126
4850	923	457	4040	700	148
4980	1018	635	4440	794	186
			4710	870	234
			4850	915	262
			4980	955	292
			5120	1010	340
			5250	1100	430
			5400	1190	755
Uni-axial Tension:					
0	0	0	0	0	0
7	2	0	7	2	0
38	5	1	38	7	1
69	10	1.5	69	12	2
100	15	2	100	18	2.5
131	19	2.5	131	23	3
163	24	3	163	28	3.5
190	29	3.5	190	34	4
220	33	4	220	39	4.5
250	39	4.5	250	45	5.5
280	45	5	280	52	6.5
295	48	-	295	55	-
310	51	5.5	310	60	7.5
340	59	6.5	325	64	-
355	63	-	340	71	7.5
370	69	7	355	78	-
376	75	8	370	84	8.5
382	81	-	302	91	9
388	86	-	388	96	-
			394	103	9.5
			400	116	-

Cumulative Strain Readings NC - 7 day.

Specimen 1 Stress (p.s.i.)	Specimen 1 Strain (microstrain) long.	Specimen 1 Strain (microstrain) lat.	Specimen 2 Stress (p.s.i.)	Specimen 2 Strain (microstrain) long.	Specimen 2 Strain (microstrain) lat.
Uni-axial Compression:					
0	0	0	0	0	0
491	86	18	491	79	22
1030	186	32	1030	181	39
1310	237	41	1310	231	51
1560	288	50	1560	290	59
1850	343	57	1850	336	70
2130	405	71	2130	394	82
2390	468	82	2390	455	98
2680	535	-	2680	522	114
2960	623	114	2960	585	136
3220	715	136	3220	656	162
3490	815	178	3360	709	191
3630	915	226	3490	755	223
3780	1080	332	3630	815	268
3795	1254	508	3700	850	316
			3780	892	343
			3845	935	394
			3910	975	432
			3975	1070	558
Uni-axial Tension:					
0	0	0	0	0	0
7	3	1	7	5	1
38	9	2.5	38	10	2
69	15	3.5	69	14	2.5
100	21	4.5	100	20	3.5
131	27	5.5	131	25	4.5
163	33	6.5	163	31	5.5
175	35	8	178	33	6
190	38	-	190	36	6.5
205	40	-	205	38	-
220	43	-	220	41	7.5
235	46	-	235	44	8
250	49	-	250	46	8.5
262	52	-	265	50	9
265	53	-	274	52	9.5
274	55	-	280	54	10
280	57	-	286	55	10.5
286	58	-	295	58	-
295	61	-	304	59	11
304	-	-	310	60	-
			316	62	11.5
			322	64	-
			325	66	-
			328	67	12
			334	68	12.5
			340	70	-
			346	72	13
			352	74	-
			355	78	-
			358	80	-

Cumulative Strain Readings NC - 28 days

Specimen 1 Stress (p.s.i.)	Specimen 1 Strain (microstrain) long.	Specimen 1 Strain (microstrain) lat.	Specimen 2 Stress (p.s.i.)	Specimen 2 Strain (microstrain) long.	Specimen 2 Strain (microstrain) lat.
Uni-axial Compression:					
0	0	0	0	0	0
352	59	11	352	51	11
756	130	14	756	130	26
1180	206	38	1180	204	39
1560	278	50	1560	272	50
2000	354	65	2000	348	65
2680	485	96	2680	477	89
3360	627	132	3360	618	126
4040	790	186	4040	792	194
4370	890	234	4370	-	254
4710	1025	322	4710	1080	558
4980	1238	457	4780	1253	-
5120	1350	533			
Uni-axial Tension:					
0	0	0	0	0	0
7	1	0	38	5	1
38	6	1	69	9	2
69	11	2	100	15	2.5
100	16	2.5	131	19	3
131	21	3	163	25	3.5
163	26	4	190	29	4.5
190	30	4.5	220	34	5
220	36	5.5	250	40	5.5
250	43	6.5	280	46	6.5
280	49	7	310	52	7
289	51	-	325	56	-
295	54	-	334	64	-
310	57	8	340	67	9
325	61	-	346	69	-
340	66	9	352	70	-
			358	74	10
			364	76	-
			370	79	10.5
			376	81	-
			385	85	-
			388	91	-

Cumulative Strain Readings 4C - 7 day

Specimen 1			Specimen 2		
Stress (p.s.i.)	Strain (microstrain) long.	lat.	Stress (p.s.i.)	Strain (microstrain) long.	lat.
Uni-axial Compression:					
0	0	0	0	0	0
491	92	24	491	76	24
1030	205	51	1030	187	46
1310	264	65	1310	243	57
1560	314	79	1560	296	70
1850	366	97	1850	370	90
2130	412	121	2130	454	112
2390	485	166	-	483	-
2680	552	228	2390	537	140
2745	590	282	2540	608	-
2810	619	336	2680	700	186
2885	660	378	2745	780	207
2960	733	590	2810	895	234
			2885	1059	284
			3020	1608	610
Uni-axial Tension:					
0	0	0	0	0	0
7	3	.5			
38	10	2			
69	16	2.5			
100	22	3.5			
131	28	4			
163	35	4.5		(Specimen broken accidently)	
190	41	5.5			
205	44	-			
220	48	6.5			
235	52	7			
250	56	7.5			
265	60	8			
280	65	8.5			
286	68	-			
292	70	9			
298	80	-			

Cumulative Strain Readings 4C - 28 days

Specimen 1			Specimen 2		
Stress (p.s.i.)	Strain (microstrain) long.	lat.	Stress (p.s.i.)	Strain (microstrain) long.	lat.
Uni-axial Compression:					
0	0	0	0	0	0
352	62	13	213	36	9
756	135	26	491	85	21
1180	214	40	756	133	32
1560	282	51	1030	182	43
2000	342	68	1310	233	54
2680	403	98	1560	280	65
3360	505	152	1850	334	75
4040	733	381	2130	389	89
			2680	508	124
			-	609	161
			3490	723	204
			3780	837	270
			3910	950	376
			4040	1107	512
Uni-axial Tension:					
0	0	0	0	0	0
38	10	1	7	2	.5
69	15	2	38	7	1.5
100	21	2.5	69	12	2
131	27	3	100	18	3
163	34	4	131	24	4
190	39	4.5	163	30	4.5
220	45	5.5	190	37	5.5
250	50	6	220	42	6.5
280	60	6.5	250	49	7
298	67	-	280	58	8
304	69	7.5	292	64	-
310	71	8	298	66	9
316	75	-	304	69	-
322	76	-	310	71	10
328	80	-	316	77	-
334	84	-	322	87	-
340	87				
343	92				
346	107				

Cumulative Strain Readings 1E - 7 days

Specimen 1			Specimen 2		
Stress (p.s.i.)	Strain (microstrain) long.	lat.	Stress (p.s.i.)	Strain (microstrain) long.	lat.
Uni-axial Compression:					
0	0	0	0	0	0
491	110	32	491	106	25
1030	248	68	1030	245	59
1310	324	78	1310	316	77
1560	384	84	1560	385	91
1705	433	107	1850	462	110
1850	474	114	2130	543	131
2000	512	124	2390	625	152
2130	551	134	2680	714	178
2260	597	146	2960	792	204
2390	643	159	3220	893	240
-	666	165	3490	1005	274
2540	694	171	3630	1060	299
-	716	180	3780	1123	325
2680	740	186	3910	1192	350
2810	787	202	4040	1263	376
2885	810	210	4170	1337	407
2960	838	218	4300	1406	437
3020	868	223	4440	1490	470
3090	895	234	4570	1600	533
3220	945	254	4570	1737	610
-	973	262			
3360	1000	274			
-	1030	284			
3490	1060	300			
3630	1130	331			
-	1170	348			
3780	1208	366			
-	1243	370			
3910	1280	386			
-	1372	419			
4105	1435	445			
-	1525	457			
4300	1575	521			

Cumulative Strain Readings 1E - 7 days

Specimen 1			Specimen 2		
Stress (p.s.i.)	Strain (microstrain) long.	lat.	Stress (p.s.i.)	Strain (microstrain) long.	lat.
Uni-axial Tension:					
0	0	0	0	0	0
38	10	2	38	3	2.5
100	23	4.5	100	9	5
163	37	7.5	163	18	8
190	43	8	190	22	9.5
220	51	9.5	220	28	11
235	58	10.5	250	34	12
265	-	-	280	41	13.5
280	65	11.5	310	47	15
295	69	12	340	54	16.5
310	73	13	370	60	18
340	80	13.5	400	67	19
352	83	-	415	70	-
364	87	-			
370	88	15			
385	92	15.5			
400	95	16			
412	99	-			
418	101	-			
430	103	17.5			
445	107	18			
454	110	18.5			
460	112	18.5			
472	115	19			

Cumulative Strain Readings

1E - 28 days

Specimen 1			Specimen 2		
Stress (p.s.i.)	Strain (microstrain) long.	lat.	Stress (p.s.i.)	Strain (microstrain) long.	lat.

Uni-axial Compression:

Specimen 1 Stress (p.s.i.)	long.	lat.	Specimen 2 Stress (p.s.i.)	long.	lat.
0	0	0	0	0	0
352	73	17	213	48	10
756	165	37	491	109	24
1180	265	59	756	169	37
1560	354	79	1030	235	52
1850	419	95	1310	297	67
2130	487	107	2000	460	108
2390	560	126	2680	633	152
2680	630	142	3360	815	207
2960	700	160	4040	1040	286
3220	781	180	4710	1258	376
3490	853	202	5400	1568	508
3780	941	228	5790	1845	623
4040	1030	257	6080	-	750
4300	1115	284	6410	-	915
4570	1222	331			
4850	1320	376			
5120	1430	402			
5400	1560	457			
5520	1620	-			
5650	1800	513			
5790	1868	546			
5920	2055	595			
6080	-	635			
6260	-	685			
6320	-	737			
6450	-	787			
6590	-	875			
6720	-	990			
6850	-	1220			

Uni-axial Tension:

Specimen 1 Stress (p.s.i.)	long.	lat.	Specimen 2 Stress (p.s.i.)	long.	lat.
0	0	0	0	0	0
38	9	2	7	1	.5
69	16	3	38	6.	2
100	23	4	69	13	3
131	30	5	100	19	4
163	38	6	131	25	5
190	44	6.5	163	32	6
220	53	7.5	190	38	7.5
250	68	8	220	46	8.5
280	85	8	250	55	9.5
310	104	8	280	66	11.5
340	144	8	310	76	13
-	154	-	328	82	-
-	156	-	340	88	15
343	201	-	352	96	-
			355	103	-
			358	108	-
			367	138	-

Cumulative Strain Readings

ME - 7 days

Specimen 1			Specimen 2		
Stress (p.s.i.)	Strain (microstrain) long.	lat.	Stress (p.s.i.)	Strain (microstrain) long.	lat.
Uni-axial Compression:					
0	0	0	0	0	0
213	55	12	213	50	13
491	125	34	491	120	30
756	194	46	756	187	44
1030	266	61	1030	258	59
1310	341	79	1310	330	70
1560	408	94	1560	394	91
1850	487	112	1850	480	107
2130	568	131	2130	552	128
2390	651	152	2390	637	152
2680	734	175	2680	725	175
2960	825	206	2960	810	204
3220	923	245	3220	887	245
3490	1025	280	3490	1020	289
3630	1085	309	3780	1140	347
3780	1150	348	3910	1213	376
3910	1210	373	4040	1282	412
4040	1280	412	4170	1348	444
4170	1325	450	4300	1427	477
4300	1367	495	4440	1520	533
4440	1418	572	4570	1613	622
4570	1510	660	4710	1723	673
4710	1630	763	4850	1850	812
4850	1830	1018	4980	-	913
			5120	-	1116
			5250	-	2030

Cumulative Strain Readings ME - 7 days

Specimen 1			Specimen 2		
Stress (p.s.i.)	Strain (microstrain) long.	lat.	Stress (p.s.i.)	Strain (microstrain) long.	lat.
Uni-axial Tension:					
0	0	0	0	0	0
38	10	2.5	38	10	2
100	24	5.5	100	24	4.5
163	39	8.5	163	38	7
190	46	10	190	44	8
220	54	11.5	220	53	9
250	61	13	250	60	10
280	69	14.5	280	67	11.5
310	78	16	310	76	13
325	82	-	340	84	14
340	86	18	370	92	15.5
352	90	-	400	100	16.5
364	94	-	412	104	-
370	96	19.5	424	108	-
382	100	20	430	110	18
394	104	21	445	115	-
400	106	21.5	460	120	19.5
412	111	22	472	124	20
424	115	22.5	484	128	-
430	118	23	490	131	21
436	120	-	502	136	21.5
442	122	23.5	508	139	-
448	125	24	514	142	22
460	130	25	520	-	-
472	135	-			
484	140	-			
490	143	27			
505	152	28			

Cumulative Strain Readings ME - 28 days

Specimen 1			Specimen 2		
Stress (p.s.i.)	Strain (microstrain) long.	lat.	Stress (p.s.i.)	Strain (microstrain) long.	lat.
Uni-axial Compression:					
0	0	0	0	0	0
352	74	26	352	74	35
756	160	52	756	163	63
1180	251	79	1180	260	102
1560	334	105	1560	345	129
2000	438	136	2000	445	167
2390	533	167	2390	549	201
2810	613	194	2810	653	239
3220	724	242	3220	760	283
3700	830	292	3630	877	336
4040	959	366	4040	1010	398
4440	1088	412	4440	1155	472
4710	1190	457	4710	1260	525
4850	1240	482	4850	1327	560
4980	1297	508	4980	1388	590
5120	1352	534			
Uni-axial Tension:					
0	0	0	0	0	0
7	3	1	7	3	1
38	-	-	38	9	2.5
69	15	3.5	69	16	3.5
100	22	4.5	100	22	4.5
131	28	6	131	28	6
163	35	7.5	163	35	7
190	41	8.5	190	42	8
220	49	10	220	48	9.5
250	57	11.5	250	55	10.5
280	64	13	280	64	12.5
310	73	14.5	310	73	13.5
340	83	16	340	84	15
370	92	17.5	370	95	17
385	98	-	400	107	19
400	105	19	430	121	21
412	-	-	460	136	23
430	117	21.5	490	153	25
460	132	23	520	172	24.5
490	149	24.5	550	190	25.5
520	165	25	568	204	-
526	169	-	574	210	-
535	175	-	582	213	16
544	181	-	588	223	-
550	185	23.5			
562	195	-			
568	200	-			
576	207	14.5			
588	207	-			

Cumulative Strain Readings 2E - 7 days

Specimen 1			Specimen 2		
Stress (p.s.i.)	Strain (microstrain)		Stress (p.s.i.)	Strain (microstrain)	
	long.	lat.		long.	lat.
Uni-axial Compression:					
0	0	0	0	0	0
213	58	15	213	59	14
491	134	33	491	137	31
756	210	50	756	212	49
1030	290	68	1030	294	68
1310	369	86	1310	372	84
1560	449	102	1560	444	102
1850	540	126	-	622	145
2130	632	150	2390	736	172
2390	748	180	2680	858	210
2680	847	208	2960	960	253
2960	975	216	3220	1117	316
3220	1118	330	3360	1200	347
3360	1200	384	3490	1285	381
3490	1285	413	3630	1385	394
3630	1383	477	3780	1520	470
3780	1490	533	3910	1693	553
3910	1600	622	4040	1860	635
4040	1758	762	4090	-	889
Uni-axial Tension:					
0	0	0	0	0	0
7	4	1	7	4	.5
38	12	3	38	12	2
69	19	4.5	69	19	3
100	28	6	100	27	4.5
131	36	7.5	131	35	6
163	44	9.5	163	44	7.5
190	52	11	190	51	8.5
220	60	12.5	220	59	9.5
250	70	14	250	73	11.5
280	78	15.5	280	78	12.5
310	90	17.5	310	89	14
328	98	-	340	101	16
340	103	19.5	370	111	16.5
355	109	-	400	124	19
370	114	21.5	430	142	21
385	-	-	442	149	-
400	129	23.5			
415	139	25			
430	146	26			
442	153	26.5			
460	163	25			
472	174	-			

Cumulative Strain Readings 2E - 28 days

Specimen 1			Specimen 2		
Stress (p.s.i.)	Strain (microstrain) long.	lat.	Stress (p.s.i.)	Strain (microstrain) long.	lat.
Uni-axial Compression:					
0	0	0	0	0	0
352	91	25	352	87	25
756	191	52	756	187	52
1180	296	80	1180	291	78
1560	396	107	1560	387	102
2000	517	143	2000	503	133
2390	618	176	2390	609	160
2810	748	226	2810	722	200
3220	875	280	3220	850	240
3630	1019	358	3630	975	290
4040	1205	432	4040	1132	348
4170	1263	462	4440	1290	406
4300	1327	496	4850	1495	508
4440	1408	541			
Uni-axial Tension:					
0	0	0	0	0	0
7	2	.5	7	3	.5
38	6	2.5	38	9	2
69	15	3.5	69	15	3.5
100	22	4.5	100	23	4.5
131	29	6	131	29	6
163	36	7	163	36	7
190	42	8	190	44	8
220	51	10	220	52	9.5
250	60	11.5	250	62	10.5
280	70	13	280	71	12.5
310	81	14.5	310	83	14
340	93	16	340	94	15
370	109	18.5	370	108	17.5
400	124	21	400	124	-
430	139	23	423	137	-
460	158	29.5	428	141	20.5
490	177	28	440	150	-
			446	154	21.5
			452	159	-
			457	164	22.5
			463	167	23
			468	172	-
			474	179	23.5
			485	187	-
			492	194	-
			497	200	24.5
			503	207	25

Cumulative Strain Readings

3E - 7 days

Specimen 1			Specimen 2		
Stress (p.s.i.)	Strain (microstrain) long.	lat.	Stress (p.s.i.)	Strain (microstrain) long.	lat.
Uni-axial Compression:					
0	0	0	0	0	0
213	57	18	213	54	14
491	133	37	491	131	32
756	206	55	756	206	50
1030	282	74	1030	282	69
1310	360	91	1310	364	89
1560	442	109	1560	452	112
1850	538	135	1850	540	138
2130	641	165	2130	641	165
2260	693	178	2390	758	199
2810	814	215	2680	885	245
3090	938	332	2960	1015	300
3220	1013	373	3220	1170	378
3360	1090	423	3490	1365	492
3490	1168	454	3630	1470	567
3630	1272	567	3780	1612	732
3780	1417	730	3910	1830	1133
3910	1683	1510			
Uni-axial Tension:					
0	0	0	0	0	0
7	3	.5	7	3	.5
38	11	2	38	13	2
69	19	3	69	19	3.5
100	27	5	100	28	4.5
131	35	6	131	36	6
163	44	7	163	45	7
190	55	8			
220	61	9			
250	69	10			
280	77	11			
295	82	-			
310	86	13			
322	89	-			
340	95	14			
362	100	-			
370	104	14.5			
385	109	-			
394	112	-			
400	115	16.5			
415	120	17			
430	125	-			

Cumulative Strain Readings   3E - 28 days

Specimen 1			Specimen 2		
Stress (p.s.i.)	Strain (microstrain) long.	lat.	Stress (p.s.i.)	Strain (microstrain) long.	lat.
Uni-axial Compression:					
0	0	0	0	0	0
352	89	25	213	55	21
756	189	51	491	117	43
1180	293	77	756	181	69
1560	394	105	1030	252	94
2000	505	136	1310	322	118
2810	750	220	1560	386	143
3220	876	269	2000	501	183
3630	1010	336	2390	616	248
4040	1175	391	2810	735	277
4300	1293	482	3360	865	351
4440	1365	528	3630	990	398
4570	1440	576	4040	1140	477
			4440	1315	592
Uni-axial Tension:					
0	0	0	0	0	0
7	3	.5	7	2	.5
38	9	2	38	9	2
69	16	3	69	15	3
100	23	3.5	100	22	4.5
131	30	4.5	131	29	6
163	37	6	163	36	7
190	45	7	190	43	8
220	53	8	220	51	9
250	62	9.5	250	59	10.5
280	73	10.5	280	68	11
310	86	11	310	79	13
340	100	13	340	92	14
370	115	14	370	106	16
400	133	15			
418	145	-			
424	153	-			
430	157	16.5			
442	166	-			
448	173	18			
454	180	-			
460	188	18.5			
466	204	-			

Cumulative Strain Readings NE - 7 days

Specimen 1			Specimen 2		
Stress (p.s.i.)	Strain (microstrain) long.	lat.	Stress (p.s.i.)	Strain (microstrain) long.	lat.
Uni-axial Compression:					
0	0	0	0	0	0
213	50	15	213	51	11
491	128	30	491	129	28
756	201	43	756	204	41
1030	284	59	1030	288	57
1310	365	77	1310	362	70
1560	433	93	2000	588	119
2000	577	126	2390	734	173
2390	750	164	2810	942	240
2810	925	212	3220	1177	366
3220	1162	306	3490	1455	469
3490	1405	398	3630	-	572
3780	1920	580			
Uni-axial Tension:					
0	0	0	0	0	0
7	4	.5	7	3	1.5
38	12	2.5	38	11	2.5
68	20	3	69	18	3.5
100	28	4.5	100	27	4.5
131	36	6	131	35	6
163	45	7	163	44	7
190	52	8	190	51	9
220	60	9.5	220	60	9.5
250	69	10.5	250	68	11
265	73	-	280	78	13
280	77	11.5	310	86	14
292	80	-	340	97	15
304	85	-	370	107	17
310	86	13	400	118	18.5
322	90	-	415	123	19.5
328	92	-	430	129	20.5
334	94	-			
340	95	14.5			
346	97	-			
352	99	15			
361	102	-			
364	104	-			
370	105	-			
373	106	-			
382	109	-			
394	112	-			
400	115	17			
418	121	-			

Cumulative Strain Readings NE - 28 days

Specimen 1			Specimen 2		
Stress (p.s.i.)	Strain (microstrain) long.	lat.	Stress (p.s.i.)	Strain (microstrain) long.	lat.
Uni-axial Compression:					
0	0	0	0	0	0
491	120	33	491	119	33
756	185	48	756	184	48
1030	254	65	1030	254	65
1310	324	80	1310	326	82
1560	391	98	1560	394	99
2000	505	128	2000	510	130
2390	624	162	2390	628	163
2810	742	204	2810	743	207
3220	878	254	3220	873	254
3630	1028	319	3630	1030	324
4040	1210	400	4040	1213	407
4300	1350	477	4170	1256	444
4440	1438	520	4300	1330	487
4570	1548	583	4440	1410	534
4635	1610	634	4570	1500	622
Uni-axial Tension:					
0	0	0	0	0	0
7	3	.5	7	3	.5
38	10	2	69	14	3
69	16	3	100	21	4
100	24	4	131	27	5.5
131	31	5.5	160	34	6.5
160	37	6.5	190	39	7
190	44	7	220	47	8
220	51	8	250	54	9.5
250	60	9.5	280	63	10.5
280	71	10.5	310	72	12.5
310	81	12.5	340	82	14
340	93	13.5	364	101	-
364	103	-	370	111	16.5
370	105	15	376	-	-
382	111	-			
388	113	-			
400	120	16			
406	123	-			
412	126	-			
418	130	-			
424	153	-			

Cumulative Strain Readings

4E - 7 days

Specimen 1			Specimen 2		
Stress (p.s.i.)	Strain (microstrain)		Stress (p.s.i.)	Strain (microstrain)	
	long.	lat.		long.	lat.
Uni-axial Compression:					
0	0	0	0	0	0
213	58	13	213	56	13
491	142	30	-	-	-
756	224	48	756	219	50
1030	316	66	1030	306	66
1310	412	84	1310	396	87
1560	502	105	1560	489	117
1850	630	133	1850	610	138
2130	747	162	2130	731	173
2390	918	218	2390	887	224
2680	1125	316	2680	1103	316
2960	1340	457	2810	1198	372
3090	-	610	2960	1320	402
3220	-	1142	3090	1579	585
			3220	-	840
			3340	-	1400
Uni-axial Tension:					
0	0	0	0	0	0
7	3	1	7	3	1
38	10	2.5	38	11	2.5
69	21	3.5	69	18	3.5
100	30	4.5	100	27	4.5
131	40	6	131	36	6
160	49	7.5	160	44	7.5
190	58	9	190	53	9
210	67	9.5	220	62	10.5
250	78	11.5	250	69	11.5
280	88	13	280	80	13
298	95	-	310	90	14
310	100	14	340	101	16
340	114	16	370	113	17.5
370	127	17.5	400	125	18.5
382	133	18	412	130	-
394	140	-	424	137	-
400	143	19.5	430	139	-
			442	147	-

Cumulative Strain Readings

4E - 28 days

Specimen 1 Stress (p.s.i.)	Specimen 1 Strain (microstrain) long.	Specimen 1 Strain (microstrain) lat.	Specimen 2 Stress (p.s.i.)	Specimen 2 Strain (microstrain) long.	Specimen 2 Strain (microstrain) lat.
Uni-axial Compression:					
0	0	0	0	0	0
213	54	17	213	57	18
491	125	37	491	129	39
756	195	55	756	197	58
1030	268	73	1030	269	78
1310	344	95	1310	343	100
1560	418	116	1560	412	120
1850	498	140	1850	495	146
2130	575	157	2130	570	165
2390	664	186	2390	666	199
2680	765	229	2680	755	234
2960	858	280	2960	855	292
3220	970	337	3220	963	351
3490	1107	402	3490	1088	407
3780	1258	496	3780	1248	497
3910	1352	559	3910	1335	559
			4040	1428	623
			4170	1530	723
Uni-axial Tension:					
0	0	0	0	0	0
7	2	1	7	1	1
38	10	2.5	38	6	2.5
69	17	4	69	14	7
100	24	5.5	100	21	6
130	32	6.5	130	28	7
160	40	8	160	36	8
190	47	9.5	190	44	9.5
220	55	10.5	220	52	10.5
250	62	11.5	250	59	11
280	72	13.5	280	68	13.5
310	83	15	310	79	16
328	87	-	340	89	17
340	91	16.5	370	100	18.5
370	101	18.5	400	113	20.5
400	114	20	430	128	22
418	124	-	442	135	-
430	127	22	451	139	-
448	137	-	460	146	24.5
454	141	-	475	160	26.5
460	144	25	490	177	27.5
472	152	-	496	192	-
478	155	-			
484	159	-			
490	162	27			
496	169	-			
502	177	-			

Cumulative Strain Readings

1P - 7 days

Specimen 1			Specimen 2		
Stress (p.s.i.)	Strain (microstrain) long.	lat.	Stress (p.s.i.)	Strain (microstrain) long.	lat.
Uni-axial Compression:					
0	0	0	0	0	0
213	90	36	213	94	42
491	223	91	491	222	97
756	346	145	756	345	150
1030	480	210	1030	484	223
1310	614	-	1310	623	293
1560	764	344	1560	765	376
1850	960	410	1850	934	439
2000	1010	442	2000	1030	477
2260	1210	507	2390	1300	592
2540	1427	583	2680	1503	673
2680	1558	622	2960	1747	762
2755	1648	647	3220	2045	864
			3490	2255	978
			3630	2490	1054
Uni-axial Tension:					
0	0	0	0	0	0
38	18	4.5	7	5	1
69	30	8	38	18	4.5
100	44	11.5	69	30	8
131	56	15	100	43	11.5
160	failure		131	57	14.5
			160	72	18
			190	84	20.5
			208	failure	

Cumulative Strain Readings

2P - 7 days

Specimen 1			Specimen 2		
Stress (p.s.i.)	Strain (microstrain)		Stress (p.s.i.)	Strain (microstrain)	
	long.	lat.		long.	lat.
Uni-axial Compression:					
0	0	0	0	0	0
491	246	100	491	249	104
756	387	157	756	392	165
1030	537	226	1030	548	231
1310	710	296	1310	722	292
1560	875	372	1560	878	350
1850	1082	439	1850	1087	396
2130	1278	508	2130	1249	488
2390	1527	592	2390	1479	564
2680	1823	688	2680	1757	655
2810	2040	762	2810	1942	706
			2960	2270	762
Uni-axial Tension:					
0	0	0	0	0	0
7	5	2.5	7	6	2.5
38	20	6.5	38	20	6
69	34	10.5	69	34	10
100	49	14	100	49	14
131	64	18.5	130	65	18
160	78	22	160	80	22
190	92	25.5	190	95	25.5
202	98	-	220	111	29
208	102	-	250	126	33.5
238	115	-			
244	120	33			
250	124	34			
265	131	-			
280	140	38			

Cumulative Strain Readings 2P - 28 days

Specimen 1			Specimen 2		
Stress (p.s.i.)	Strain (microstrain) long.	lat.	Stress (p.s.i.)	Strain (microstrain) long.	lat.
Uni-axial Compression:					
0	0	0	0	0	0
213	91	35	213	100	29
491	203	80	491	219	66
756	321	126	356	340	107
1030	452	172	1030	472	148
1310	572	226	1310	594	188
1560	683	283	1560	717	229
1850	830	254	1850	860	289
2130	968	403	2130	982	344
2390	1112	464	2390	1128	391
2680	1267	523	2680	1295	452
2960	1440	597	2960	1450	503
3220	1610	668	3220	1608	563
3360	1697	707	3490	2010	634
Uni-axial Tension:					
0	0	0	0	0	0
7	5	2	7	5	2
38	17	5.5	38	17	5.5
69	29	9	69	29	9
100	41	12.5	100	42	13
131	54	16	130	55	16.5
160	65	18.5	160	67	6
190	77	23	190	83	3
208	84	-			
220	88	25.5			
250	99	-			

Cumulative Strain Readings

3P - 7 days

Specimen 1			Specimen 2		
Stress (p.s.i.)	Strain (microstrain) long.	lat.	Stress (p.s.i.)	Strain (microstrain) long.	lat.
Uni-axial Compression:					
0	0	0	0	0	0
213	108	59	213	144	57
491	335	130	491	325	124
756	519	205	756	516	199
1030	725	262	1030	727	280
1310	987	381	1310	974	384
1560	1240	457	1560	1218	442
1850	1552	558	1850	1558	546
2000	1770	614	2130	1918	660
			2260	2115	729
Uni-axial Tension:					
0	0	0	0	0	0
7	7	3.5	7	8	2
38	25	9	38	26	6
69	42	13	69	44	9.5
100	62	17.5	100	63	13
131	81	21.5	131	85	17.5
160	97	25	160	102	21
190	117	29	150	121	24.5
220	136	33	220	141	28.5
250	155	37.5	241	151	-

Cumulative Strain Readings

3P - 28 days

Specimen 1			Specimen 2		
Stress (p.s.i.)	Strain (microstrain)		Stress (p.s.i.)	Strain (microstrain)	
	long.	lat.		long.	lat.
Uni-axial Compression:					
0	0	0			
213	104	41			
491	242	96			
756	379	155			
1030	522	218			
1310	668	293			
1560	815	366			
1850	982	424			
2130	1149	487			
2390	1313	552			
2680	1510	619			
2960	1703	690			
3220	1920	787			
Uni-axial Tension:					
0	0	0	0	0	0
7	6	2.5	7	6	2.5
38	20	6	38	18	6.5
69	33	10.5	69	32	10.5
100	47	14	100	47	-
131	63	18.5			
160	75	22			
178	85	-			
190	90	27			
220	104	-			

<u>Cumulative Strain Readings</u> 4P - 7 days

Specimen 1			Specimen 2		
Stress (p.s.i.)	Strain (microstrain) long.	lat.	Stress (p.s.i.)	Strain (microstrain) long.	lat.
Uni-axial Compression:					
0	0	0	0	0	0
213	185	57	213	177	62
491	432	136	491	427	150
756	716	226	756	715	262
1030	1057	355	1030	1098	391
1310	1570	457	1310	1630	558
			1450	-	635
Uni-axial Tension:					
0	0	0			
7	11	2.5			
38	35	7			
69	61	11.5			
100	87	17			
131	113	21.5			
160	134	25.5			
190	162	30.5			
220	188	36			

Cumulative Strain Readings

4P - 28 days

Specimen 1			Specimen 2		
Stress (p.s.i.)	Strain (microstrain) long.	lat.	Stress (p.s.i.)	Strain (microstrain) long.	lat.
Uni-axial Compression:					
0	0	0			
73	43	22			
352	203	80			
491	290	110			
632	380	138			
756	456	167			
888	549	201			
1030	645	238			
1180	742	284			
1310	843	325			
1435	930	366			
1560	1015	386			
1705	1130	427			
1850	1245	467			
2000	1357	503			
2130	1468	543			
2260	1600	587			
2390	1745	635			
2540	1908	694			
2680	2060	742			
Uni-axial Tension:					
0	0	0			
7	6	2.5			
38	24	7			
69	40	11.5			
100	58	16.5			
131	74	21			
160	90	25.5			
190	106	30			

Cumulative Strain Readings 5P - 7 days

Specimen 1			Specimen 2		
Stress (p.s.i.)	Strain (microstrain) long.	lat.	Stress (p.s.i.)	Strain (microstrain) long.	lat.
Uni-axial Compression:					
0	0	0	0	0	0
213	218	74	213	215	73
352	362	117	352	361	116
491	519	165	491	520	162
632	695	216	632	712	218
756	882	284	756	918	286
888	1110	372	888	1147	366
1030	1430	457	1030	1467	432
1180	1810	558	1180	1550	525
1310	2345	762	1310	2000	635
			-	-	838
			-	-	1192
Uni-axial Tension:					
0	0	0	0	0	0
7	15	3.5	-	12	2.5
38	28	8	-	37	7
69	71	14	-	61	11.5
100	102	20	-	86	16.5
130	132	26	-	112	21
160	160	31.5	-	135	25.5
178	177	-	-	155	-
187	185	-			

Cumulative Strain Readings

5P - 28 days

Specimen 1			Specimen 2		
Stress (p.s.i.)	Strain (microstrain)		Stress (p.s.i.)	Strain (microstrain)	
	long.	lat.		long.	lat.
Uni-axial Compression:					
0	0	0			
73	47	20			
213	135	55			
352	226	89			
491	319	126			
632	412	158			
756	505	191			
888	600	231			
1030	714	280			
1180	827	324			
1310	935	376			
1435	1015	396			
1560	1130	436			
1705	1260	480			
1850	1400	532			
2000	1550	583			
2130	1620	639			
2260	1890	695			
2390	2090	782			
Uni-axial Tension:					
0	0	0			
7	8	3			
38	28	9			
69	46	14			
100	65	19			
130	85	25			
160	100	29			
190	120	36			
205	127	-			
214	137	40			

APPENDIX D

STATISTICAL TREATMENT OF EXPERIMENTAL DATA

STATISTICAL TREATMENT OF EXPERIMENTAL DATA

1. MEASUREMENT OF VARIABILITY

The strength of concrete test specimens can be assumed to fall into some pattern of the normal frequency distribution curve as illustrated in Fig.D1. When a great deal of care has been taken in the manufacture and testing of concrete test specimens and variation between individual results is small, the frequency curve will be tall and narrow as shown in Fig.D1, curve A. Conversely, the frequency curve will be low and elongated if great variation exists between the individual test results and their mean as shown in Fig.D1, curve B.

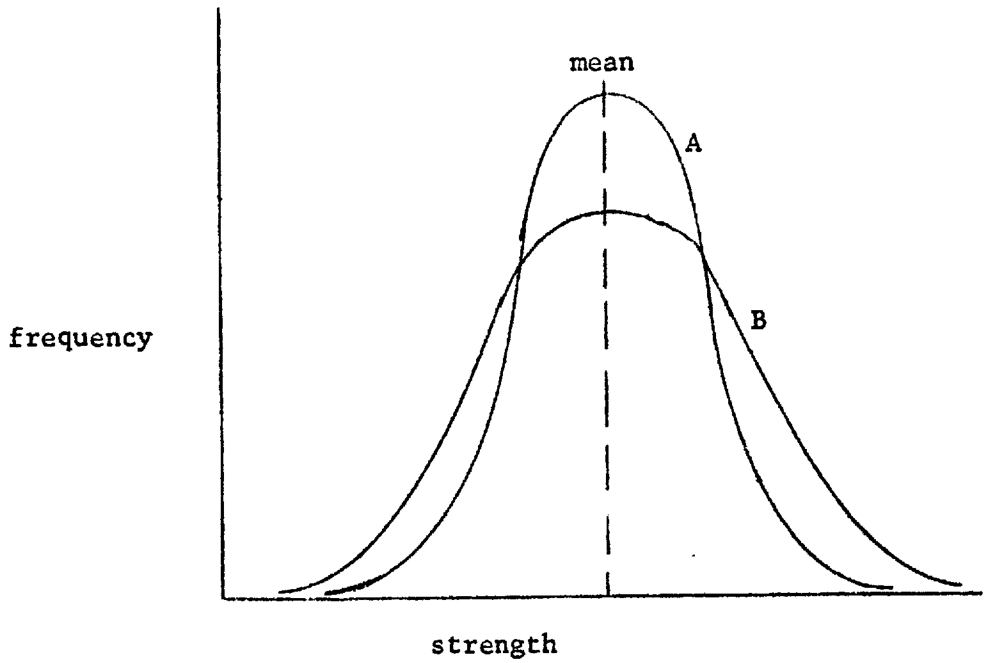


Fig. D1 Typical Frequency Curves.

A significant feature of a distribution is the dispersion or spread of values about the mean. The most generally recognised measure of dispersion is the root - mean - square deviation of the test results from their average. Referring to Fig.D1, the standard deviation, σ, is the radius of gyration of the area under the theoretical probability curve about the centre. The standard deviation is found by extracting the square root of the average of the squares of deviations of individual results from their average.

Average: $\bar{X}$ = the average of all tests.

$$\bar{X} = \frac{X_1 + X_2 + X_3 + \dots \dots X_n}{n} \qquad \dots\dots\dots\dots\dots\dots(1)$$

where X_1, X_2, X_3 . . . . X_n are the results of individual tests and n is the total number of specimens tested.

Standard Deviation - σ

$$\sigma = \sqrt{\frac{(X_1 - \bar{X})^2 + (X_2 - \bar{X})^2 \dots .(X_n - \bar{X})^2}{n}} \dots\dots\dots\dots(2)$$

Where n is less than 30, the standard deviation is a particular case, denoted by S.

$$S = \sqrt{\frac{(X_1 - \bar{X})^2 + (X_2 - \bar{X})^2 + \dots .(X_n - \bar{X})^2}{n - 1}} \qquad \dots\dots\dots(3)$$

This in tern reduces to,

$$S = \sqrt{\frac{n}{n-1}} \sqrt{\frac{\Sigma X^2}{n} - \bar{X}^2} \qquad \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots(3A)$$

It is seen that equation (3A) approaches equation (2) as $n \rightarrow \infty$.

Coefficient of Variation - V

In the interpretation of a value of σ, it is sometimes necessary to involve a comparison of the value with that of the mean, e.g. in comparing the variability of tension and strength tests which would have significantly di ferent values of σ for the same 'variability' since the means are different in general by a factor of 10. The standard deviation expressed as a percentage of the mean is called the coefficient of variation, denoted by V,

$$V = \frac{\sigma}{\bar{X}} \times 100 \text{ per cent}$$

Because V is calculated from all the test results obtained from all batches of concrete, it is called the between-batch coefficient of variation.

<u>Range</u> - R

is defined as the difference between the highest and lowest value of a set of data, i.e. from one batch. For small samples, the range is a relatively sensitive measure of general variability. However, it is not efficient in the statistical sense as it does not consider the intermediate results.

<u>Within-batch Coefficient of Variation</u> - V_1

The values of V_1 are estimated from the range of a set of data. The within-batch standard deviation and the associated coefficient of variation can be conveniently calculated as follows:

$$S_1 = \frac{1}{d_2} \bar{R}$$

$$V_1 = \frac{S_1}{\bar{X}} \times 100$$

where S_1 = within-test standard deviation.

$\frac{1}{d_2}$ = a constant depending on the number of test results obtained from each batch.

$\bar{R}$ = average range of batches of companion specimens.

$\bar{X}$ = average strength.

Tables of $1/d_2$ are given in A.S.T.M.,S.T.P., 15C[93].

For 2 and 3 specimens per sample, the values of $1/d_2$ are 0.89 and 0.59 respectively.

2. CORRELATION METHODS

A problem that arose frequently in this work was to determine the best form of representation of an apparent relation between two variables, e.g. $E_D - E_C$. The method used was a linear regression analysis.

The Correlation Coefficient:

To test for the significance of an apparent linear relationship, the correlation coefficient r is calculated[111],

$$r = \frac{\Sigma(x - \bar{x})(y - \bar{y})}{\sqrt{\Sigma(x - \bar{x})^2 \Sigma(y - \bar{y})^2}}$$

If $r = \pm 1$, a perfect linear relationship exists between the variables. Minus 1 indicates a perfectly linear relation with a negative slope and plus 1 indicates the reverse. If $r = 0$, no relationship exists between the variables. Tables are provided in most statistical texts[109,98] which allow a level of significance to be associated with r.

The Regression Equation:

The regression equation of y on x is obtained from:

$$y = a + b(x - \bar{x})$$

where $a = \frac{\Sigma y}{n}$

$$b = \frac{\Sigma(y - \bar{y})(x - \bar{x})}{\Sigma(x - \bar{x})^2}$$

$$\bar{x} = \frac{\Sigma x}{n}$$

Residual Variance about the Regression Line:

The regression line calculated above gives only the best estimate of the value of the quantity in question. It is desirable to have confidence limits for the regression line. The confidence limits chosen were the 95 per cent lines, i.e. 95 times out of a 100 the points on the graph will lie within the limit lines. A property of the limit lines chosen is that they are fixed at a distance 2 times the standard deviation or error (S_E). S_E may be calculated from[98]:

$$S_E = \sqrt{1 - r^2}\sqrt{\frac{\Sigma(y - \bar{y})^2}{n - 2}}$$

The regression equations and the corresponding limit lines have been calculated for various relations in this work. They are plotted on the various graphs.