UNIAXIAL TENSILE CREEP AND FAILURE OF CONCRETE

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by

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To My Parents

ABSTRACT

The work in this thesis consists of a study of the behaviour of concrete under uniaxial tension with regard to both strain development under external loading and short- and long-term failure.

A method was first developed for the accurate short-and long-term uniaxial tensile testing of concrete and has been applied throughout the investigation.

As a result of this work, a mechanism has been proposed within a framework of a seepage-plastic theory to describe the tensile creep and failure of concrete. A rheological model has also been suggested to describe this mechanism.

The tensile creep of air drying concrete was larger than that expected in compression but smaller when the concrete is stored sealed or under water. The creep of sealed and immersed concrete reached limiting values within two months while that of air drying concrete continued for a much longer time. In other respects tensile creep behaviour was similar to that in compression.

The creep of sealed concrete was about half that of immersed concrete and as low as 0.1 of that of air drying concrete.

A relationship has been obtained between ultimate strength and the period of sustained loading. This relationship is similar to that previously obtained for concrete in compression but the reduction of strength due to long-term loading is greater than that for compression and can be up to 50 per cent of the short-term strength.

This relationship is considerably influenced by the storage conditions of the specimen and the type of aggregate employed. The long-term strength of sealed concrete has been found to be up to 50 per cent higher than that of water-immersed concrete made with gravel aggregate while the short-term strength was about 15 to 20 per cent higher. This was not found to be so for concrete made with crushed granite aggregate.

A limiting positive ultimate strain of 70 to 90 microstrain due to external loading was observed for all gravel concrete tests, short-or long-term but higher values were measured on granite concrete.

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INTRODUCTION

Hydraulic cements as used for making concrete have been known for many generations. Their properties and performance when mixed with water is still not fully understood. The behaviour of concrete is even more complicated by the behaviour of aggregates it contains. When under stress cement paste and aggregate will have to work together and share the load. Steel is used also where necessary to improve structural efficiency by taking an important share of the load.

To understand and predict the performance of a structure constructed of plain, reinforced or prestressed concrete, it is very difficult, if not impossible, to consider it as a whole. The factors that could be at work at the same time are very numerous and the contribution of each could not be understood from tests on the whole structure. For this reason it has been the practice of most investigators to separate these factors and to study them in relative isolation. From such studies the ultimate aim is to combine the information and attempt to apply it for predicting the behaviour of the actual structure.

Creep of contrete is one of these factors, and investigators have tended to study it under the simplest of conditions and try to separate it from other factors.

A great number of investigators have studied the creep of concrete particularly because of its vital importance to the behaviour of prestressed concrete structures. Almost all the work has been concerned with the uniaxial compressive state of stress. Unfortunately no full agreement on an explanation of creep mechanism has yet been reached although a number of alternative theories have been proposed.

The tensile properties of concrete and especially those concerned with creep have been mostly overlooked. This could be due to the fact that in reinforced concrete the tensile strength of the concrete is of little importance and steel is introduced to take the tensile force. In addition, studies of tensile behaviour generally present more experimental difficulties compared with those of compressive behaviour.

Recently, however, more attention has been given to tensile properties and its role has been more appreciated. In all plain concrete structures, in structures where cracking is not permitted, in partial prestressed concrete and in road structures the tensile properties of concrete are of considerable interest. Moreover, the failure of concrete whether under tensile or compressive loading is essentially a tensile failure and an understanding of its tensile properties should lead to a better appreciation of its behaviour under all types of loading.

Uniaxial tension is the only simple type of test loading that imposes a uniform tensile stress over the section. A study of the tensile properties under such loading is necessary before moving into a complex type of loading, and once this is fully understood it should be possible to predict the behaviour under more complicated state of stress. However, the difficulty in applying uniaxial tensile stresses and the avoidance of eccentricity perhaps hindered many investigators in the past.

The present work is concerned with the development of an easy and accurate method of short-and long-term loading concrete specimens in uniaxial tension up to failure, investigation into the uniaxial tensile creep under sustained loading and a study of the short-and long-term uniaxial tensile strength of concrete.

The tensile behaviour of loaded specimens while immersed in water, sealed or air drying is investigated in detail.

Theories of creep are examined in connection with the experimental findings and a proposal for a mechanism to describe tensile creep is presented. An explanatory rheological model is used to define tensile behaviour. The proposed mechanism takes into account both the short-and long-term failure of concrete in uniaxial tension. The tensile stress-strain curve, cracking and failure have also been studied.

The review of the literature does not discuss all the considerable amount of work published on compressive creep but only includes that literature which has a bearing on the tensile behaviour of concrete.

CHAPTER I

BACKGROUND TO PRESENT WORK

I.1. THE STRUCTURE OF CONCRETE

Concrete is a multiphase material consisting of cement, aggregate (coarse and fine) and water. The cement and water react chemically to produce cement paste which provides the binding material. Aggregate could be considered as inert particles. The complexity of concrete is mainly due to the complexity of the product of hydration of cement paste.

It is neither the object nor the capacity of this chapter to discuss the chemistry of cement or the hydration product, but only a very brief summary of its composition and hydration which could be relevant to the present work is presented.

I.1.a. Composition of Portland Cement

Portland cement is the product of a mixture of limestone and clay heated gradually to 1400°C. and then left to cool. The clinker produced is powered with an added amount of gypsum ($CAO\ SO_4.2H_2O$). The particle sizes are of the order of 1-100 micron. There are four major compounds in Portland cement:

1) Tricalcium silicate, $3\ CAO.SIO_2$ or C_3S
2) Dicalcium silicate, $2\ CAO.SIO_2$ or C_2S
3) Tricalcium aluminate, $3\ CAO.Al_2O_3$ or C_3A
4) Tetracalium alumino ferrite, $4\ CAO.Al_2O_3.FE_2O_3$ or C_4AF

The percentages by weight are approximately 40, 30, 11 and 12 respectively. Among the other minor components is gypsum which is added to regulate the setting time of the cement paste.

I. 1.(b) Hydration of Portland Cement

When Portland cement is mixed with water chemical reactions start almost immediately and continue almost indefinately if free water is available. Hydration products are gradually deposited and the hydrated compounds gain strength at different rates.

The hydrated products of C_3S stiffen within a few hours and gain their major strength within a few weeks. The reactions in C_2S are slower; setting goes on for no definite time but the stiffness is reached within a few days. Its gain of strength is not appreciable before a couple of weeks, but after a year or so its nearly the same as that of C_3S. C_3A and C_4AF react quickly with water but their hydrated products produce little strength and the added gypsum, while seeming to regulate their setting time, appears to have little effect on the rate of hydration of the calcium silicates C_3S and C_2S.

The characteristics of Portland cement are changed by varying the compound compositions. Fineness is also important as it increases the specific area of the cement grains thus enlarging the area of contact with water which respectively increases the rate of hydration.

Cement paste, directly after mixing, consists of cement grains dispersed in water. Reactions between the two start almost immediately and hydration products are deposited in the water-filled spaces. Once the gap between the cement grains is bridged and relatively solid interconnection exists the volume and shape of cement paste remain constant.

As every 1 cc of cement produces 2.2 cc[1] of cement hydrates at full hydration a volume of water filled spaces of 1.2 cc must be available before full hydration can take place. This corresponds to a water-cement ratio of 0.38 by weight. However, it was found that a water vapour pressure of 0.80 or more within the concrete is necessary for the hydration process to continue[1].

The absolute volume of the hydration products is less than that of cement plus water, although its bulk volume is greater. The diminution in absolute volume is due to the drop in the specific volume of the chemically combined water (or water of hydration). This causes a demand of water to fill the spaces and unless the water is available a reduction in the water vapour pressure will occur. In sealed specimens an initial water-cement ratio of 0.50 is required if the water vapour pressure is to remain above 0.80 and the rate of hydration to equal that of concrete immersed in water.

The chemically-combined water or the water of hydration is equal to 0.23 of cement by weight (which is also termed as the non-evapourable water). The water that occupies the pores in the hydrated cement paste is called the gel water. This water is adsorbed and the large free binding energy available on the surface of the gel compresses it to a specific volume of 0.9[2]. The other type of water is the capillary water and this is the free water that occupies the volume of cement paste which has not been filled by the product of hydration and, being beyond the influence of the surface forces of the gel, it is under no stress at saturation. The boundry between the capillary water and gel water is defined in such a way that capillary water dries out at a relative humidity of between 50-100% and gel water at relative humidity between 0-50%.

Hardened cement paste looks under the microscope like an amorphous gel. This was shown by X-ray examination to be very finely crystalline. The complicated nature of hardened cement paste made most investigators carry out their work on hydrates having fewer compounds. They found that it most resembles the naturally-occuring mineral tobermonite and Brunauer and his colleages called the hydrates of C_3S and C_2S tobermorite gel.

The hydrated products of C_3S and C_2S form nearly 75% of the total weight of hardened cement paste and have the major responsibility for its physical and mechanical properties. The main important characteristics of these hydrates is their enormous specific surface area which mainly provides Portland cement paste with its cementing properties. The surface area of hardened cement paste was measured by Powers and Brownyard and their figure was 210 m^2/gm.[3]

In recent years electromicrographs[4,5,6,7] showed that the tobermonite gel consists of extremely thin crumpled sheets of fine needle-like crystals. The thickness of these sheets depends mainly on the amount of lime as an increase in this will reduce the proportion of thin layers. The average thickness of these layers varies between 10-100 A° and the average width of the spaces between these particles which form the gel pores is 15-20 A°.

I. 2. THE STRENGTH OF CONCRETE

The strength of concrete is considered to be its most important engineering property. Being related directly to the structure of hardened cement paste, strength provides us with a good picture of the general quality of our concrete. The factors influencing the strength of concrete are many but the water content and porosity are the two most influential. Aggregate content and quality seem to have little direct effect on the strength. Aggregate is generally considered as inert material of a stiffer and stronger nature than its surroundings.

Both the degree of hydration and porosity of the concrete depend on the initial water-cement ratio of the mix in fully compacted concrete. Although the water is necessary for the hydration process its amount must be limited as the higher the water-cement ratio the more porous the concrete. Some hydration must take place before any strength is produced. Full hydration could be achieved with an initial water-cement ratio of 0.38 (see page 9), but full hydration does not necessarily lead to maximum strength. Abrams[78] produced a compressive strength of 40,000 lb./in.^{2} in compression from a cement paste having a water-cement ratio of 0.08 where the majority of cement grains remained unhydrated (pressure had to be used in this particular case to consolidate the mix). The high strength may be attributed to the very thin layers of cement hydrates surrounding the unhydrated cement grains[2].

Formulae have been devised to define the strength of mortar and concrete in relation to its constituents. These include -

Feret's formula:[8]

$$S = K \left(\frac{c}{c+e+a} \right)^2$$

S is the strength of concrete, c, e, and a are the absolute volumes of cement, water and air voids respectively and K is a constant.

Powers equation was:

$$S = S_g . X^n$$

Where S_g is the characteristic strength of cement gel, n is a constant averaging about 3, and X is the gel space ratio. Both formulae show increase in strength with increase in the degree of hydration and decrease in water-cement ratio.

The exact mechanism by which cement gel gains its strength is not fully understood. Two different theories were originally put forward to explain hardening or gain of strength of the cement gel - the first by H. Le Chatelier[9] related the gain of strength to the interlocking between the crystals in the cement gel - while the second is a colloidal theory in which W. Michaelis[9] considered hardening to be due to the precipitation of a colloidal gel from a saturated solution (the gel is soft and fills the spaces between cement grains and the strength is produced by the hardening due to the inner attraction of water by cement grains).

Recent studies do not deviate very much from these two theories, and Powers[10] relates the source of strength to two kinds of cohesive bonds:

1) Forces of attraction between the solid surfaces of the gel particles separated by the small gaps (gel pores) (van der Waals type).
2) Chemical bonds where the particles are linked together by chemical reactions.

The reason for the high compressive and low tensile strength of concrete has not yet been fully explained. Brunauer[11] considered that the tensile strength is limited to overcoming the surface forces whereas to cause failure in compression the chemical bonds must also be broken.

The effect of aggregates on the strength of concrete may more likely be concentrated on the quality of bond with cement paste. This bond strength was reported by Alexander[12] to vary between 50-100% of the cement paste strength. Hsu and Slate[13] emphasise the influence of the moisture content of the specimens at the time of testing on the tensile bond strength of the paste aggregate. They found that the mortar-aggregate tensile bond strength varied from 33-67% of the tensile strength of the mortar (saw-cut aggregate surface was used).

The maximum size of aggregate in the mix was found to influence the strength of concrete. Reducing it increases the strength[14] especially when the water-cement ratio is low[15,16,17]. Alexander[12] quoted an increase of 50% of the 28 day compressive strength of concrete having a water-cement ratio of 0.35 when the maximum size is reduced from 6 in. to 3/8 in., whereas the corresponding increase is 15% when the water-cement ratio is 0.6.

The tensile strength is expected to be more sensitive to aggregate properties and quantity. This was supported by the work of Hughes and Chapman[14], whose results also indicated that increasing either the size or the roundness of the aggregate decreases the tensile strength.

The high sensitivity of the tensile strength of the concrete to the curing condition and the moisture condition at the time of testing has been reported by Alexander[12]. Any disturbance that may occur between the time of casting the concrete and the time of testing could affect its tensile strength.

Hsu and colleagues[13,18] showed how the volume changes of the concrete, whether due to hydration, temperature, swelling, shrinkage or external load could induce high stresses on the boundary between the cement and the aggregate. This may cause bond cracks, or cracks in cement paste, depending on the distance between the particles of aggregate. Hsu and Slate demonstrated this using idealised mathematical models, and supported it with microscopic X-ray examination.

The effects on the tensile strength of many of the factors discussed could differ depending on whether flexural, splitting or uniaxial strength is investigated. For instance, exposing a beam specimen to drying for some time before testing would be expected to show its maximum effect on the flexural strength where the failure initiates at the maximum stress fibre on the bottom of the beam and where also the tensile stress due to the differential shrinkage is maximum. Sealing the concrete specimen, which in some cases reduces the rate of hydration, may increase its tensile strength. This will be demonstrated in chapter VII.

The measured tensile strength of concrete is much smaller than that theoretically estimated from molecular cohesion and calculated from the surface energy of a solid. The large difference could be attributed to the high stress concentration at local points, under relatively low average stress over the whole section, caused by the presence of flaws. Cracks initiate at these points and propagate leading to failure.

In tests on concrete in compression internal cracking and microcracking have been detected by various means[13,19]. Using the ultra sonic pluse velosity technique Jones[19] found that the onset of cracking was detected at about 35% of the ultimate load when using cubes for his test, and at 65% when using bobbin shaped specimens.

He also found that concrete containing smooth gravel began to crack at a lower average compressive stresses than that containing coarser textured aggregate. This could perhaps be related to the weaker aggregate-cement bond of the smoother aggregate. The extension of this aspect of failure to tensile loading is discussed in Chapter VII.

I. 3. SHRINKAGE OF CONCRETE

The contraction of concrete due to the evaporation of some of its moisture is generally called drying shrinkage. But shrinkage occurs from the time the water is added and before the concrete has hardened, due to the absorption of water by the cement particles. This has little effect on the property of concrete. Volume changes also occur without any loss or gain of moisture from the atmosphere. This is caused by the process of hydration and is generally called autogenous volume change. Another form of volume change is carbonation which is described later.

The shrinkage does not take place uniformally over a section. The surface of the concrete dries out first followed by the interior, thus causing differential shrinkage stresses, (tensile on the outside and compressive in the inside of the specimen). As the shrinkage is a phenomenon of the cement gel, the aggregate, behaving elastically, opposes this shrinkage and acts as a restraining body causing further stresses to be set up. In reinforced concrete structures, steel will also act against these stresses and resist them, while undergoing compression and imposing tension in the concrete. When the structure is loaded, further stresses are imposed which may either act against or be added to the stresses caused by the shrinkage. In a prestressed concrete structure, shrinkage along with creep causes a relaxation of the cable stresses, and reduces the compressive stresses imposed on the concrete. Thus a knowledge of the shrinkage deformation is necessary before any attempt is made to evaluate these stresses.

The factors influencing the shrinkage are many and almost any change in the concrete composition, constituents, curing and surroundings will effect the shrinkage. These differ in magnitude and importance and generally one factor would interfere with the other. In general wet curing of concrete, humid atmosphere and low water cement ratio would reduce shrinkage.

I. 3. a. Autogeneous Shrinkage and Swelling

As has been discussed previously (see page 9), the absolute volume of the material in cement gel diminishes with the process of hydration and water is drawn from the capillary pores to fill the newly formed gel pores. This moisture adsorption by the gel pores walls would lead to swelling if the capillary water is replaced but, in the case of sealed specimens, there would be no water available to replace the removed water and shrinkage will occur perhaps due to capillary tension. The degree of autogeneous shrinkage or swelling depends on the composition of the concrete, method of curing and size of specimen.

1. 3. b. Carbonation Shrinkage

Though generally carbonation shrinkage is included with drying shrinkage its cause and pheonomenon are distinctly different.

Carbonation is the irreversible shrinkage resulting from the chemical reactions that take place between carbon dioxide (in the atmosphere) and the calcium hydroxide mainly at the surface of the concrete. Its rate depends on the concentration of carbon dioxide in the atmosphere, water contact, size of specimen and the relative humidity of the ambient medium. The effect of carbonation is nearly negligable at 25% and 100% relative humidity and greatest at 50%[29].

I. 3. c. Drying Shrinkage

As its name implies, drying shrinkage occurs due to the evaporation of moisture from the concrete. This could happen almost as soon as the relative humidity of the ambient atmosphere drops below 100%. The water held in the large capillaries is the first to evaporate and as defined before (see page 10) all capillary water evaporates when the relative humidity drops below 50%. The second water to evaporate is the gel pores water. This water occupies smaller gaps and is adsorbed to the solid surfaces of the gel particles, undergoing compression (see page 9) which makes it more difficult to evaporate and its evaporation yields the greatest part of the drying shrinkage. It finally disappears when the relative humidity drops to zero.

The evaporation of either or both of these types of water will cause shrinkage, the degree of which will depend on many factors.

Many investigators have studied these factors and the literature pertaining to the subject is very vast. A brief conclusion only will be given below.

The following observations have mostly been established:

a) Shrinkage changes with the type or fineness of cement. A change in either of these two may lead to variation in the structure of cement gel, shrinkage being a pheonomenon related to cement gel would be expected in turn to be influenced. However, the effect of cement properties on the shrinkage of concrete is small[21].

b) The rate of shrinkage increases with increasing water-cement ratio. The extent of shrinkage has also been found to increase with water-cement ratio[22,23,24] as this will increase the amount of water available for evaporation. Increasing the water content would reduce the volume of restraining aggregate and so would be expected to increase shrinkage. The water content is not believed to be a primary factor[21].

c) Aggregate content is perhaps the most important factor, as aggregate restrains the amount of shrinkage that otherwise might have been expected. Thus shrinkage reduces with increasing aggregate-cement ratio and its modulus of elasticity.

The microcracks that may occur as a result of high stresses at the cement aggregate interfaces may also help to reduce the amount of shrinkage.

d) The rate of shrinkage decreases with time and increases with reducing relative humidity of the ambient atmosphere. Good evidence of this has been given by L'Hermite[25].

e) The larger the specimen the less the shrinkage. This is more pronouned at early ages as the rate of shrinkage of larger specimens seems to increase at later ages and the gap between the shrinkage of the different specimen sizes narrows[25]. The reason may be due to the change in surface-volume ratio, the distance which the water would have to travel before it evaporates and the restraints offered by the core of specimens due to the differential shrinkage stresses.

The mechanism by which shrinkage occurs was studied by a number of investigators[24, 79, 80]. The most likely explanation is that shrinkage occurs as a result of the increase in "Van der Waals" forces and hydrostatic tension due to the evaporation of water from the gel and capillary pores causing the structure of cement gel to attract in order to restore the hygrometic equilibrium. During the action of shrinking, the strucutre of cement gel may alter, pores may change their shape and dimension and new chemical bonds between the particles of cement gel brought closer together may be developed. This, as well as microcracks at the aggregate cement surface, may account for the observed permenant shrinkage or the irreversable part on rewetting which may be as much as one third of the overall shrinkage. The role of adsorbed water will be covered when discussing creep mechanism.

I. 4. CREEP OF CONCRETE

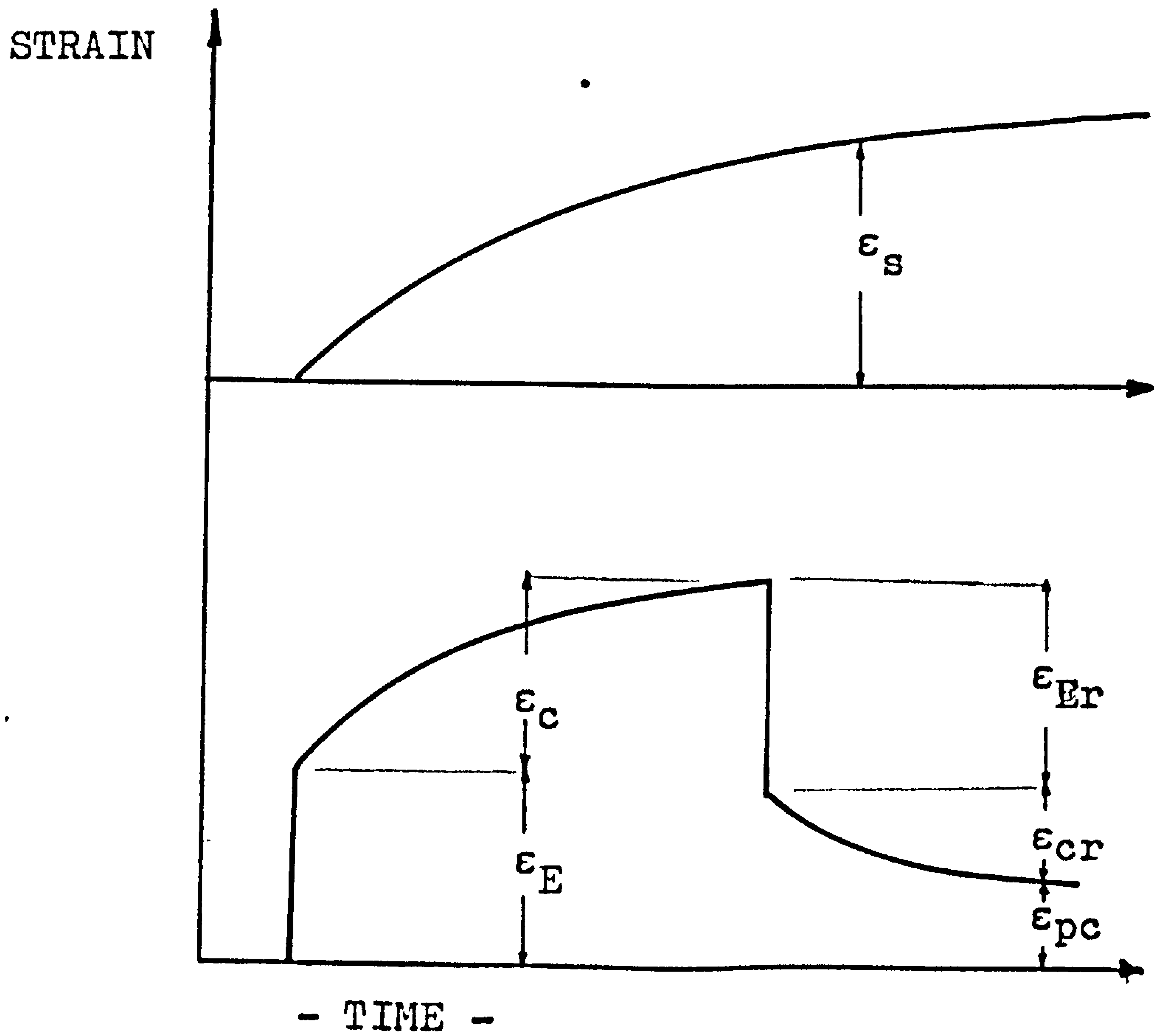
I.4.a. Definition of Strains

When a specimen of cement paste, mortar or concrete is subjected to a sustained load, an elastic strain followed by an inelastic strain will be exhibited. The elastic strain takes place almost instantaneously while the inelastic strain is time-dependant. The inelastic time-dependant strain taking place due to the application of the load will be called creep. The change in elastic modulus of the material while creep is being observed will generally be neglected unless otherwise stated.

The time-dependant strain due to other factors besides external load (generally shrinkage strains measured on unloaded control specimens) will be deducted. Although the applied load might interfere with the rate and degree of shrinkage strains, this effect will generally be included in the creep strain. The term "total strain" is used when shrinkage is not deducted.

When, after a period of sustained loading, the load is removed, a part or whole of the elastic strain will be recovered followed by a time-dependant recovery of a part or whole of the creep strain. These two recoverable strains will be called elastic recovery and delayed elastic or creep recovery respectively. The irrecoverable part of the creep strain will be called permanent creep.

The term "specific creep" or "specific" (creep and elastic) strain" will be used to define the creep or (creep and elastic) strain per unit stress. The figure below summarises these definitions (Figure I. 1):



ε_s	=	Shrinkage or swelling strains (deducted from those below)
ε_E	=	Elastic strain on loading
ε_c	=	Creep strains
ε_{Er}	=	Elastic strains on unloading (elastic recovery)
ε_{cr}	=	Creep recovery
ε_{pc}	=	Permanent creep after unloading and allowing for recovery

Fig. I.1. Definition of Terms

In all cases the absolute difference of the total strain and the strain in the control specimens is to be considered regardless of whether the load is compression, tension or whether the control specimens are shrinking or swelling.

The creep is considered to have reached a constant limit when the value of the total time-dependant strain in the loaded specimens minus that of the unloaded control specimens remains constant for a reasonable period of time. The rate of creep is then considered to be nill. It should be appreciated that the action of the load could change the shrinkage strain by a value equivalent to that of the creep but of opposite sign. However, this possibility is since taken into account by the stated definition of creep.

I.4.b. Uniaxial Compression Creep

The behaviour of creep has long been recognised and the amount of literature pertaining to the subject is considerable. The factors influencing the creep of concrete are numerous and have mostly been investigated for sustained uniaxial compression.

A review on the effect of these factors may be found in papers like Neville[60] and others[55,61] or in theses like Illston[65] and Elbaroudi[46] and only the general conclusions of the established facts based on these reviews are to be included in the present thesis to provide the information necessary for the examination of the creep theroies and for the comparison with uniaxial tensile creep.

The factors influencing creep may be divided in two categories:

Intrinsic - including,

- type of cement
- water-cement ratio
- type of aggregate
- aggregate-cement ratio
- grading, shape and size of aggregate
- compaction
- shape and size of specimen.

Environmental - including,

- level of stress
- age at loading

- time under load
- curing conditions

As the creep is basically a phenomenon related to cement gel it is to be expected that all the factors influencing cement gel would bear some effect on creep. In fully compacted concrete it has been observed that:

a) Creep varied with type and fineness of cement[62,63]. This may be related to the effect they both have on the degree of hydration and structure of cement gel.
   It has also been observed that cements producing more shrinkage also undergo more creep[64].

b) Increasing the water-cement ratio leads to an increase in the creep if other factors remain constant[26, 27]. The reason may be that increasing the water-cement ratio results in a more porous, less rigid cement gel and in the presence of a large amount of free water in the hardened paste.

c) Increasing aggregate-cement ratio reduces creep[28,29] (when the aggregate is made of stiffer material than cement paste). Aggregate, in this case, acts as a restraining body. The higher its modulus of elasticity the more restraints it offers and the less the creep and shrinkage. Cracks and microcracks at the interface between the cement paste and aggregate may contribute to creep under high stresses. Neville put forward a formula relating creep of concrete to volume fraction of aggregate:

$$\varepsilon_c = \varepsilon_c^{\bullet}(1 - V_a)^n$$

ε_c = creep of concrete

$\varepsilon_c^{\bullet}$ = creep of cement paste

V_a = volume fraction of aggregate

n : ranges between 1.7 and 2.1 with time

d) The larger the specimens used for observation the less the creep (Ross[30] and Hanson[81]). This may be attributed to the fact that movement of water would be restricted and the sensitivity of the specimen to the changes in the ambient humidity is reduced.

e) Specific creep of a given concrete is nearly constant for stresses not exceeding about 50% of the ultimate strength. This is the average level of stress at which cracking starts to occur in the short-term loading of concrete in uniaxial compression and linearity would not be expected after that level of stress.

f) The older the concrete at the time of loading the less the total creep. This may be related to the rigidity and stability gained by cement gel with time.

g) The rate of creep decreases with time. This may be related to the restoration of hygrometric aquilibrium which was disturbed by the sudden application of load and to the more stable distribution of load to the composite elements of the concrete taking place with time. The development with time of stronger and more rigid cement gel may also act as a contributory factor. Many formulae have been proposed relating creep to time. A convenient and practical one is the hyperbolic formular suggested by Ross(31).

$$\varepsilon_c = \frac{t}{a + bt} \quad \text{where}$$

ε_c = creep strain

t = time after loading

a and b are constants

and other formulae of the form: $\varepsilon_c = A(1-e^{-Bt})$ where

t = time after loading, and

A and B are constants; are also simple and fairly representative.

h) Wet curing results in less creep. This will depend on the ambient humidity of the atmosphere surrounding the loaded specimens. In general wet curing produces a more rigid cement gel with less pores and less free water and so should lead to less creep. The presence of water-filled pores in concrete lacking hydration may be the cause of high creep. It was found by many investigators(32,33,34,35) that the removal of this water almost eliminated creep.

i) Drying concrete creeps more than concrete stored in water. This has been realised and appreciated by a number of investigators(5,36,37,38) and has led some(38) to separate creep into two components, basic creep and drying creep. The former occurs under conditions of no moisture interchange with the ambient humidity and the latter is equal to that creep occuring under drying condition minus the former. Formulae considering the effect of shrinkage have been suggested like that of L'Hermite(39):

$$\varepsilon_c = \varepsilon_{cw}(1 + k\varepsilon_s) \quad \text{where}$$

ε_{cw} = creep under free shrinkage condition

ε_s = shrinkage at the given relative humidity

k = constant depending on the concrete

I. 5. UNIAXIAL TENSILE CREEP

As previously mentioned, little work has been carried out on tensile creep and in particular uniaxial tensile creep. The results obtained from the indirect tensile test may not apply to uniaxial tension and will not be included in this review.

There are certain difficulties to be experienced when investigating the uniaxial tensile properties of concrete and unless these are overcome the results may not be representative. These difficulties may either not exist or be easily avoidable in uniaxial compression. They are mainly the outcome of the low tensile strength of concrete (about one-tenth of the compression) and the difficulty of applying uniaxial tensile load to the specimen. As a result of the low tensile strength the applied stresses (a certain ratio of the ultimate strength) must also be low and thus produce correspondingly small strains. For example, a concrete of a uniaxial tensile strength of 300 lb/in.^{2} loaded to one-third of its ultimate strength, 100 lb/in.^{2}, will exhibit an elastic strain of about 33 microstrain (i.e. 33×10^{-6}).

Depending on the concrete under test and moisture condition, the creep strain after a period of sustained laoding will be a certain ratio of elastic strain either above or below unity. In some cases, as will be seen later, the ratio of the ultimate creep strain to the elastic strain could be as low as one half.

It can therefore be realised that unless the measuring devices are very accurate and stable with time the errors involved in the measurements, and especially when comparing different concretes, may lead to a false conclusion.

Applying a uniaxial tensile load is not very simple and often results in a large eccentricity of the load. This eccentricity, apart from yielding a wrong measured strain, may lead to an incorrect estimation of the load at failure and to a large variation of results as shown by Evans[40]. This error may be maximised if the measurement is taken at the surface of the specimen.

The variation in the modulus of elasticity of the tested piece from one section to another may also contribute with a further error as demonstrated by Elvery[41]. This error may be reduced by casting the concrete in the appropriate direction.

On the other hand, shrinkage strains are generally much larger than the tensile creep strains and the variation in the measured shrinkage of two control specimens may in certain cases exceed the creep strain.

These difficulties therefore must be considered when discussing the subject and their role should be appreciated before drawing any conclusion.

Tensile and compressive creep are both phenomena related to cement gel. The factors affecting the compressive creep generally affect the cement gel, and so would also be expected to affect tensile creep. But the way and the degree by which these factors influence the former may not necessarily follow the same pattern. This, however is to be corroborated.

In 1936 Durton[42] reported some results on uniaxial tensile creep and his curves are shown in Figure I.2. He conducted the tests under three different moisture conditions:

a) specimens cured and loaded in water,

b) specimens cured and loaded in air, and

c) specimens cured in water for two months then loaded in air.

The results indicated that the specimens cured and loaded in water underwent the smallest creep, followed by those cured and loaded in air, and the specimens cured in water then loaded while drying underwent the largest creep. These findings followed the same pattern as for compression.

Results obtained by Glanville and Thomas[28] in 1939 are shown in Figure I.3 and these indicated close agreement in the values of tensile and compressive stresses in an uncontrolled humidity atmosphere. Ross[43] in 1954 investigated tensile creep and his results are shown in Figure I.4 and Table I.

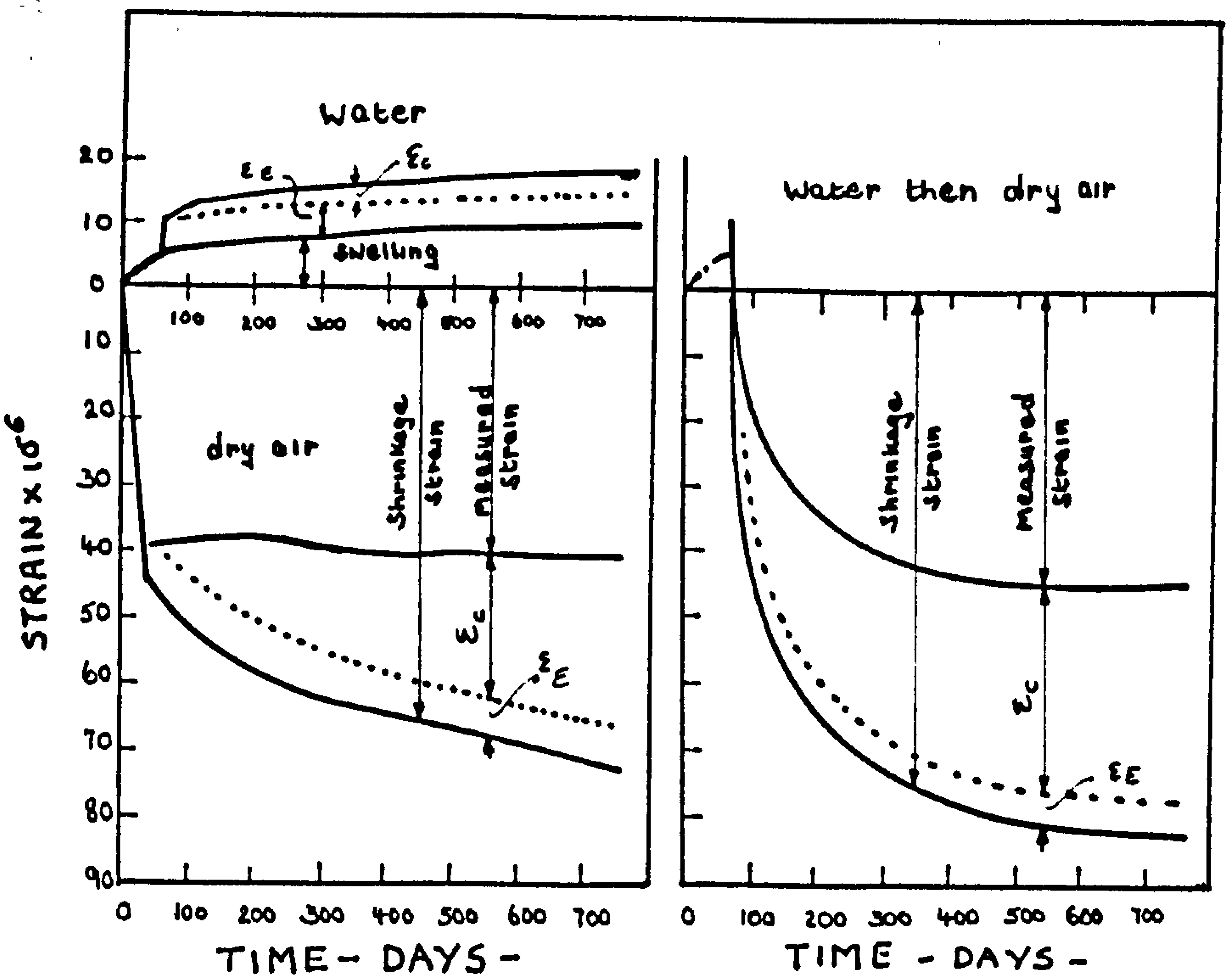


Fig. I.2. The Effect on Strain of Shrinkage, Swelling, and Sustained Tension (Durton[42])

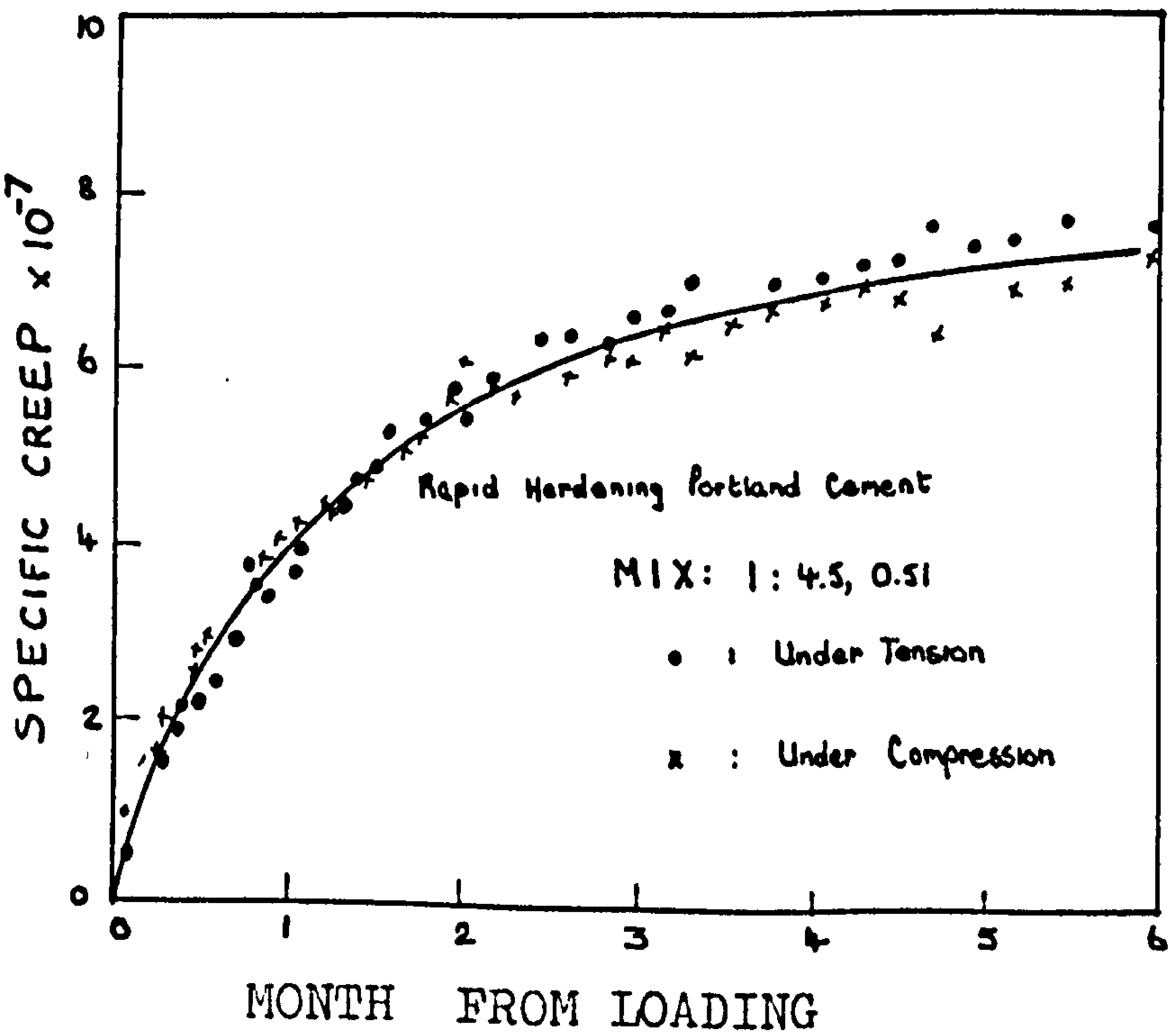


Fig. I.3. Creep of Concrete in Tension and Compression (Glanville and Thomas (28))

He used 1:2:4 concrete with ordinary Portland cement and Thames ballast, $\frac{3}{8}$ maximum size aggregate and a water-cement ratio of 0.6. Specimens were cured wet until four days before loading when they were exposed to a laboratory atmosphere. The tensile stresses were applied through internal pressure to hollow cylinders.

Ross related the difference in the tensile and compressive creeps to the change in the relative humidity. Choosing 60-70% for normal humidity he suggested that tensile creep would exceed compressive creep at higher humidities and vice versa, and connected this influence of humidity on the relative values of tensile and compressive creep with a seepage theory.

The examination of a more recent work by L'Hermite(44), Mamillan and Lefeure (1965) on tensile creep where the specimens were left to dry out for six months at 50% relative humidity, then loaded to 16, 24 and 26 kg. cm.^{2}, indicated that tensile creep nearly ceased after 10 days of loading except in specimens loaded to 26 kg. cm.^{2}. It also indicates that the relationship between stresses and tensile creep strains is non-linear. They reported values of creep strain after a period of three days loading in relation to the elastic loading of 16 kg. cm.^{2} : 22%.

24 " : 33%.

26 " : 35%.

Their result is shown in Figure I.5.

The specimens were prism-shaped and the tensile load was applied through a steel plate stuck to the concrete using epoxy resin. The strain measurement was taken by means of extensometer.

Illston[45] (1965) investigated the effect of the level of stress, age at loading and moisture condition on the tensile creep and he suggested that the main characteristics were much the same as those in compression with three major observed differences:

1) The rate of specific creep was initially greater in tension.

2) The absence of delayed elastic strain in tension.

3) The limiting delayed elastic strain in tension was observed to be the same whether the concrete was drying at a relative humidity of 65% or kept wet.

Figures I.6 and 1.7 show some of his results in tensile creep. Illston used cylindrical specimens, the tensile load was applied by means of a steel spider cast into the concrete. The strains were measured by means of the Whittemore demountable strain gauge.

Hansen's[77] results also indicated a higher initial tensile creep than compression but the gap narrowed with time. El-Baroudi's[46] (1940) curves on the tensile creep of concrete (mix) proportions 1 : $1\frac{2}{3}$: $3\frac{1}{2}$ by volume, water-cement ratio of 0.54 by weight normal Portland cement) are shown in Figure 1.8. These curves show large differences between drying creep and creep of concrete kept wet. These results support Durton's findings, but contradict Illston's.

Ohna and Shibata (1959)[47] investigated the effect of rate of loading on the extensibility of concrete in uniaxial tension. Although their results were not directly comparable with those already mentioned, they give an indication of how the extensibility was affected by the storage conditions. Their results are shown in Figures 1.9, 1.10 & 1.11.

The three Figures relate the tensile strain to the loading speed after reaching different levels of stress. Figure 1.9 represents specimens cured in their moulds for two days, then left to dry in an atmosphere of 55% R.H. Loading started at the seventh day. The same condition applies to Figures 1.10 and 1.11, except that the specimens related to Figure 1.10 were sealed for the first two days and the loading was started at the fourth. The specimens referred to in Figure 1.11 sealed for the first 13 days and the loading was started on the fourteenth day.

Electrical resistance strain meter was used for measuring the strains and the load was applied by means of bars cast into the concrete. Prism-shaped specimens were used.

These results indicated that the strain increased almost linearly (logarithmic scale) with decreasing loading speed for the specimens loaded at the seventh day (five days after drying), but the maximum strain when loading started at the fourth day (two days after drying began) was reached with a higher loading speed. This was emphasized even more with the specimen sealed for 13 days and loaded at the fourteenth day (one day after drying began).

The authors suggested that the reason may be due to the wet concrete's having a larger extensibility than dry concrete. However, the reason could also be explained as being due to the larger shrinkage rate taking place in the early days after unsealing the specimens, thus encouraging more creep.

Unfortunately no certain conclusion could be drawn from the existing work as differences between the results of the various investigators were great.

Although, perhaps, the reviewed work indicated that tensile creep may be affected by the same factors as compressive creep it did not give any definite conclusion on the way these factors may affect it and to which degree.

On the other hand, most of the authors agreed with the difficulties encountered in applying tensile stresses and measuring the small tensile strains which affected their accuracy.

Therefore, it is thought that the discussion would be more worthwhile if it were postponed until the author's work has been presented.

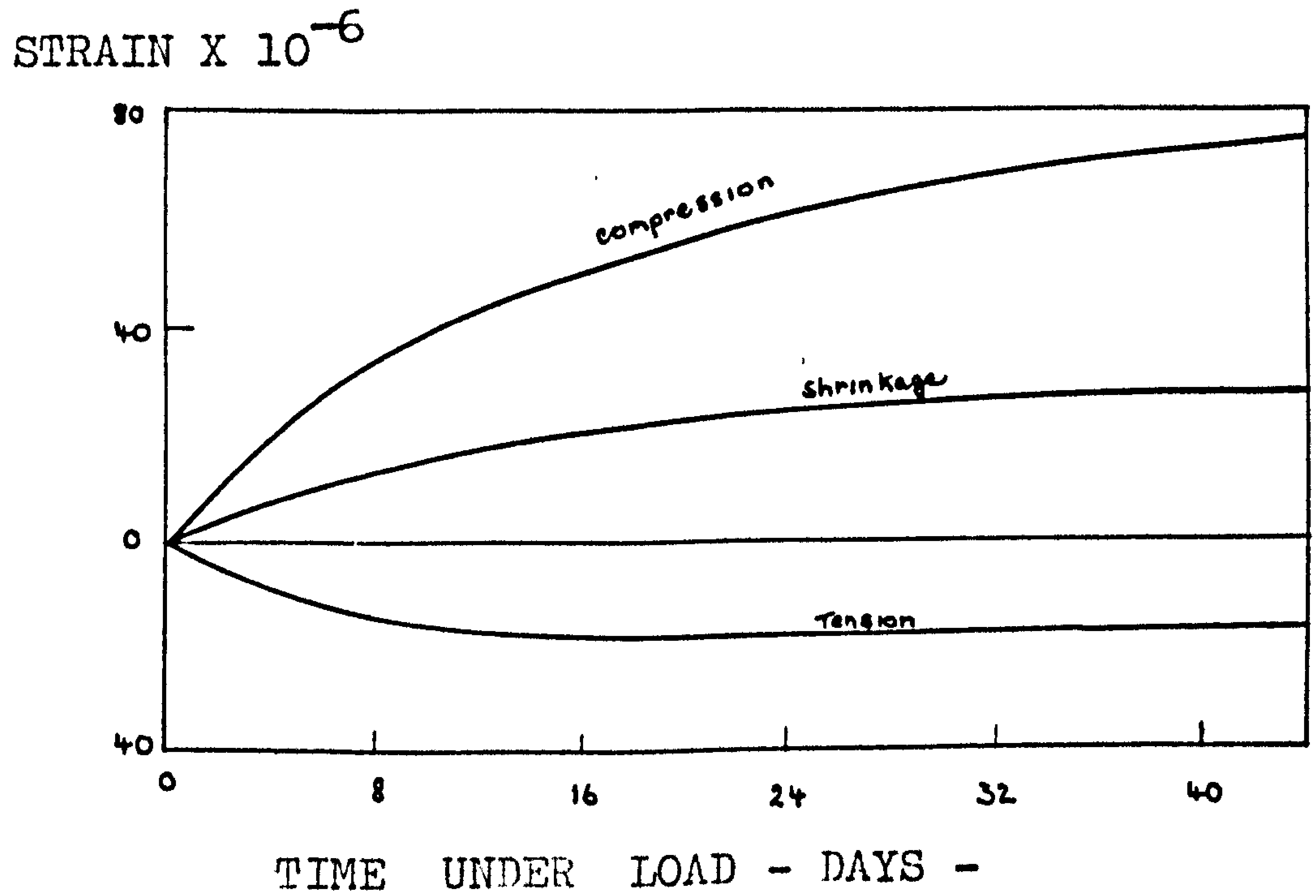


Fig. I.4. Creep of Concrete in Tension and Compression (redrawn from Ross(43))

TABLE I

Test No.	Period loaded (days)	Mean relative humidity (percent)	Creep Strains x 10^6			
			Under two-dimensional stressing, specimen (a)		Under one-dimensional stressing	
			In compressive direction	In tensile direction	Compressive Stress, specimen (b)	Tensile Stress, specimen (c)
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	32	60	—*	5	22	28
2	44	65	53	38	47	41
3	60	70	110	17	44	65
4	60	80	32	111	27	105
5	40	87	7	40	20	43
6	60	57	96	10	64	46

*Strain gauge defected

From Ross(43)

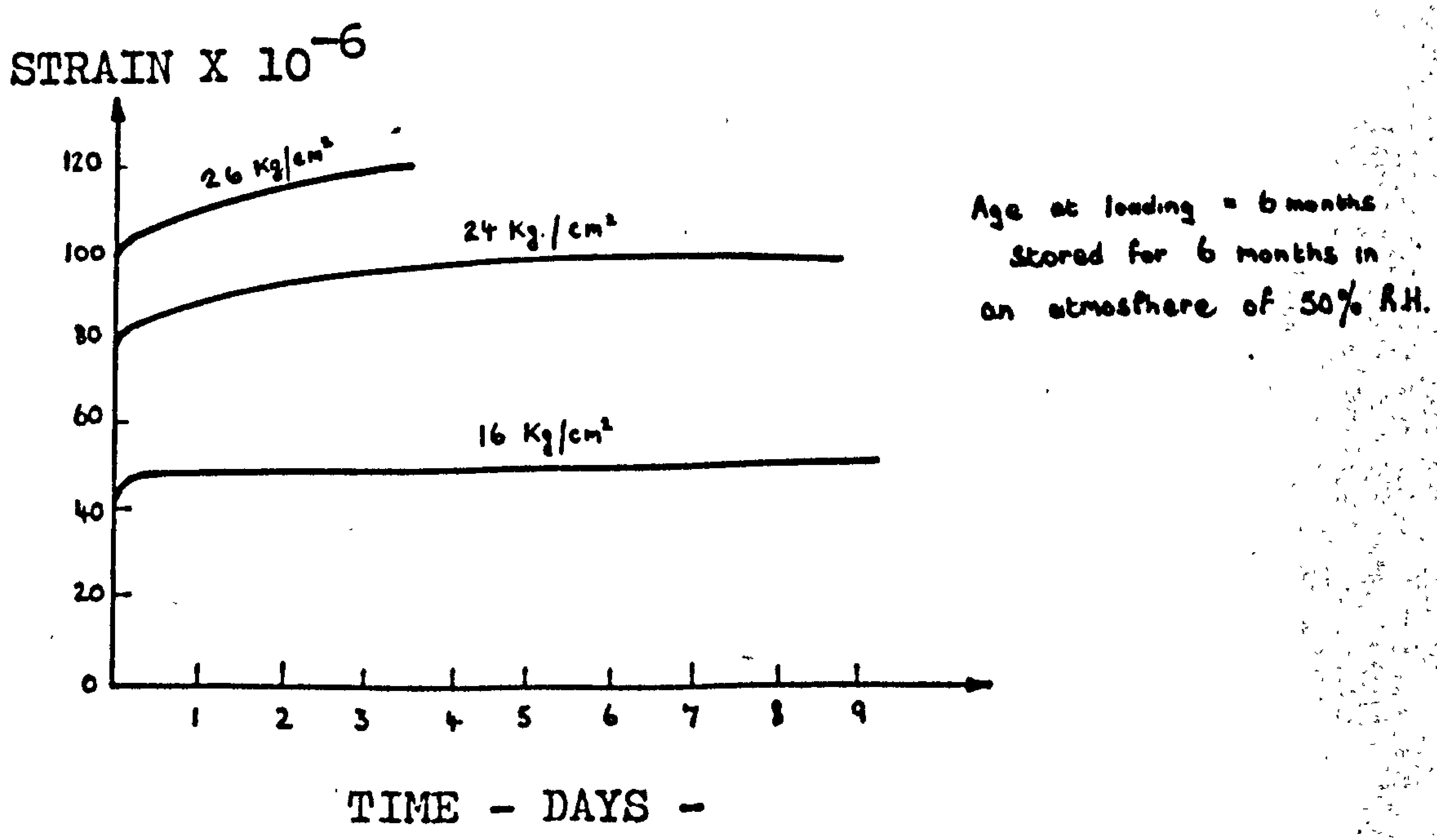


Fig. I.5. Creep of Concrete in Tension (L'Hermite, Mamillan and Lefeure(44))

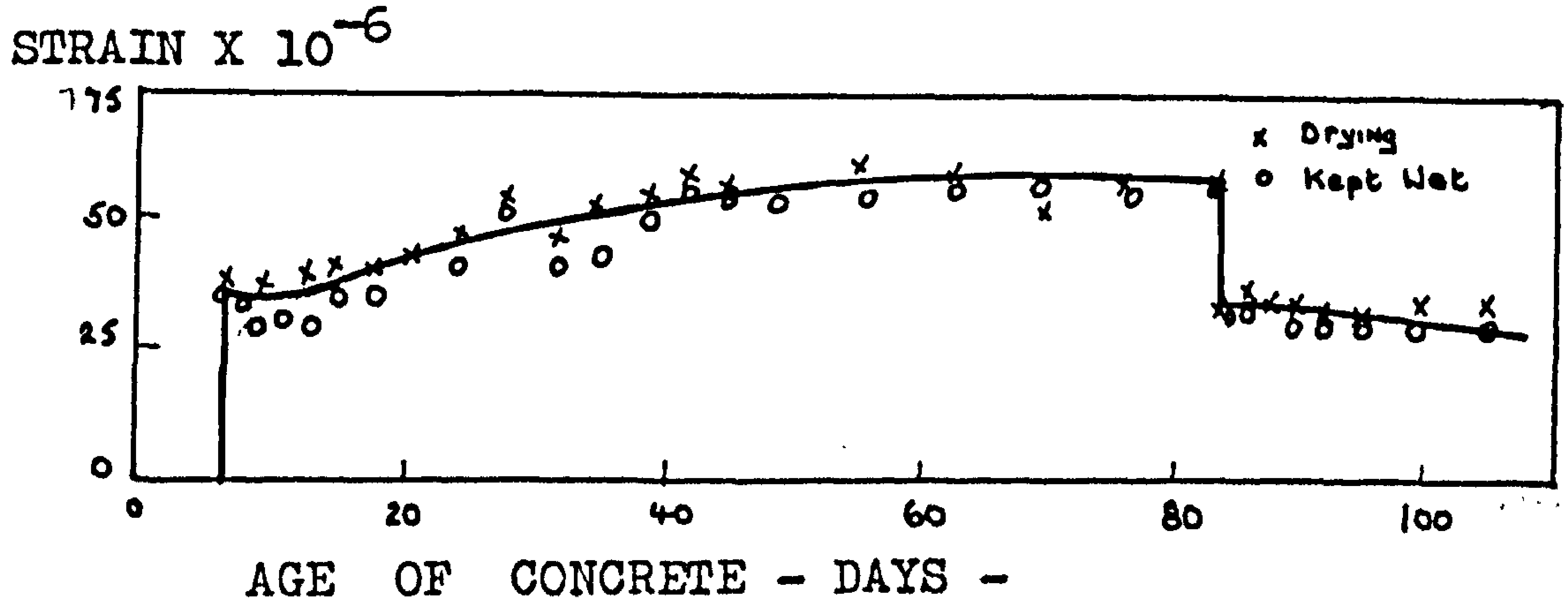


Fig. I.6. The Effect of Humidity of Storage on the Strain of Concrete in Tension(45)

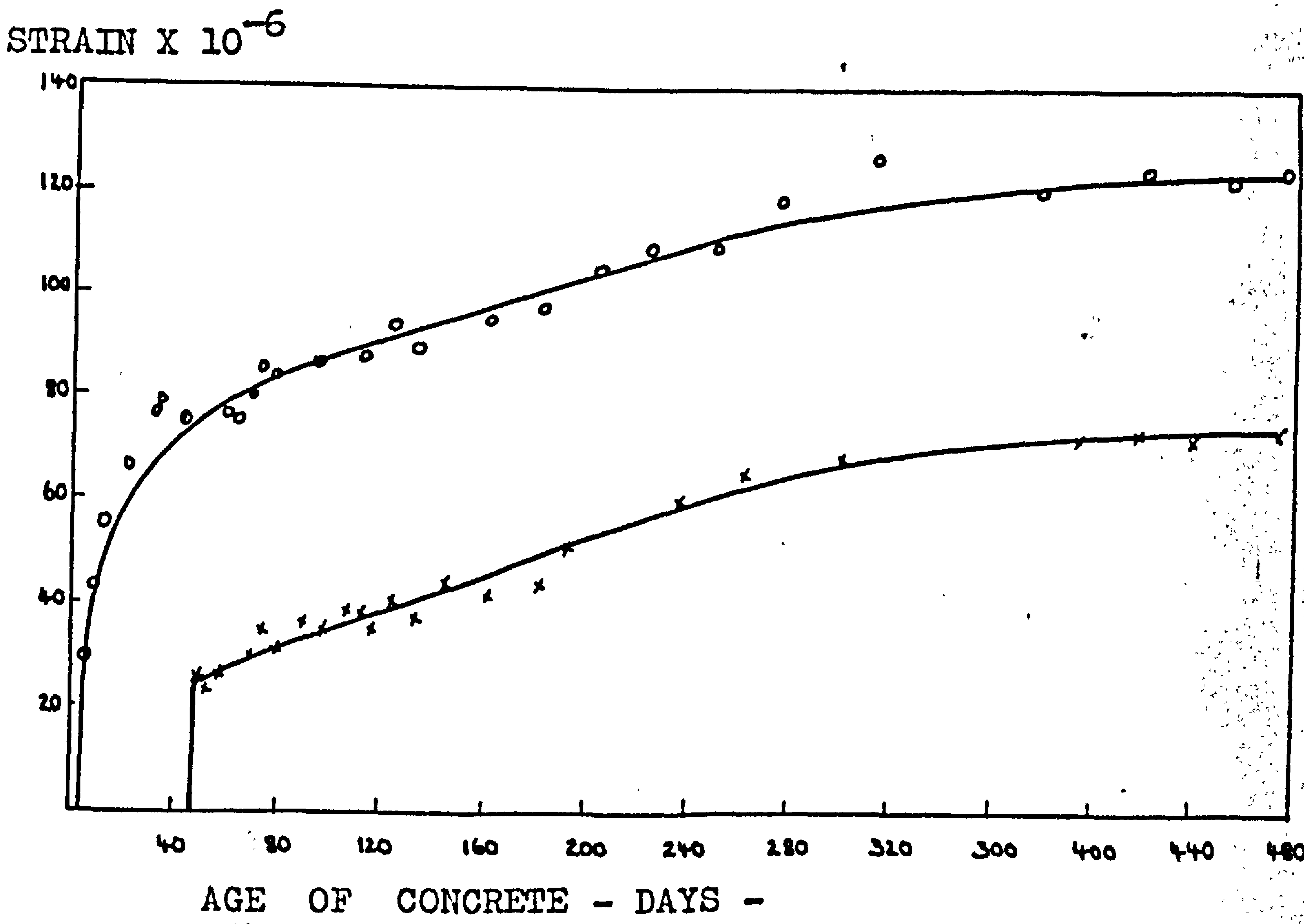


Fig. I.7. The Effect of Age of Loading on the Strain of Concrete in Tension (Illston(45))

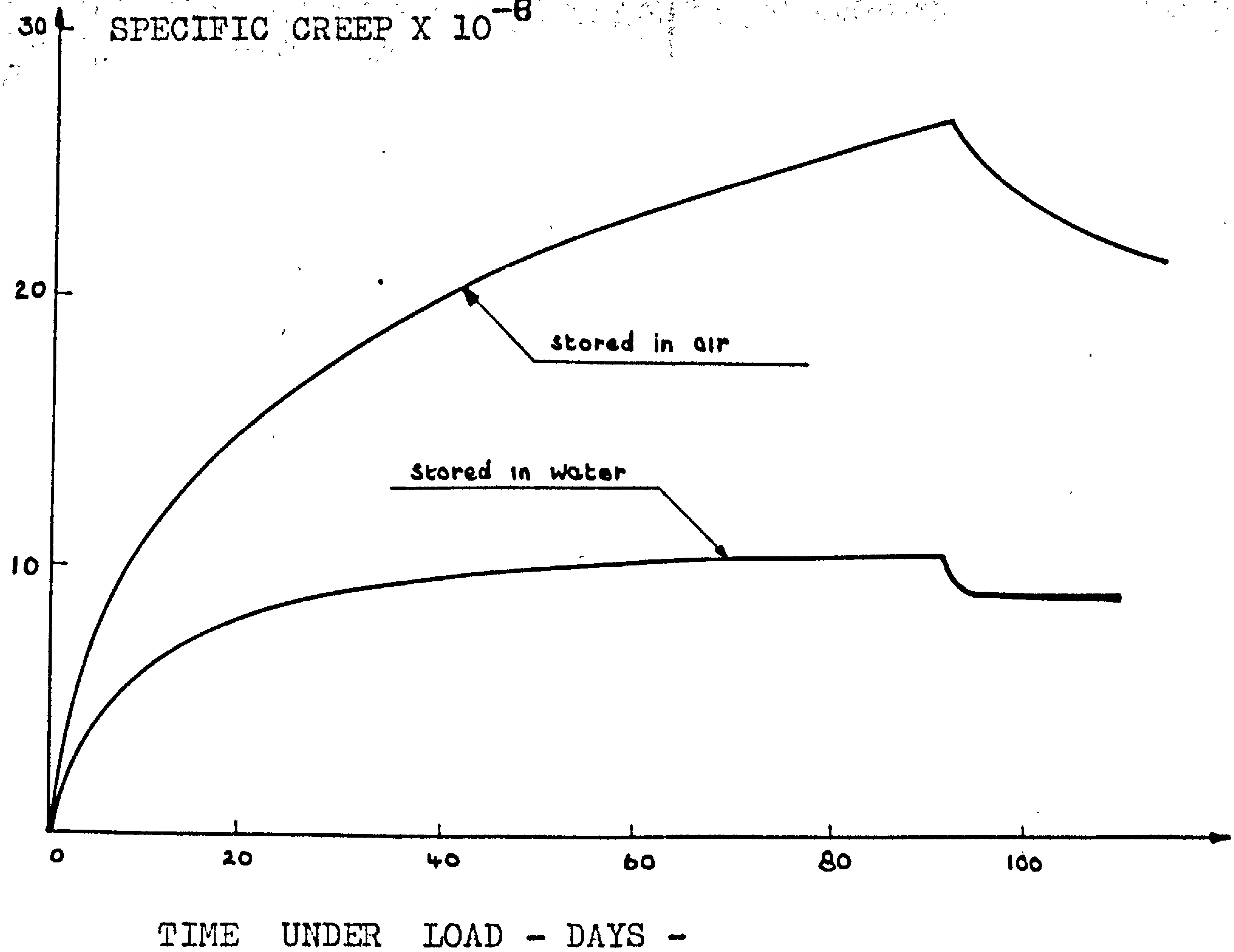


Fig. I.8. Elbaroudi's (46) Curves for Tensile Creep

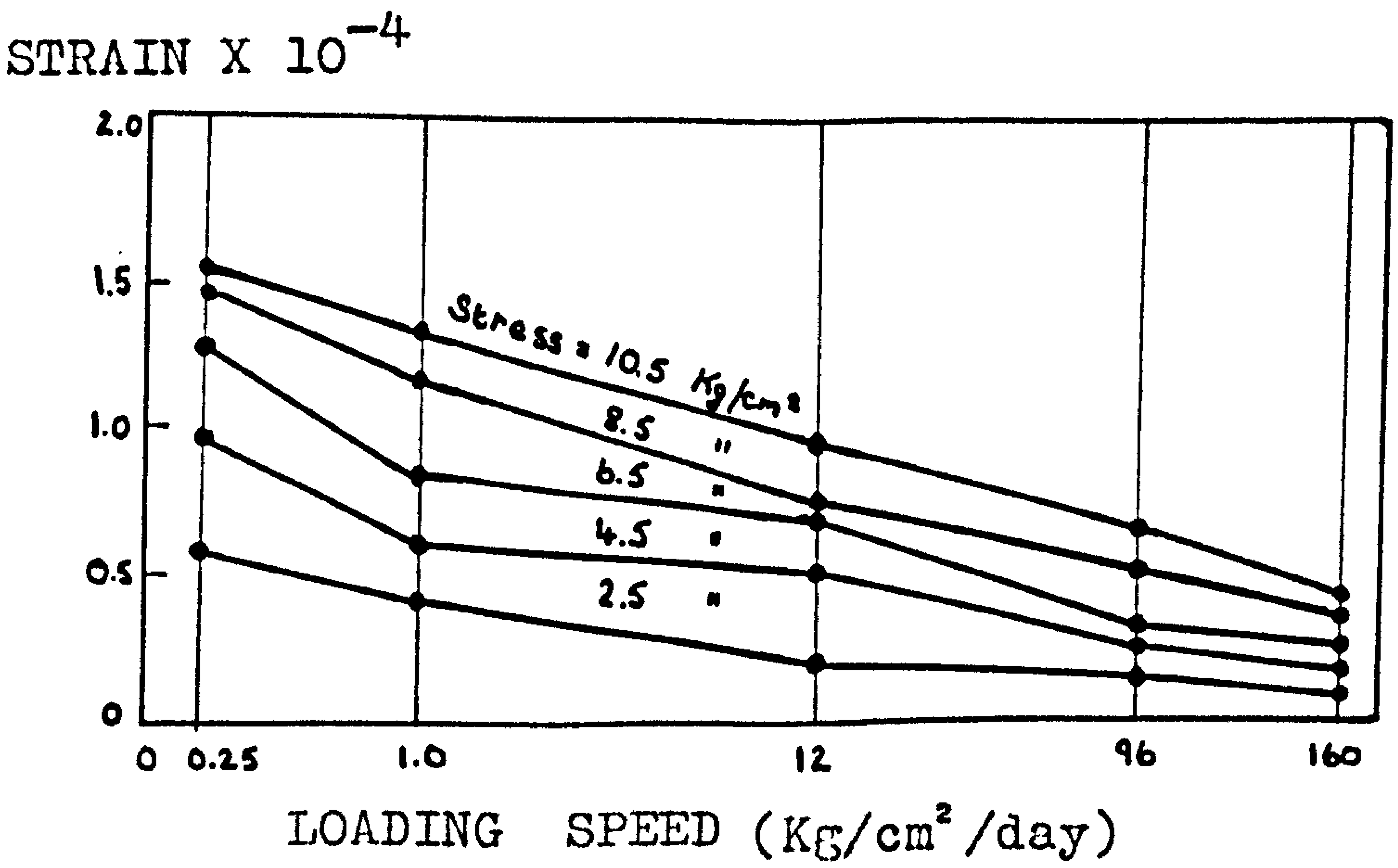


Fig. I.9. The Effect of Rate of Loading on Tensile Strain (Ohno and Shibata (47)

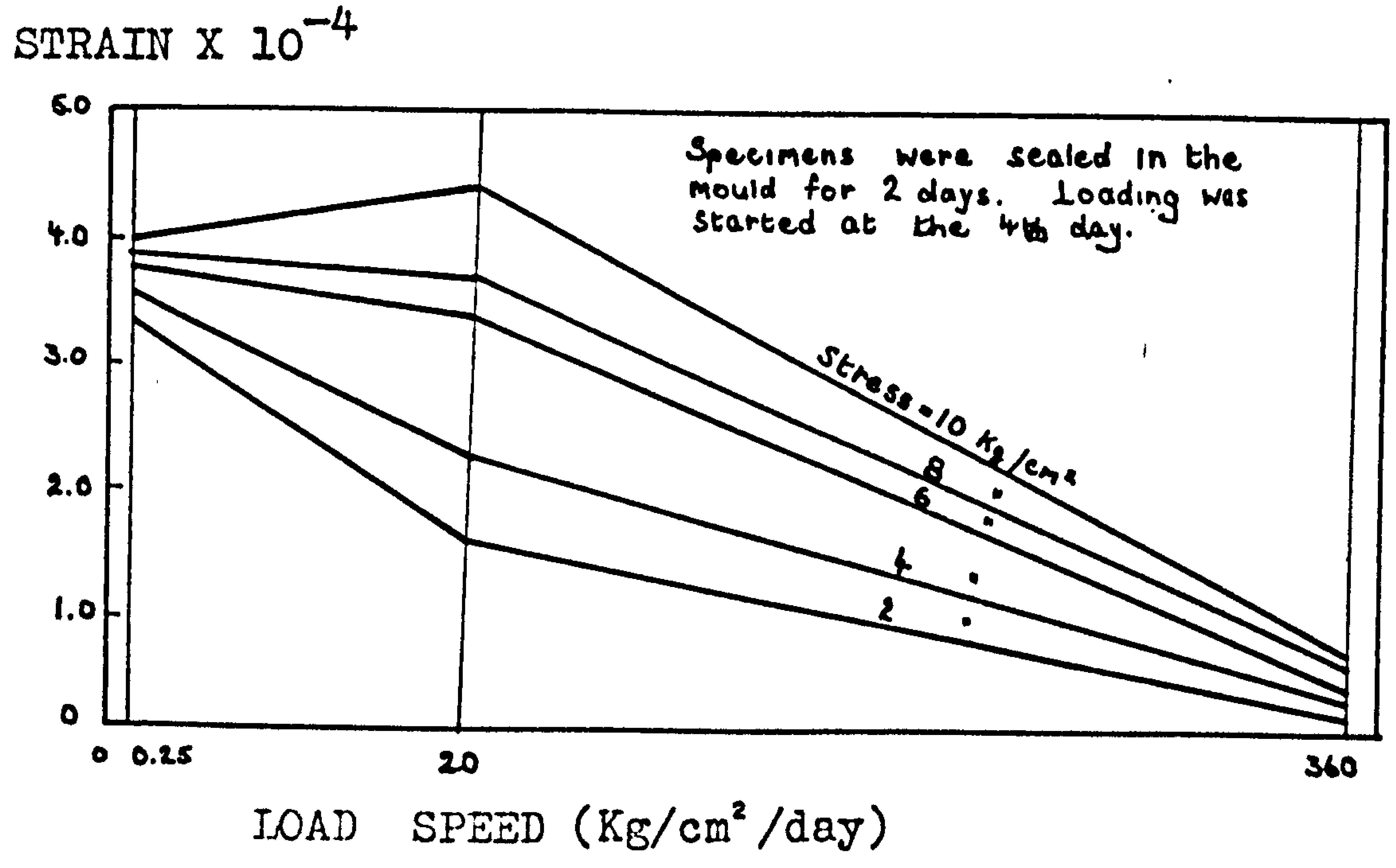


Fig. I. 10 The Effect on Strain of Rate of Loading (Ohno and Shibata(47))

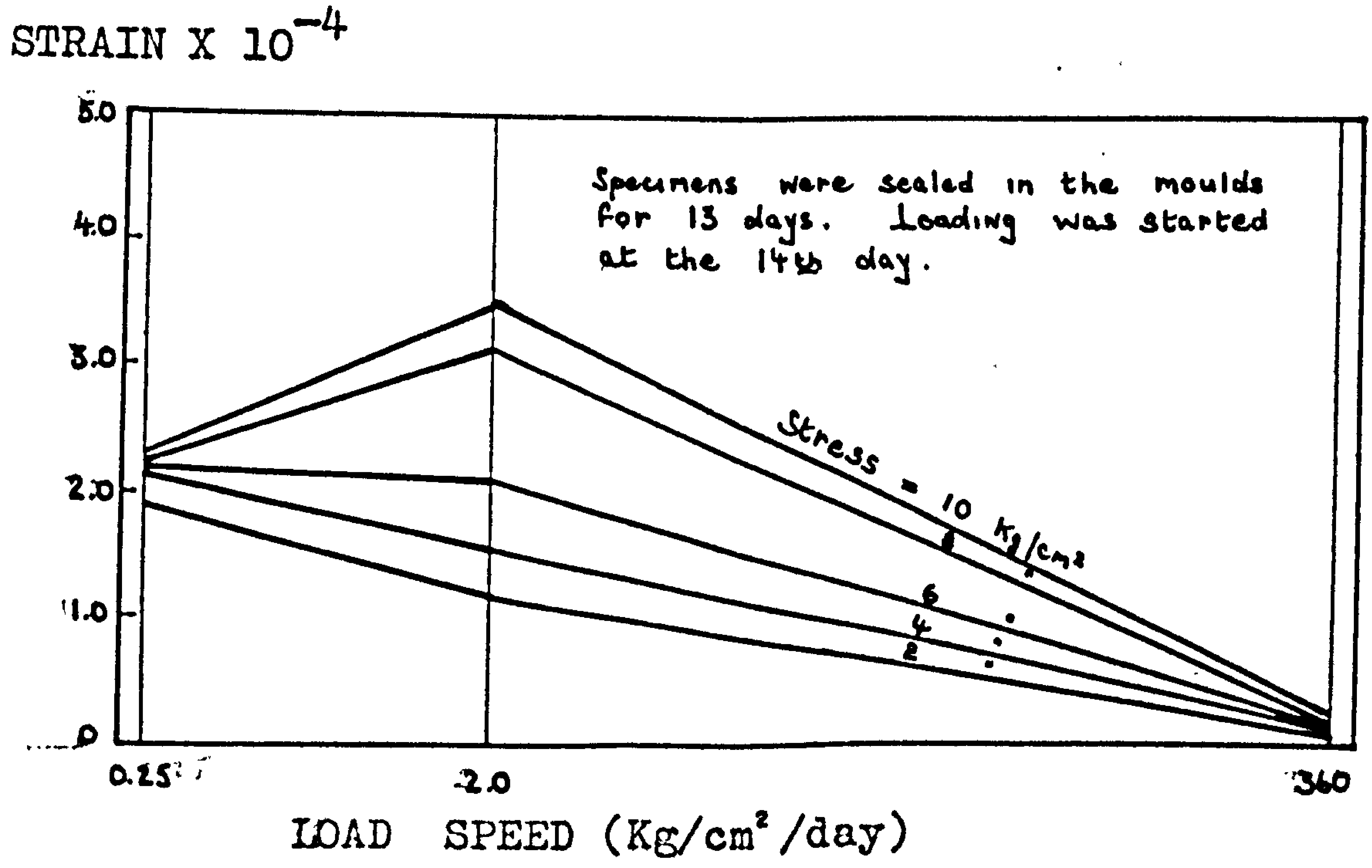


Fig. I. 11 The Effect of Rate of Loading on Strain (Ohno and Shibata(47))

I. 6. THEORIES OF CREEP

Since there is no direct evidence as to how creep occurs, the explanations and theories of its mechanism have differed with different investigators and with different experimental findings. These theories are, in general, hypothetical based on some knowledge of the cement gel structure and on the experimental observation. Their degree of certainty depends on their logicality in connection with the structure of concrete, on the statistical backing of experimental results and on the number of supporting investigators.

A theory is generally thought to be valid when the creep behaviour, taking into account the various factors influencing it, could be explained within its framework. It generally remains acceptable until a new theory explaining wider range of results and accounting for the effects of those factors that could not be explained by the former theory is introduced. The former theory can then either be disregarded or may be broadened, elaborated and interpreted in such a way so that it could account for the new findings.

On the other hand, a mechansim based on two or more theories may be sought to explain creep. Such a mechansim predicting uniaxial tensile creep will be proposed and presented in Chapter VI.

The theories of creep could be summarised under four headings:

a) Differential shrinkage theory.

b) Plastic theory.

c) Viscous theory.

d) Seepage theory.

The most popular are the viscous and seepage theories.

I. 6. a. Differential Shrinkage Theory

This theory could only be applied when shrinkage is taking place. If we consider a drying specimen, the shrinkage occurs first on the surface and then moves gradually inwards to the core. This differential shrinkage sets up compressive stresses in the core of the specimens and tensile stresses on the outside. A state of strain balancing these two takes place changing as the shrinkage continues to occur. If now a compressive stress is applied a new state of strain will be exhibited and it could be demonstrated[48], that, for a non-linear stress-strain relationship, this new state of strain occurring after the application of the external stress is greater than the sum of that taking place due to shrinkage and stress separately. Thus leading to a time-dependant strain.

This theory cannot explain the creep of sealed and immersed concrete. Neither can it explain the creep of drying concrete in an ambient medium of high relative humidity and subjected to low applied stress as, in this case, the relationship between stresses and the (elastic + creep) strains will generally be linear. However, it is possible in certain cases that creep could be enhanced by the effect of the differential shrinkage (or what is called "Pickett effect")

I. 6. b. Plastic Theory

This theory relates the creep strain to that occurring due to the gradual breakdown of the concrete. As cement paste is of a hetrogeneous nature high stresses could be expected to occur at local places while the average stress over the whole section is relatively low. This may be emphasized at the microscopic level when perhaps points of high stresses above the observed strength of the material occur under very low stress/strength ratios. Microcracks may occur at these points and the load distributed to the neighbouring elements. These microcracks and redistribution of stress might not occur immediately after application of load but gradually with time and so account for a time-dependant strain.

In specimens drying out shrinkage stresses may add to the concentration of stresses and to the development of further microcracks. Glucklich[59] studied the effect of microcracks on the time-dependant strain and demonstrated it by a rheological model.

The Plastic Theory is limited and cannot alone account for the creep occurring after a long period of loading, for the creep under low stresses or for the linearity of stress-creep strain.

However it may, in conjunction with other theories like the Seepage Theory, form a sound mechanism for explaining creep and failure especially in uniaxial tension. This will be discussed in a later chapter (VI).

I. 6. c. Viscous Theory

The Viscous Theory is based on the assumption of passive or viscous movement between the particles of the cement gel. It assumes that the stresses taking place in the gel due to the application of the load are not large enough to squeeze the water out, but sufficient to move the particles of the gel relative to one another.

Ruetz[35], for instance, after examining the results obtained on tensile and torsion creep concluded "that the creep mechanism in hardened cement paste is a pure shear process". He suggested that the passive movement takes place in the interconnecting water layers of a few molecules thickness in the sub-microscopic gel. Illston[45] also suggested that "creep strain is primary viscous movement caused by the reorientation of the structure of cement stone under load". However, he did not exclude the possibility of some deformation caused by water movement or Seepage Theory.

The linear relationship between stress and creep (at low stresses) is considered to support the Viscous Theory, but it could also be explained by others. Also, the results of Hansen[49] showed that the time-dependant deflection of a beam (preloaded and allowed to recover) under sustained loading was the same as that which had not been subjected to previous loading which is also to be expected from a viscous movement.

On the other hand no change of volume due to sustained loading and a Poisson's ratio of 0.5 would be expected with a viscous movement but the tests indicated that Poisson's ratio was small (a zero creep Poisson's ratio was suggested by Ross[43]). It is also not very easy to explain the large creep accompanying the large shrinkage by the Viscous Theory.

I. 6. d. Seepage Theory

Seepage Theory, contrary to the Viscous Theory, is based on the assumption that movement of water within the gel must take place to restore the hygrometic equilibrium when disturbed by the application of load. This theory relates the phenomenon of creep very much to that of the shrinkage.

Shrinkage occurs due to the evaporation of water; a state of hygral disequilibrium with the ambient atmosphere occurs due to the drop of vapour pressure within the concrete and some of the water imbedded in the gel structure as absorbed water is drawn out. This imposes compressive stresses on the elastic phase surrounding this water (generally flaky and needle-shaped particles) and results in a diminution of volume. When a compressive load is applied more compression will be imposed on that medium and its hygrometric equilibrium is again disturbed and this requires a further movement of water from within that medium. This movement takes place gradually until the hygrometric equilibrium is restored. If and when it is disturbed again by the action of external stress (applied load) or by internal stress (shrinkage and swelling stresses) re-adjustment of the water molecules in the medium that has stabilised under the action of the preceding stresses will have to take place resulting in new volume change that will again be stabilised with time under the new condition of adjusted stresses. This theory is supported by many investigators(55,66,67).

Powers(66) while supporting this theory emphasized the importance of understanding the thermodynamics of adsorption and desorption for obtaining an explicit explanation. He also introduces the role of adsorbed water in the places of hindered adsorption as a structural element of cement gel able to maintain static resistance and he called it load-bearing water. Powers relates the cause of creep to the adjustment of the molecules of adsorbed load-bearing water under the action of stress. The work by Glucklich and Ishai(50) indicated the dependancy of the phenomenological nature of creep on the moisture condition of the specimen.

On the other hand, Maney's[51] tests have indicated that the loss in weight of loaded test pieces was the same as that of the non-loaded specimens. This finding has been presumed as contradictory to the Seepage Theory as it was considered that for the Seepage Theory to be correct, creep must be accompanied by a variation of the quantity of included water. Another objection to the Seepage Theory was made by Ruetz[35] who suggested that a "uniaxial tensile test is the simplest method of checking the Seepage Theory". Its occurrence, he concluded, refutes the Seepage Theory defining it as the hypothesis that water is extruded under external pressure. He also suggested that the results of tests of torsion creep did not agree with the Seepage Theory. This aspect will be discussed in Chapter VI.

I. 7. RHEOLOGICAL MODELS

I. 7. a. Introduction

At the present state of knowledge of volume change behaviour of concrete it seems unlikely that any mathematical solution will be explicitly representative. However, many of the existing mathematical expressions have proved their validity within the limits and conditions for which they were formulated, but could not generally account for the many variables that may affect particular aspects of the behaviour of concrete. Such behaviour, sometimes new and unpredictable, is still being observed by investigators.

It has been a common practice in concrete to use rheological models when trying to apply mathematics for predicting the inter-relationship of stress-strain and time. These models consist of a combination of simple mechanical elements which have defined criteria. The model is no more than a means of describing the behaviour so as to assist in producing an expression to fit the experimental findings. It does not, however, provide a real picture of the medium it is representing, but it makes the analysis relatively simple. The model is flexible in the sense that if it does not produce the wanted expression, reorganisation of the elements composing it, adding a new element or omitting one could be made. Such models have been proposed and used by many investigators for predicting creep strain and only the basic elements composing these models and a few examples of the different approaches will be given in the proceeding pages.

1. 7. b. Basic Elements

There are three basic elements used in rheology:

1) Hook element
2) Newton element
3) St. Venant element

The Hook element shown in Figure I.12 is represented by a spring and this accounts for elastic deformation. It interrelates stress and strain using Hook's law.

$$\sigma = E.\varepsilon$$

σ = stress

ε = strain

E = modulus of elasticity

The Newton element shown in Figure I.13 is represented by a dashpot and this accounts for viscous deformation. It interrelates stress and rate of strain in terms of time using Newton's law of viscosity:

$$d\varepsilon/dt = \sigma/\eta$$

$d\varepsilon/dt$ = the rate of strain

η = coefficient of viscosity

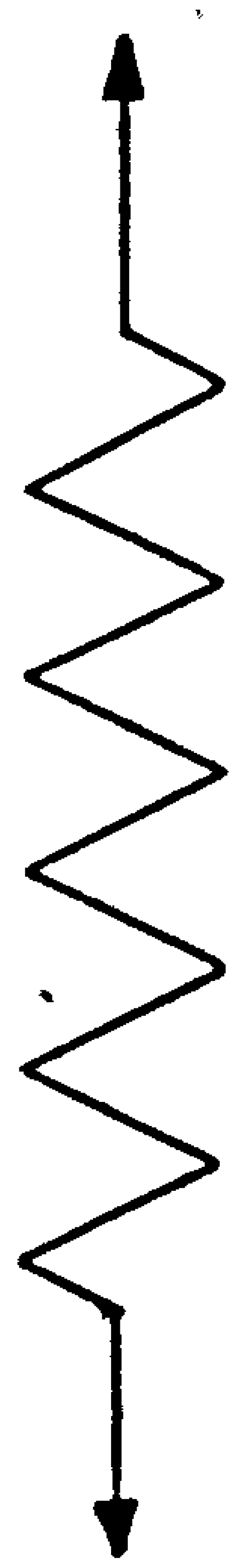


Fig. I.12. Hook Element

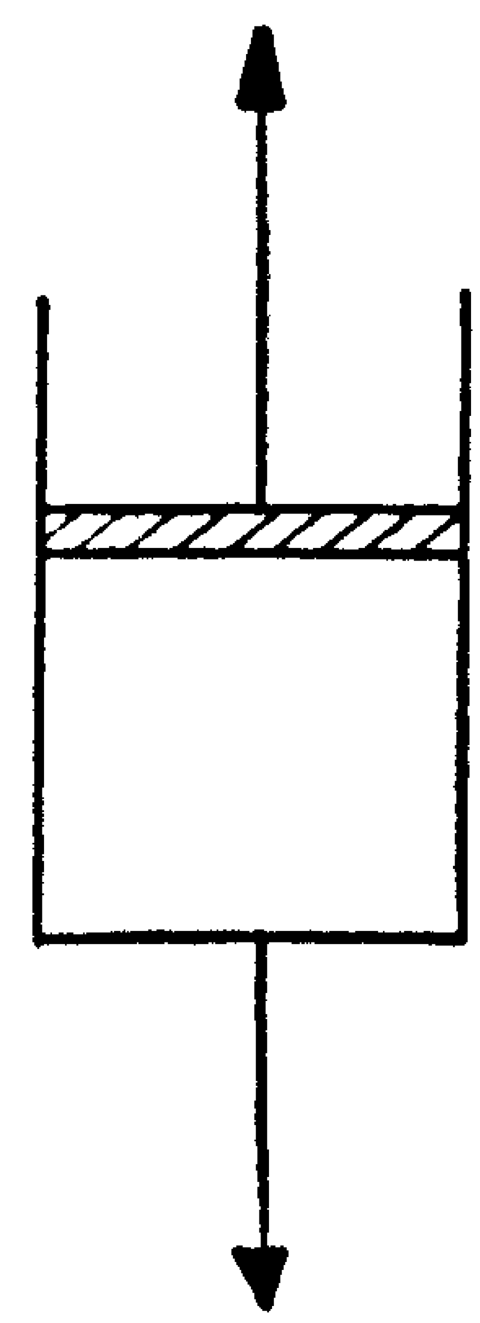


Fig. I.13. Newton Element

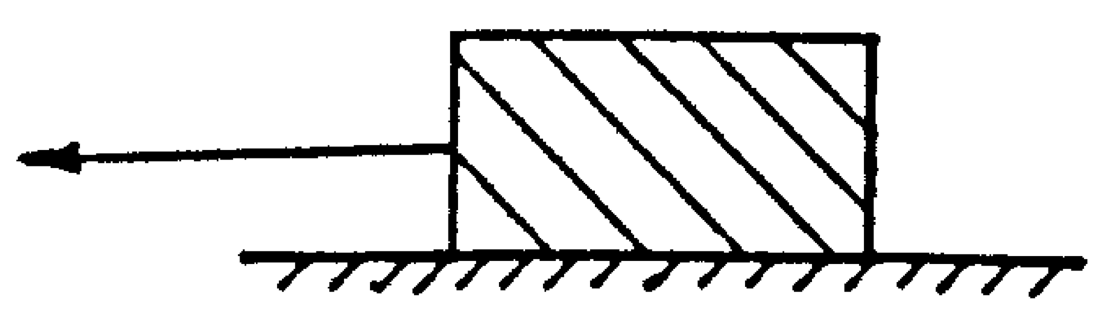


Fig. I.14. Friction Element

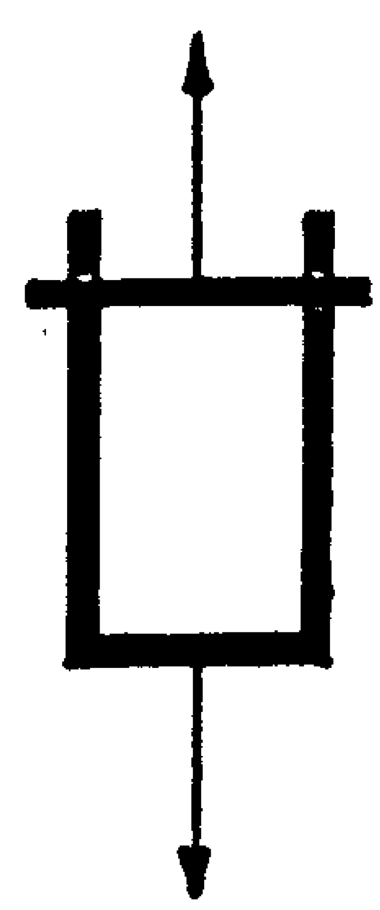


Fig. I.15. Shear Pin Element

The St. Venant element shown in Figure I.14 is generally represented by a frictional element and accounts for a plastic movement. It is stress-dependant.

Another element is sometimes used to account for a breakdown and is represented by a shear pin as seen in Figure I.15. It is independant of strain but stress-dependant.

I. 7. c. Rheological Models

Combination of the basic elements produce different models. Simple and direct combination of two of these elements are found in Kelvin and Maxwell elements shown in Figure I.16.

They are both a combination of Hook and Newton elements. In the former they are in parallel while in the latter in series. Kelvin element would account for a recoverable time-dependant strain while Maxwell represents a recoverable elastic strain followed by an irrecoverable time-dependant.

The rheological model (Burger body) shown in Figure I.17 combined Kelvin and Maxwell elements and was assumed to represent well the creep behaviour. It can produce the strain characteristics of the typical strains occurring under sustained loading and unloading, namely:

Elastic

Time-dependent

Elastic recovery

Time-dependent recovery, and

Permanent creep.

It does not, however, take into account the different modulus of elasticity of the composite material, the change in time of the rigidity, storage conditions, etc. These, however, were dealt with by the different investigators in various ways.

For example the general differential equation for this model (derived by Reiner) (quoted by Hansen[52]) is:

$$\frac{\beta_k}{E_k}\cdot\frac{\delta^2\varepsilon}{\delta t^2} + \frac{\delta\varepsilon}{\delta t} = \frac{\beta_k}{E_m E_k}\cdot\frac{\delta^2\sigma}{\delta t^2} + \frac{\beta_m E_k + \beta_m E_m + \beta_k E_m}{\beta_m E_k E_m}\cdot\frac{\delta\sigma}{\delta t} + \frac{1}{\beta_m}\cdot\sigma$$

where ε = strain

σ = stress

t = time (seconds)

E = modulus of elasticity (Kg/cm^2)

β = modulus of viscosity ($Kg\ s/cm^2$)

This was solved by Hansen[52] assuming sustained loading, and expressing the rheological constants in terms of the concrete composition by using two other models (figure I.18) to introduce the effect of the different modulus of elasticity of the composite

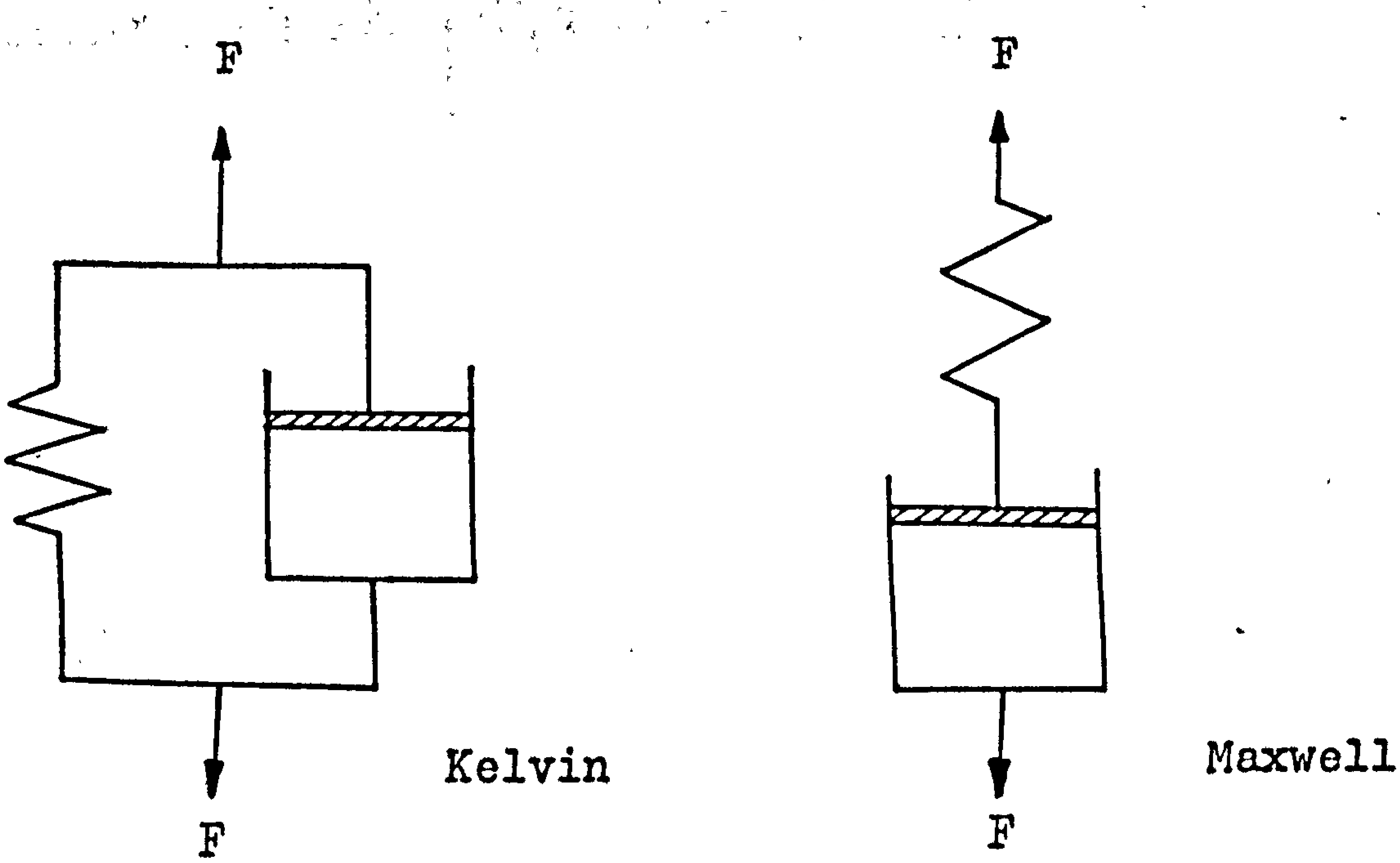


Fig. I.16. Kelvin and Maxwell Elements

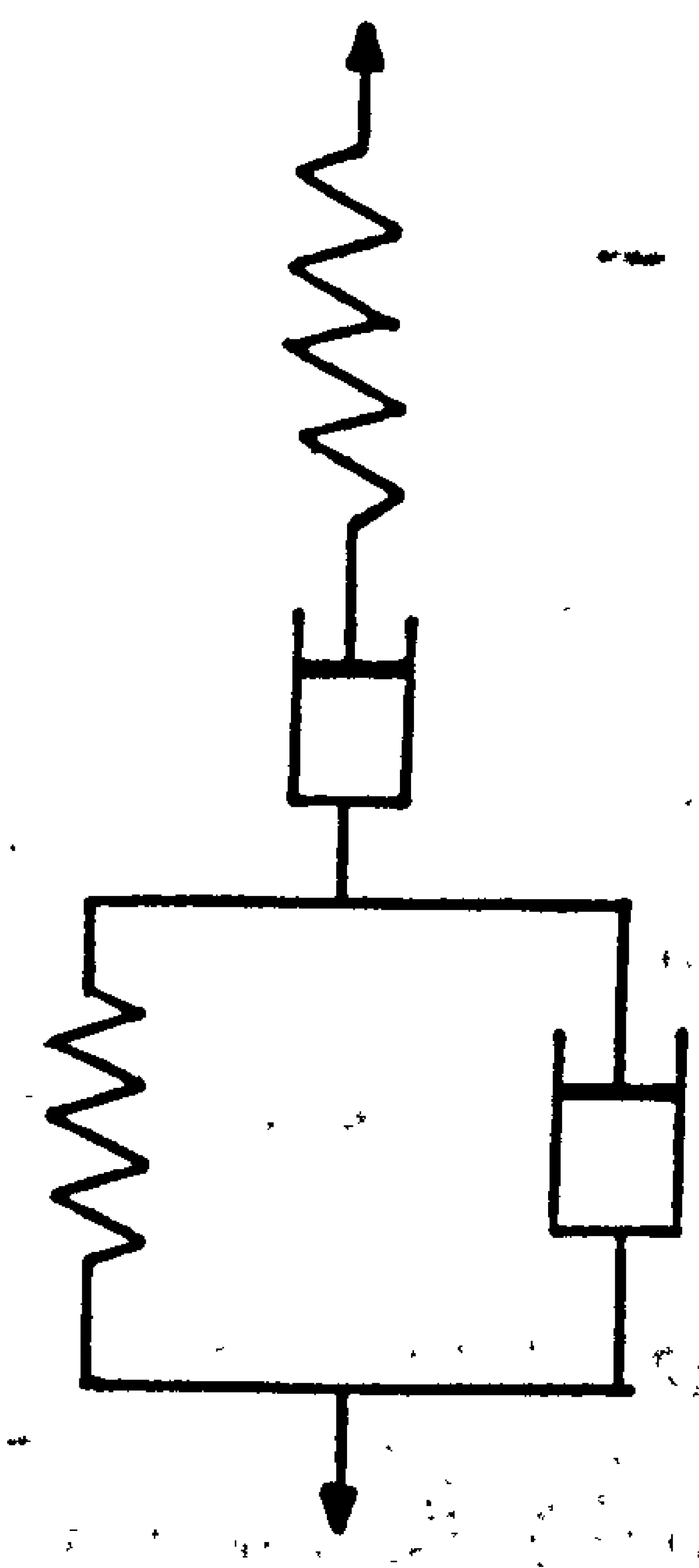


Fig. I.17. Burger's Body

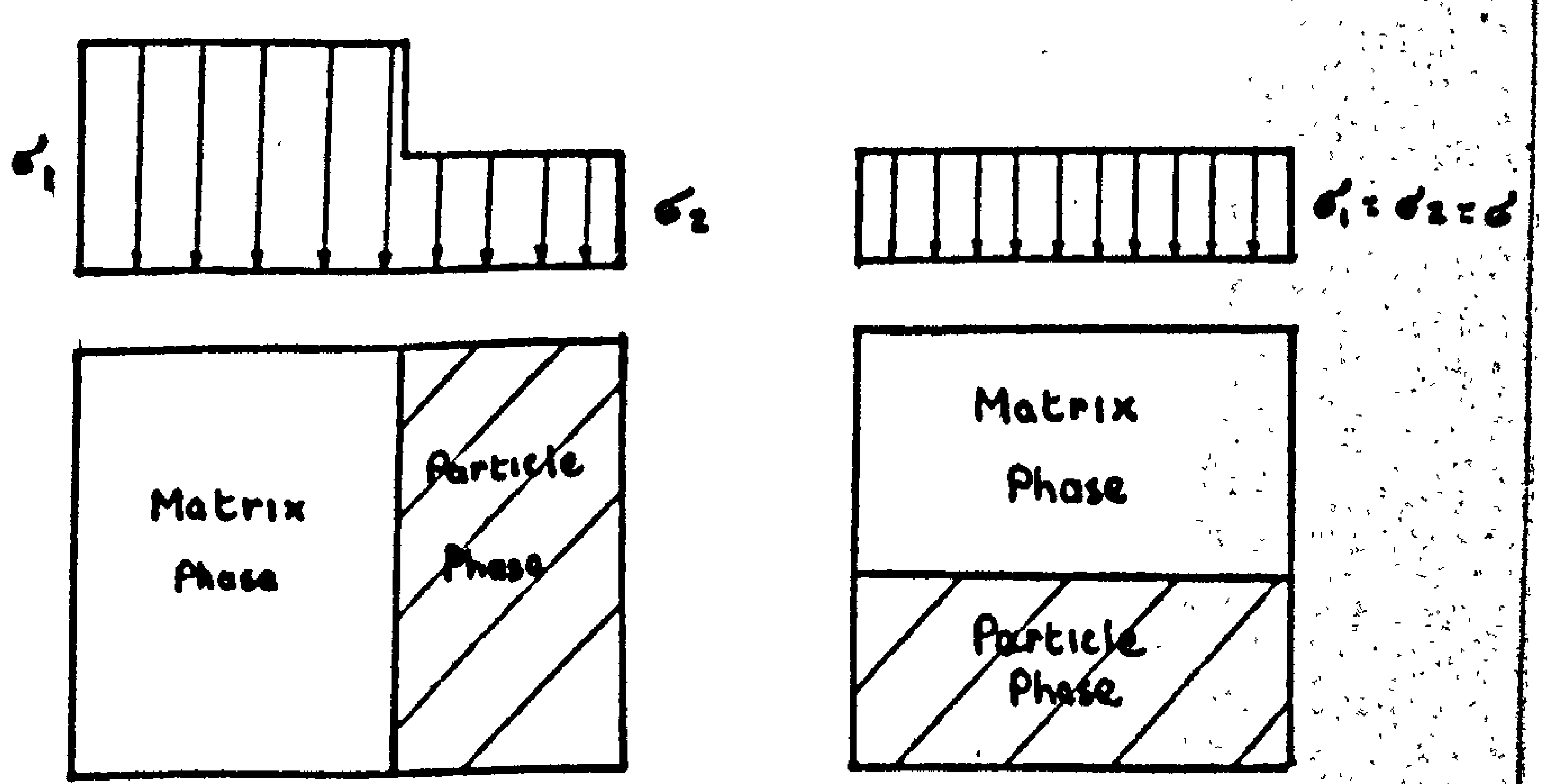


Fig. 1.18 Solid Models

material. His analysis resulted in the equation:

$$\frac{\varepsilon_c,\ t_1}{\sigma} = \frac{\beta(0.31g_o+a)V}{(Nk_1+0.31)g_o}(1 - e^{-m(t_1-t_o)}) + \alpha_1 aV_1 \log_e \frac{t_1}{t_o}$$

where ε_c, t_1 = creep strain at time t_1

σ = stress (Kg/cm^2)

t_1 = age of concrete at time t, (days)

t_o = age of concrete when loaded

a = water-cement ratio, corrected for bleeding

V_1 = volume concentration of cement paste in mortar or concrete

g_o = degree of hydration of cement at the time of load application

α_1, β, m = experimental coefficients

k_1 = weight ratio of non-evaporable water to cement when all cement is completely hydrated

N = constants related to cement composition.

Hansen claimed that his expression gives good agreement for various concretes loaded and stored in water.

The models in Figure I.18. postulate that concrete is a two-phase material (coarse aggregate embedded in a matrix of mortar, sand embedded in a matrix of cement paste, pores embedded in a matrix of cement gel . . . etc.) which is generally an acceptable assumption.

Each phase of the two considered phases is assumed to be homogeneous and isotropic.

Other approaches, trying to include the effect of hydration and stability gained with time, were made by considering a non-linear stress/time-dependant strain in Hook and Newton elements, (stress softening spring or time-thickening dashpot). Elaborations of Burger's body wo were made by adding on in series one or more Kelvin elements like the model used by Freudenthal and Roll[53] Fig.I.19 or by adding dashpot in pa~~r~~rallel and a spring in series (by Hansen[54]) seen in figure I.20. It is noticeable that in Burger's body or any other model consisting of a single dashpot the creep will continue for an indefinite time. The single dashpot, however, was used to account for the non-recoverable creep. This was overcome by Glucklich using flaps in the dashpot of the Kelvin element which operated to prevent the dashpot returning thus accounting for a permenant creep.

A complete model of creep (discussed by L'Hermit[55]) proposed by Toroja & Paez is shown in Figure I.21. This model, contrary to those mentioned, can take into account the effect on creep of ambient humidity by varying the level of water in the tank.

It is composed of elements A, B, C, D and E.

Element A: Spring, elastic and recoverable

B: Spring moving inside a cylinder and transmitting load to its walls, elastic but allows for some residual deformation through its friction element

C, D & E: In addition to B action, they allow for viscous movement in the cylinder. The three elements are connected to a common tank of water by tubes diminishing in size so as to represent a progressive difference in viscosity. The authors varied the constants of the elements to make it represent a multiphase material.

Glucklich(59) proposed a model which he used to demonstrate the effect of microcracking on the time-dependant strain of concrete. The model is shown in Figure I.22. It comprises two elements in series; an imperfect Hookean body and an imperfect Kelvin body. By introducing a large number of friction element of different coefficient of friction in series with the springs, Glucklich allowed for microcracking to take place while elastic and viscous movement are also being exhibited. The microcracks occur in time while the load is transferred to the spring from the dashpot. In another paper with Ishai(56) Glucklich, using a simple model, attempted to account for the effect of water content on creep.

England(57) used a solid model to predict, basically, the relationship between time and creep or time and shrinkage for concretes of various mixes containing any aggregate. This could only be done after knowing the properties of the constituents (including creep and shrinkage data for the matrix). The model treats the concrete as a two-phase material (aggregate embedded in a matrix of mortar or cement paste).

A more complete solid model was used by Illston[58] to predict elastic and creep strains after conducting some control tests. The model takes into account the composition of the concrete and the variation in the hydration products occurring with time. This provides a more realistic picture of the material it represents. The model does not indicate how the strains occur but postulates that each of the components has certain characterists which would have to be defined. The model is shown in Figure I.23.

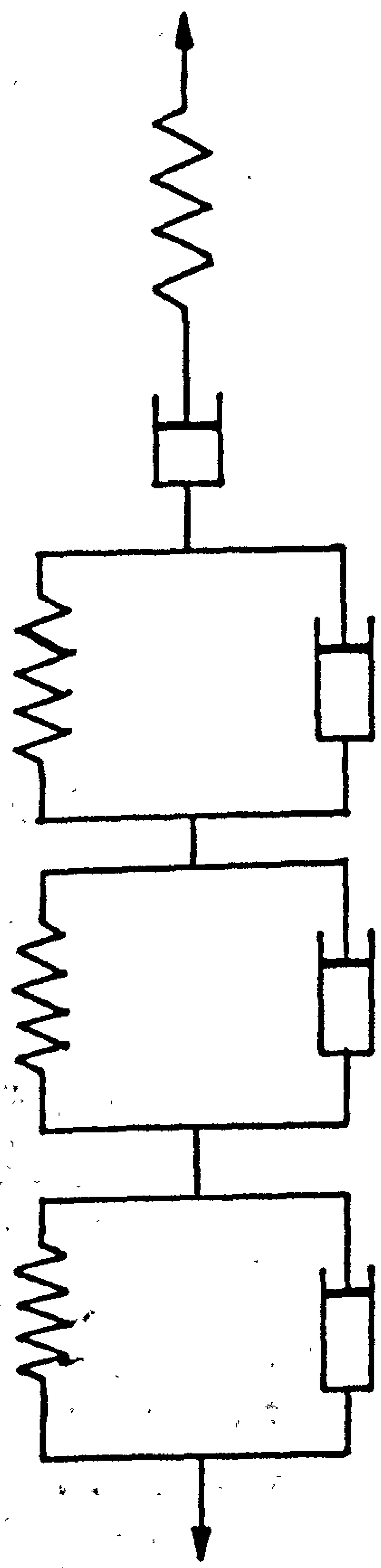


Fig. I.19. Model for Concrete (Freudenthal and Roll(53))

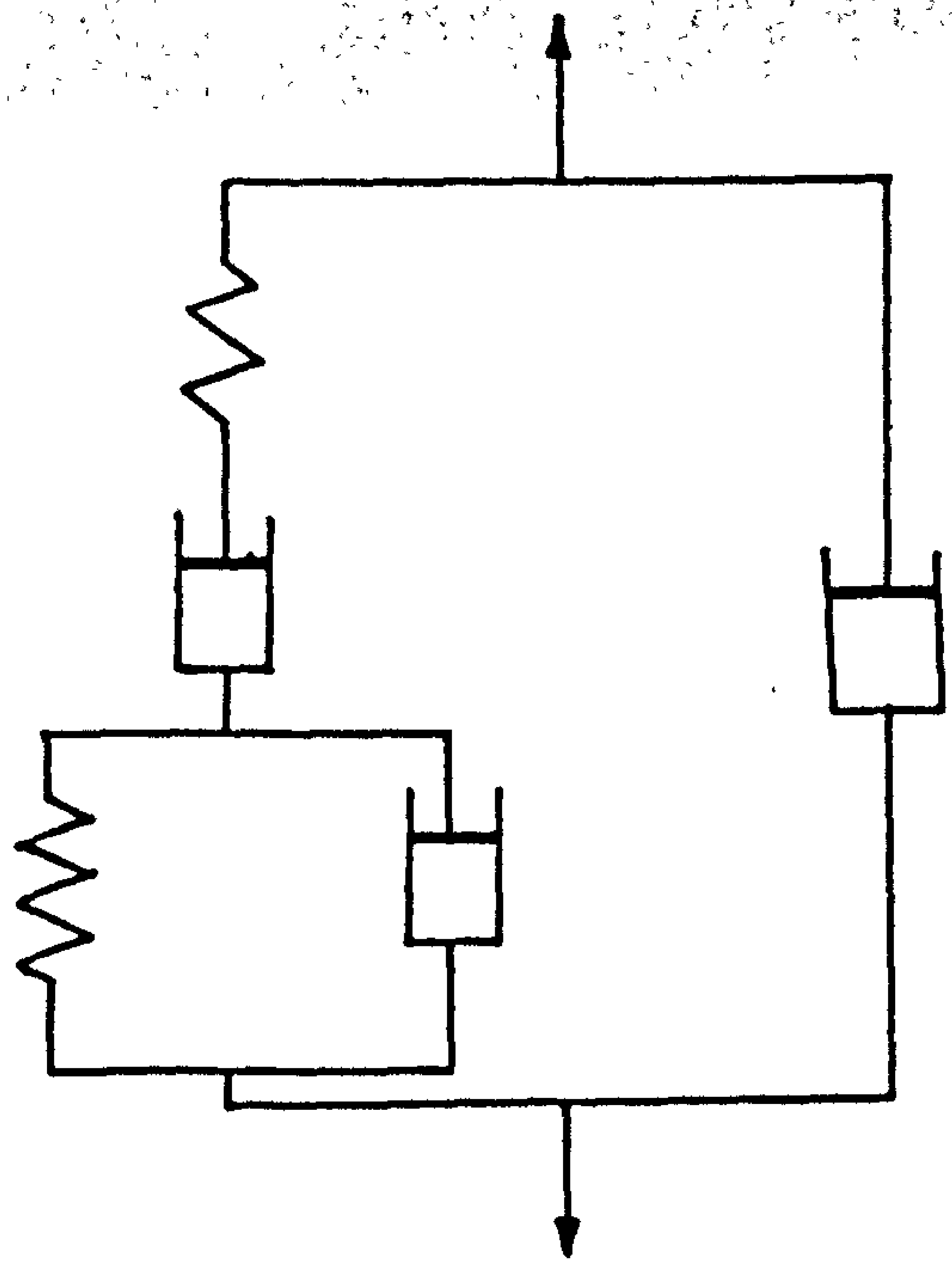


Fig. I.20. Model for Concrete (Hansen(54))

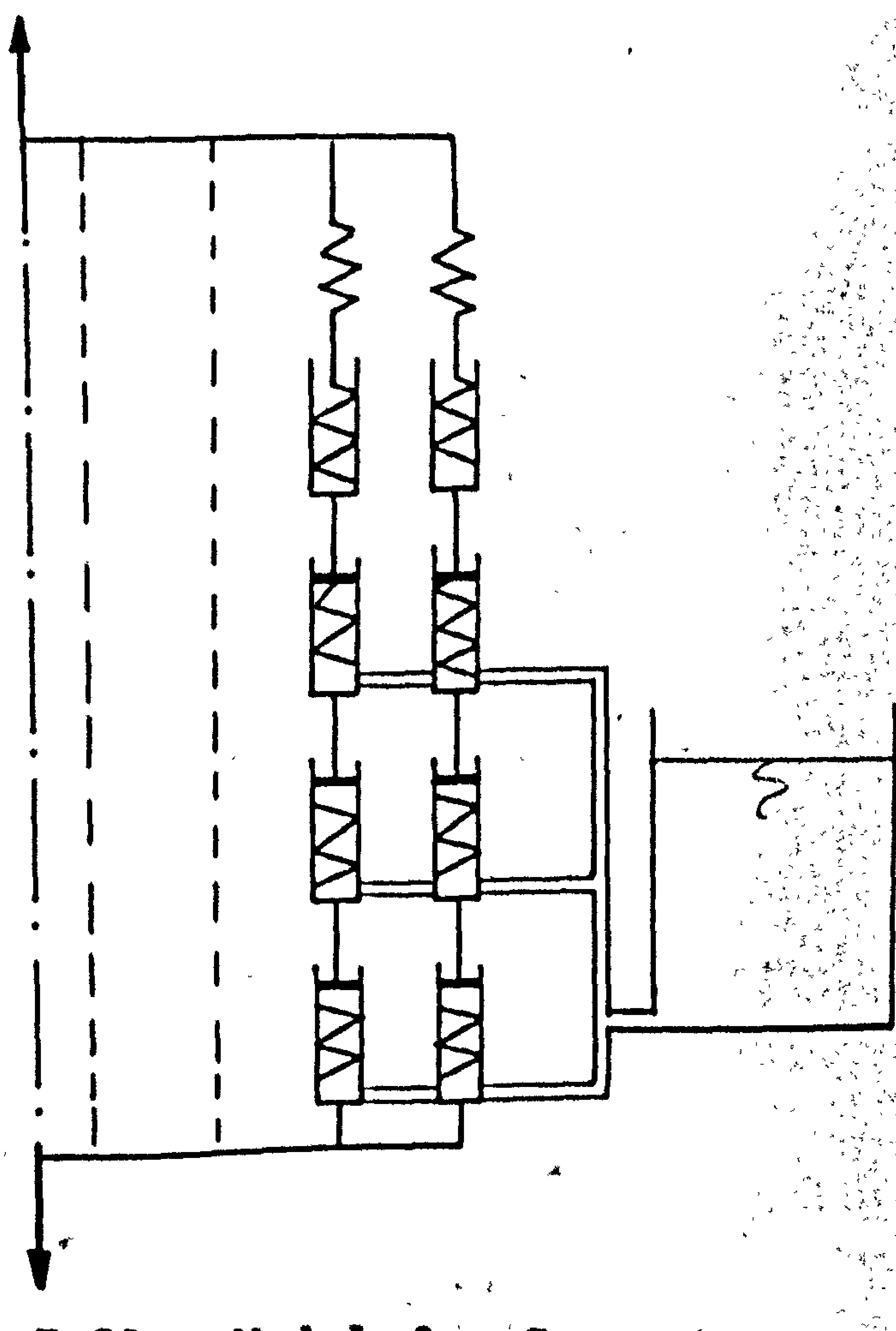


Fig. I.21. Model for Concrete (Toroja and Paez(55))

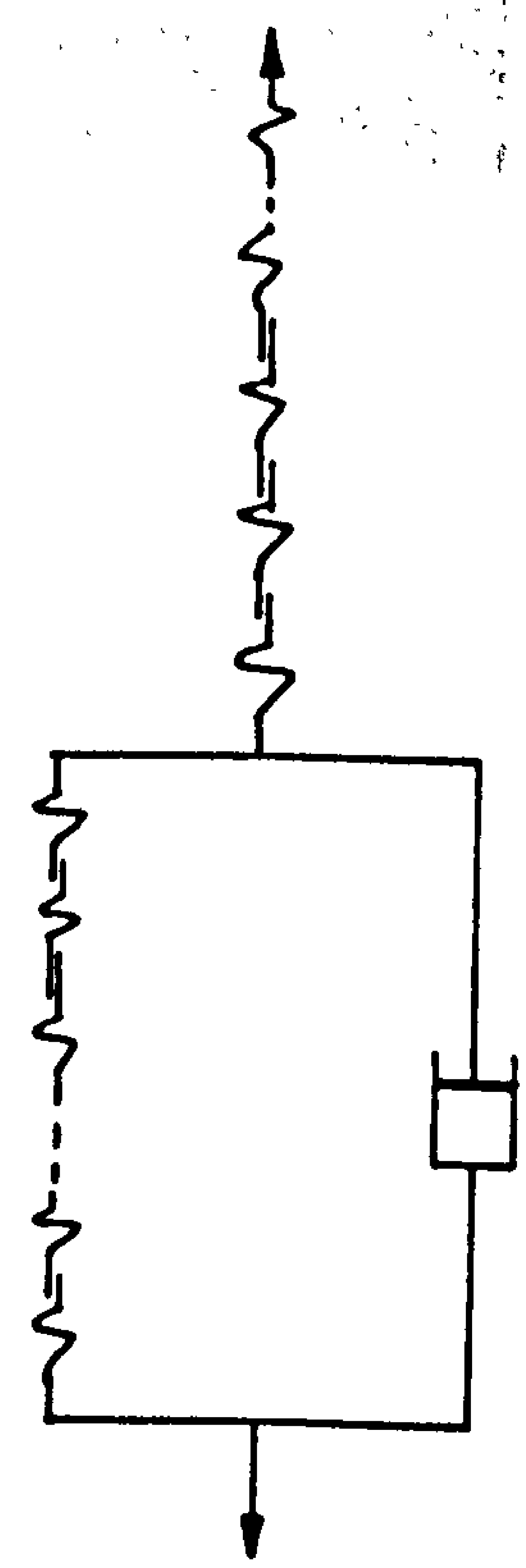


Fig. 22. Model for Concrete (Glucklich[59])

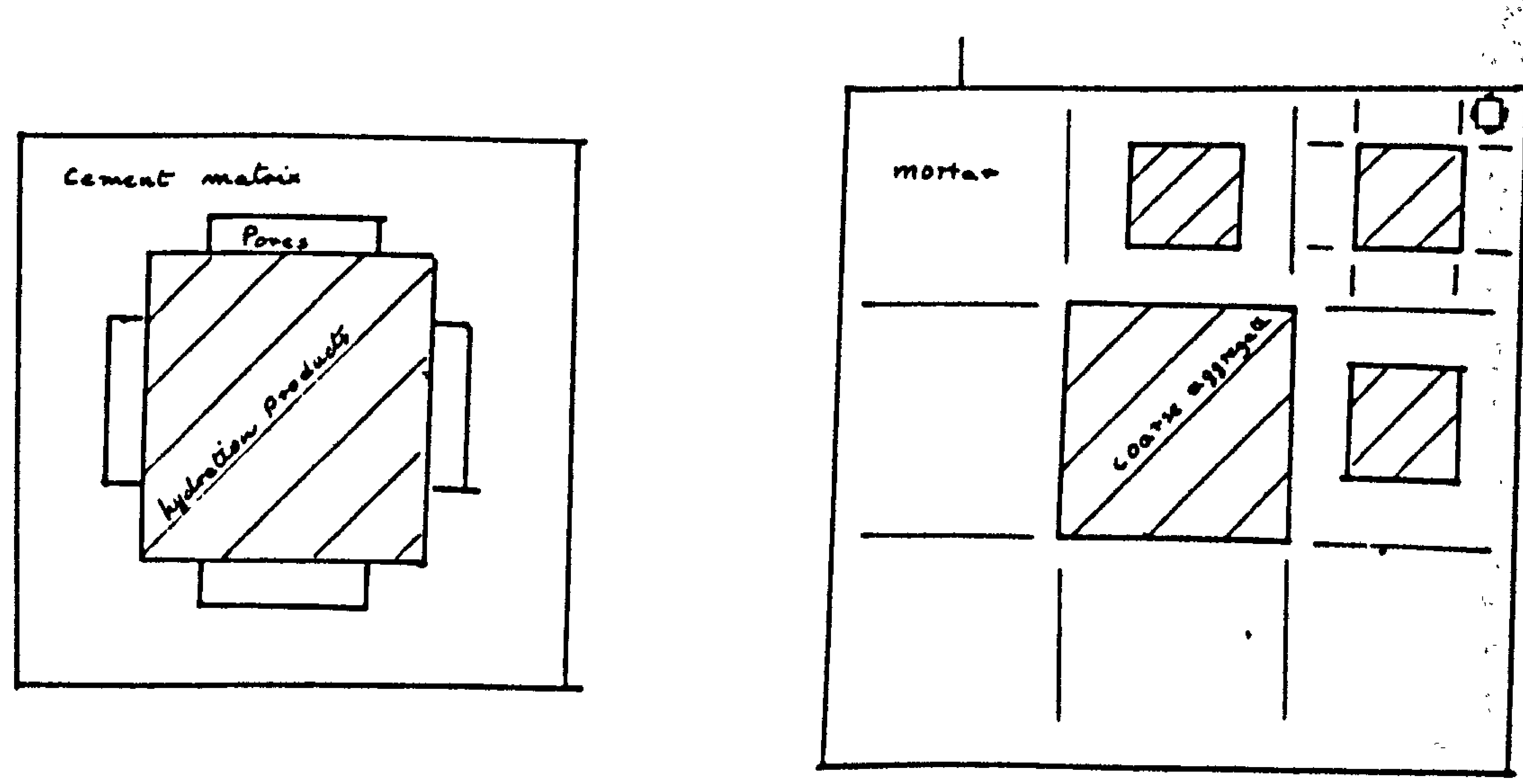


Fig. I.23. Solid Model for Concrete (Illston[58])

C H A P T E R II

SCOPE OF PROGRAMME

II. 1. INTRODUCTION

The lack of existing information on which a conclusion on the tensile creep behaviour may be based justified, it was thought, an explicit investigation into this property as well as other properties of concrete in uniaxial tension. Such a programme of investigations would provide useful practical information for the Engineers dealing with structure where tensile properties of concrete are directly used and should offer an important contribution to the better understanding of the complex nature of concrete.

At the outset of the present work the aim was to investigate the behaviour of concrete under long-term uniaxial loading. It was later extended to cover a study of the stress-strain relationship and the short and long-term strength of concrete.

II. 2. THEORETICAL CONSIDERATION

The theories of creep are generally examined against the experimental data widely available on compression. There is, however, no such data available on uniaxial tensile creep to allow for a similar examination.

For a theory to be valid it is thought that it must be able to explain tensile creep as well as compressive creep behaviour within its framework. The present programme was thus directed towards finding this data on uniaxial tensile creep and applying it to examine the various theories of creep. An accurate study of this property might help in deciding the validity of one or another of the existing theories and could lead to a new outlook on the problem involved.

The factors that may influence tensile creep, as in compressive creep, are very numerous. It has not been possible for the present programme to cover all the factors involved and has only coverd those which were thought to be the most important.

From the review of literature presented in Chapter I, it can be realised that water, whether as an intrinsic factor or environmental, plays a major role in determining the properties of concrete including that of creep. It has been established in compression, for example, that increasing the water-cement ratio or the water-aggregate ratio enhances creep, restricting the movement of water within the concrete reduces creep considerably and that creep almost ceases on the removal from within the concrete of most of its evaporable water.

The two most popular theories, namely seepage and viscous theories, are basically concerned with adsorbed water. The seepage theory clearly relates the creep phenomenon to the movement of water from and within the concrete and to the hygrometric equilibrium of the medium. Whereas, although the viscous theory relates the creep to the passive movement of the gel particles relative to one another, most of its supporting investigators seem to localise this movement to the layer of adsorbed water (which is only a few molecules thick) separating these particles. They also seem to believe that the thickness and diffusion of this water would evidently influence this movement of the gel particles.

The differential shrinkage theory does not relate creep directly to water movement but again, the movement of water from within the concrete causes the shrinkage and subsequently results in differential shrinkage between the core and the outside of the concrete specimen setting up the differential stresses on which the differential shrinkage theory is based.

The plastic theory is perhaps the only theory of creep that is directly independent, in its principle, from the water aspect. However, the water content would alter the structure of the cement gel, the degree of heterogeniety of the concrete, the strength and the internal stresses due to volume changes on hydration. The moisture condition of storage would also affect the state of internal stress and tend to re-arrange the distribution of stresses within the particles of the multiphase material. Thus, an indirect effect on creep based on a plastic theory of these influences is clearly visualised.

The present work has, therefore, been concentrated on the investigations of those factors relating mostly to this aspect of water in its broad definition. The effect of the same factors studied when investigating tensile creep were explored when investigating uniaxial tensile strength.

Special attention was given to the investigation of the effect of sealing and immersing on the tensile creep, short and long-term tensile strength of concrete and the modulus of elasticity.

The preliminary investigation on the stress-strain curve resulted in a large tensile strain and falling branch in the curve.

This curve was published by Evans [(69)] and also by Hughes[(68)] during the course of the present investigation and after the author had observed this effect.

Further investigation into this aspect showed that the falling branch could be due to a crack propogation and was the outcome of a shift of the axis of loading with respect to the uncracked part of the failing section.

II. 3. EXPERIMENTAL CONSIDERATION

The wide deviation of results and opinions in the reported literature on uniaxial tensile creep and indeed on all uniaxial tensile properties of concrete is perhaps due to the lack of an existing accurate method for the long and short-term loading of concrete up to failure in uniaxial tension.

This emphasized the need for such an accurate method in order to produce conclusive results and to avoid obtaining such deviation.

The development of the method occupied the first stage of the present programme. This method is presented in Chapter III.

The experimental investigation included:

- Investigation into tensile creep.
- Investigation into the short-term strength of concrete, cracking and the stress-strain curve, including the falling branch.
- Investigation into the long-term strength, of concrete, its relation to the short-term and the upper limit of positive strain.

1) Investigation into Tensile Creep

The experiments on tensile creep include investigation into the factors shown in Table II. 1.

Control Specimens

The shrinkage and swelling strains were measured

TABLE II.1

Main Variable	Range	Secondary Variable 1	Range	Secondary Variable 2	Range	Constant Conditions	Experiment No.	Remarks
Ambient Humidity	S . W . A	L.O.S. lb/in²	65 130	Time Under Load (Days)	30 140 — 600	A/C, W/C A.T, C.T, A.Gr A.A.L.	1, 2	2 - 3 Specimens for each Test
Age of Loading	1 3 7 28	A.H.	S W	-	-	A/C, W/C, A.T, C.T, A.Gr, T.U.L, L.O.S.	3	
Level of Stress	75, 100 185 90, 125 160	A.H.	W S	A.A.L. "Days"	3 20	W/C, A/C, A.T, C.T, T.U.L, A.Gr.	4, 5	
Water Cement Ratio	0.42 0.50 0.60	A.H.	S W	L.O.S	82 110 142 182	A/C, C.T., A.T, T.U.L, A.A.L.	6, 7, 8	
Aggregate cement Ratio	4.5 6 7.5	A.H.	S W	-	-	W/C, C.T, A.T, T.U.L A.A.L. A.G.	9, 10, 11	
Aggregate Type	Gravel, Crushed Granite	A. H.	S W	W/C	0.60		7, 11, 12	

ABBREVIATIONS

W/C : Water Cement Ratio
A/C : Aggregate Cement Ratio
A.H. : Ambient Humidity
L.O.S. : Level of Stress
A.T. : Aggregate Type
C.T. : Cement Type
A.A.L. : Age at Loading

S: Sealed
W: Immersed in Water
A: Air Drying
T.U.L: Time Under Load
A.Gr: Aggregate Grading

on control specimens for every mix and curing condition and these measurements were deducted from those made on the loaded specimens in order to obtain creep strains.

The strain development in all the specimens of a certain mix and curing condition were observed before loading commenced to check whether this development was similar for the specimens to be loaded to that of specimens to be used as control. When the rate of strain development showed different trends in certain specimens, these specimens were generally disregarded from that particular test.

Non-Destructive Checking

Specimens were generally checked non-destructively using the resonant frequency test. The differences between the readings of specimens of the same mix and curing conditions should not be more than those expected from a normal concrete fully compacted and vibrated for the same time. Specimens which gave readings outside the acceptable range of difference were generally disregarded.

The difference between control and loaded specimens was further minimised when investigating all parameters (except age at loading) by commencing loading at the age of about 28 days where the rate of strain in sealed and immersed specimens becomes very small.

Drying specimens were not generally used since better control could be achieved with sealed or immersed specimens when investigating most parameters.

2) Investigation into the Short-Term Strength of Concrete

The factors investigated in this series are mostly the same as those covered when dealing with tensile creep. Particular attention was given to the effect of sealed and immersed curing on the strength of concrete made with different mixes. This included the effect of immersing and drying on the strength of sealed concrete.

The specimens were generally tested at the ages 1 - 3 - 7 - 28 and sometimes 90 days. The relationship between short-term strength and the age of concrete was plotted for sealed and immersed specimens of different water-cement and aggregate-cement ratios.

Specimens in which accoustic strain gauges were embedded were also used and the stress-strain curves of the different concrete mixes and curing conditions were produced. The effect of sustained loading on the shape of the curves and on the onset of cracking was also investigated.

3) Investigation into the Long-Term Strength of Concrete in Uniaxial Tension

This investigation is conducted in two ways:

a) By loading specimens to different stress/strength levels and keeping the load sustained until failure occurred, and

b) By loading specimens to a certain stress (generally lower than the above) for a limited period and then loading to failure in the same manner as when investigating short-term strength.

The first of these was given priority and curves showing the relationship between strength and time under load as well as the relationship between maximum strains, time and strength were obtained for concrete made with gravel or crushed aggregate, cured wet and loaded while kept immersed. A few tests were also conducted on sealed concrete.

4) Investigation into the Complete Stress-Strain Curve Including the Falling Branch

It was noticed when testing some immersed specimens that large strains were recorded at stresses approaching the ultimate strength of the concrete. For this reason an investigation directed towards finding the ultimate strain which may be developed in the concrete before failure was carried out. Using the rig described in the next chapter and a special technique for loading the specimen an apparent falling branch in the stress-strain curve was obtained. Further work to corroborate this finding was made by mounting surface acoustic strain gauges at opposite sides of the specimens to observe the eccentricity that might develop after the peak of the stress-strain curve is reached and hence to find the true ultimate strain.

One type of concrete cured wet and loaded while still immersed was used in this series.

C H A P T E R   III

DEVELOPED METHOD FOR THE ACCURATE SHORT-AND LONG-TERM TESTING OF CONCRETE IN UNIAXIAL TENSION

INTRODUCTION

The direct tensile testing of concrete, although of considerable theoretical and practical interest, has generally been avoided by most investigators due to the difficulty encountered in applying the load uniaxially and producing failure within the desired zone. When investigating long term properties there is the additional problem that many specimens would be required under load at the same time and the economic factor and compactness of the test become very important. In many previous investigations or for acceptance tests it has been found more convenient to use indirect tensile testing techniques.

III. 1. REQUIREMENTS OF METHOD

For long-and short-term studies of the behaviour of concrete in uniaxial tension, as for the present programme of work, the following requirements were considered to be necessary:

1. The load should be truly axial so as to produce uniform stress across the section.
2. The arrangement should be suitable for both short-and long-term loading of concrete up to failure.
3. The end attachment should be easily fixed to wet, dry, or sealed specimens.
4. The end attachment should be simple and recoverable.

5. The cost of making specimens and end attachments should be low.

6. The system should allow specimens to be subjected to wet, dry or sealed conditions when under load.

7. The strain measuring system should be stable for a long period of time.

8. The loading arrangement should ensure that the load is maintained at a constant value as time-dependant strains occur.

III. 2 EXISTING METHODS

When the present programme began in 1965 a survey was made of the existing methods of loading specimens and measuring strains in tension. Methods of testing concrete in direct tension have been reported in sientific literature for many years, indeed in 1865 Grant[70] devised a method for testing briquettes. Many other methods have since been devised and they were recently summarised in a RILEM[71] report. In this report the methods in use in various laboratories were divided into four different categories, depending upon the method of clamping the end of the test piece, as follows:

1. by means of embedded steel bars,
2) by lateral gripping,
3. by gluing,
4. by means of wings or truncated cones.

The survey made of the existing methods resulted in the following conclusions:

a) The traditional briquette test was most unsuitable in its present form as it produced large variation of results. According to Evans(40) "The strain on one side of the briquette was sometimes as much as double that on the other side, with the result that the tensile strength is reduced considerably".

b) The embedded bars technique required elaborate work to ensure good bond and central location of the bars. A more suitable setup was made by Illston(65) using a steel spider cast into the concrete and located by means of special jigs. This, however, was still complicated and unless the shape of the specimen was enlarged at the end, failure would be encouraged to take place at the weak area at the end of the spider. Also, any use of embedded steel may require, from a practical point of view, the wasting of the embedded pieces with the used specimens. The specimen also requires extra length sufficient to accommodate the embedded bars and to provide the necessary clearance between the end of the bars and the tested zone in order to produce flat ends in the specimen which adds more complication to the technique.

c) The gluing technique (i.e. applying the load to steel plates stuck to the two ends of the specimen) has recently been used by many investigators(72,73). It was, however, tried by the author at the beginning of the programme and found unsatisfactory for the following reasons:

i) The intense care needed for preparing the surfaces of the concrete and the steel before gluing them.

ii) The glue did not adhere readily to wet surfaces and, even with the recent improvement made by Hughes & Chapmen(73), the final rate of success was no better than 67%.

iii) The comparatively long time required for the adhesives to harden during which specimens may have to be stored under different conditions from those required by the investigation.

iv) Cleaning the steel plates is very difficult.

v) The failure is likely to take place at the end of the specimen unless it is shaped to avoid this possibility.

vi) The possibility of bond failure or failure in the disturbed region of the concrete at the interface with the steel plate always existed and would require special precaution when subjecting the specimens to sustained loading (and especially dead weight loading).

d) The lateral clamping technique reported by Ward[74] to be satisfactory for short-term loading would be very costly and impractical when large numbers of specimens are loaded simultaneously for a long period of time.

An interesting technique was used by Ross[43] where he applies the load through internal pressure to hollow concrete cylinders. This technique would overcome the eccentricity (if the thickness/diameter ratio is small enough) and the possibility of end failure. But the difficulty encountered in the strain measurements, the undefined zone of failure, the relatively large diameter required, and the cost and complication would outweigh the advantages when large numbers of specimens are to be used.

III. 3. BASIS OF PRESENT METHOD

A consideration of the shapes of specimens available led to the selection of a circular cross-section as the most suitable and simplest to set up for concentric loading. The desirability of sealing some specimens from the time of casting and the need to produce a large number of specimens suggested the use of disposable moulds.

The present method consists of using cylindrical specimens tapered outwards at their ends with end caps tapered internally to follow the shape of the specimen ends, leaving an annular space which can be grouted to provide a positive grip under tension as shown in Figures III.1. & III.2.

After a trial of alternative arrangements, it was decided that p.v.c. tubing which was available in the form of rainwater pipes could be easily modified to provide disposable moulds. These moulds could also be used to provide an efficient sealing arrangement for those specimens which were required to remain sealed.

The shape of the specimen, shown in Figure III.1. satisfied the requirements listed above. Each end of every specimen is cast against a steel plate in which an accurately positioned circular recess has been machined. This forms a slight protrussion on the end of the concrete which ensures accurate location of the steel end caps used for applying the tensile load. These end caps are shown in Figure III.2.

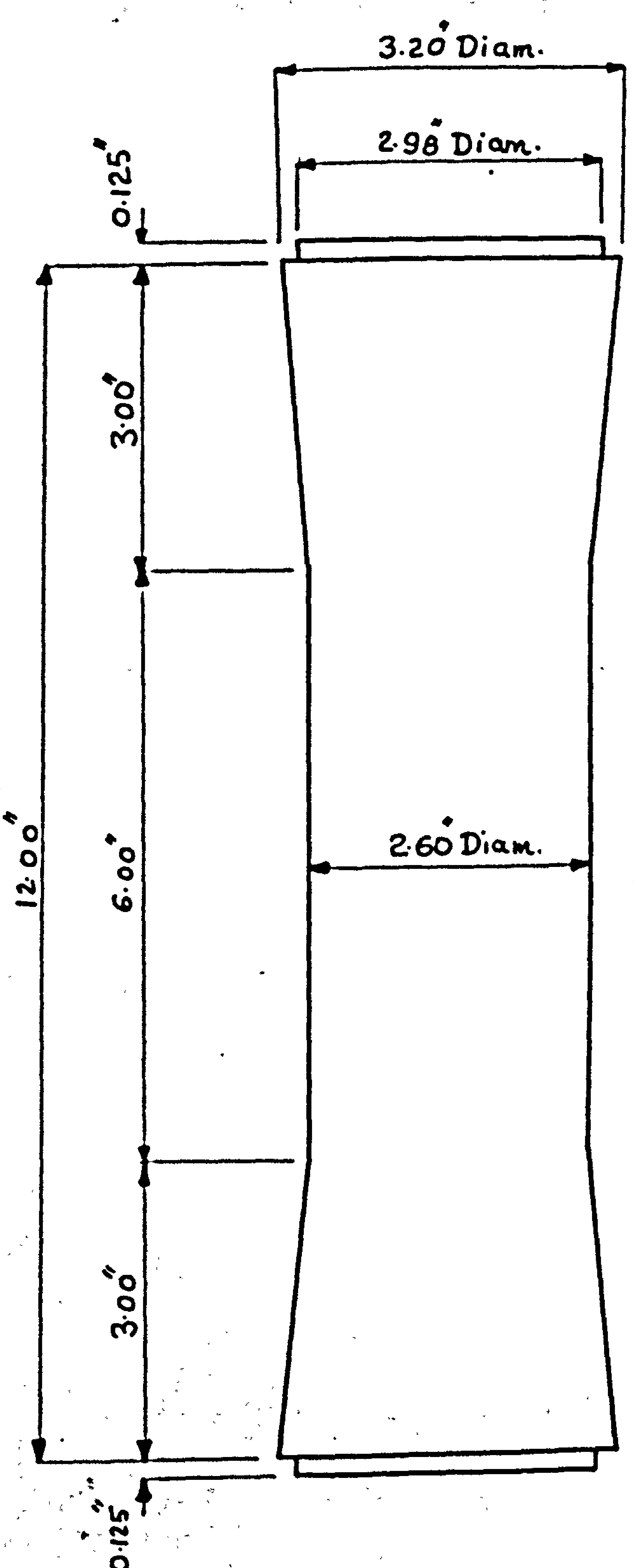


Fig. III.1. Shape of tensile specimen

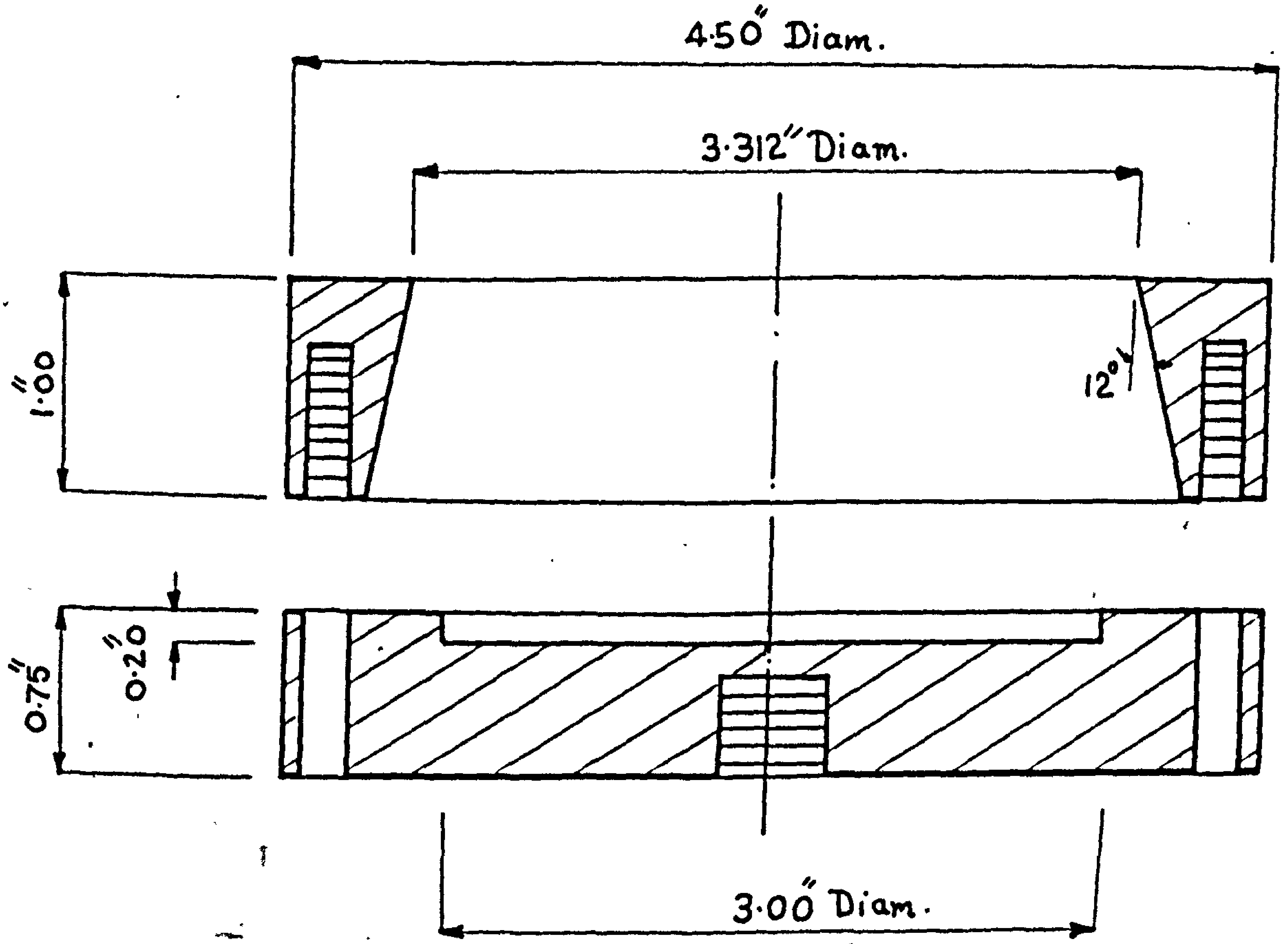


Fig. III.2. Cross section through end cap.

Flexible Bowden cables have been chosen to apply load to the end caps because these minimise end movements. Convenient screwed connections are readily available for these cables.

Vibrating-wire strain guages were considered to be the most appropriate means of measuring creep strains and the R.R.L. type[75] was found to be very suitable for embedding in the centre of specimens along the longitudinal axes.

A dead load system is used to apply tensile load through a simple lever. A rig has been built to enable up to three specimens to be connected in series to each lever system. This arrangement is simple and reliably maintains a constant load independently of the development of strain in the specimens.

III. 4. MOULDS FOR SPECIMENS

A suitable $13\frac{1}{4}$ in. length of $2\frac{1}{2}$ in. p.v.c. rainwater pipe was cut and formed into the required shape over two steel formers used successively while one end of the pipe was immersed in boiling water. It was found necessary to use two formers of slightly different sizes to allow for the slight shape recovery which occurred on cooling.

The two were identical except that the second former (No. 2) was 0.020 in. narrower than the first former (No. 1). Each former consisted of a tapered base and a rod fixed accurately to its centre along the perpendicular axis. Photographs of these formers are shown in Figure III.3.

In order to control the direction of deformation and to apply a steady axial load to the p.v.c. pipe, a weight, (No. 3) with a central hole through it was placed with a sliding fit on tb the rod.

This weight consisted of a solid steel cylinder of a diameter slightly smaller than the p.v.c. pipe.

A second weight (No. 4) of the same shape and dimensions as the base of former (2) was used when the second end of the pipe was being shaped. A photograph of these weights is shown in Figure III.3.

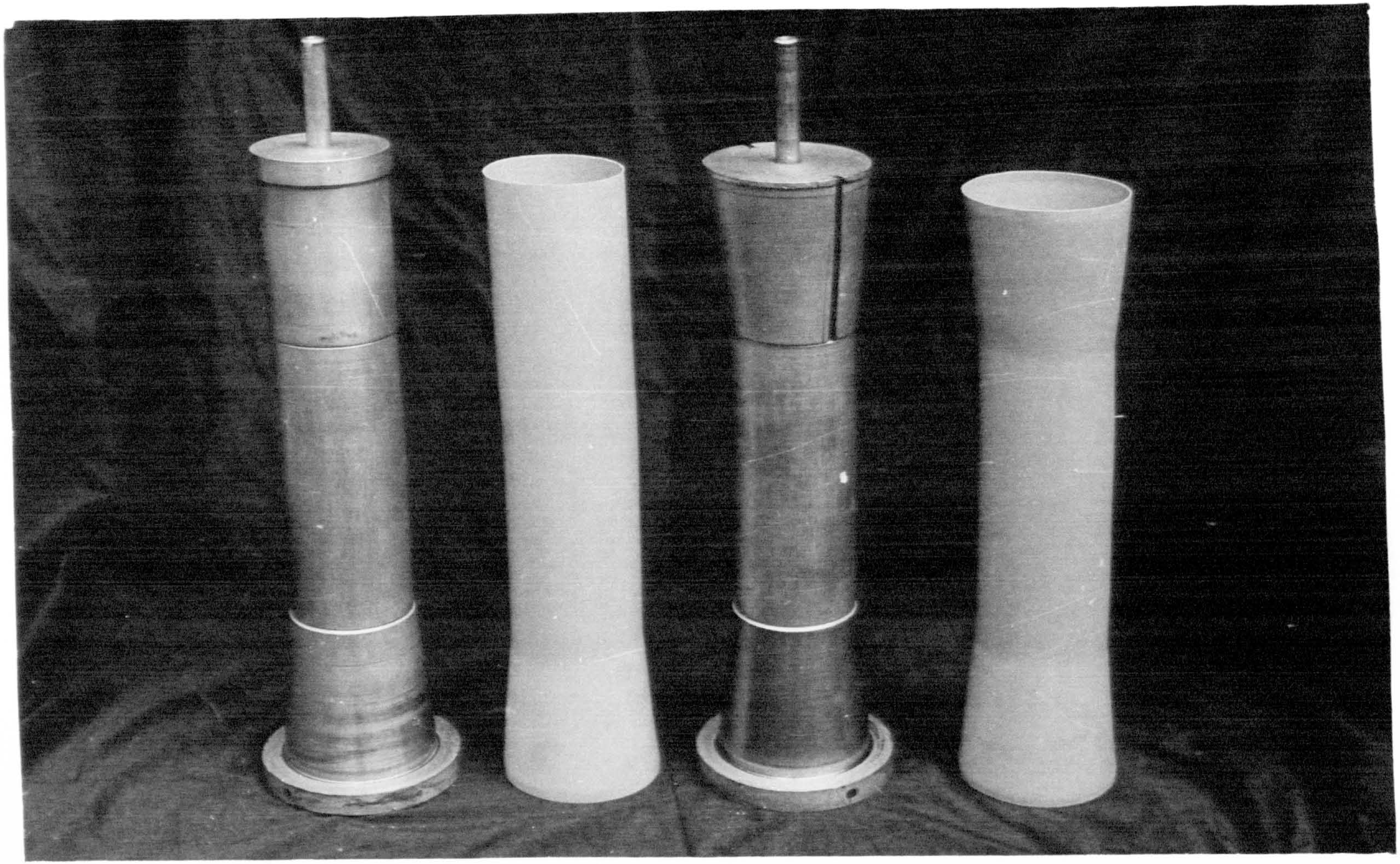


Fig. III.3. Former for shaping p.v.c. moulds.

<u>The Procedure for Forming the p.v.c. Moulds is as follows:</u>

Former No. 1 is first placed in the boiling water. The length of p.v.c. pipe is placed over it and weight No. 3 is placed over the pipe guided by the rod as shown in Figure III4. Additional pressure could be added by hand as required. When the pipe reaches the bottom of the former base the whole assembly is removed from the boiling water using an insulating handle screwed to the top of the rod. The pipe is then cooled slightly by cold water and placed over former No. 2 to cool further, with weight No. 3 still in position.

Former No. 1 is then returned to the boiling water and the pipe that has already been shaped at one end is reversed, placed over former No. 1 and weight (No. 4) is placed over it guided by the rod, as before. The $13\frac{1}{4}$ in. length pipe reduces, after shaping, to about $12\frac{1}{2}$ in. The ends are then cut square on a lathe to the required 12 in. length.

The average time taken to produce each mould, as part of a continuous production run, is not more than two minutes. Material cost at the time of writing is about 1/9d.

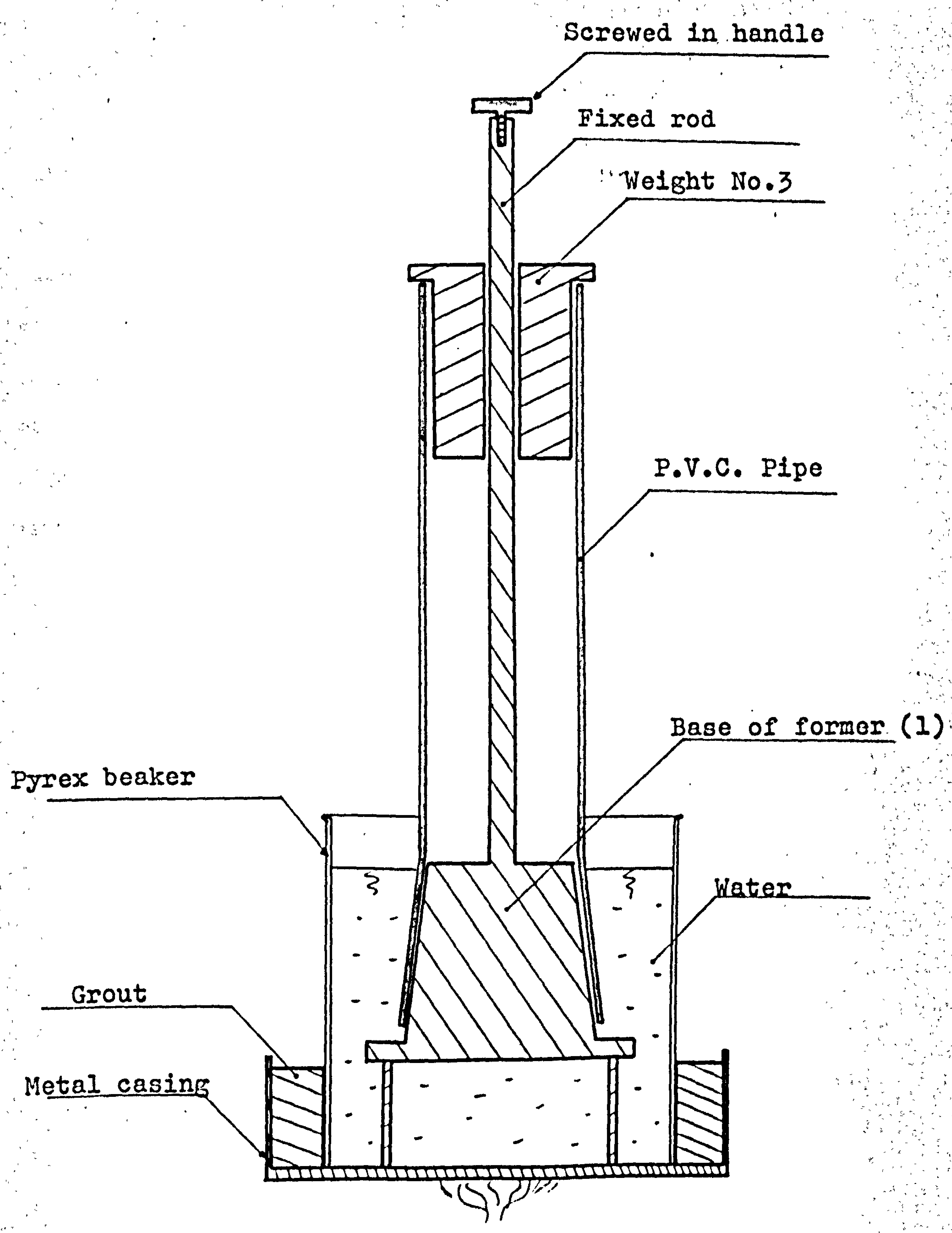


Fig. III.4. Details of mould making.

III. 5. CASTING THE SPECIMENS

To enable the specimens to be cast by vibration and to allow internal strain guages and ends to be positioned accurately, the moulds are held firmly in frames which can be clamped to a vibrating table. These frames can accommodate four moulds and consist of a steel base-plate carrying twelve vertical rods which are connected, in groups of three, to four sets of top and bottom rings, each pair of rings being used to hold a p.v.c. mould firmly in position (see Figure III.5). The base plate and rings are recessed to form the protrusions on the end of the concrete specimens.

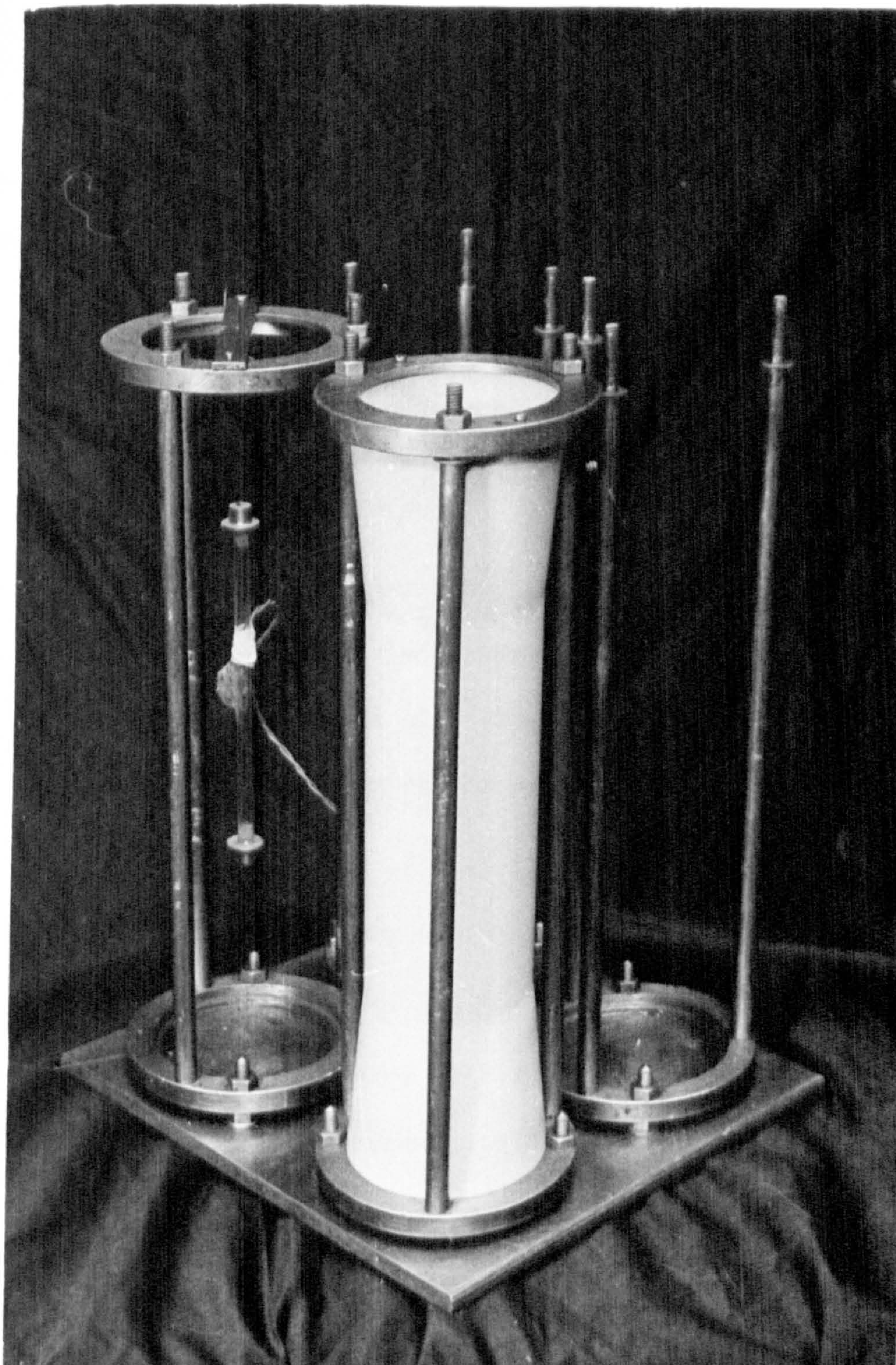


Fig. III.5. Frame for clamping p.v.c. moulds during casting.

III. 6. STRAIN GAUGES

R. R. L. type[75] vibrating wire guages to be embedded along the axis of the specimens were chosen as means for measuring the strain for the following reasons:

1) As the tansile strains are considerably small the accuracy of the guage becomes very important. The R.R.L. type guage offered a very good accuracy (strains as low as $\frac{1}{2} \times 10^{-6}$ could easily be detected).

2) Once the concrete has set the guage will be held firmly inside the concrete by the steel flanges and no movement can take place except that due to the deformation of the concrete.

3) The likelihood of the different human errors involved with positioning a demountable gauge at various times is eliminated.

4) The gauge is very stable with time and unlike the resistance strain gauge it suffers no drift.

5) Since the gauge may be positioned along the axis inside the specimens, measurements of average strains are recovered in spite of any variation in strain across the section.

6) Once the gauge has been cast it is very easy to use, and specimens need not be touched or removed from where they are stored (e.g. water tanks) for taking strain measurements.

The gauge has very small overall stiffness and introduces only a negligible amount of interference with the homogeneity of the concrete.

The flanges of the R.R.L. type gauge developed by Potocki[75] were reduced in diameter so that their effect on the properties of the concrete specimens was minimized. All parts of the modified gauge were made in the laboratory and assembled by the author, so the cost was reduced considerably.

The gauge had an overall length of $5\frac{1}{2}$ in. over which a high tensile steel wire of 0.01 in. size was stretched. The energising coil was fitted in a recess of 0.08 ± 0.005 inches deep at the centre of the tube. This arrangement left a distance of 0.04 to 0.05 between the coil and the wire. The gauge is shown in Figure III.6.

The gauge works on the principle that the variation of tension in the wire due to the movement of the flanges holding its ends, alters the frequency of lateral vibration which is set and detected electrically by means of the energising coil.

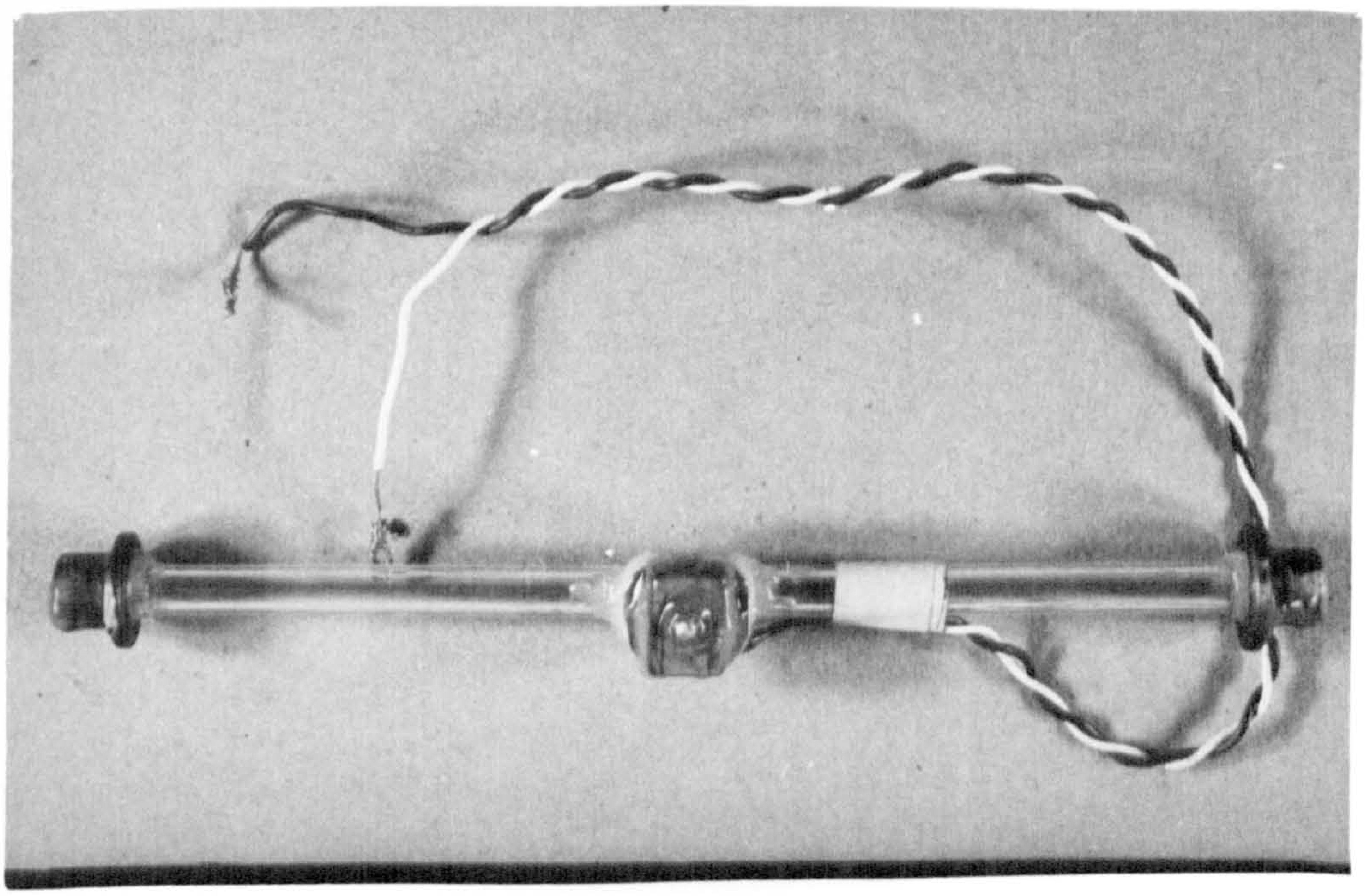


Fig. III.6. Modified R.R.L.[75] type accoustic gauge.

Fig. III.7. Specimen in place in end cap before and after grouting.

The gauge is connected to a "Maihak" instrument in which the frequency of the guage wire is compared with that of another similar wire whose tension can be adjusted by a micrometer screw device. The change of strain is detected by a comparison between the gauge wire and the reference wire.

As an alternative, a Deakin-Phillips instrument was used to record the period of time for the wire to vibrate for either 100 or 1,000 cycles. The strains could then be obtained using the tables provided by the manufacturers.

III. 7. END ATTACHMENTS

The end caps are made in two parts as shown in Figure III.2. This allows them to be removed readily from discarded specimens and cleaned for re-use. One part of each end cap has a circular recess on one side and a threaded hole on the other side, both accurately located. Details and dimensions are shown in the Figure.

The specimen is located in the recess by means of the protrusion on its ends and the annular gap between the specimen and the inside of the ring part of the end cap is packed with grout or mortar (see Figure III.7) A suitable mortar can be made with Prompt cement*, sand (passing a B.S. No. 25 sieve) and water in the proportions 1:1:0.5. This mortar hardens rapidly enough to enable the specimens to be handled within 10 minutes and tensioned after about 30-60 minutes. It is, however, advisable to wait longer when testing strong concrete.

The diameter of the end caps is 2 inches larger than the smallest part of the specimen and this conveniently allows a water-jacket to be fitted around the specimen when required. This can be made from transparent plastic sleeving stretched over the outside of both end caps and fastened with either adhesive tape or rubber rings.

* Obtained from Prompt Cement Products Limited, London, S.E.5.

III. 8. LOADING ARRANGEMENTS

A frame was built to accommodate up to 30 specimens for long-term studies of tensile behaviour. These specimens are suspended in 10 lines with 3 specimens in each line, as shown in Figure III.8. The weight of the lower specimens acting on the upper specimens is negligible compared with the applied loads and can be accounted for when loading concrete at very early ages.

The load is applied to the lines by suspending weights from the ends of the levers located at the bottom of the frame. Each load line was calibrated by temporarily suspending a proving ring in place of one of the specimens.

For the short-term testing to failure the assembled specimens and end attachments could readily be placed in a tensile testing machine. A special loading rig was modified to be used for short-term testing (see figure III.9). The rate of loading was maintained constant by using a small lamp fixed near the ring and this flashed at intervals of 2, 4 or 8 seconds while the load was applied manually in equal increments following the flashes.

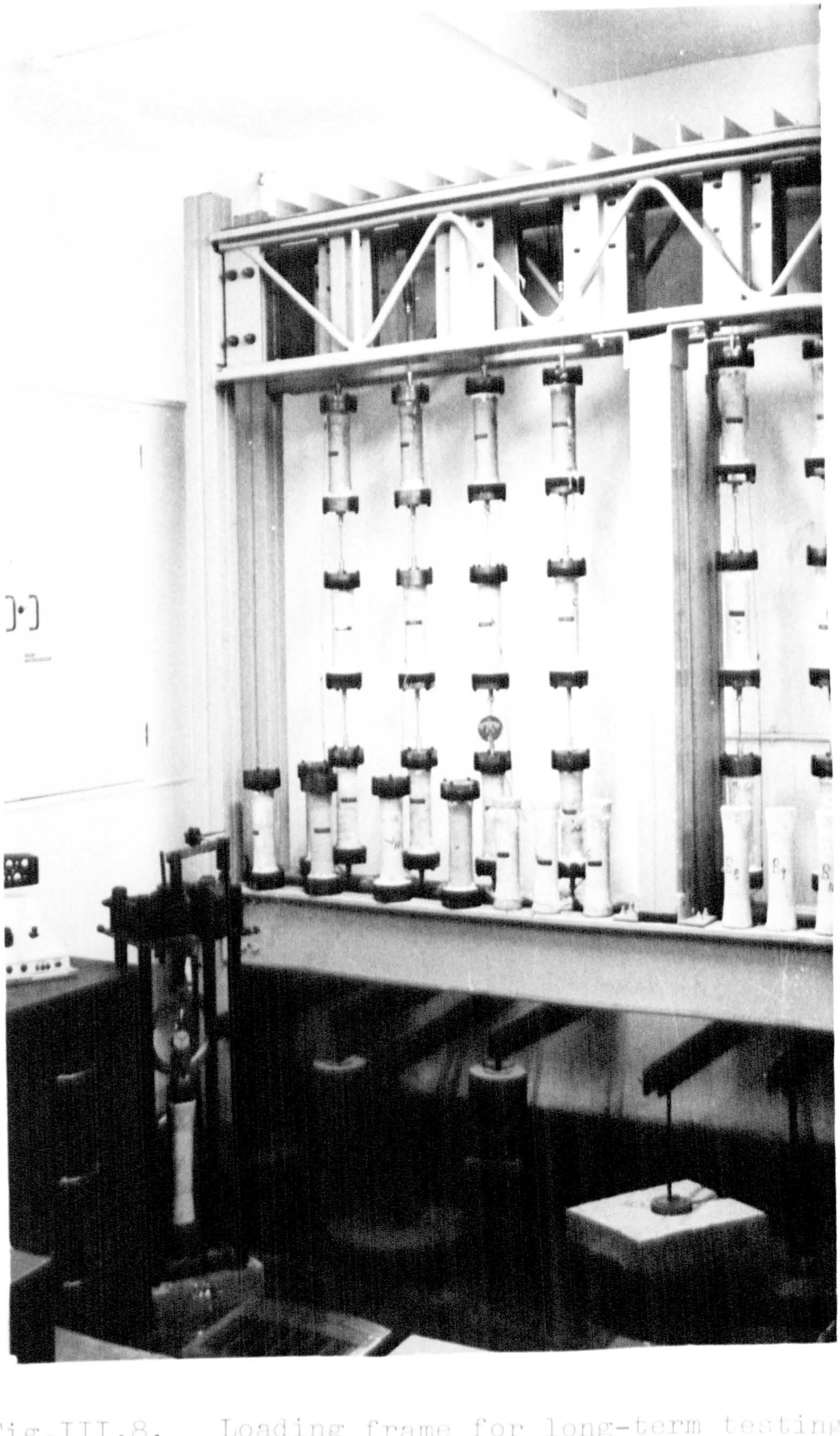


Fig.III.8. Loading frame for long-term testing.

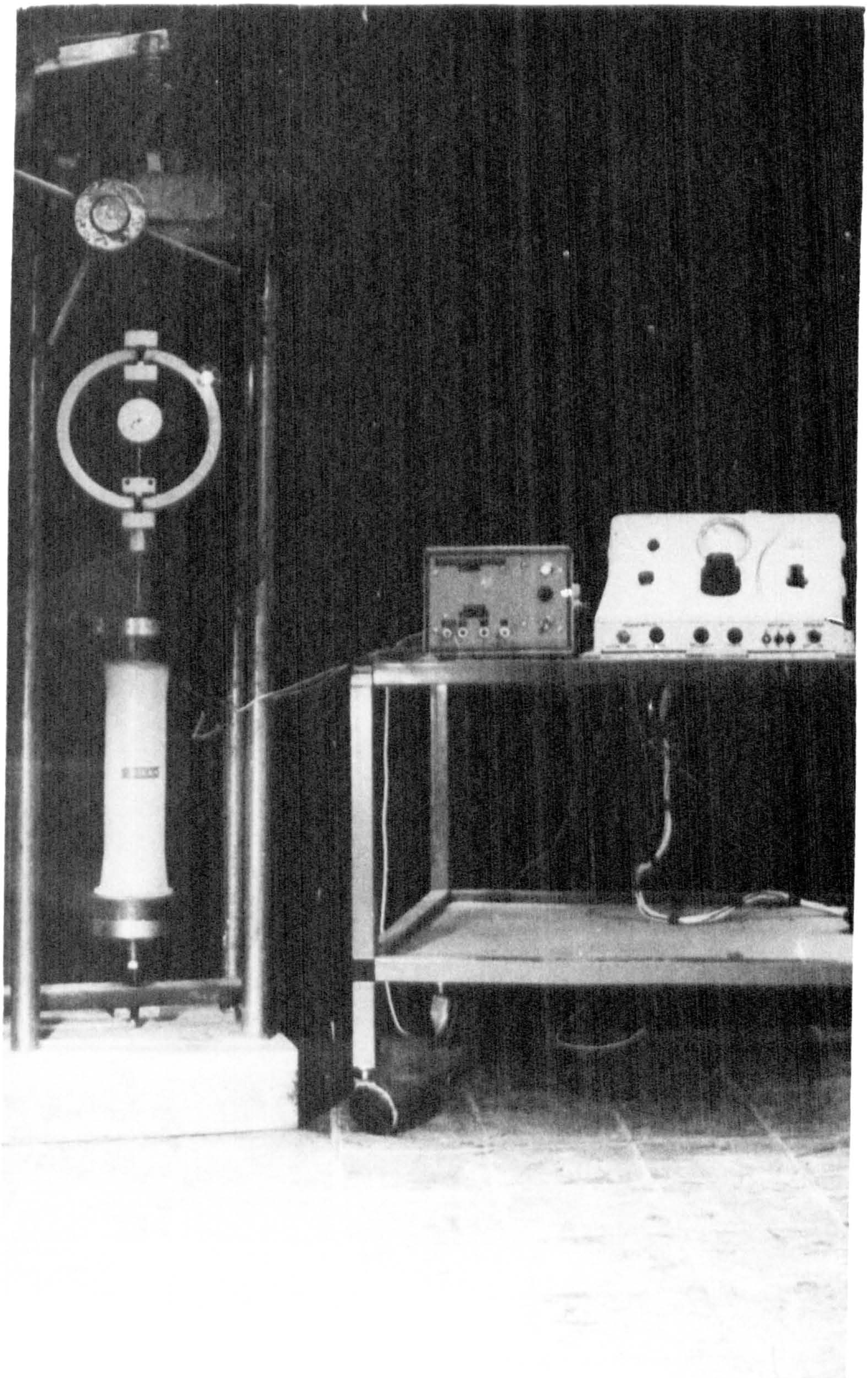


Fig. III.9. A rig for short-term testing.

III. 9. PERFORMANCE OF METHOD

During the course of development of this method, its performance has been judged by four particular considerations, namely the location of the fracture with respect to the length of the specimen, the precision of the loading position relative to the axis, the variation in test results and practicability of operation.

In the early stages of development about 40% of the specimens tested to failure fractured near one of the end attachments. It was, however, realised that the end failure occurred least when testing concrete that had been cured in water until before testing and most when testing specimens that had been cured, sealed in their p.v.c. jackets. In the latter case, prior to connecting the ends, a ring of the p.v.c. was cut on the top and bottom of the specimens to allow the grout to be placed.in contact with the concrete. This however wetted the concrete locally near the end and seemed to have contributed to end failure. Also, at that time the inside of the end caps was not greased and so movement between the end caps and the concrete specimens when load was applied might have taken place between the concrete and the grout, instead of between the grout and the steel. This reduced the effective surface area to which the load was being transfered from the end caps, thus increasing the circumfexential compressive stresses. Under these conditions it is probable that the combination of compressive and tensile stresses locally may have developed a higher principle tensile stress than that occuring at the centre of length of the specimen.

The end failure was overcome by increasing the internal angle of the end cap rings, by using a thin film of grease on the inside of the end caps and by using Prompt cement as described.

It was also found after this modification that it was not necesary to cut a ring of the p.v.c. tubing near the end before connecting the end caps. The grout could be packed directly between the cap and the p.v.c. tubing as the degree of contribution to the strength that the p.v.c. can offer within the small strains is negligible. This arrangement, however, provided very effective sealing.

Measurements of strain by external vibrating wire gauges fixed on opposite sides of the specimens have confirmed that the effective eccentricity of the load was very small. This was not particularly easy to measure accurately because the estimation of eccentricity from surface strain readings depends upon the difference between two measurements made on opposite sides of the specimens. Small errors are introduced into the readings by the surface contact conditions between the gauge and the specimen and these errors tend to exagerate the estimate of eccentricity. However, in spite of this, it was clear that the maximum eccentricity is not more than 1% of the diameter of the specimen and this would lead to errors in estimating surface stress of less than 4% of the average tensile stress in the specimen.

The curves that have been obtained for the study of relationship between strength, strain time, water/cement ratio and other parameters presented in the following chapters indicate that the method gives results with remarkably low variation compared with that expected from most of the established methods for the mechanical testing of concrete.

The method is thought to be practical in the sense that end caps could be connected to sealed, dry or wet concrete, the procedure of connecting the ends is simple, relatively quick and requires no special jigs for the alignment. Sealing is effected by leaving the p.v.c. jackets on and these can easily be cut away using a circular saw. Immersion in water while under load could be done using the plastic tubing already described. Finally if the specimen is not to be tested after connecting the ends, these could be disconnected without causing any damage to the specimen.

Fig. III.10 & 11 show typical failures in gauged, ungauged and sealed specimens.

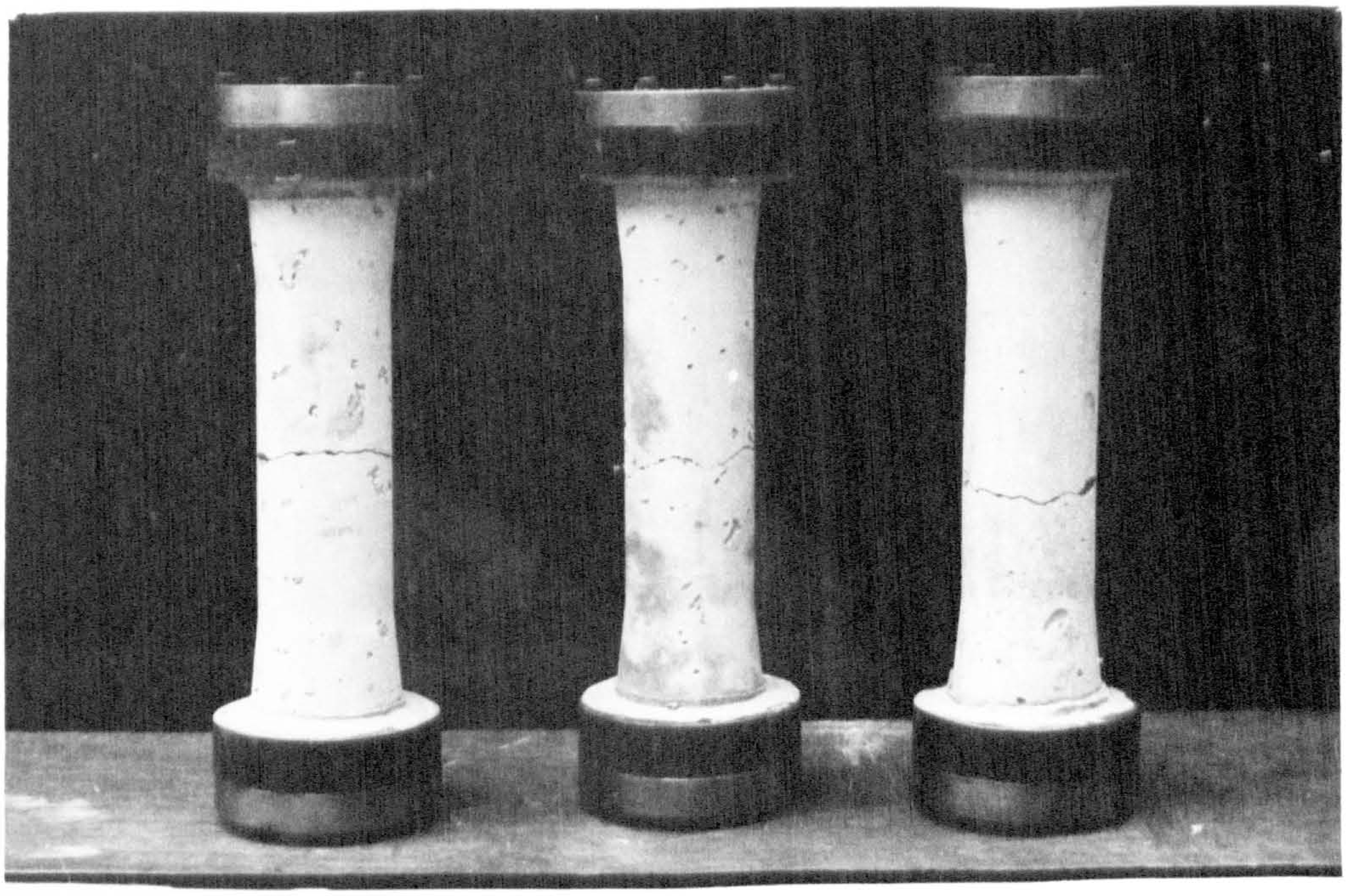


Fig. III.10. Typical tension fracture in specimens.

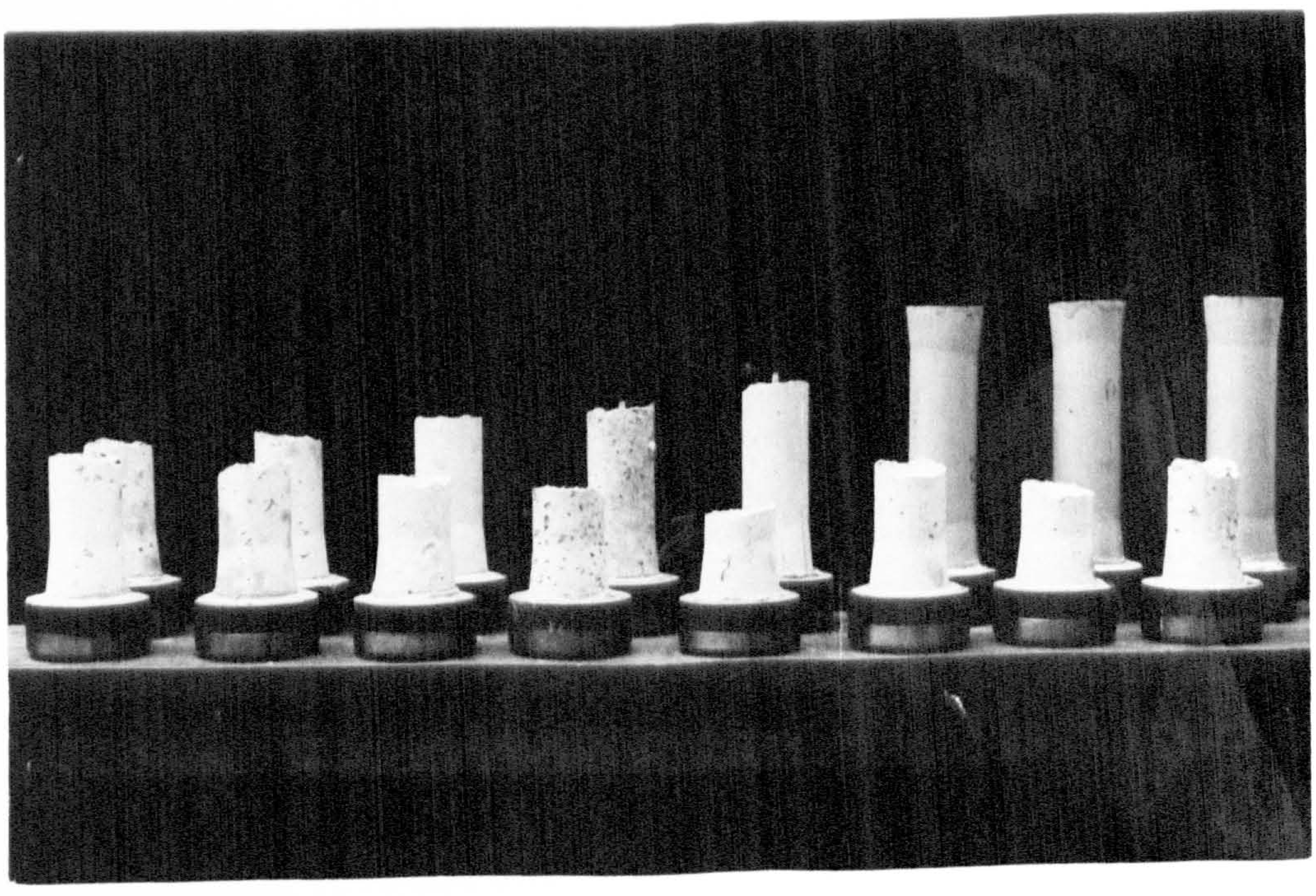


Fig. III.11. Typical tension fracture in gauged and ungauged specimens.

CHAPTER IV

MATERIALS, MIX DESIGN AND CREEP MEASUREMENT

INTRODUCTION

The method developed for loading the specimens in uniaxial tension has been presented in the preceding chapter. A brief description of the casting procedure, materials and mix design used throughout the test programme will be discussed below.

IV. 1. MATERIALS

IV. 1. a. Aggregate

Washed Ham river gravels were used mainly throughout the test programme. Crushed granite aggregate was only used for two mixes when the effect of type of aggregate on creep was investigated, but when not stated the former was used. The maximum size aggregate used was $\frac{3}{8}$ in. The grading followed that which corresponded to No. 1, 2 and 4 of the Cement and Concrete Association curves shown in Figure IV.1.. Curve No. 2 was used by most of the mixes.

IV. 1. b. Cement and Water

A single batch of ordinary Portland cement "typical cements" stored in air-tight steel drums was used throughout the test programme. The cement quality satisfied the B.S. 12:1958 requirements.

The following strength results were provided by the Blue Circle Cement Company who supplied the cement in special batches selected as "typical":

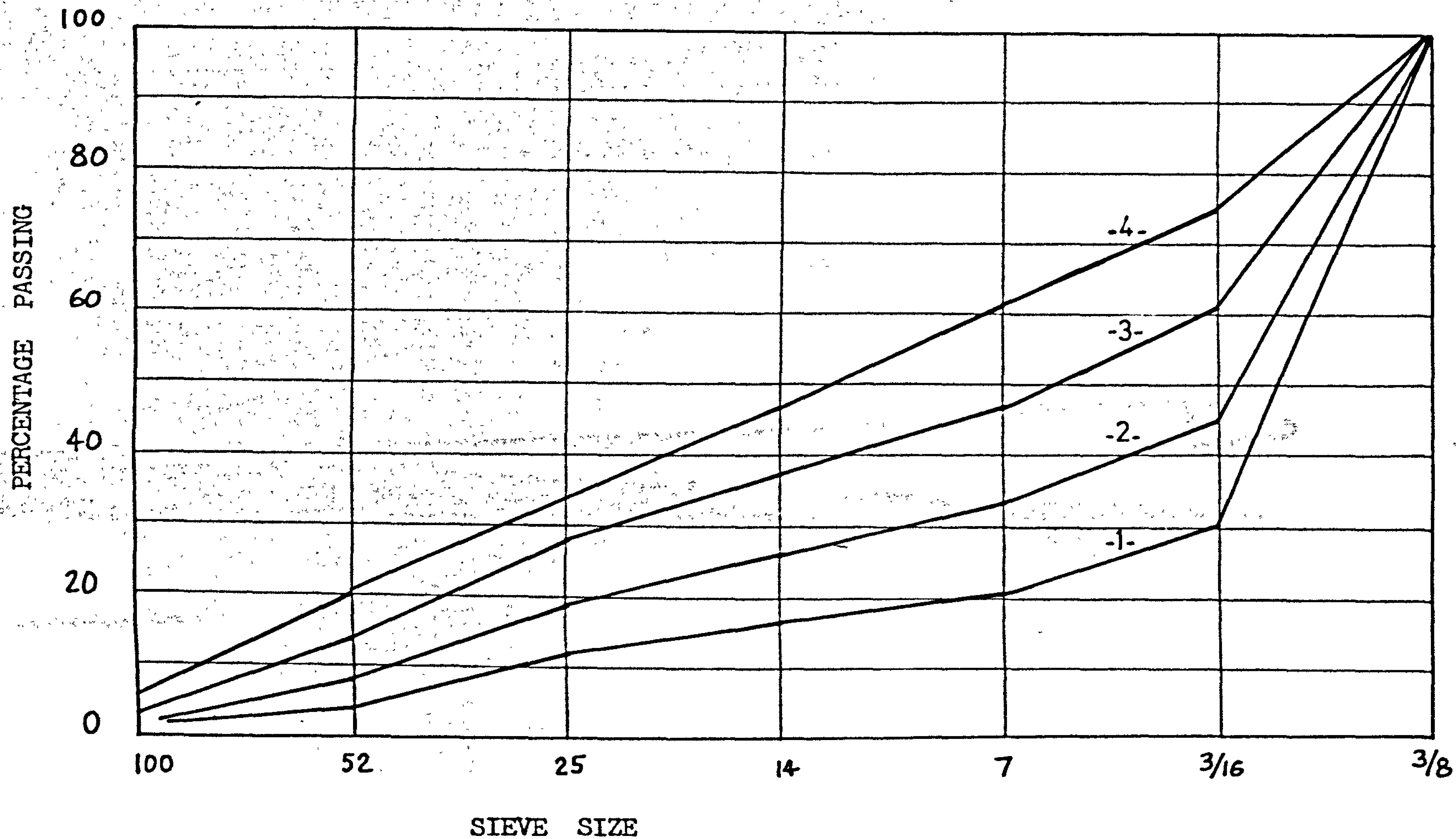


Fig. IV.1. Cement and Concrete Association Standard Curves for $\frac{3}{8}$" aggregate (From Text Book "Concrete Practice" by R.H. Elvery(76))

Vibrated Mortar (lb/in^2)

3 days	:	4260
7 "	:	6185

Concrete Compressive Strength (lb/in^2)

3 days	:	2230
7 "	:	3215
28 "	:	5130

Ordinary tap water is used in all the mixes.

IV. 2. MIX DESIGN

Eight mixes have been employed which included the following:

Mix 1	-	1:4.5, 0.50, Gr. No. 2.,	Gravel
2	-	1:4.5, 0.42, Gr. No. 1.,	"
3	-	1:4.5, 0.60, Gr. No. 2.,	"
4	-	1:4.5, 0.70, Gr. No. 4.,	"
5	-	1:6, 0.60, Gr. No. 2.,	"
6	-	1:7.5, 0.60, Gr. No. 2.,	"
7	-	1:4.5, 0.50, Gr. No. 2.,	Crushed Granite
8	-	1:4.5, 0.60, Gr. No. 2.,	" "

IV. 3. NUMBER OF BATCHES & SPECIMENS

Twenty batches were cast and used for the experiments throughout the programme. Each batch generally consisted of 20 specimens, a number of which were gauged, and 3 four-inch cubes. The gauged specimens were used when strain measurements were required, while the ungauged specimens were generally used for strength tests.

Specimens from the same batch were sometimes employed for different series of tests and were either cured wet, sealed or by air drying.

Mix No. 1 was employed for all the investigations where the effect of environmental factors were considered. The reason for this choice was that this mix provided a relatively high proportion of cement paste (the only part of the material responsible for creep) while still used practically in industry. It has a medium workability which suites best the casting operation. The other mixes were employed when investigating the effect of the intrinsic factors.

IV. 4. CASTING PROCEDURE

The p.v.c. moulds were held firmly in position by means of steel frames made specially for this purpose as described in Chapter III. Gauges were fitted to a number of these moulds along their axes as shown in Figure III.5. The gauge wires were passed out through a hole provided at the top side of the p.v.c. mould which was then sealed with Plasticine.

Prior to casting, the moulds, fixed to steel bases in groups of four, were placed symmetrically and clamped to the vibrating table. The gauges were checked before casting to ensure that they were operating properly.

The materials were proportioned by weight to an accuracy of $\pm$ 1 gram. The materials were then placed in the pan of a Liner Currflow paddle mixer of 1 cu. ft. capacity. Mixing was continued for three minutes. Vibration was continued until full compaction was achieved. The gauges were checked again after casting in order to allow for the recovery of any gauge that might have stopped operating due to the vibration.

The first strain reading was registered about one hour from adding water to the mix. The moulds were dismantled at 24 hours.

IV. 5. CURING

Specimens were either unsealed at 24 hours by cutting the p.v.c. moulds with a circular electric saw and immersed in water or kept sealed in their p.v.c. jackets in which case the exposed ends were covered with wax. Both sealed and immersed specimens were stored in the same room as that of the testing frame. The temperature was partially controlled and it varied between 19°c. - 21°c.

The temperature of the room where the specimens were stored for the first 24 hours was not controlled and the temperature changes between winter and summer might have caused some variation in the early strength of the concrete. In general when comparison was sought between the concrete of different batches these were cast at short intervals between one another to minimise this effect. Moreover, most of the experiments were conducted on specimens which had been curing for 28 days.

In the case of curing while air drying, the specimens were stored in an atmosphere of relative humidity varying between 55 and 80%. By employing sealed and immersed specimens for most of the investigation the problem of the large variation in the ambient humidity did not arise.,

IV. 6. TEST PROCEDURE

About 2 to 3 hours before testing, end caps were connected to the specimens using Prompt cement mortar as a grouting material as previously described. The grout was left to harden for a period of 1 to 2 hours before testing, depending on the expected strength of the concrete used. The end caps could be connected to wet, sealed or dry specimens. The operation of connecting the end caps to 10 specimens took about 15 minutes.

In the case of immersed concrete the specimens were covered with damp rags over this period. The plastic sleeves, specially prepared for this purpose, were then fitted around the end caps and sealed at the bottom using rubber rings. The gap was then filled with water. This arrangement allowed for individual immersing and the specimen could be hung under the same line of loading as that of its sealed companion.

The specimens were generally hung in position and the gauge wires connected to specially prepared channels along side each line of loading which were in turn connected to the "Maihak" apparatus. This arrangement allowed for a quick registering of strains to be made after the application of load. The control specimens used for measureing strains due to volume changes other than that caused by the external load were also connected to their respective channels. The first reading could be registered in as short a period as 1-2 seconds from loading.

In the case of air drying concrete, the control unloaded specimens, were hung next to the loaded ones so as to subject them to the same humidity conditions.

CHAPTER V

TENSILE CREEP EXPERIMENTS

V. 1. INTRODUCTION

The present chapter deals entirely with the various aspects of uniaxial tensile creep of concrete and covers an investigation into the effects of the factors discussed in Chapter II.

For comparison with compressive creep reference is made to the availability of information reported in scientific literature, and the summary presented in Chapter I. Established figures on specific compressive creep are also used when a numerical comparison is thought to be worthwhile.

As previously reasoned in Chapter II, the emphasis of the present study is directed towards an investigation into the effect on tensile creep of the various factors relating to the aspect of water. For example, the effect on creep of the moisture condition of the surrounding medium and the water content of the mix.

V. 2. EXPERIMENTAL INVESTIGATION

Specimens from 19 batches were employed throughout the investigation.

The batch generally consisted of 20 specimens plus a number of cubes and sometimes a few 4 x 4 x 20 in. beams and 6 x 6 in. cylinders. The cubes, beams or cylinders were only used in order to check the quality of the mix employed.

When investigating the effect on creep of certain parameters, such as the aggregate-cement ratio, special effort was made to cast all the batches concerned with the investigation within as short a period of time as possible. This arrangement allowed for specimens of the different mixes to be loaded at the same time and under the same line of loading (three specimens could be loaded in one line; see Chapter III).

V. 2. a. Investigation into the Effect of Ambient Humidity

Three different moisture conditions were considered; sealing, immersing and air-drying - in a relative humidity averaging 65%. The lack of humidity-controlled cabinets limited this study to these three conditions only. However, the relative humidity was recorded during the test period in the case of air-drying specimens so that the effect on the creep and shrinkage could be checked. The smoothness in the shrinkage curves of unloaded specimens when compared with these of the loaded specimens may also provide an idea on the effect of the rate of drying on creep.

Specimens from three batches of identical mix were used to compare the creep under these three moisture conditions. These batches were C_1, C_2 and C_{14}. Batches C_1 and C_2 were cast mainly for this purpose, while batch C_{14} was concerned with the study of the effect on creep of the age at loading. However, to allow for the validity of such a comparison the mix chosen for C_{14} and the age at loading of certain specimens were the same as those used for C_1 and C_2. Batches C_1 and C_2 provide the necessary specimens for experiments 1 and 2 (see Table II.1) which deals with the two conditions, sealed and air-drying, while specimens from C_{14} were considered for the immersed condition.

The study of both sealed and immersed conditions was also included when investigating the other parameters throughout the test programme. These are reported when the specific parameter is considered.

MIX DETAILS (C_1, C_2 and C_{14})

Water-Cement Ratio = 0.50

Aggregate-Cement Ratio = 4.5

$\frac{3}{8}$" maximum size of Aggregate

C.&C.A. grading curve No. 2.

Experiment 1:

For this experiment 20 specimens were made, 12 of which were gauged, together with 3 control cubes.

Half of the specimens (i.e. 6 gauged and 4 ungauged) were unsealed at 24 hours and immersed in water, while the rest were kept sealed in their p.v.c. jackets and stored in the same room as those immersed in water. The exposed ends of the sealed specimens were covered with wax so as to ensure proper sealing.

At the age of 6 days all the unsealed specimens curing in water were removed from the storage tank and end caps were connected to both sealed and unsealed specimens. Two sealed and two unsealed, ungauged specimens were first tested to failure at a constant rate of loading of about 100 $lb./in^2$ per minute to determine the ultimate tensile strength of the particular mix at the age of 6 days. The strength results were as follows:

	Tensile Strength $lb./in^2$	
1.	285	(Sealed)
2.	263	(Immersed)

The unsealed specimens were kept wet for 6 days and then left to air-dry. The loading was started at the age of 7 days.

The arrangement followed in loading the specimens allowed for the first reading to be taken in less than 3 seconds after loading as previously described.

Two levels of stresses 70 and 140 $lb./in^2$ corresponding to 25 and 50% respectively of the ultimate strength were employed. Two identical specimens were used for every level of stress and moisture condition. This number of specimens was thought to be satisfactory as all the specimens used were checked nondestructively and their strain developments were observed for the period of time before loading. As previously mentioned, those specimens not functioning properly were excluded from the test. Two unloaded control specimens for each moisture condition were stored next to the loaded specimens which were used for measuring the strains occurring due to shrinkage, swelling or temperature volume changes. The load was maintained for a period of 140 days on all specimens. The procedure for unloading was similar to that for loading and the first reading was registered about 2-3 seconds after removing the load.

As previously mentioned, the stress was maintained constant by means of suspending a dead weight through a lever arm system.

Two of the gauged specimens, one sealed and due to be loaded to 70 lb./in^2 and the other unsealed and due to be loaded to 140 lb./in^2 were excluded from the experiment as their gauges failed to function properly, thus leaving only one specimen for each of these tests.

For this reason, and because the results began to indicate vast differences between the creep of sealed and drying specimens, another experiment, No. 2, basically a repetition of experiment No. 1, was conducted in order to corroborate the findings of the former.

<u>Experiment 2:</u>

The same number of specimens and the procedure used for

Experiment 1 were also used for this experiment using batch C_2. The specimens were cured and loaded in the same manner as in Experiment 1. The specimens loaded to 70 lb./in^2 were unloaded at the age of 140 days, while those loaded to 140 lb./in^2 were left under load for 600 days.

Recovery of strain after unloading was observed for periods ranging between 60-100 days for both Experiments 1 and 2.

The results of Experiment 2 support entirely those of Experiment 1.

The results of both experiments are plotted and shown in Figures V.1, V.2, V.3, V.4,V.5. Every point on the curve represents the average of two specimens generally employed for each test, unless otherwise stated. The strains due to volume changes in the unloaded sealed and drying specimens are plotted against time and shown in Figures V.1, V.5 & V.9. The initial strain in Figure V.9 represents the strain developments over the first 24 hours after casting.

The creep curves of immersed or wet concrete were obtained from Experiment 3, reported later, using batch C_{14}. In this experiment two specimens, cured wet, were loaded at the age of 7 days while kept immersed in water by using plastic sleeves as demonstrated in Chapter III. The two specimens were each subjected to a stress of 130 lb./in^2.

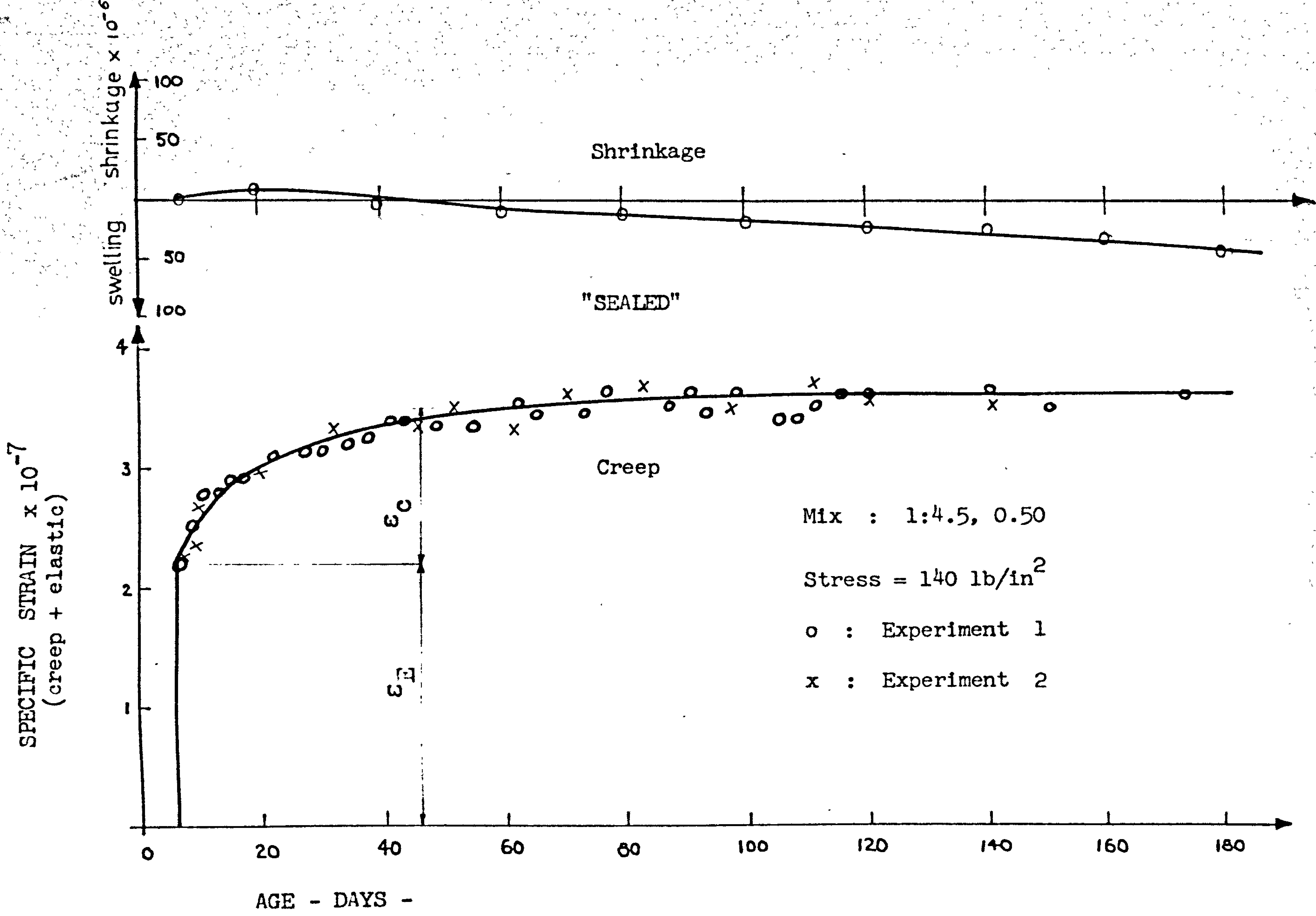


Fig. V.1. Creep and Shrinkage of Sealed Concrete

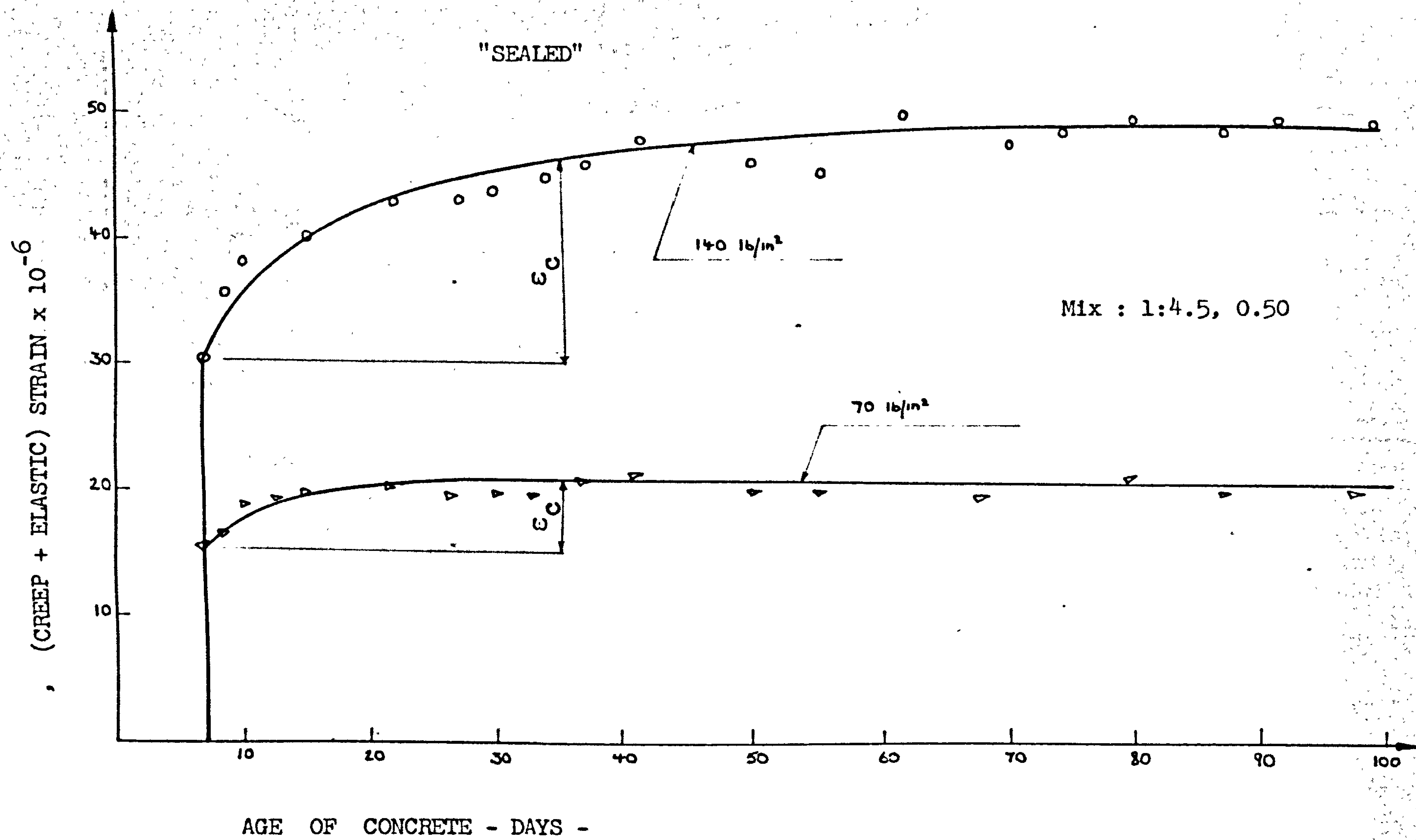


Fig. V.2. Creep of Sealed Concrete

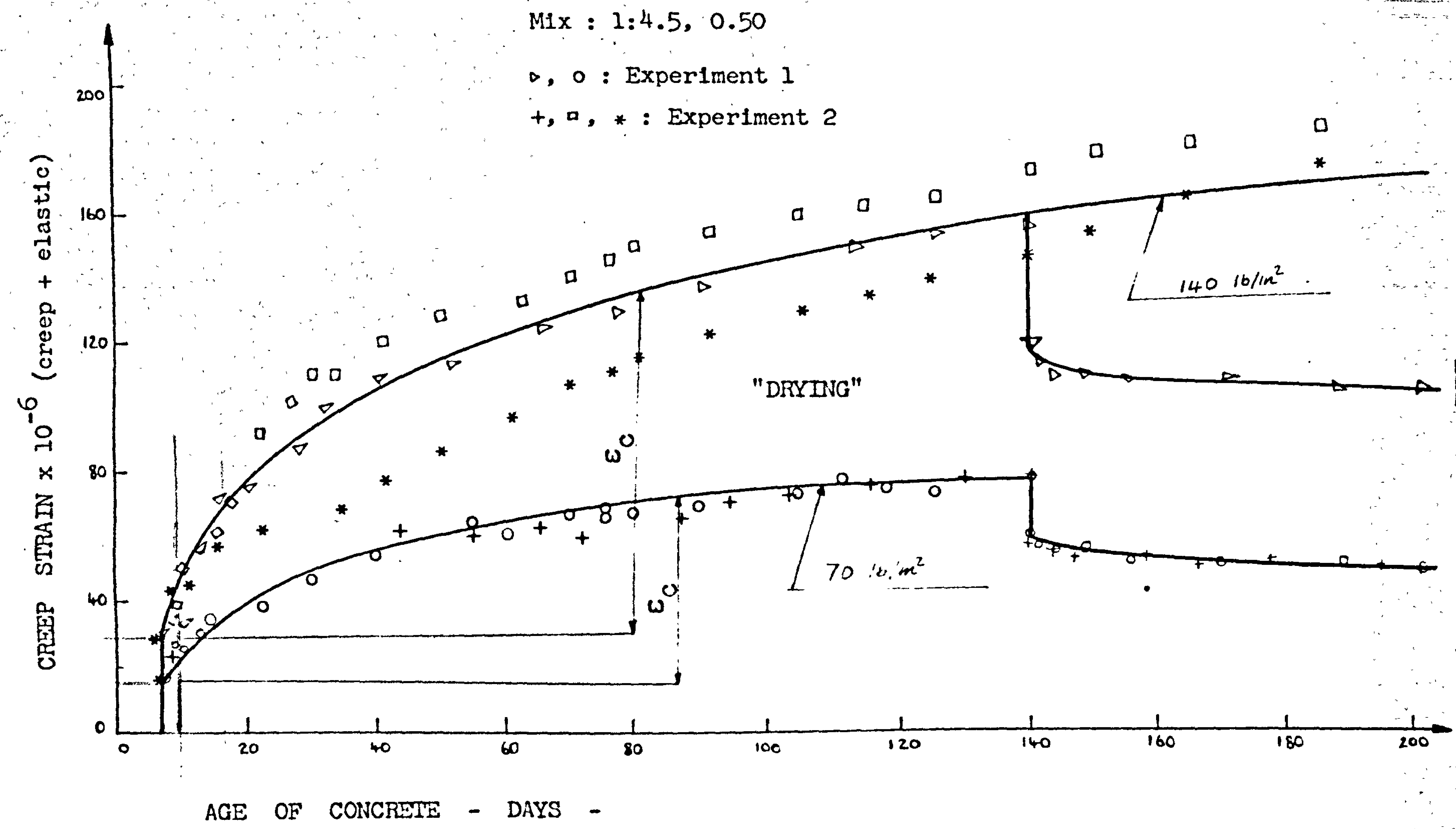


Fig. V.3. Creep of Drying Concrete

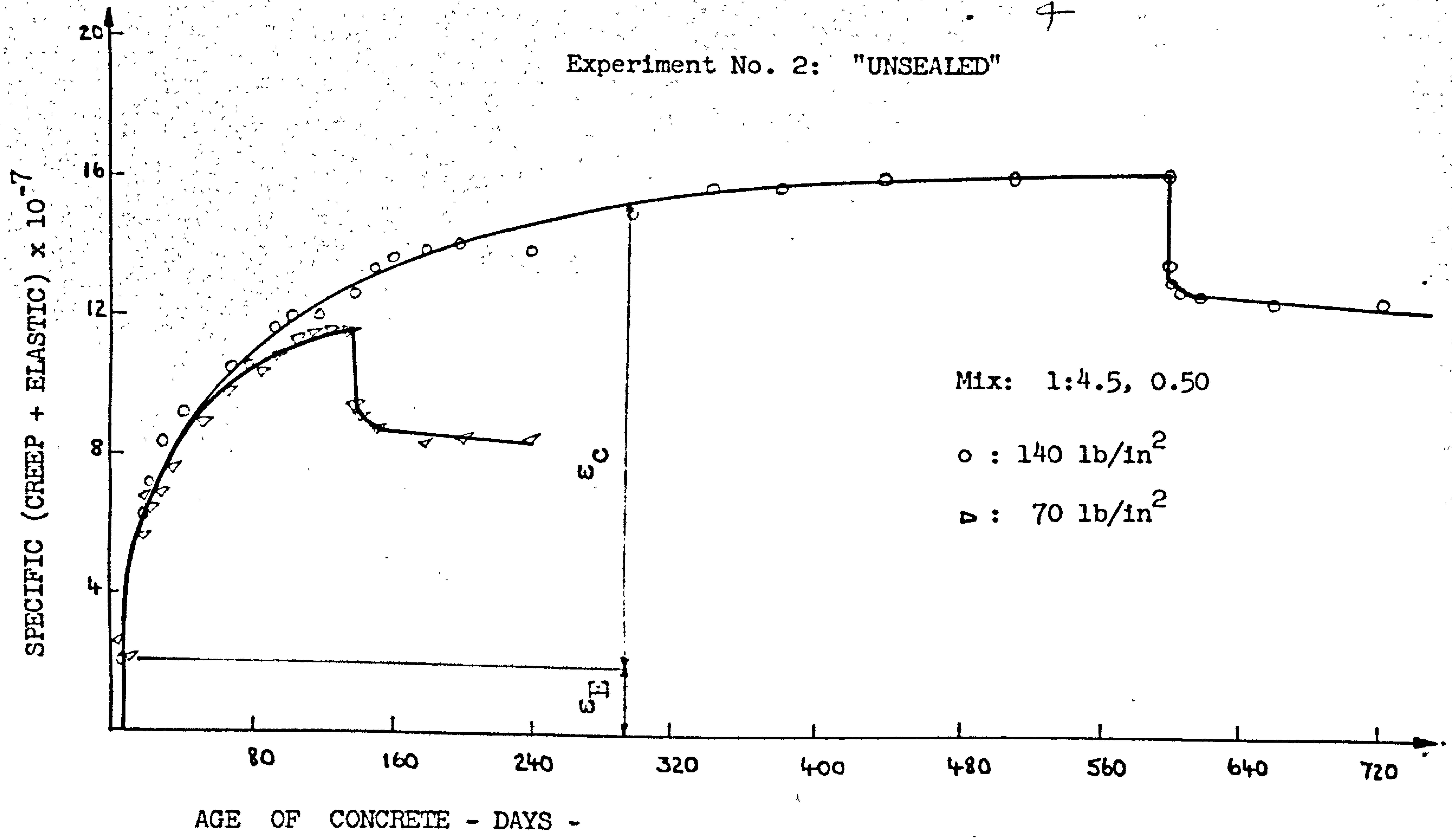


Fig. V.4. Creep of Drying Concrete

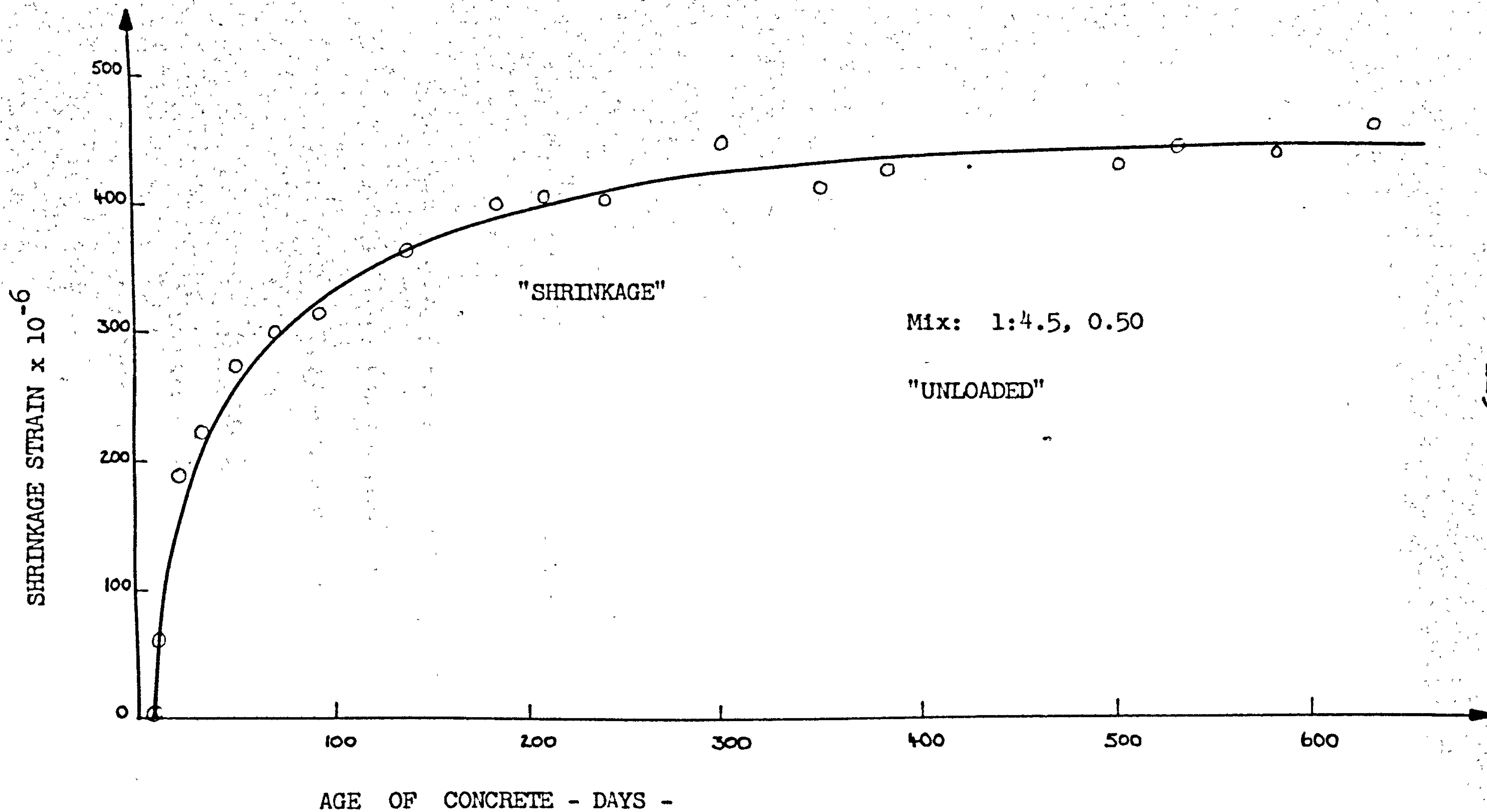


Fig. V.5. Shrinkage of Air Drying Unloaded Specimens

One of the two specimens was unloaded after about 30 days and the other was kept under load for about 140 days. However the creep strains of the two specimens were almost identical for the first 30 days. There is no reason why any change should occur at a later time, particularly as the creep of immersed specimens almost reaches a limited value after about 30-40 days of loading.

The strains due to volume changes of unloaded immersed concrete were observed on three control specimens. The creep results of this test are shown in Figure V.6.

V. 2. b. Investigation into the Effect of Age at Loading

Experimental Information

In this investigation specimens were loaded at the ages of 1, 3, 7 and 28 days. Two moisture conditions, sealed and immersed were considered.

For immersed condition, a batch (C_{14}) was cast mainly for this experiment.

In the case of sealed condition, the information available from the other experiments on specimens having the same mix as C_{14} and loaded at the ages of 3, 7 and 28 days were considered.

MIX DETAILS

Aggregate-Cement Ratio = 4.5

Water-Cement Ratio = 0.50

$\frac{3}{8}$ in. Maximum Size Aggregate

C. & C.A. Grading Curve No. 2.

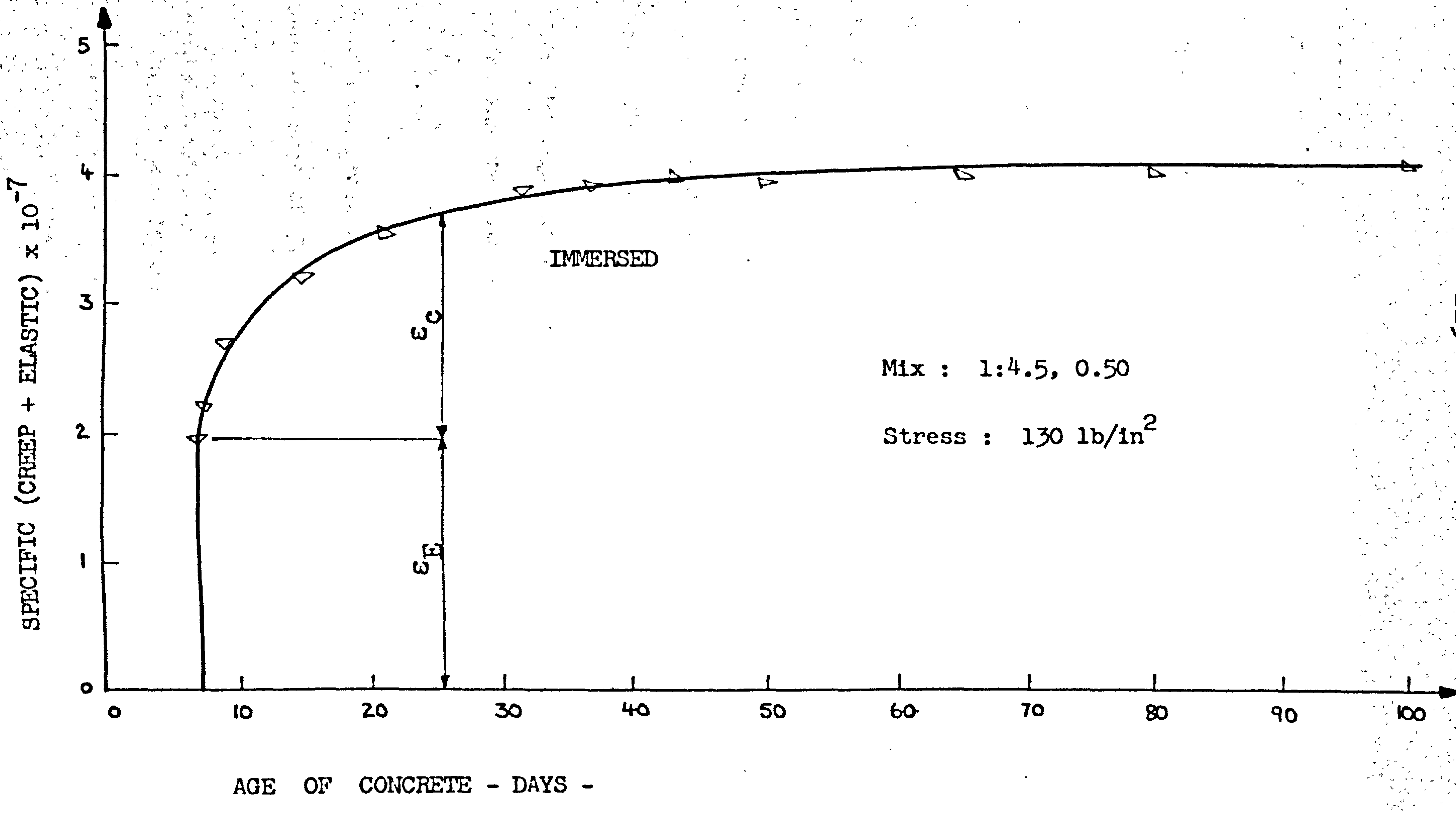


Fig. V.6. Creep of Immersed Concrete

Experiment No. 3.

This experiment was made using 20 specimens (12 gauged and 8 ungauged) and 3 cubes which were cast using batch C_{14}. The specimens were unsealed after 24 hours and immersed in water. End caps were connected to two gauged and two ungauged specimens after they had been checked nondestructively. The specimens were kept wet while connecting the end caps by covering the exposed parts with damp sleeves. However the time required for this operation (i.e. the time needed for connecting end caps to one side of the specimens and leaving the grout to set, turning them over and repeating this for the other side) is less than 15 minutes. Once the grout had set and before the specimen was hard enough for testing, the plastic sleeve (mentioned earlier) could be placed around the end caps and the gaps between the plastic sleeve and specimens filled with water.

The two ungauged specimens were tested to failure at the age of 28 hours and the two gauged specimens were loaded thereafter to 70 lb./in^2. The stress corresponded to about 50% of the ultimate.

The same procedure was maintained for the other specimens loaded at the ages of 3, 7 and 28 days.

The stress was maintained at 70 lb./in^2 for those loaded at 3 days but for those loaded at 7 and 28 days it was increased to 130 lb./in^2. This was in order to maintain the stress near 50% of the ultimate strength at the ages of 7 and 28 days. The comparison is, however, always made on specific creep.

The strain in unloaded concrete due to swelling volume change was observed on 3 specimens. One gauged specimen was found to be faulty and excluded from the test.

The load was maintained over all the specimens for a period of 30 days. Recovery of strain was observed over a further period of 40-70 days.

As in Experiment 1, first reading after loading and unloading was registered in about 2-3 seconds.

The results of Experiment 3 are plotted and shown in Figure V.7. Figure V.8 shows the results of sealed specimens obtained from other experiments and redrawn for comparison. Figure V.9 shows the development of strain with time in unloaded sealed and immersed concrete from the age of 1 hour.

V. 2. c. Investigation into the Effect of the Level of Stress

Experimental Information

Two experiments were carried out, the first employing sealed and immersed concrete loaded to three levels of stress at the age of 3 days, and the second employing only sealed concrete loaded to 3 levels of stress at the age of 20 days.

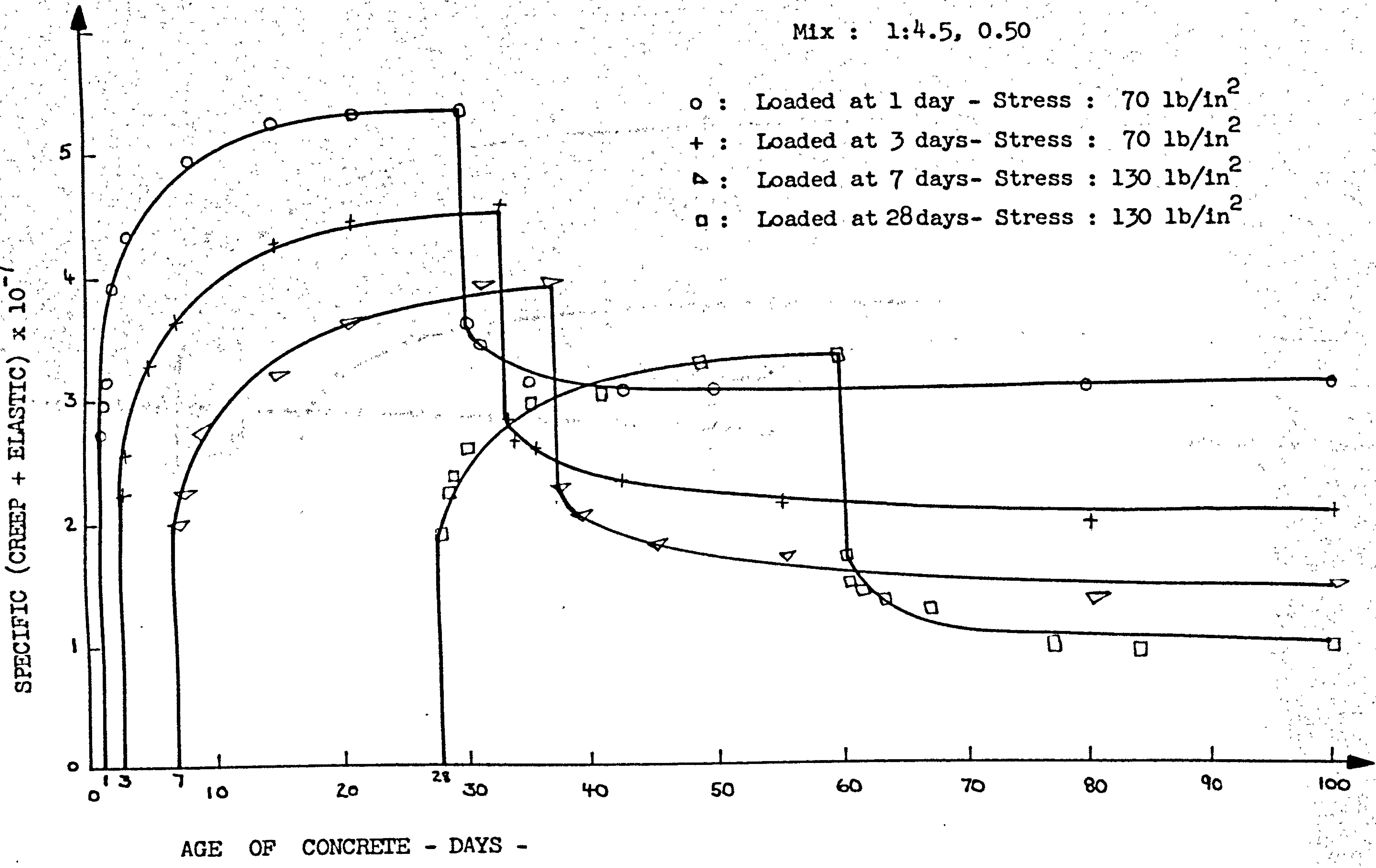


Fig. V.7. Creep of Immersed Concrete Loaded at Different Ages

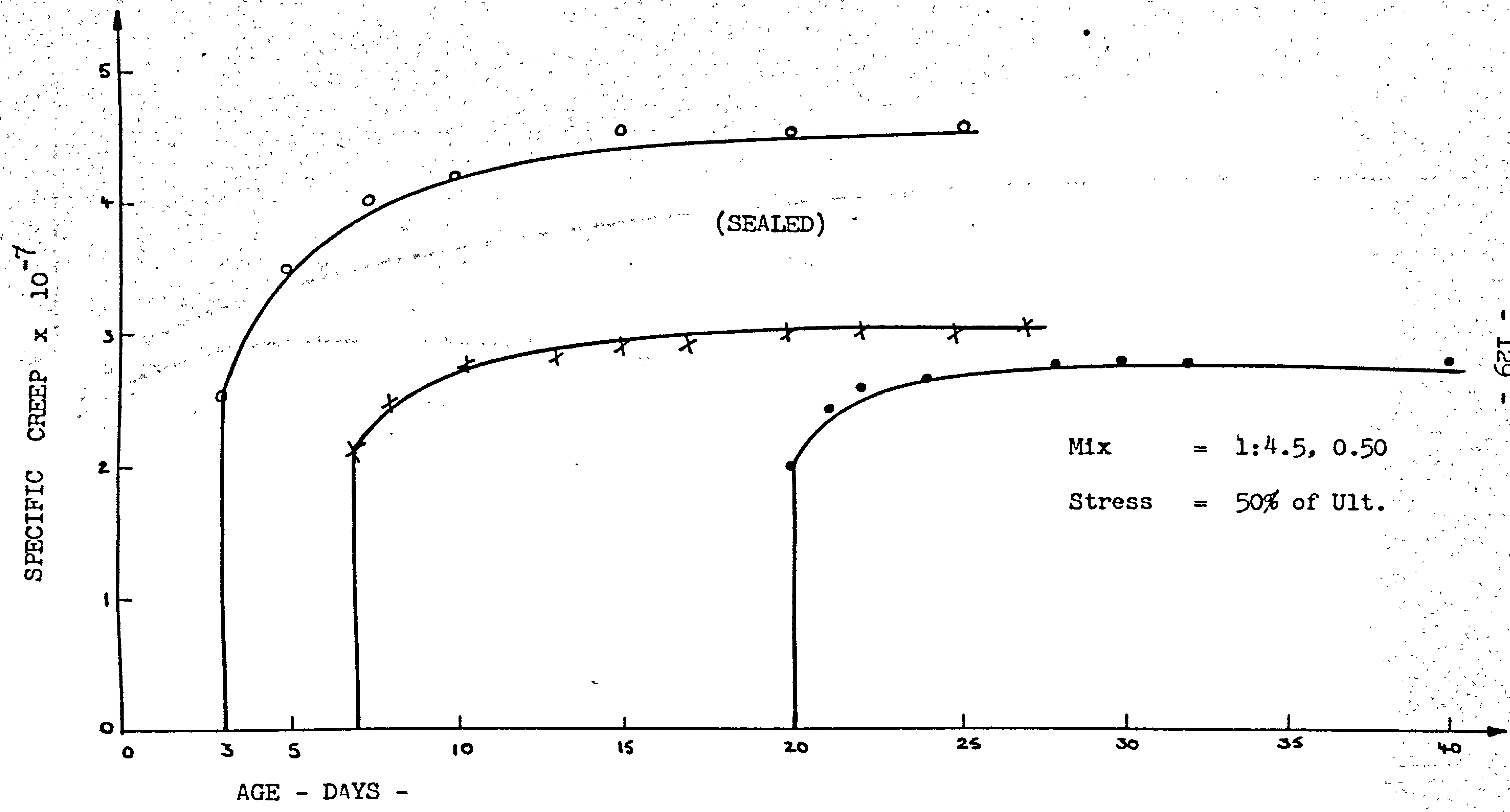


Fig. V.8. Creep of Sealed Concrete Loaded at Different Ages.

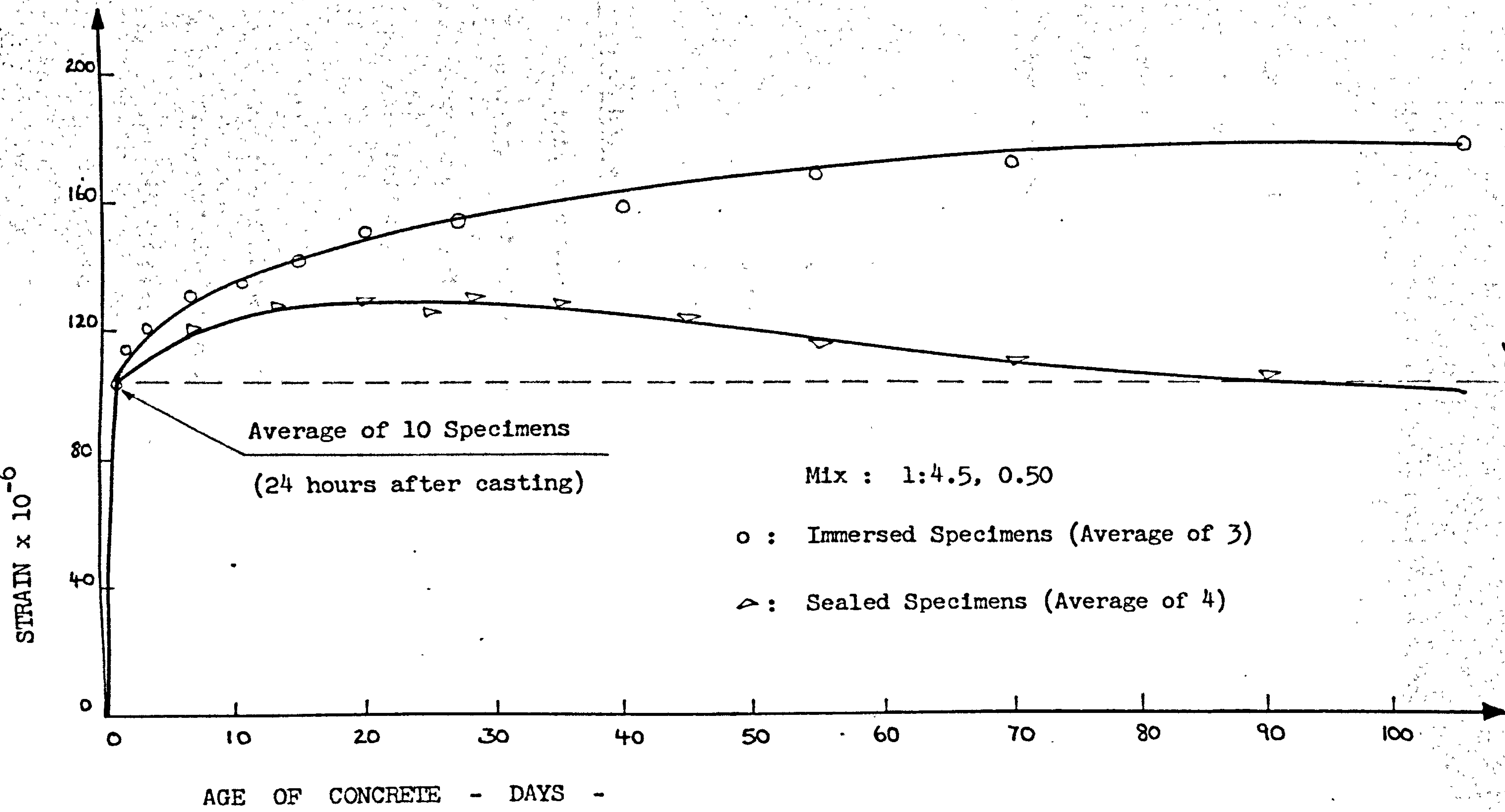


Fig: V.9 Strain Development in Unloaded Specimens

The levels of stress were 75, 100 and 135 lb./in^2 for Experiment 4, and 80, 135 and 168 lb./in^2 for Experiment 5.

The same mix design was employed for both experiments using batches C_{10} and C_4 respectively.

The level of stress was also investigated for mixes of different water-cement ratio. These experiments are reported later, but some of their results are shown here for comparison.

MIX DETAILS

Aggregate-Cement Ratio = 4.5

Water-Cement Ratio = 0.50

$\frac{3}{8}$ in. maximum size Aggregate

C. & C.A. Grading No. 2.

Experiment No. 4.

For this experiment 20 specimens (16 gauged and 4 ungauged) and 3 cubes were cast and left standing for 24 hours. Ten of these (8 gauged and 2 ungauged) were then unsealed and immersed in water. The rest were left sealed. All specimens were stored in the same room.

At the age of three days, end-caps were connected to all the specimens following the procedure described earlier. The four ungauged specimens (2 sealed and 2 immersed) were first tested to failure to check the ultimate strength before loading commenced. The strength results were as follows:

Tensile Strength lb./in^2

(means of 2 results)

Sealed	:	200
Wet	:	185

Using two specimens for each test (i.e. for each level of stress and moisture condition) 12 specimens were loaded at the age of 3 days to 70, 96 and 124 lb./in^2. The sealed specimens loaded to 70 lb./in^2 were excluded from the test due to failure of their gauges to function properly. Two sealed and 2 immersed, unloaded specimens were stored next to the loaded ones. These were used as control specimens for observing the strains developing with time due to volume changes other than those caused by external loading.

The load was maintained for 20 days on all the specimens. Unloading took place at the age of 28 days and recovery of strain was observed for a further period of 25 days.

The results of this experiment are plotted and shown in Figures V.10, V.11, V.12. & V.13.

Experiment 5

For this experiment 16 specimens (14 gauged and 2 ungauged) and 3 cubes were cast using batch C_5. The specimens were kept

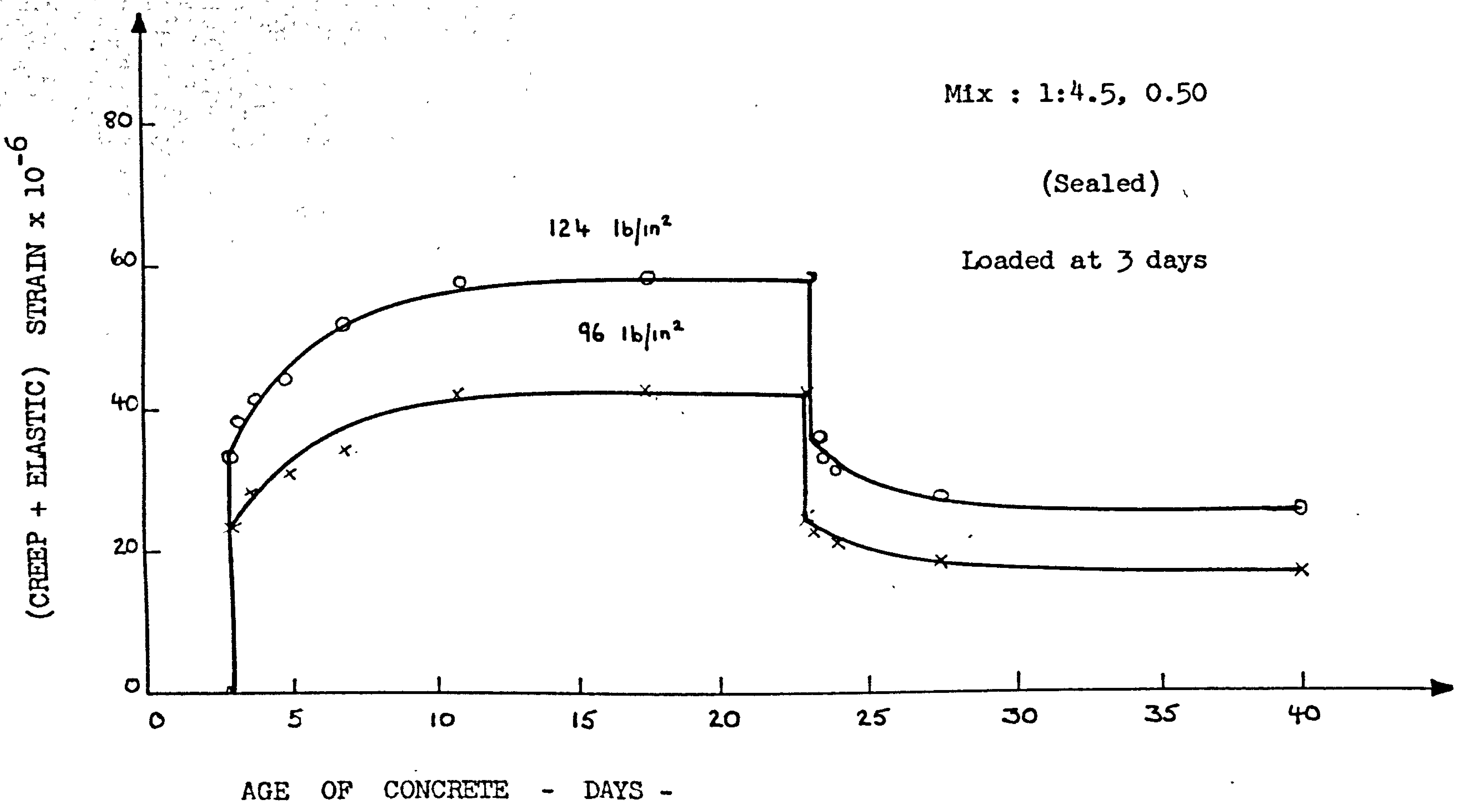


Fig: V.10. Effect of Level of Stress on Creep of Sealed Concrete

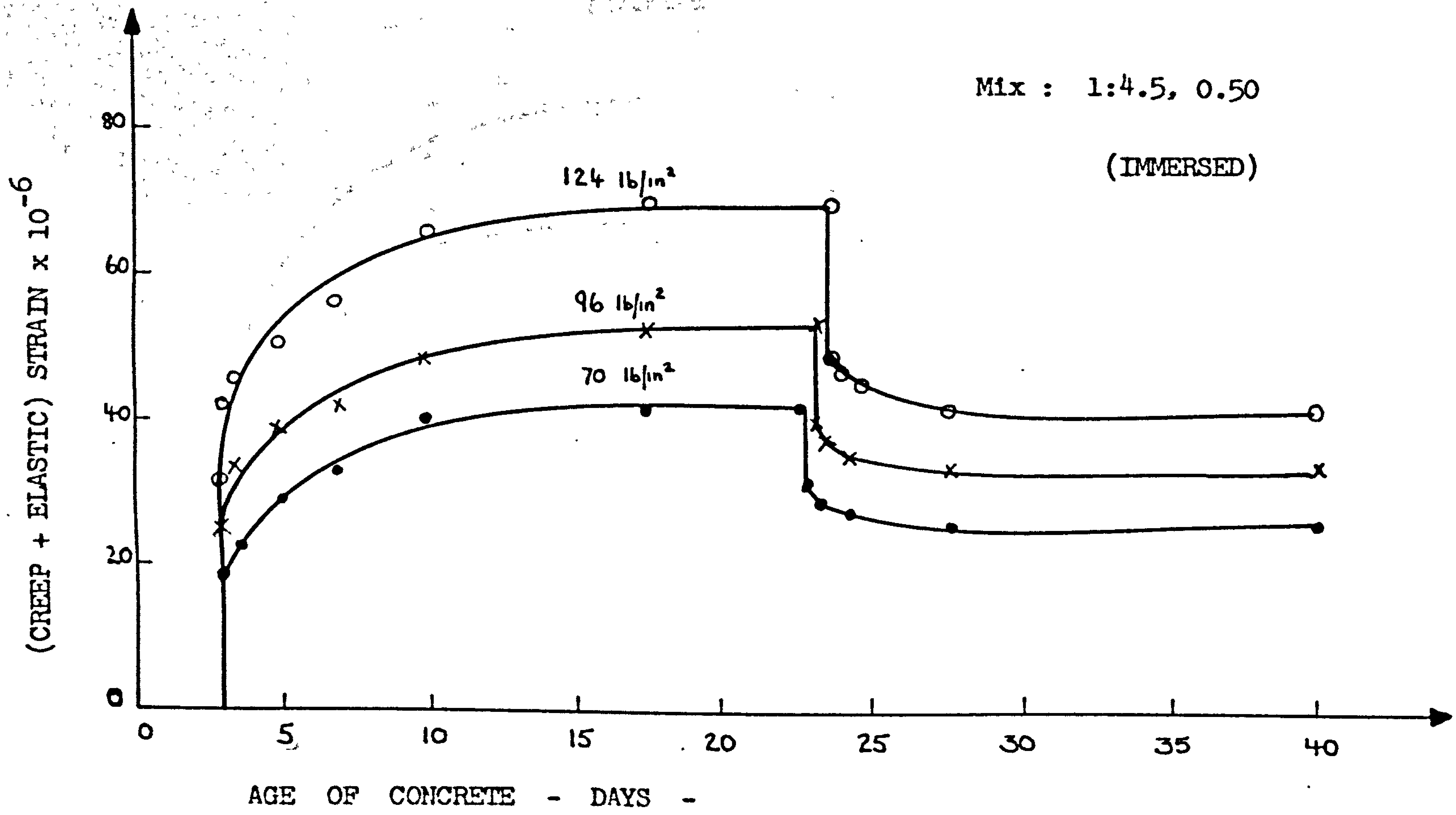


Fig: V.11. Effect of Level of Stress on Creep of Immersed Concrete

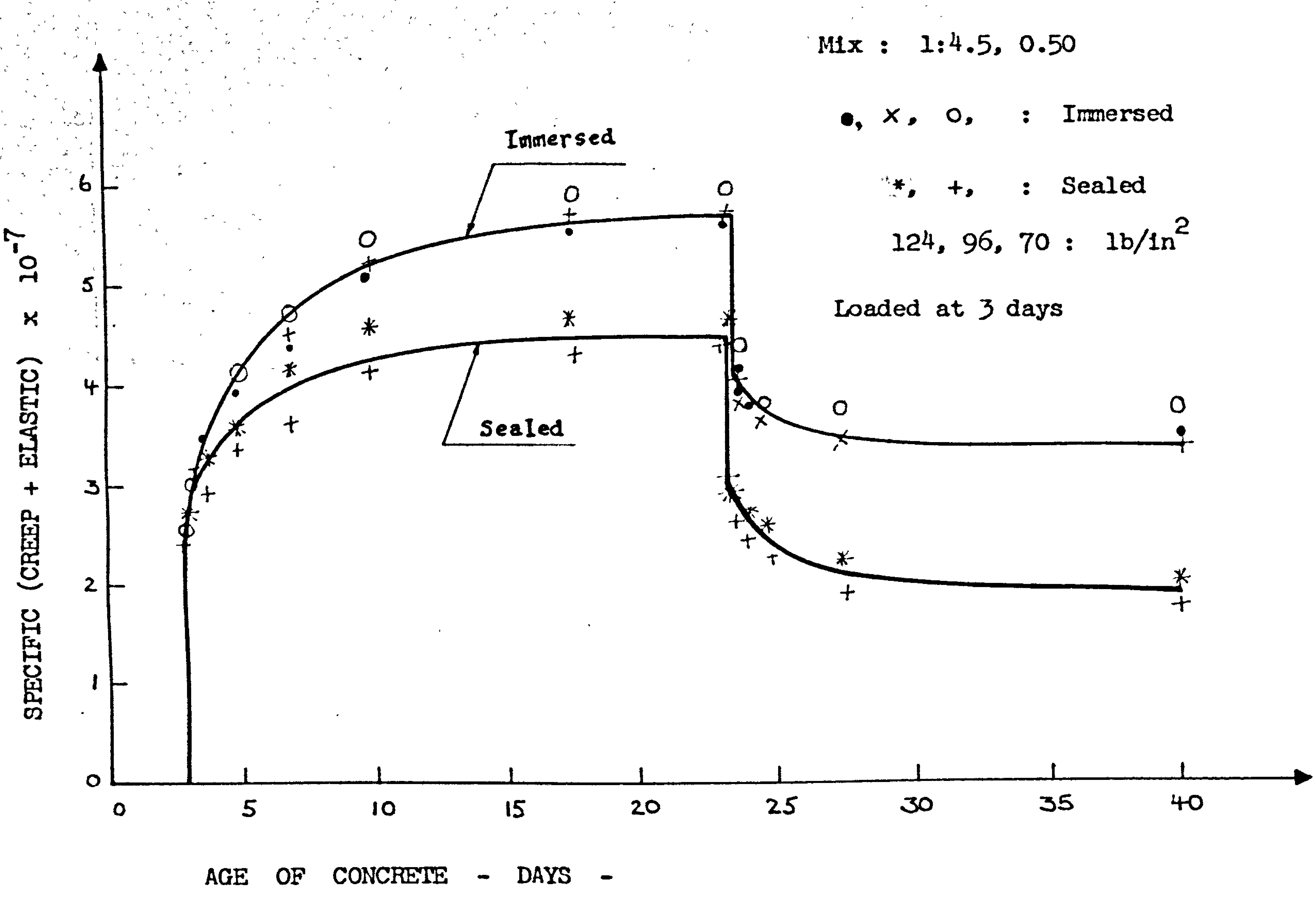


Fig: V.12. Creep of Sealed & Immersed Concrete

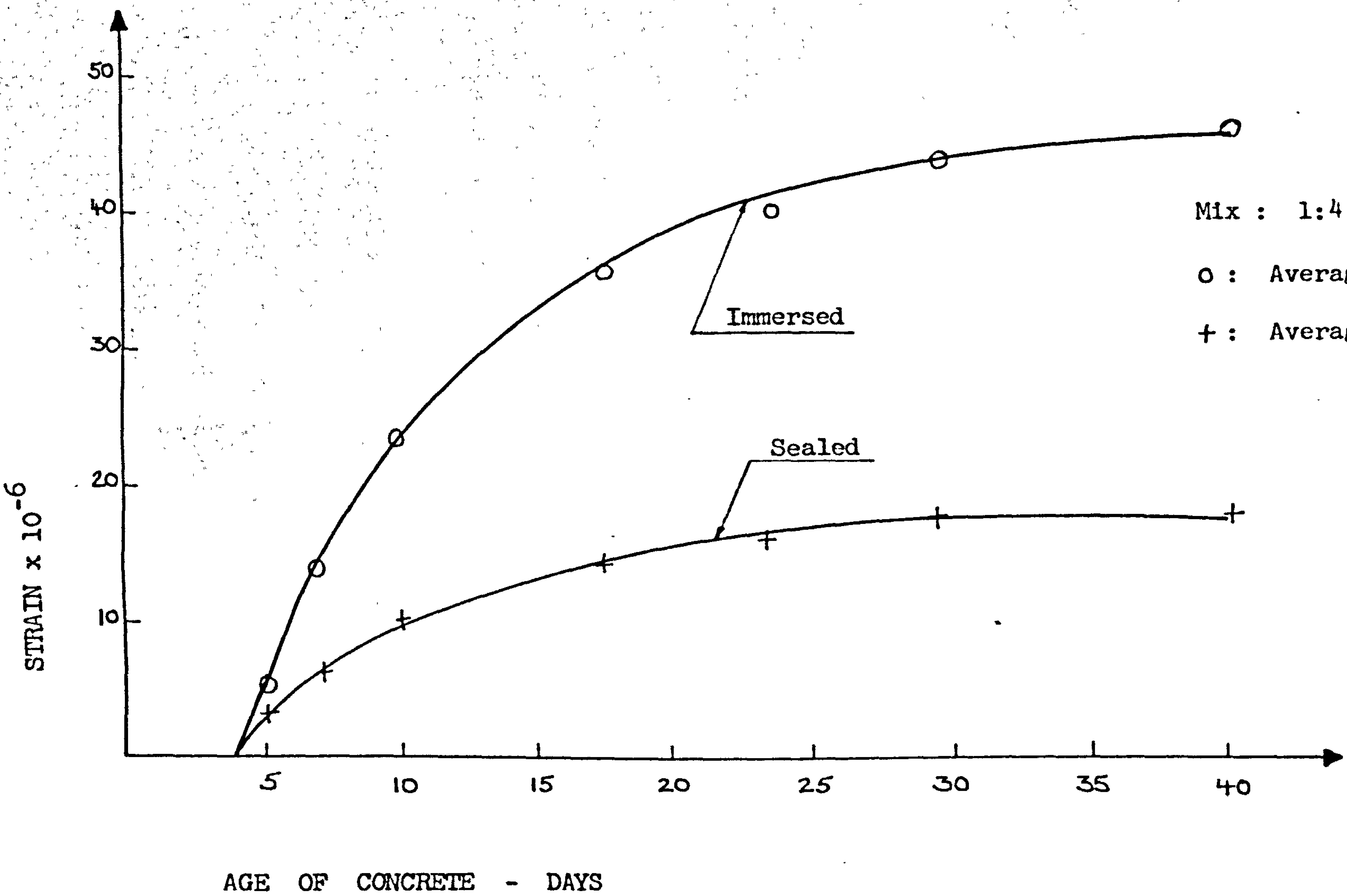


Fig: V.13. Strain Development in Unloaded Specimens sealed in their p.v.c. jackets.

At the age of 20 days, end caps were connected to 9 gauged and 3 ungauged specimens. The ungauged specimens were first tested to failure to give an average strength of 290 lb./in^2 and the 9 gauged specimens were then loaded to 80, 130 and 168 lb./in^2 using three specimens for each test. The four gauged specimens left were used as control. One gauged specimen did not function properly.

The load was maintained for 20 days on all specimens. Unloading took place at the age of 40 days and recovery of strain was observed for a further period of 20 days after unloading. The results of Experiment 5 are plotted and shown in Figure V.14.

Figure V.15, V.16 & V.17. show the effect of level of stress on the creep of concrete made with the same aggregate-cement ratio, but with water-cement ratios of 0.40, 0.60 and 0.70 and loaded at the age of about 30 days.

V. 2. d. <u>Investigation into the Effect of Water-Cement Ratio</u>.

<u>Experimental Information</u>

A range of water-cement ratio between 0.42 - 0.70 has been covered. The mixes employed had water-cement ratios of 0.42, 0.50, 0.60 and 0.70. The aggregate-cement ratio was constant for all the mixes. Two moisture conditions, sealed and immersed were employed.

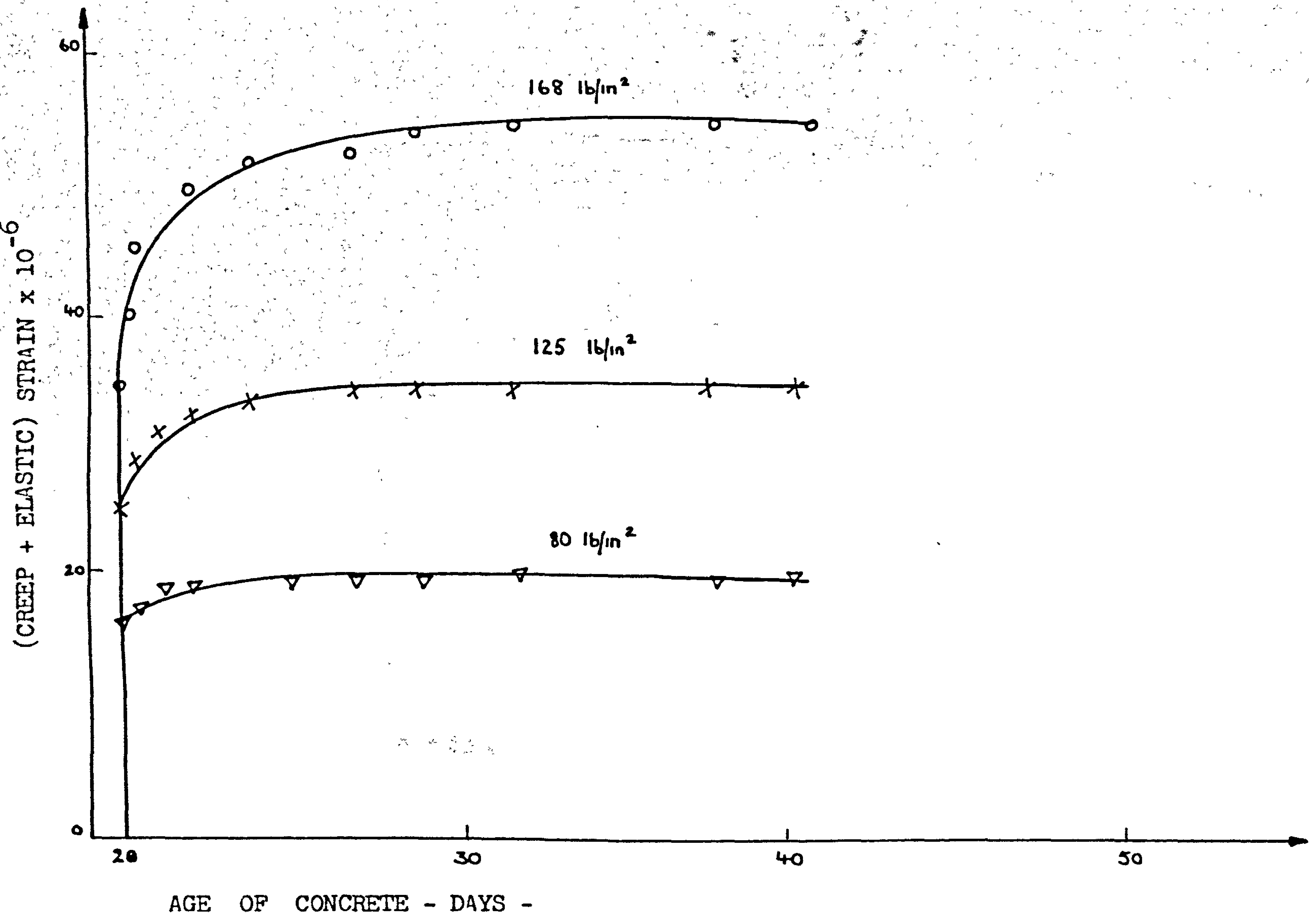


Fig. V.14. Creep of Sealed Concrete.

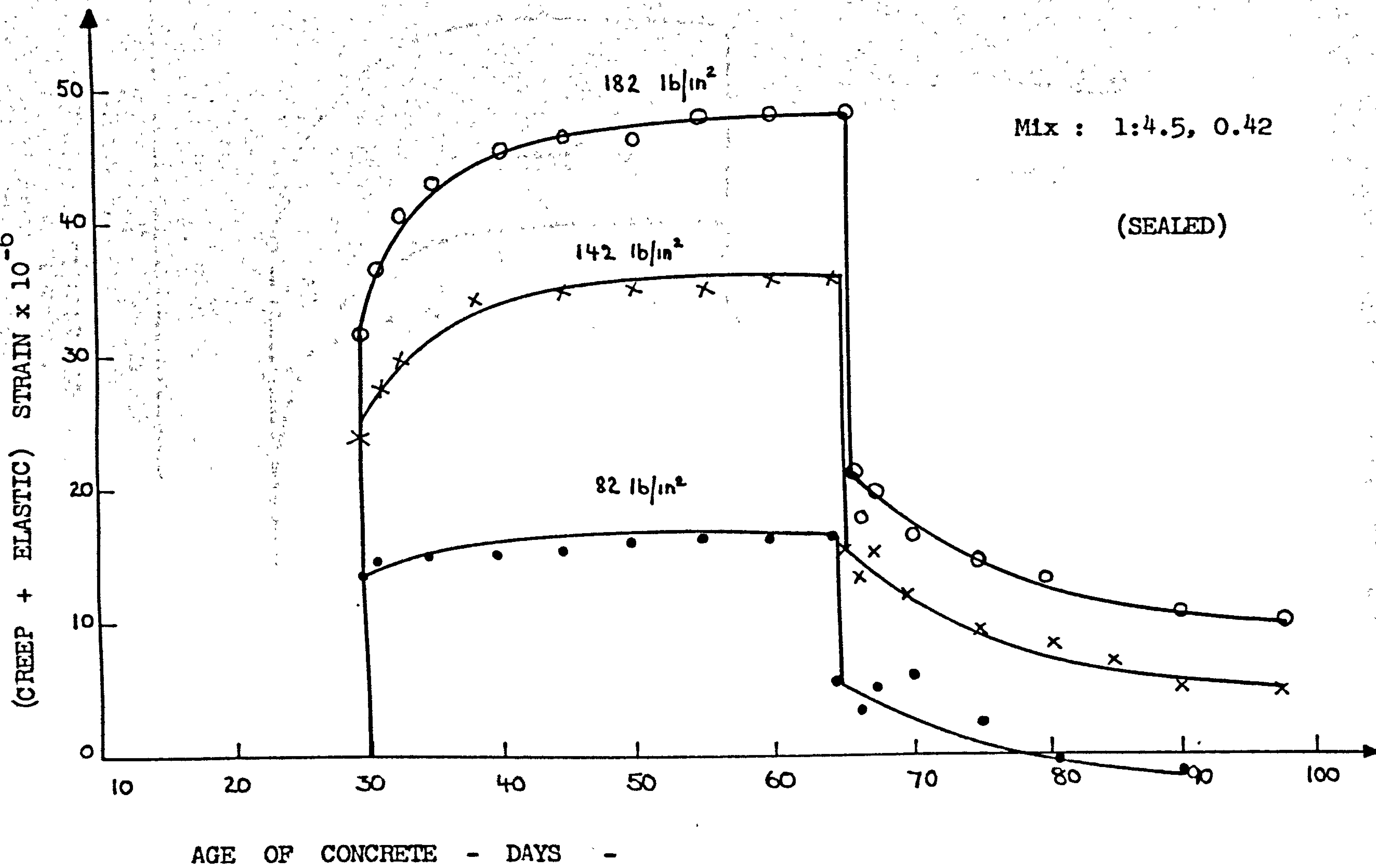


Fig: V.15. Effect of Level of Stress on Creep of Sealed Concrete Made with w/c = 0.42

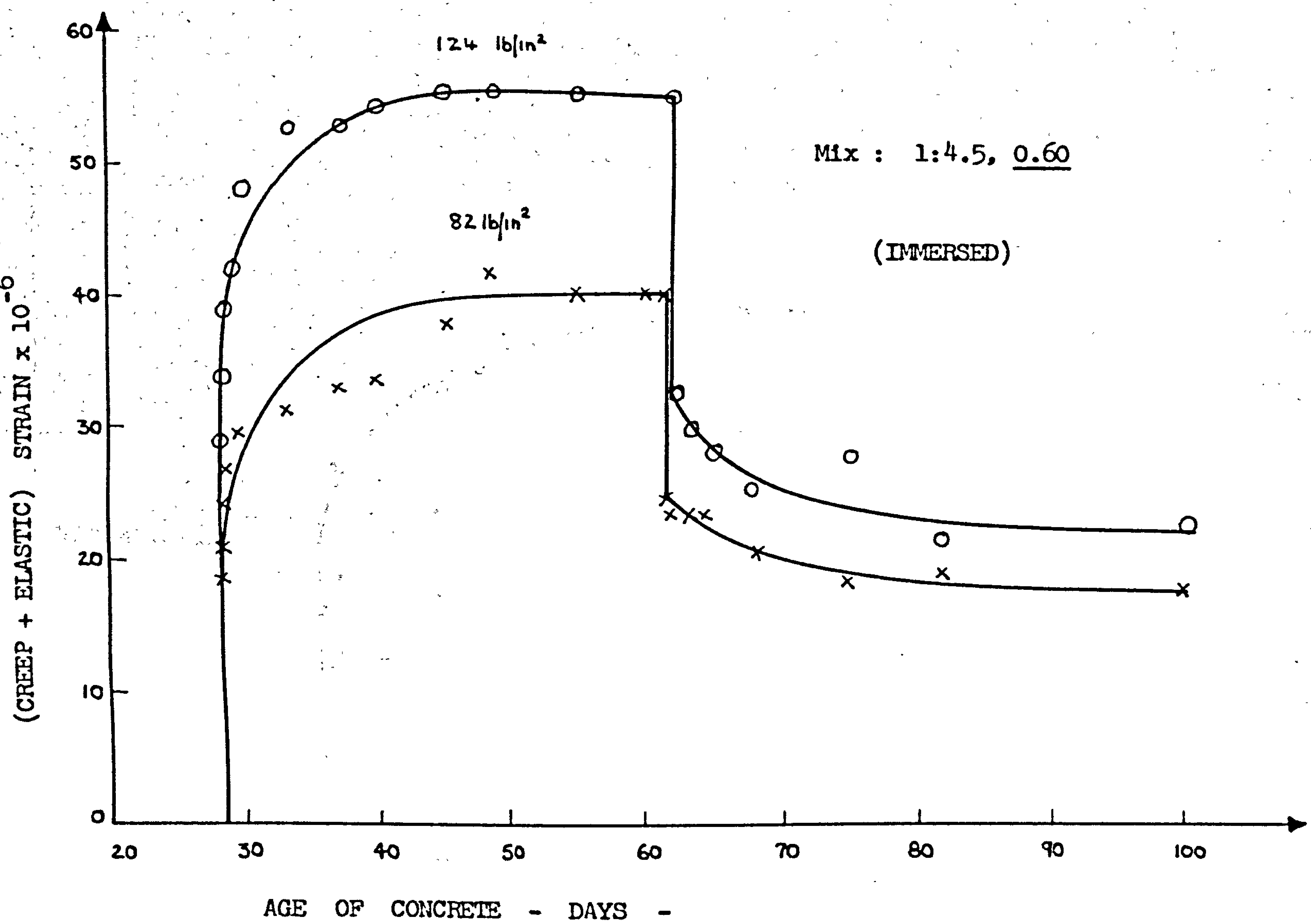


Fig: V.16. Effect of Level of Stress on Creep of Immersed Concrete made with w/c = 0.60

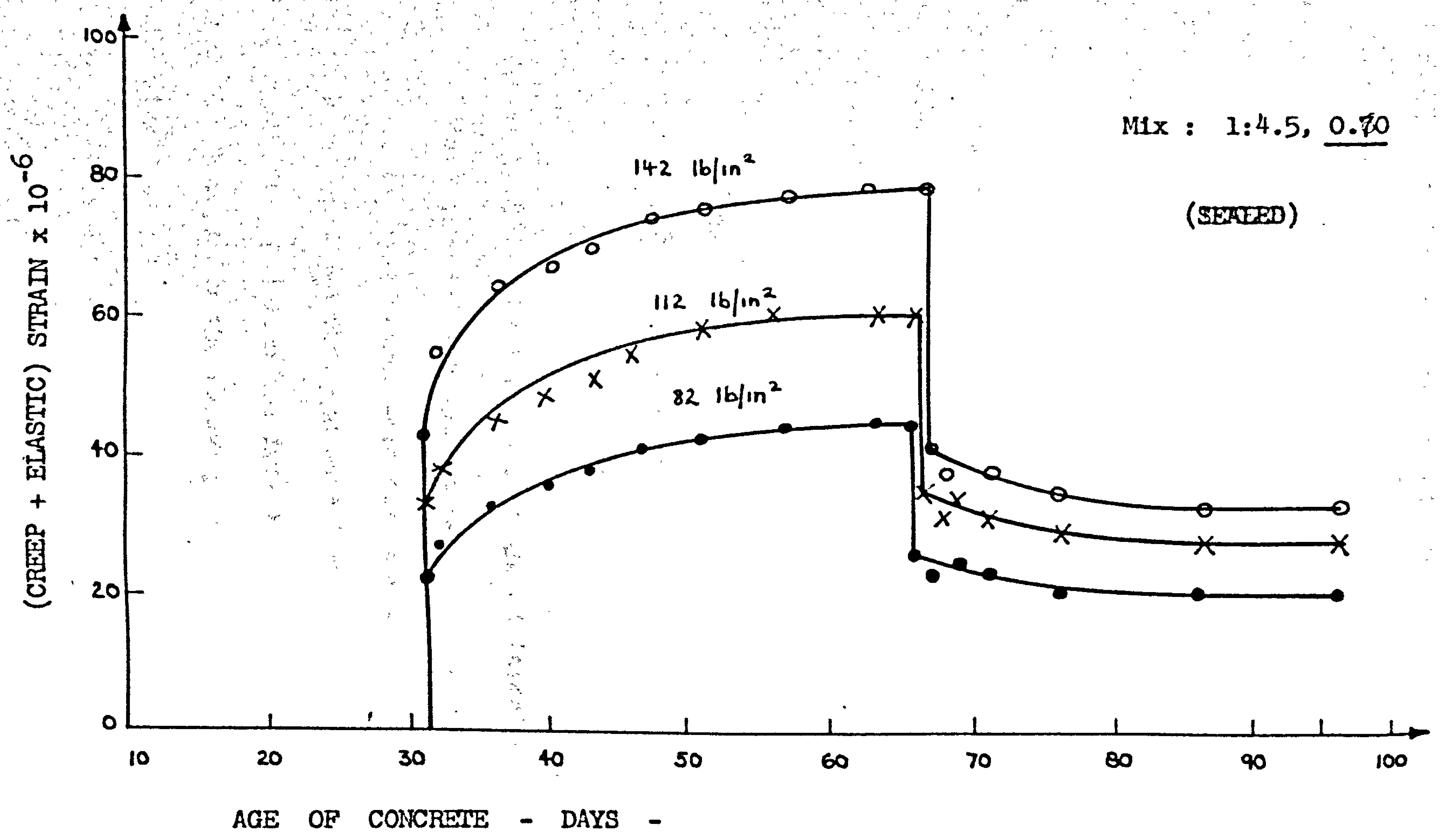


Fig: V.17. Effect of Level of Stress on the Creep of Sealed Concrete Made with w/c = 0.70

Three experiments, No. 6, 7 and 8, using batches C_{11}, C_{12} and C_{13} respectively were carried out mainly for this purpose. These included water-cement ratios of 0.70, 0.60 and 0.42 for C_{11}, C_{12} and C_{13} respectively. The results on mixes having water-cement ratio of 0.50 is obtained from other experiments investigating different aspects, but which satisfied the conditions necessary for the comparison.

MIX DETAILS

In all three experiments the concrete had an aggregate-cement ratio of 4.5. The water-cement ratios and aggregate gradings varied as follows:

0.70	:	for Experiment 6
0.60	:	for Experiment 7
0.42	:	for Experiment 8

Procedure

In order to allow the specimens of the different mixes to be loaded at the same time and for some to be hanging under the same line of loading, the three mixes were cast one after the other at an interval of 2 days between them and were loaded at the ages of 28, 30 and 32 days, depending upon the date of casting. The load was maintained for 35 days and the recovery was observed for 30 days after unloading.

Experiment 6

This experiment used 20 specimens (12 gauged and 8 ungauged) and 3 cubes. Resonant frequency checking showed that 5 specimens (4 gauged) were faulty. The reason was thought to be due to mis-handling the specimens when dismantled (24 hours after casting).

Owing to this, this experiment was devoted to sealed condition only. The specimens were therefore kept sealed in their p.v.c. jackets.

At the age of 32 days three ungauged specimens were tested to failure and loading started thereafter. Three levels of stress were employed, 82, 110 and 142 lb./in^2 using 2 specimens for each level. Three gauged specimens were used as control.

Experiment 7 & 8

Twenty specimens (12 gauged and 8 ungauged) and 3 cubes were cast for experiments 7 & 8, using batch C_{12} and C_{13} respectively. Six gauged and 4 ungauged specimens of each batch were unsealed after 24 hours and immersed in water. The rest were kept sealed and stored in the same room as those immersed in water.

Loading started at the age of 28 and 30 days for Experiments 7 & 8 respectively. Three ungauged specimens of each of the two batches were first tested to failure. Three levels of stress, 82, 117 and 142 lb./in^2 were employed. One sealed and one immersed specimen in each experiment were used for each level of stress. Three sealed and three immersed specimens were used as controls for each experiment.

The use of one loaded specimen only for a test was justified:

1) Because the strain development curve for that specimen was known for 30 days and was compared with that of the unloaded specimens used for control. The strain observed on the control specimens, which was deducted from that of the loaded specimens for the 30 day period of loading was small.

2) Because the nondestructive check made on all the specimens ensured that only those which had been made satisfactorily were used and this precaution resulted in very close similarity between the properties of loaded and unloaded specimens.

However, the main purpose of these experiments was to investigate the effect of water-cement ratio on creep. It was therefore relevant to determine the average specific creep of the specimen at the 3 levels of stress used in relation to the different values of water-cement ratio.

The results of Experiments 6, 7 and 8 are plotted and shown in Figures V.18, V.19 & V.20.

V. 2. e. Investigation into the Effect of Aggregate-Cement Ratio

Experimental Information

Three aggregate-cement ratios were considered for this investigation, namely 1:45, 1:6 and 1:75. The water-cement ratio was kept constant. Two moisture conditions, sealed and immersed were employed.

Three Experiments, 9, 10 and 11, were carried out using batches C_{17}, C_{18} and C_{19} respectively.

As in Experiments 6, 7 and 8, the batches used for Experiments 9, 10 and 11 were also cast at 2 day intervals in order to allow for the specimens of different experiments to be loaded at the same time. Loading started at the ages 28, 30 and 32, depending on the time of casting. The load was maintained for a period of 30 days and recovery was observed for 40-50 days after unloading.

MIX DETAILS

All three experiments employed a constant water-cement ratio of 0.60, $\frac{3}{8}$ in. maximum size aggregate and C. & C.A. grading curve No. 2. The aggregate-cement ratios were as follows:

1:6	in Experiment 9
1:7.5	in Experiment 10
1:4.5	in Experiment 11

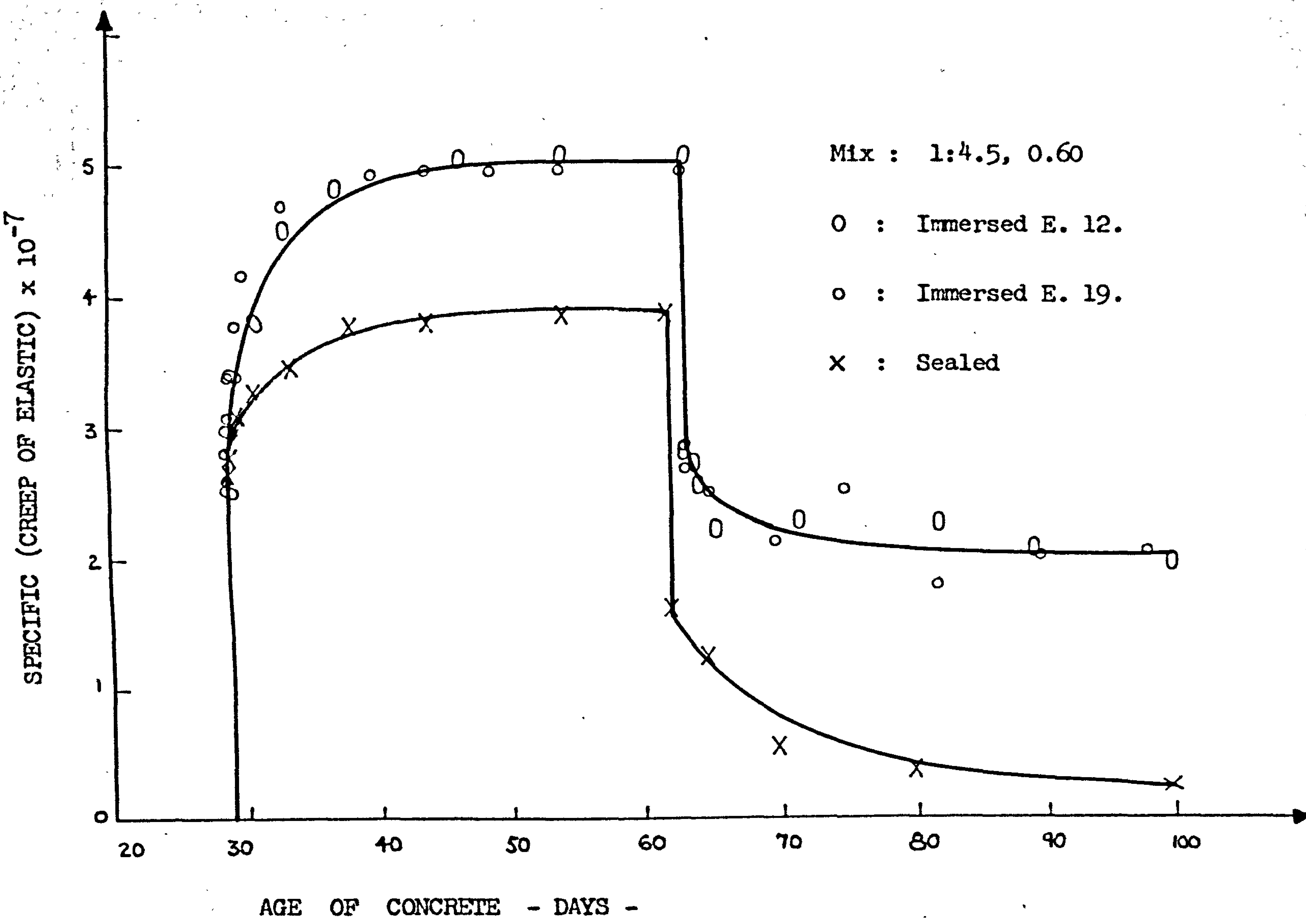


Fig: V.18 Creep of Sealed & Immersed Concrete

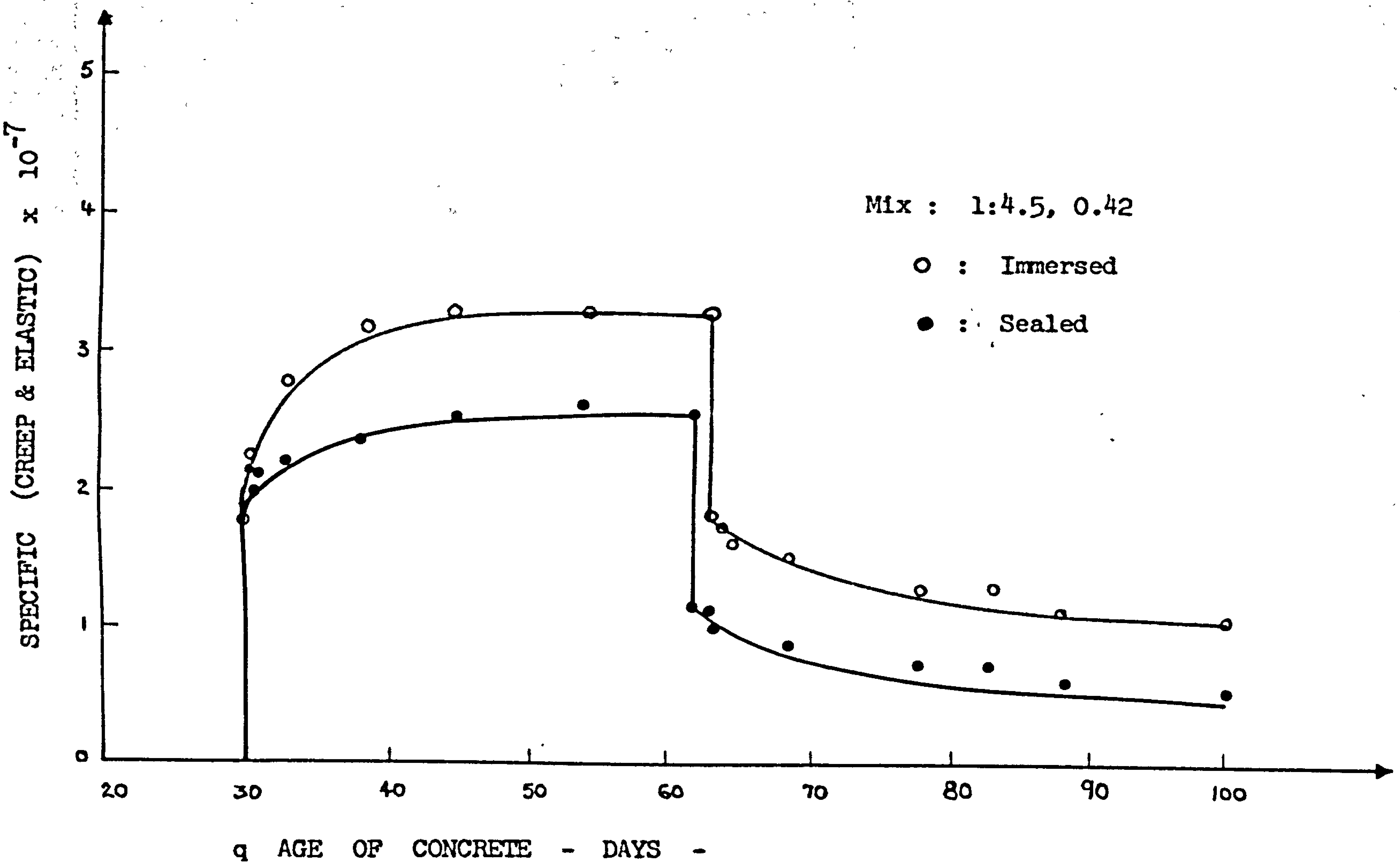


Fig: V. 19. Creep of Sealed & Immersed Concrete

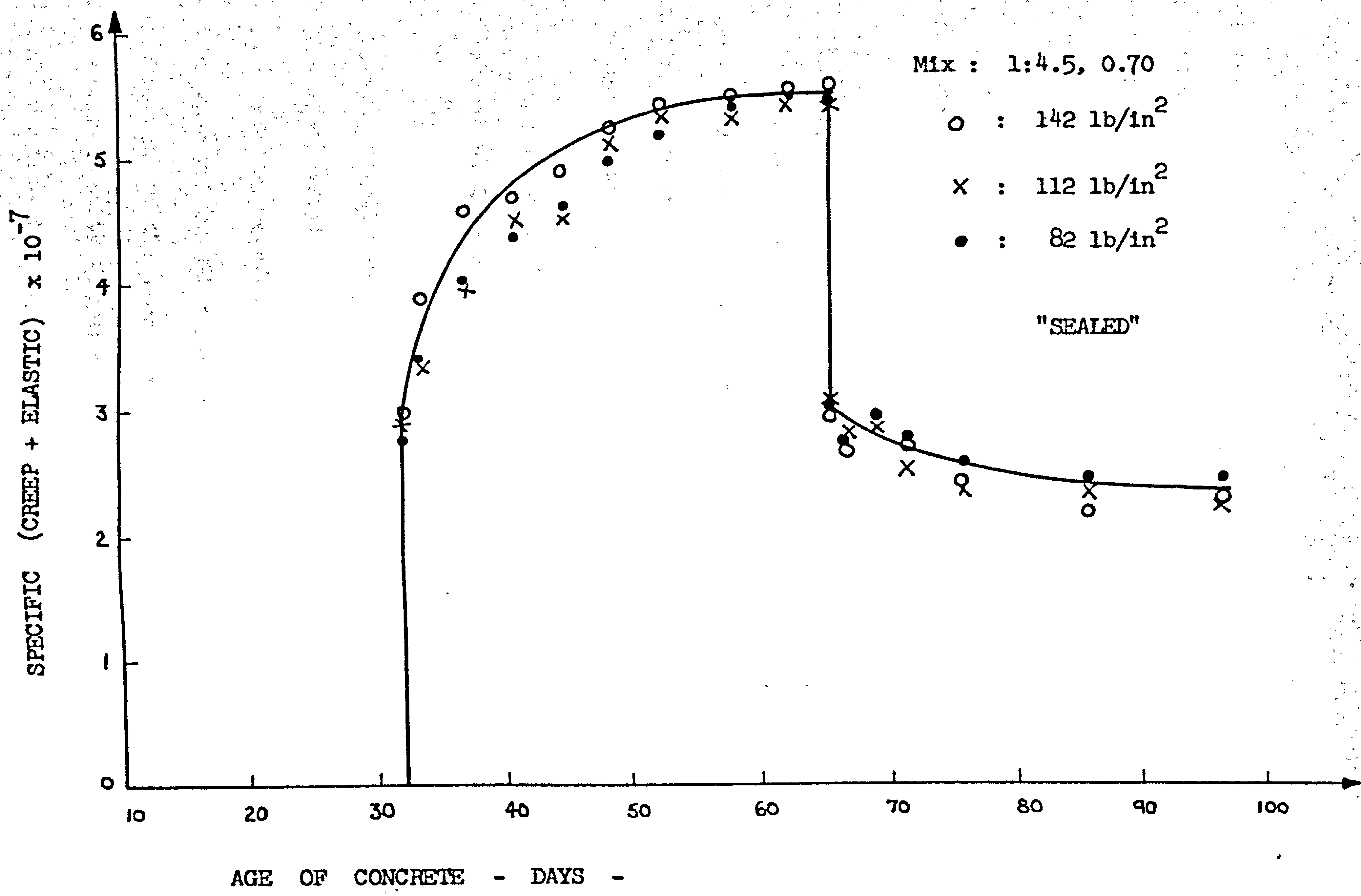


Fig: V.20. Creep of Sealed Concrete

Experiment 9

For this experiment 20 specimens (8 gauged and 12 ungauged) and three cubes were cast using batch C_{17}. Four gauged and 6 ungauged were unsealed and immersed in water after 24 hours from casting. The rest were kept sealed. All specimens were stored in the same room as those of Experiments 10 and 11.

Two sealed and 2 immersed specimens were loaded to 112 lb./in^2 and 2 sealed and 2 immersed specimens were used as control.

Experiments 10 & 11

The same number of specimens and procedure as in Experiment 9 were maintained for these experiments, except that loading started at the age of 30 and 28 days for experiment 10 and 11 respectively and that the stress used in Experiment 10 was 128 lb./in^2.

The results of Experiments 9, 10 and 11 are plotted and shown in Figures V.21, V.22 & V.18.

V. 2. f. Investigation into the Effect of Type of Aggregate on Creep

The preceding investigations had all been conducted on mixes using gravel aggregate. Experiment 12, however, using batch C_{15} was carried out on mix employing Leicestershire granite aggregate in order to compare the creep between it and those employing gravel aggregate. Granite was used only for the coarse aggregate, i.e. $\frac{3}{8}$ - $\frac{3}{16}$ in., the same fine aggregate was maintained as before.

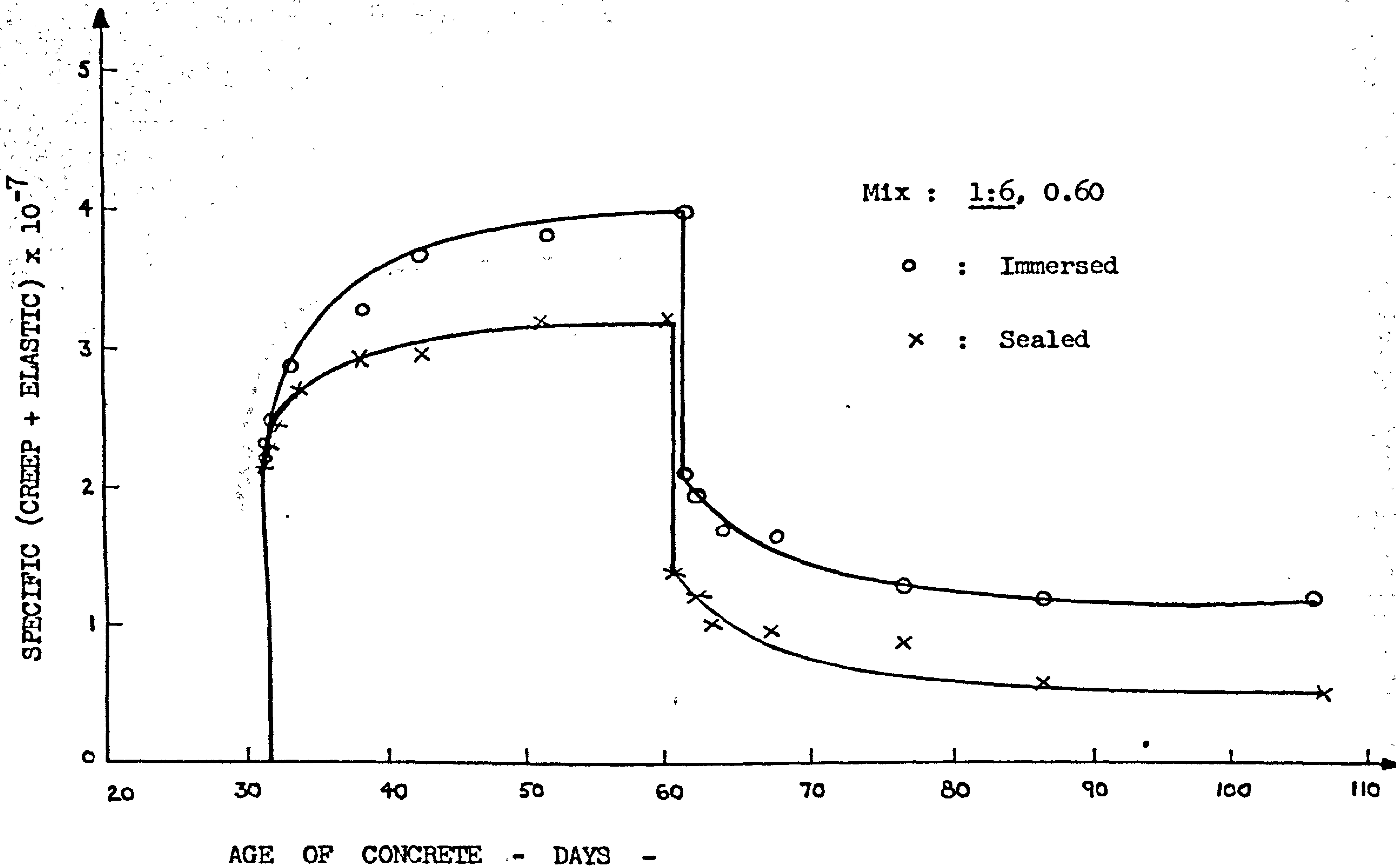


Fig: V. 21. Creep of Sealed & Immersed Concrete

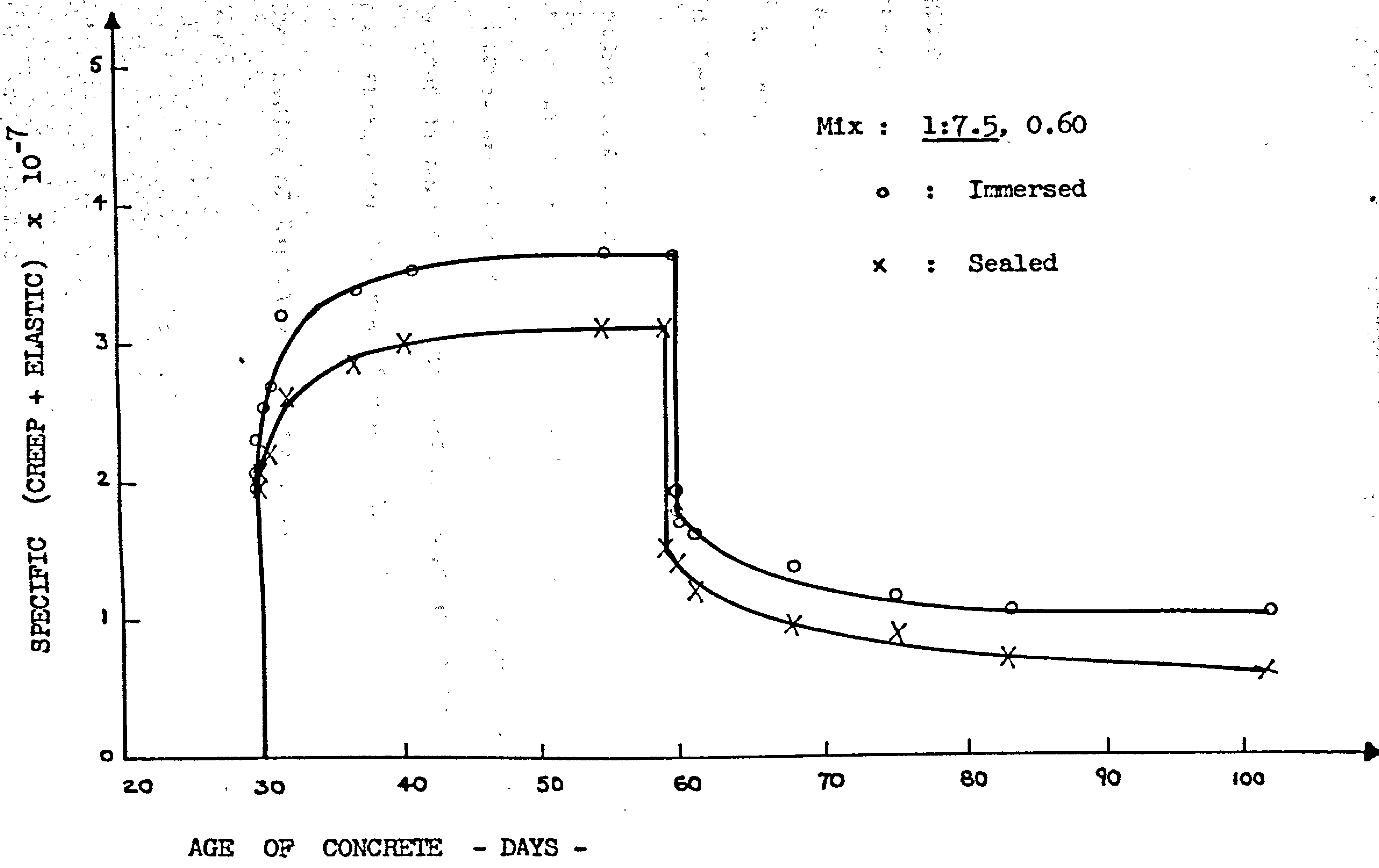


Fig: V. 22. Creep of Sealed and Immersed Concrete

MIX DETAILS

Aggregate-Cement Ratio = 4.5

Water-Cement Ratio = 0.60

Grading No. 2. Maximum Size Aggregate $\frac{3}{8}$ in.

Crushed Croft Leistershire Granite Aggregate $\frac{3}{8}$ - $\frac{3}{16}$ in.

$\frac{3}{16}$ in. River Gravel

Experiment 12

Sixteen specimens (6 gauged and 10 ungauged) and 3 cubes were cast. All the gauged and 6 ungauged specimens were unsealed and immersed in water 24 hours after casting. The rest were kept curing sealed in their jackets.

Loading started at the age of 30 days after testing 2 immersed specimens to failure to give an average strength of 260 lb./in^2. One level of stress was employed (122 lb./in^2) using two specimens. The rest were used as controls. The load was maintained for 30 days. Recovery of strain was observed for a period of 40 days.

The results of Experiment 12 are plotted and shown in Figure V.23.

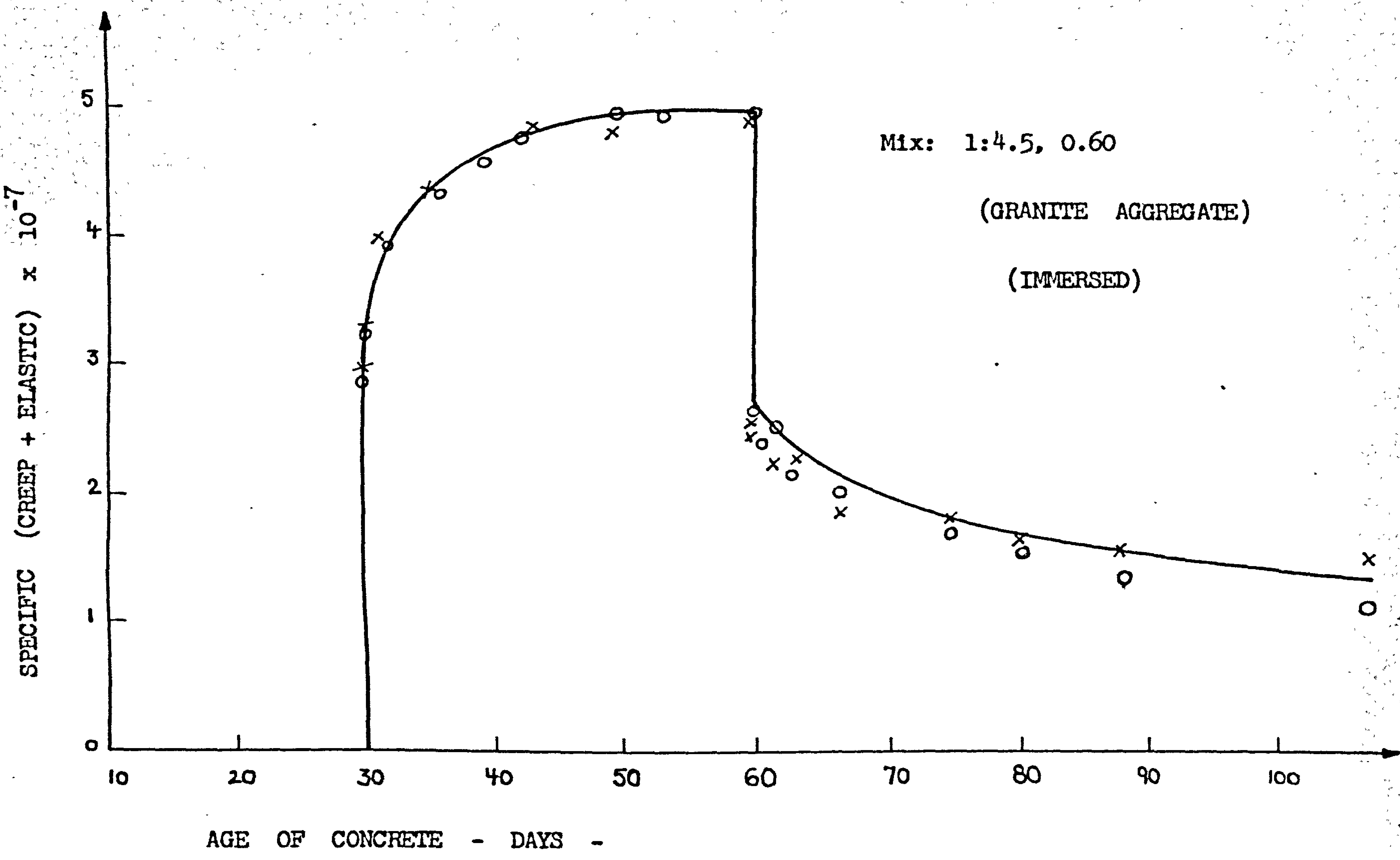


Fig: V.23. Creep of Concrete Made with Granite Aggregate

CHAPTER VI

DISCUSSION OF CREEP RESULTS AND PROPOSED MECHANISM OF CONCRETE BEHAVIOUR

VI.1. DISCUSSION OF CREEP RESULTS

VI.1.a. Effect of Ambient Humidity

The results of the experiments on the creep of sealed, wet and air drying concrete are summarised in Figures V.1. to V.7.

Sealed Specimens

Figure V.1. shows the relationship between the specific (creep and elastic) strains and time of sealed concrete loaded to 50% of ultimate at the age of seven days and for the unloaded control companions. It is noticed that the rate of creep decreases rapidly with time and the creep reaches a practically limiting value after several weeks of loading. It can also be noticed that the ultimate creep in this case constitutes a fraction of the elastic strain on loading below unity (about 0.66). The strain development in the unloaded control specimens (shown at the top part of Figure V.1.) over the period during which most of the creep strain had taken place is very small. A similar trend is observed on specimens loaded to 25% of their ultimate except that the ratio between the ultimate creep and elastic strain is nearly half that of the former value and that the ultimate creep is reached earlier which suggests a non-linear stress-creep relationship (see Figure V.2.) This point is discussed later.

Air Drying Specimens

Figure V.3. shows the relationship between the creep of air drying concrete and time for concrete specimens loaded to 25 and 50% of their ultimate at the age of seven days. The rate of creep in this case did not decrease as quickly as in the case of sealed concrete and the creep was still being observed when the specimens were unloaded at the ages of 140 and 600 days. However, for those specimens loaded to the lower stress (25% of ultimate), the rate of creep reached a low value after three months of loading. The ratios of creep to elastic strains after a period of loading of 140 days were 4.0 and 4.4 for concrete loaded to 25 and 50% of their ultimate respectively. Both concrete specimens loaded to 25% and 50% of their ultimate showed relatively small creep recovery on unloading ($\varepsilon_{cr} = 0.4\ \varepsilon_{Er}$). However, this recovery was still continuing when the test was terminated.

Large scattering of results for concrete loaded to 50% of ultimate compared to those loaded to 25% of ultimate and deviation from linearity increasing with time are observed in Figure V.3. while the creep of concrete specimens loaded to 25% of their ultimate seemed to be reaching a limiting value after 3-4 months of loading, the creep of those loaded to 50% was still continuing at a high rate (see Figures V.3. and V.4). These observations may indicate the occurrence of time-dependant microcracks which is particularly pronounced in specimens loaded to high stresses.

On unloading, at 140 days, the elastic recovery was observed to be more than the elastic strain on loading. This was particularly noticeable in specimens loaded to 50% of their ultimate. The following values were recorded:

$$\varepsilon_{Er} = 1.33\ \varepsilon_E \qquad (50\,\%\text{ of ult.})$$

$$\varepsilon_{Er} = 1.1\ \varepsilon_E \qquad (25\,\%\text{ of ult.})$$

The corresponding values for the two specimens loaded to 50% of their ultimate for 600 days were as follows:

Specimen 1 - $\varepsilon_{Er} = 2\ \varepsilon_E$

Specimen 2 - $\varepsilon_{Er} = 1.3\ \varepsilon_E$

When these two specimens were finally tested to failure their strength were 65% and 85% of that of the unloaded control specimens for specimens 1 and 2 respectively.

All these findings seem to support the opinion of time-dependant microcracks development.

The creep strain after 600 days of loading specimens to 50% of their ultimate was as much as seven-times the elastic strain on loading (specific creep = 1.4×10^{-6}). This value is much higher than the corresponding values generally obtained in compression although a direct comparison between concretes of the same mix, loaded at the same age and stored under the same humidity conditions was not obtained.

The specific creep of concrete specimens loaded to 25% of their ultimate for 140 days was 0.75×10^{-6}. This value is still higher than that obtained in compression but the trend and shape of the curves are similar.

Figure V.5. shows the strain developments in the unloaded air drying concrete specimens. It can be noticed that the shrinkage strain during the loading period is more than double the creep strain of specimens loaded to 140 lb/in^2. The shrinkage strain in the unloaded specimens during 600 days was 450×10^{-6} while the corresponding strain of the loaded specimens was 255×10^{-6} to give a creep strain of 195×10^{-6}.

Immersed Concrete

A typical creep curve of immersed concrete is shown in Figure V.6. The trend and shape is similar to that of sealed concrete. The rate of creep seems to diminish quickly with time to reach almost zero after 30-50 days of loading. The creep strain is, however, higher than that of the sealed concrete but much lower than of the air drying. The ultimate value in this case was a fraction of the elastic strain equal to 1.1(i.e $\varepsilon_c = 1.1\ \varepsilon_E$). The creep results for immersed concrete were very consistent and the two specimens employed for that test gave almost identical readings.

Conclusion

The creep of sealed, immersed and air drying concretes are compared in Figures VI. 1. and Vl. 2. which cover the creep for early age after loading as well as the long-term creep. Comparison of these results with those published for compression creep leads to the following conclusions:

1. The tensile creep curve of air drying concrete loaded to low stresses is similar to that of compression creep.

2. The specific tensile creep of air drying concrete was observed to be higher than that of compressive creep especiallyat a relatively high stress-strength ratio.

3. The elastic recovery of air drying concrete was higher than the elastic strain on loading while the opposite is generally obtained in compression.

4. The creep of both sealed and immersed concrete both reach limiting values after several weeks of loading, the limiting creep of immersed concrete being nearly twice as much as that of sealed concrete.

5. The specific creep of air drying concrete is much larger than that of sealed and immersed concrete for specimens that had not been dried out prior to

loading. The creep of air drying concrete after a loading period of 6 months was observed to be as much as nine-times that of sealed concrete and 4.5 times that of immersed concrete under the conditions of the test pefgrmed. These figures were much smaller over the first period after loading as seen in Figure VI. 2.

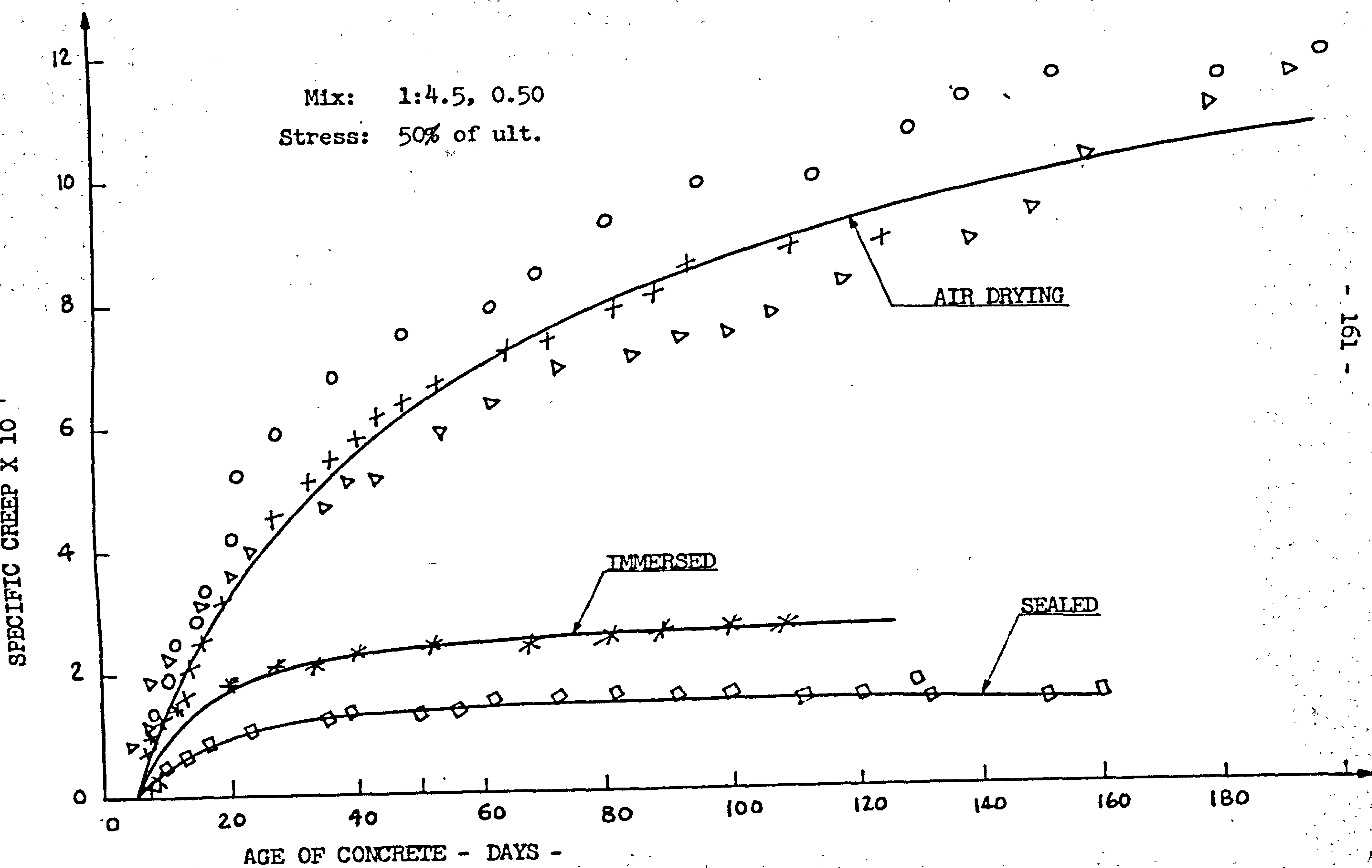


Fig. VI.1. Creep of Concrete Stored Under Different Moisture Conditions

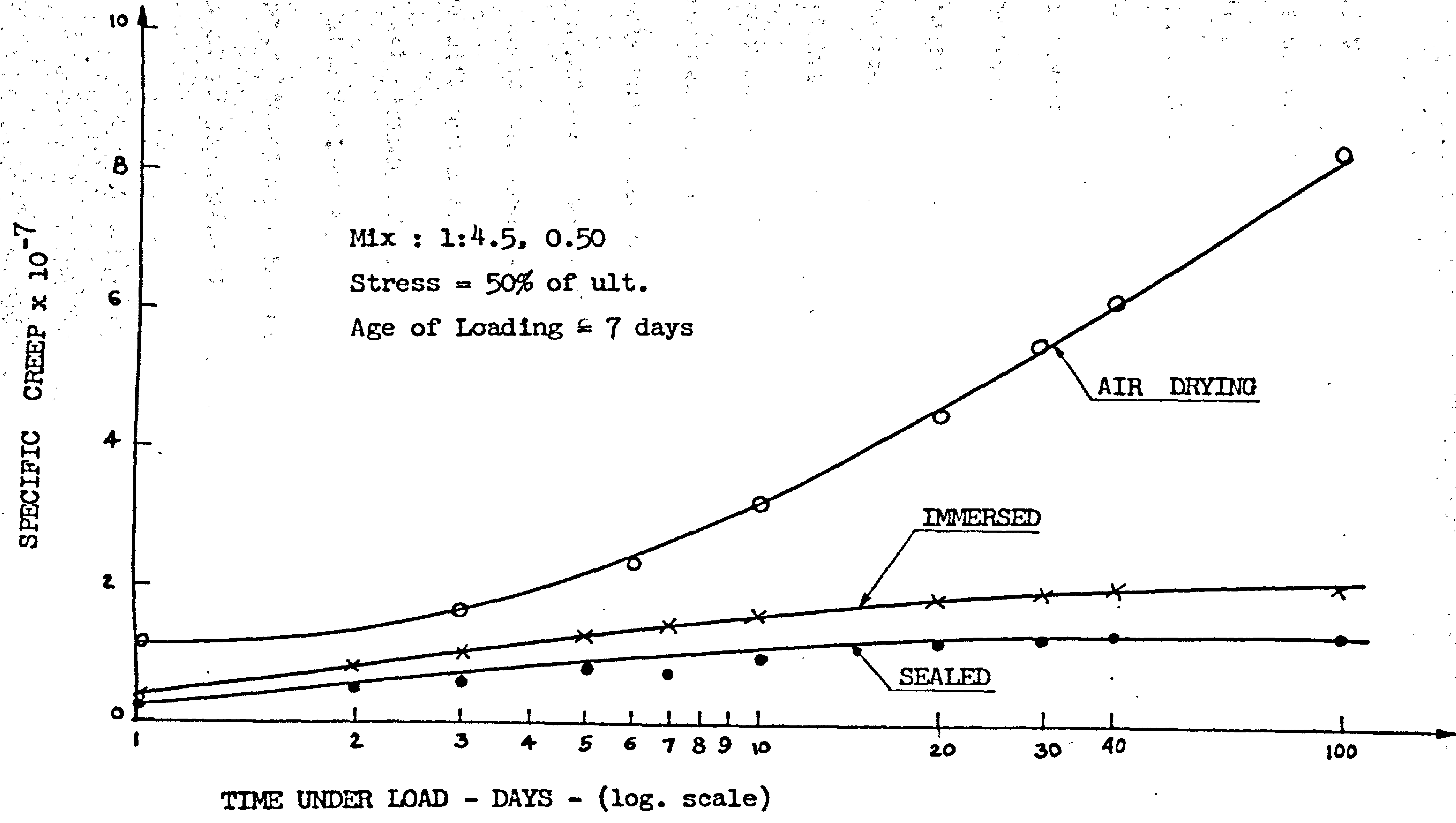


Fig. VI.2. Creep of Concrete Stored under Different Moisture Conditions at early days after loading.

VI.1.b. The Effect of Age at Loading

The results of this experiment are summarised in Figures V.7., V.8., VI.3. and VI.4. The general trend obtained is the same as in compression, that is the creep reduces with increasing the age at loading. The creep of all the immersed concrete specimens loaded at the different ages (1,3,7 & 28 days) seems to reach a limiting value after several weeks of loading. This appears to be reached earlier with concrete loaded at 1 and 3 days. The elastic recovery is obsêrved to be smaller than the elastic strain at loading for all the tests. The ratio of $\frac{\varepsilon_{Er}}{\varepsilon_E}$ is smaller the younger the concrete at the time of loading which is to be expected beoause the increase of modulus of elasticity is greater at the early ages. The elastic recovery was even observed to be smaller than the corresponding elastic strain measured on the control specimens. This suggests that the load may have activated the hydration at local points due to the reorganisation of water film within the concrete and the accessibility of water to the opened-up cracks at loading causing them to be partly filled with hydration products. The ratio of creep recovery to creep for equal periods of loading increased with increase of age at loading. It is also noticed that the specific creep recovery of the concrete specimens loaded at 1 day was the smallest while that for specimens loaded at 28 days was the largest.

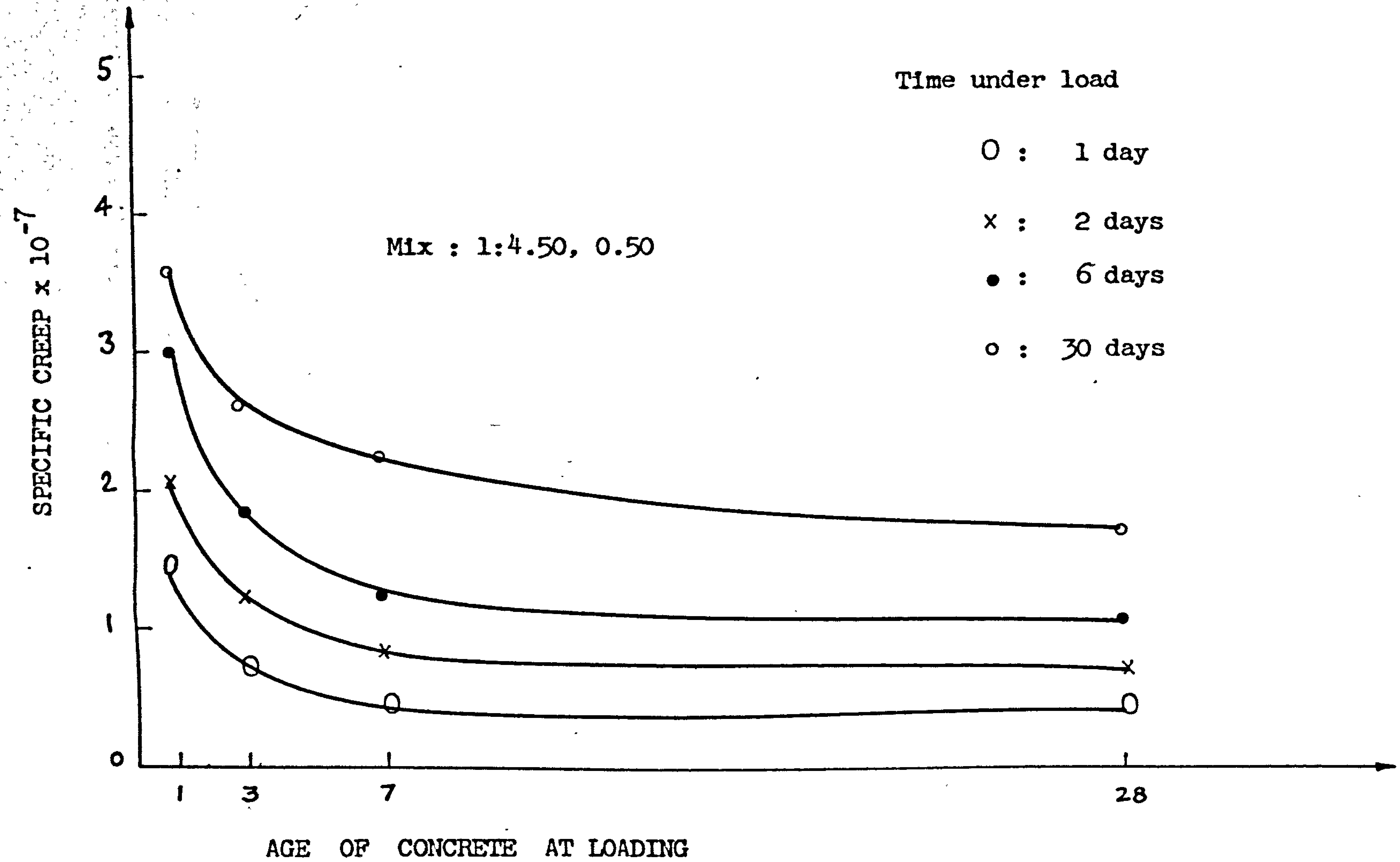


Fig. VI.3. The Effect of Age at Loading on Creep of Immersed Concrete

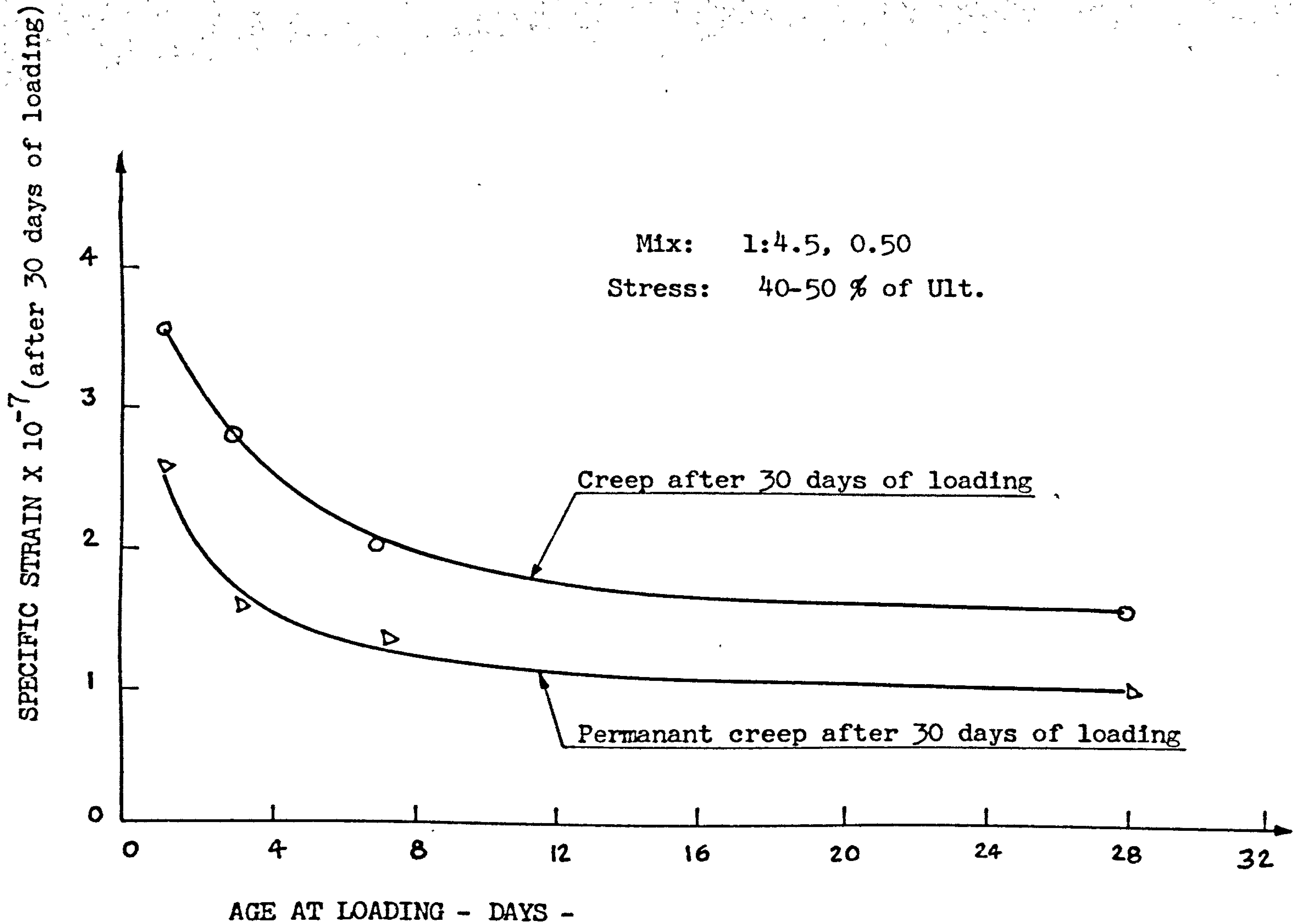


Fig. VI.4. The Effect of Age at Loading on the Creep of Immersed Concrete.

Conclusion

The following conclusion can be drawn from the tests on the effect of age at loading on the tensile creep of immersed concrete:

1. The creep decreased with increasing the age at loading
2. The elastic recovery was smaller than the elastic strain at loading particularly for concrete loaded at early ages.
3. The elastic recovery was smaller than the elastic strain measured on specimens which had previously been unloaded when used as controls.
4. As previously observed the creep of concrete specimens loaded at the different ages reached a limiting value after a period of several weeks and this decreased with decreasing age at loading.
5. The ratio of creep recovery to creep was smaller the younger the concrete at loading.

VI.1.c. The Effect of Level of Stress

The results of tests to measure the effect of level of stress on tensile creep are shown in Figures V.10. to V.17.

Figure V.10 & V.11 show the creep of sealed and immersed concrete respectively, loaded to different levels of stress at an age of 3 days. Both figures indicate a linear relationship between creep and applied stresses although the stress at some levels exceeded 60% of the ultimate strength at the time of loading. This linearity can be seen in Figure V.12. where the specific creep is plotted against time. Figure V.12. shows also that the average specific creep of sealed and immersed concrete are nearly the same for the first 1-2 days after the application of load but their curves start to diverge gradually thereafter to reach a wide gap after 20 days of loading, the creep of immersed concrete being the larger. It can also be noticed that the creep recovery of the sealed concrete is larger than that of the immersed concrete and that the permanent creep of immersed concrete is almost double that of the sealed.

Figure V.14. shows the effect of level of stress on the creep of sealed concrete having the same mix as those represented by Figures V.10., V.11. & V.12. but loaded at the age of 20 days. The levels of stress are about 65, 50 & 30% of the ultimate strength at the time of loading. The two upper levels being the same as for those loaded at the age of three days. In this experiment no linearity was found between creep and stress and the specific creep increased with increasing stress.

It is, however, noticed that the elastic recovery is much smaller than the elastic strain at loading and the difference could not be accounted for by the increase in the modulus of elasticity with time between the ages of 20 and 40 days alone. The average ratio of ε_{Er} to ε_E was 1 : 0.75.

Figures V.15. & V.17. show the effect of level of stress on the creep of sealed concrete having the same aggregate-cement ratio but water-cement ratios of 0.42 & 0.70 respectively, while Figure V.16 shows the effect of level of stress on the creep of immersed concrete with 0.60 water-cement ratio. It is noticed that the creep-stress relationship is linear for the immersed concrete and for the sealed concrete made with a water-cement ratio of 0.70 but is non-linear for the sealed concrete made with a water-cement ratio of 0.42. It is also noticed that the creep recovery of the sealed concrete made with a low water-cement ratio was more than that made with a high water-cement ratio. The elastic recovery was smaller than the elastic strain at loading in all the three experiments.

Conclusion

In general it can be concluded that:

- the creep increased linearly with the applied tensile stress for both immersed concrete and sealed concrete loaded at early ages and with high water-cement ratio.

The sealed concrete loaded at later ages (20 & 30 days) and made with water-cement ratios of 0.50 & 0.42 gave non-linear stress-creep relationship in which case the specific creep increased with the applied stress.

- the elastic recovery was smaller than the elastic strain at loading in both sealed and immersed concrete made with different water-cement ratios.

- the creep recovery was relatively higher in sealed concrete than immersed concrete especially for that made with low water-cement ratio.

VI.1.d. The Effect of Water-Cement Ratio

Figures V.18, V.19. and V.20. show the specific creep plotted against time for concretes having the same aggregate-cement ratios and water-cement ratios of 0.42, 0.60 and 0.70. The same mix but of water-cement ratio equal to 0.50 has been covered earlier and is compared with the others in Figure VI.5. The creep of sealed and immersed concrete is shown to increase with increasing water-cement ratio. It is noticed from Figures V.18. and V.19. that the creep recovery is relatively larger in sealed concrete than in immersed concrete. This is particularly clear in Figure V.18 where the creep of sealed concrete was almost recovered over a period equal to that of the loading.

It is also realised from these figures that the creep of sealed and immersed concrete reached a limiting value after few weeks of loading except in the case of 0.70 water-cement ratio. The creep recovery of the concrete made with a water-cement of 0.70 was also still continuing when the test was terminated. Sealed concrete only was used with this water-cement ratio.

Figure VI.5. shows the specific creep after 30days of loading plotted against water-cement ratio. It is noticed that the increase in the specific creep was much more pronounced in the case of the higher range of water-cement ratios than the lower range particularly for sealed concrete.

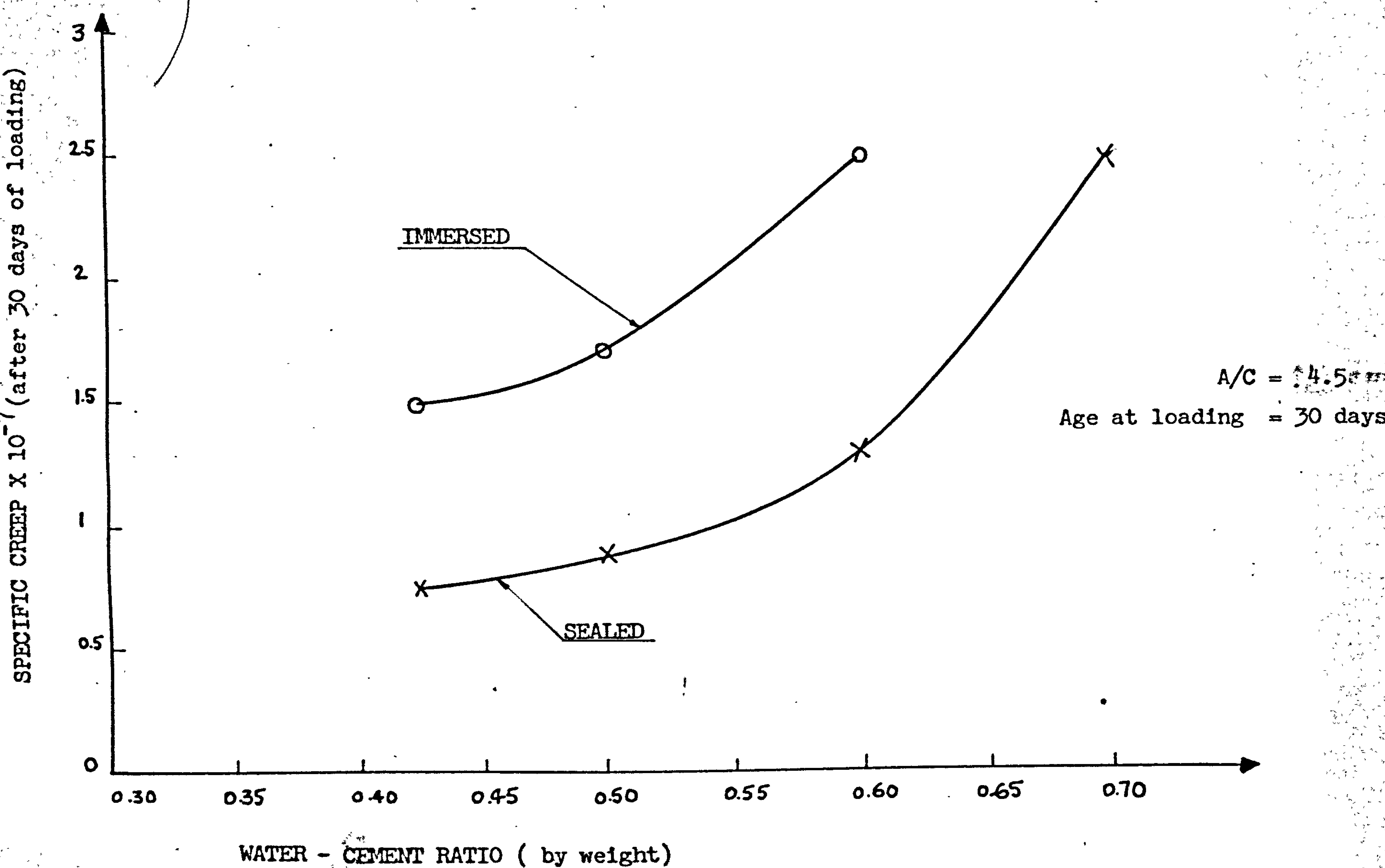


Fig. VI.5. The Effect of Water - Cement Ratio on the Creep of Concrete.

Conclusion

It can be concluded that:

1. The tensile creep, as in compression, increases with increasing the water-cement ratio of the mix. The increase being more pronounced at the higher range of water-cement ratio than at the lower range.

2. The creep of both sealed and immersed concrete made with the different water-cement ratio reaches a limiting value after several weeks from loading. This period was observed to be longer for sealed concrete made with high water-cement (0.70).

3. The elastic recovery was always considerably smaller than the elastic strain.

VI.1.e. The Effect of Aggregate-Cement Ratio

The preceding investigations were all carried out on concrete made with an aggregate-cement ratio of 4.50. Two more aggregate-cement ratios, 6 and 7, were also used in order to compare the effect of the three ratios on the tensile creep. The results of these two experiments are shown in Figures V.21. and V.22.

As observed earlier the creep of immersed concrete was higher than that of the sealed concrete but the difference decreased with increasing aggregate-cement ratio (see Figure VI.6.)

The ratio of creep recovery to creep is seen to increase with increasing aggregate-cement ratio especially in immersed concrete. It is also noticed in Figure VI.7 that the specific creep of sealed concrete after 30 days of loading did not decrease very much with increasing aggregate-cement ratio while the effect is noticeable on the creep of immersed concrete.

VI.1.f. The Effect of Type of Aggregate

Only one type of aggregate, that is gravel aggregate was used throughout the experiments which have been discussed.

Figures V.23, however, shows the specific creep of immersed concrete made with granite aggregate and a water-cement ratio of 0.60. This can be compared with Figure V.18 which shows the corresponding specific creep of concrete made with gravel aggregate. The specific

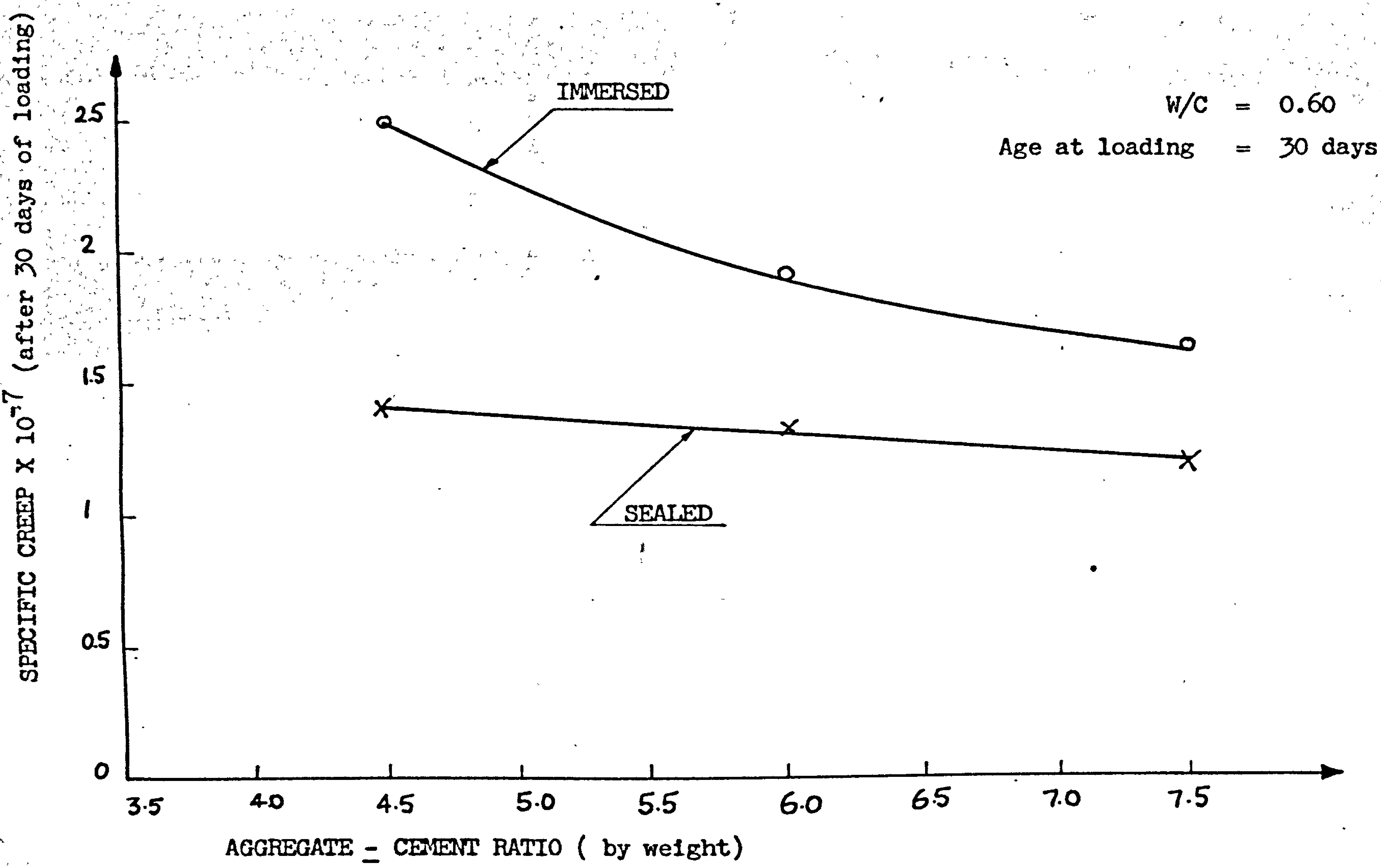


Fig. VI.6. The effect of aggregate - cement ratio on the creep of concrete.

creep of the concrete made with granite aggregate was observed to be smaller than that made with gravel aggregate while the creep recovery is larger. The creep and creep recovery as fractions of the elastic, strain and elastic recovery and the ratios of creep recovery to creep for both concretes were as follows:

Concrete made with Granite Aggregate	$\varepsilon_c = 1.05\ \varepsilon_E$	After 30 days of loading
	$\varepsilon_{cr} = 0.42\ \varepsilon_{Er}$	
	$\varepsilon_{cr} = 0.38\ \varepsilon_c$	
Concrete made with Gravel Aggregate	$\varepsilon_c = 0.70\ \varepsilon_E$	
	$\varepsilon_{cr} = 0.55\ \varepsilon_{Er}$	
	$\varepsilon_{cr} = 0.63\ \varepsilon_c$	

The specimens used in both cases were loaded to 50% of their respective ultimate strength.

The difference may be due to the better bond characteristic offered by the crushed granite aggregate. However, as only one batch was made with granite aggregate there is not enough experimental evidence for a conclusion to be made.

General Conclusion

The following points may be concluded on the tensile creep experiments:

1. The creep of air drying concrete is much larger than that of sealed or immersed concrete.

2. The creep of immersed concrete is nearly double that of sealed concrete while the relative creep recovery is less.

3. The elastic recoveries of sealed and immersed concrete are more than the elastic strain on loading or that measured on unloaded control specimens while the reverse was generally observed on air drying concrete.

4. The creep of sealed and immersed concrete reached a limiting value after several weeks of loading while that of air drying concrete was still continuing after one year.

5. The creep decreased with increasing age at loading

6. The relative creep recovery was larger the older the concrete at loading.

7. The tensile creep generally increased linearly with the applied stress except for sealed concrete made with low water-cement ratios. However, the creep in the latter was very small.

8. The creep increased with increasing water-cement ratio particularly at the higher ranges.

9. The creep decreased with increasing aggregate-cement ratio. This was more pronounced in immersed concrete. However, if we plot specific creep against cement paste fraction by volume, as shown in Figure VI.7., we find that changing the water-cement ratio and the aggregate-cement ratio fall into completely distinct and independent curves. The effect of water-cement ratio being much more influential.

The points concluded above are explained in the mechanism discussed in the following pages.

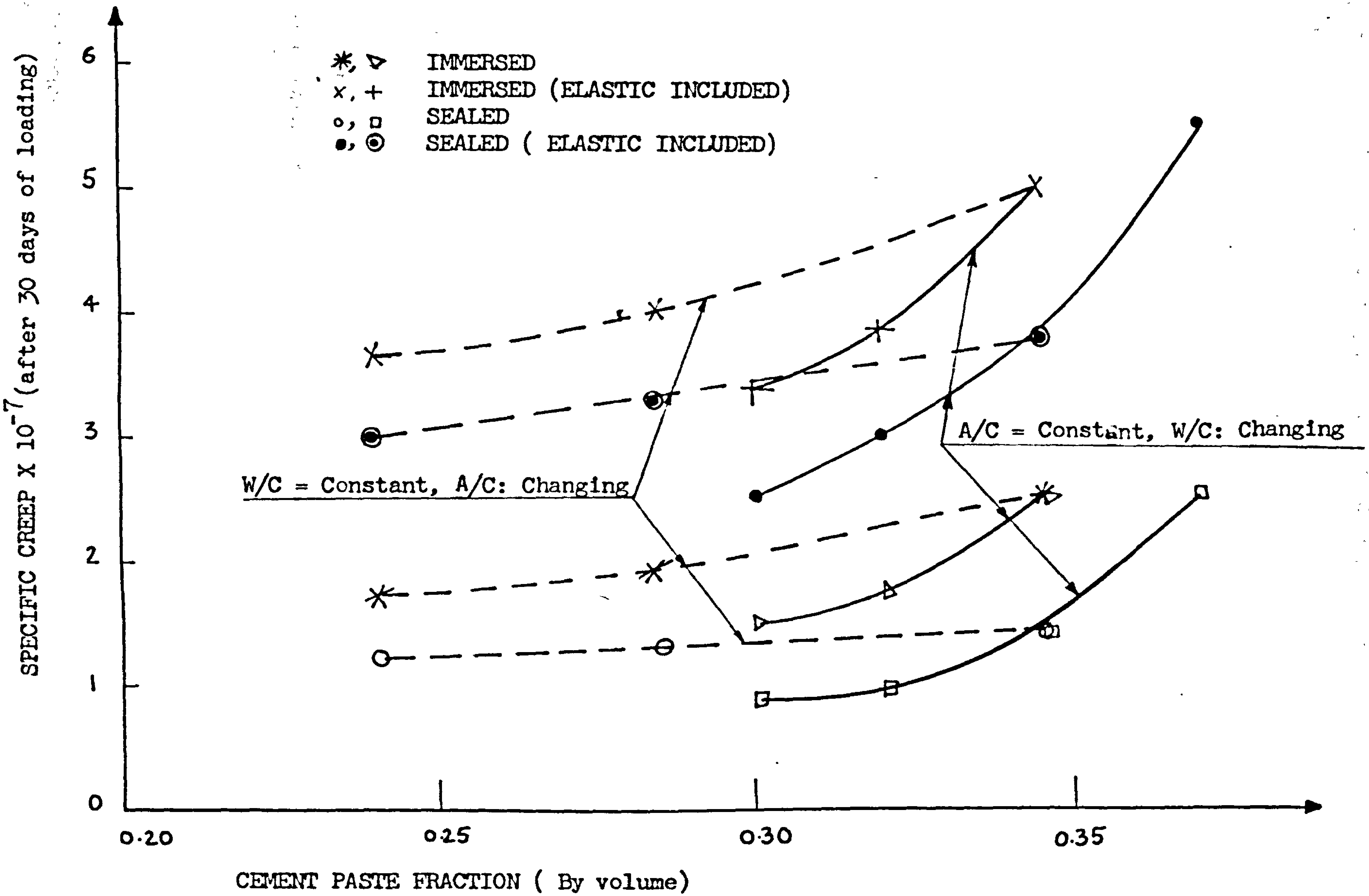


Fig. VI.7. The Effect of A/C & W/C on Creep.

VI.2. PROPOSED MECHANISM

The following mechanism is based on the present knowledge of the internal structure of concrete and on the observations made on the experimental results obtained by the author and other supporting experimental evidence within a framework of a Seepage-Plastic theory.

VI.2.a. Structural Details

As previously mentioned, the hydration of cement produces a coherent mass of which primary particles are of colloidal dimensions. This material, knownas cement gel, is the body responsible for creep, shrinkage and swelling in concrete. The particles of cement gel are probably sheets of impure colloidal tobermorite having an average thickness of about 30Å. The interstitial spaces between these particles have an average width of 15-18Å. However, solid links are believed to exist between the particles which provide the cement gel with rigidity and limited swelling capacity. The solid links are formed in the narrow spaces between the cement grains where the hydration products are first deposited resulting in much higher density of tobermorite crystallites[82]. The larger voids between the cement grains are filled with a lower density tobermorite gel.

The gel pores and the narrow spaces between the gel particles are filled with water at saturation. If we assume that the surface of the gel particle is open to an atmosphere of saturated vapour, a thickness of up to 5 molecules of water (13Å) would build up over that surface[83]. This thickness varies with the ambient vapour pressure. . This means that if the distance between the gel particles is less than 26A° there will not be enough space for an unobstructed adsorprtion to occur over that surface. These places which have thickness below 26A° form the areas of hindered adsorption[83]. The pressure exerted over these areas (swelling pressure) is balanced by the tensile stresses over the areas of solid links and by the surface forces (Van der Waals type). The ultimate equilibrium would involve an overall balance of stresses caused by surface forces of attraction (which vary with the distance between the surfaces and the nature of the filling), disjoining pressure and chemical bonds. When aggregate is added to the cement paste its rigidity and bond characteristics with cement paste will also contribute to the final state of equilibrium of the concrete.

The description of capillary voids, gel pores and the concrete as a whole which was discussed in Chapter I will be postulated in the discussion of the creep mechanisms.

VI.2.b. Tensile Creep Mechanism

On subjecting a concrete specimen to a sudden tensile load, the stress first exhibited over its section will be considered distributed over the following components:

1. Coarse aggregate
2. Fine aggregate and unhydrated cement
3. Solid links in dense cement gel

4. Adsorbed water filling the narrow distances between the particles of metastable cement gel.

5. Surface forces.

At a time t, after a certain period of sustained loading has elapsed, the distribution of the stress over the composite material would differ from that which preceded it until a final equilibrium has been reached and creep ceased to occur. The load,however, will always be considered to be supported by the same components (the five stated above) with a change in its distribution. This is envisaged as follows:

The aggregate and solid links bridged between cement particles are considered stable and behave elastically (that is exhibit no creep). However, the adsorbed water (which was in a supposed state of equilibrium with the ambient vapour pressure surrounding it prior to loading) will not remain stable after the application of load and will gradually adjust its thickness in the direction of stabilisation with the ambient vapour pressure within the concrete and finally with the vapour pressure of the surrounding medium if the concrete is unsealed.

Immersed Concrete

The voids inside the concrete are full of water. The water film tension over the solid surfaces of cement gel particles is released and the distance between the particles of cement gel is a maximum. The water in the areas of hindered adsorption undergoes compression.

The aggregate behaving as restricting inert material will be mostly under tension and the bond between the aggregate and cement paste is in its weakest state. Few bond cracks have already existed and the extent of these may determine the state of stress over the bonded areas. A state of stress in equilibrium with the internal forces is supposed to exist.

When a tensile load is applied, all the components (including the load-bearing water) behave elastically for a short period of time. The existing bond cracks and microcracks would open up and some new ones may form which are able to close up on quick removal of the load.

With time and due to the disturbed hygral equilibrium, some water would seep to the narrow spaces in order to restore the state of equilibrium. This will cause a release of some of the load which was supported by the water to the other components of the structure. In doing so, it increases the tensile stresses already existing in the aggregate and also the bond stresses between the aggregate and cement paste and perhaps causes some new bond cracks to occur. This operation continues until a state of internal equilibrium is reached with the external force.

While the creep is occuring, the hydration may be activated over the areas of newly-formed or opened-up bond cracks and microcracks. New links between the pores of the structure may also be established. These cause a change of the structure and prevent full recovery of strain on unloading.

High average stress on loading may cause the bond stress between the aggregate and cement paste to reach a higher value than that of the bond strength between the two materials, after a certain creep strain has occured. When this is reached over a critical area it may spread out and cause failure.

This bond stress may either be reached within a short period of time by applying a high average stress, a long period with a lower stress or may never be reached. It will, however, be mostly dominated by the strain in the aggregate, which is assumed to behave elastically, as the bond stress between it and the cement paste is proportional to the strain in the aggregate. The ultimate strain before failure may thus depend on the nature of bond between the aggregate and cement paste and on the modulus of elasticity of the aggregate. This may not be the case with sealed or air drying concrete.

Sealed Concrete

A similar mechansim to immersed concrete is envisaged with the following exceptions:

1. The voids in the concrete may not all be filled with water.

2. The specimen is not at maximum swelling and the distance between the gel particles is not at maximum.

3. The state of bond stress between the aggregate and cement paste differs from that of the immersed condition and will depend on the shrinkage or swelling that had taken place at the time under consideration.

4. The aggregate may therefore be under lower tensile stresses or even under low compressive stresses.

5. Bond cracks and microcracks may be less extensive since the volume changes are smaller.

6. The compressive stresses over the load-bearing water should be less than in the case of immersed concrete as the vapour pressure of the ambient medium is lower.

When a tensile load is applied an elastic strain is developed in the components of the composite material and this should not differ much from that of the immersed condition in the case of low stresses with few microcracks occuring, provided the water cement ratio is high enough to allow the same degree of hydration as in the immersed conditions.

The adsorbed load-bearing water, behaving elastically for a very short period after the application of load, soon attracts water from its surroundings, as the surfaces in the region of hindered adsorption become capable of adsorbing more water. However, as the ambient atmosphere within the concrete is sealed from the outside, the final equilibrium is reached with a decrease in the vapour pressure within the concrete until it balances the forces over the adsorbed water caused by the external load. This decrease in vapour pressure results in forces over the adsorbed water preventing it from reaching a state of expansion as in the case of immersed concrete and thus resulting in less creep.

The small expansion of the adsorbed water results in a release of load to the surrounding elastic phase which is less than the corresponding load released in the case of immersed concrete. The bond stresses around the aggregate are not as high as in the case of a larger degree of volume change and the likelihood of gradual bond cracking due to the relief of some of the stresses to the aggregate is less. This should also lead to less creep strain.

Air Drying Concrete

The creep mechansim in this case, although basically the same as in the other two cases, may be affected by other factors due to the shrinkage strains and the consequences of differential shrinkage stresses and internal stresses due to the restraint offered by the elastic phase.

After shrinkage has taken place and the vapour pressure with the concrete has dropped to 0.75-0.85 the majority of adsorbed load-bearing water may be in a tension state balanced by the surrounding elastic phase which would have to undergo compression. The aggregate, while restraining the shrinkage, is mostly in a compression state. A few bond cracks might have developed around the aggregate particles in the areas of tensile stresses and the degree of these may determine the state of stress in the uncracked areas. Other microcracks may have occured in the cement gel itself due to the restraint offered by the aggregate and the solid links or due to the differential shrinkage stresses. These cracks may be located in the comparitively large distances between the gel particles since the Van der Waals forces will have been overcome at these points. The water vapour pressure is assumed to be changing as drying continues.

When a tensile load is applied an elastic strain is exhibited the degree of which would depend on the number of cracks that had occured due to shrinkage stresses and the rigidity of the components forming the concrete by the time of loading.

As in the other two cases, the water in the narrow spaces is disturbed and would require adjustment of thickness to allow for the new state of stress and to stabilise with the vapour pressure surrounding it. When shrinkage is taking place at the same time, the film tension over the adsorbed water in the system would also exert stresses over that load-bearing water which result in a strain of opposite direction to that caused by the load. The load can therefore be considered as a force acting against the forces of shinkage and so reducing the strain compared with that expected if no load had been applied.

However, the two forces, although working in different directions, both exert ensile stresses over the load-bearing water and in many places may cause rupture especially at the surface of the specimen where the tensile stresses are at maximum due to the addition of differential shrinkage stresses. The continuous changing of the water vapour pressure of the atmosphere makes the differential shrinkage stresses more influential and helps to produce further microcracking.

The microcracks occur in the unloaded specimens due to the shrinkage stresses alone, while in the loaded specimens due to the combined shrinkage and external load. As the concrete is of a heterogeneous nature, the dimensions of the particles and the distances between them change from one point to another, but it is possible to envisage statistically a certain proportionality between the microcracks and the applied stresses. This proportionality does not necessarily exist and may especially not exist at the higher level of stresses where the microcracks could be joining up to form bigger cracks which may ultimately lead to failure.

However, another mechanism may be at work in the core of the specimen due to the differential shrinkage stresses. The differential shrinkage sets up compressive stresses in the core causing compressive creep to occur and consequently the water to seep to the outer parts of the specimen and finally to evaporate. This may accelerate the stabilisation of stresses across the section. The application of the tensile load in this case acts against this operation and may level out the compressive stress in the core, reduce it or even change it to tension. In all cases this slows the movement of the water from the middle to the outside. This would result in a decrease in shrinkage strains or in a creep strain.

In the case of a concrete that had been drying for a good period (4-6 months) and which had almost reached a stable value at the time of loading the creep strains would be expected to be much smaller for the following reason:

The time-dependant shrinkage stresses would practically have ceased and the stress over the loaded specimens would be the result of shrinkage stresses that had already existed together with the stresses due to the applied load. As the shrinkage has nearly ceased these stresses would not change very much with time and the only change would be due to the release of some of the stresses in the elastic phase caused by the readjustment of the thickness of the thin layers of adsorbed water left in the system. However, this could still cause some time-dependant microcracks to occur the degree of which would depend on the level of applied stress.

The water needed for the adjustment of the thickness of the load-bearing adsorbed water should be proportional to the water already existing in the system and to the stress already acting on this water.

Loading a dried specimen would thus be expected to yield a small creep.

VI.2.c. The Effect of Water-Cement Ratio on Creep

Water-cement ratio controls the distance between the cement particles while in suspension just after mixing. The products of hydration which are deposited in the spaces originally occupied by water will have more spaces to accommodate them in the case of higher water-cement ratio and thus will be less dense. The solid links bridging the short distances of the cement particles will also be fewer. The pores filled with water or air should be larger and the channels within the concrete may be linked more directly. The quantity of adsorbed water should also be greater and the distances between the particles of cement gel filled with adsorbed water should be larger.

The result is that when the load is applied, and due to the fewer solid links in the concrete a higher share of the load would first be supported by the adsorbed load-bearing water which will result in a greater creep.

VI.2.d. The Effect of Aggregate-Cement Ratio

Increasing the relative quantity of aggregate does not appreciably alter the structure of cement gel. The creep characteristics would therefore not be expected to change. On the other hand increasing the aggregate-cement ratio leads to a decrease in the relative volume of cement gel in the whole concrete and thus a decrease in the share of load taken by the active material. Therefore, on the application of a load, the greater the aggregate-cement ratio the smaller the intensity of stress supported by the cement gel and consequently that supported by the load-bearing water.

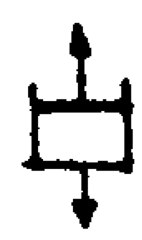
While creep is occuring, a relatively large proportion of solid material (aggregate in this case) would restrict the expansion of the load-bearing water by taking a large share of the load released due to this expansion. This should reduce the stresses over the adsorbed water rapidly, thus reducing the rate of creep as well as its amount. A stiffer aggregate should have a similar effect.

While increasing the aggregate-cement ratio results in a reduction of creep similar to that brought about by a reduction of water-cement ratio, the mechanism is completely different. Decreasing the water-cement ratio is a preventive measure in which the structure of cement gel is altered to a more stable one while an increase of the aggregate proportion is a restrictive measure.

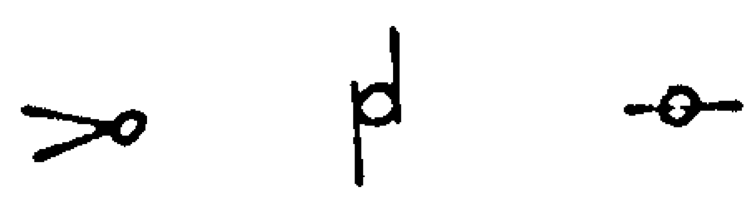
VI.3. EXPLANATORY MODEL

The proposed model aims only to clarify the mechanism discussed earlier. It does not, however, represent the actual structure.

The elements used in this model describe the following actions:

Viscous flow 

Elastic strain 

Cracking points, capable of re-healing 

The adsorbed load-bearing water and surface tension between its solid boundaries are represented in the assembly A. This assembly deforms elastically on loading followed by a delayed elastic strain, all recoverable.

Element B, demonstrates cracking and healing with time inside the cement gel.

Element C, represents the solid links and the dense stable and elastic material in the cement gel and is purely elastic.

Element D, represents the aggregate surrounded by the cement gel. Elastic and bond cracks can be exhibited. Healing of bond cracks is also possible.

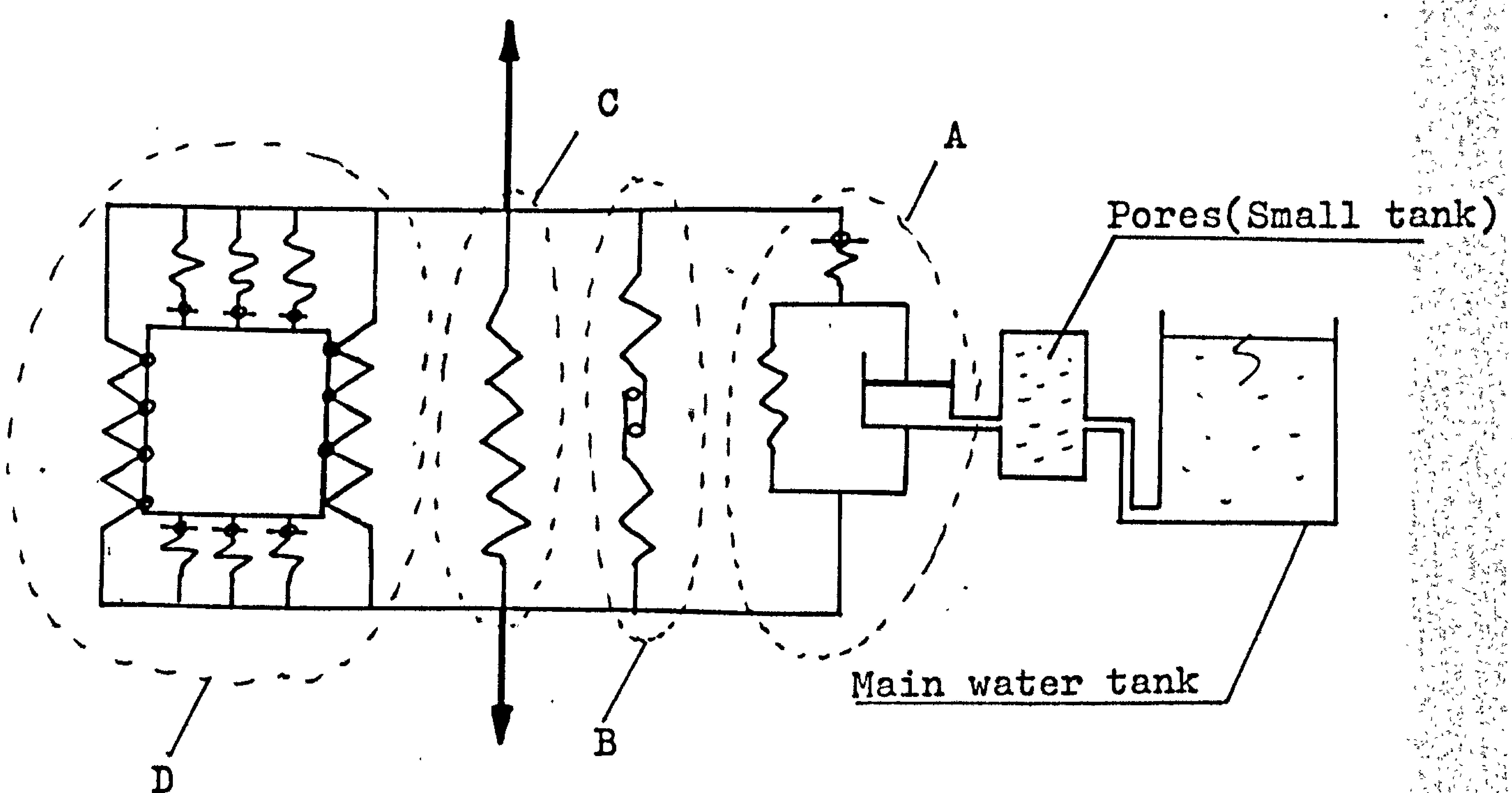


Fig. VI.1. Explanatory Model

The elements used could have been reduced to fewer elements without changing the behaviour of the model but the proposed arrangement is thought to make the model more representative of the actual structure.

A decrease in the water-cement ratio or an increase in the maturity of the concrete may be represented by increasing the number of elements of type C, and the rigidity of assembly A, while an increase in the aggregate-cement ratio may be accounted for by increasing the number of elements of type D.

Sealed Concrete

The model in this case can be imagined to be isolated from the main water tank and the vapour pressure in the pores (small tank) is constant.

On application of the load, all the elements share the load in proportion to their respective stiffness. The disturbance in the water vapour pressure over the load-bearing water is restored with time by the migration of some of the pores water (small tank) to the viscous flow element A until the equilibrium is reached. Being isolated from the main tank (the surrounding atmosphere) the water vapour pressure inside the pores (small tank) would have to change to a lower degree.

The movement of water from the pores to the load-bearing region represented in the model by the viscous flow and Hook elements in parallel in the assembly A causes this assembly to expand and release some of the load to the other elements which share it according to their stiffness. This may cause cracking or bond failure at some points which are accounted for in elements B and D. These cracks may heal again in which case they obstruct the creep recovery.

Failure may also take place in element A if the stress due to both the external load and the shrinkage exceed the load capacity of some of the load-bearing water. This may particularly occur when the concrete is drying out while under load.

Drying Creep

This can be represented in the model by reducing the vapour pressure at the small tank. This can be brought about by lowering the position of the main tank. In this case the film of adsorbed load-bearing water may also be in tension balanced by the rigid structure which can be represented by a tension in the upper spring of assembly A and compression in the other elements. When a tensile load is applied the tensile stress in the upper spring of assembly A is increased while the compression in the other elements is either reduced or reversed to tension, depending on the shrinkage stresses. The possibility of cracking in element A is maximum in this case and this may occur depending on the changing shrinkage stresses (which could result from changing the level of water in the main tank) and the applied load. On the other hand the action of tensile stress while possibly inducing cracks in the elements already under tension could prevent some from occuring in those under compression. As the creep is considered to be the strain recorded over a loaded specimen minus that of the unloaded control specimens, both forms of the mentioned microcracks would contribute to a larger creep strain.

The core of the specimen, however, is generally under compression due to the differential shrinkage, and so, for concrete in the core, all the elements in the model are considered compressed in this case including those of assembly A. The applied tensile load will then oppose the stress acting on the elements and may either reduce it or reverse it depending on the state of stress prior to loading. The tensile creep could then be imagined as reducing the compressive creep that might have occured had the load not been applied (that occuring in the core of the unloaded specimens).

Immersed Concrete

For this condition the main tank is imagined to be at a certain level higher than that of the viscous element. Swelling pressure would set the upper spring of assembly A in compression and the rest in tension. The tensile load while reducing the compressive stresses in assembly A increases the tensile stresses in the rest of the model. As creep occurs more tensile stresses are released to elements B, C and D and the possibility of bond failure and micro-cracking is a maximum. However, healing is also very probable due to the saturated condition of the system.

CHAPTER VII

THE SHORT & LONG-TERM TENSILE STRENGTH OF CONCRETE

VII.1. INTRODUCTION

The strength of concrete has been the subject of study of many investigators, but most studies have been devoted to the compressive strength and the direct tensile stength has been given considerably less priority - as was discussed in Chapter I.

Although the tensile and compressive strength of concrete are affected by almost the same factors, it is realised that the degree may differ. The differences are mostly concentrated in the bond characteristics between aggregate and cement paste, volume changes and moisture condition. These effects would also differ according to the shape of specimens and type of testing.

The present work does not deal with the differences between compressive and tensile strength or the effect of the numerous factors involved. It onlydeals with the effect of some of these factors on direct tensile strength, which have not been fully appreciated, such as the different effects of sealing and immersing on the short and long-term tensile strength of concrete. The present Chapter also contains short-term tensile strength results on concrete of different water-cement ratios, aggregate-cement ratios and type of aggregate.

VII.2. Short-term Tensile Strength Experiments

This work included the effect of sealing; immersing; sustained loading; water-cement ratio; aggregate-cement ratio; type of aggregate and age on the direct tensile strength of concrete.

The mixes and batches employed here are those used in the investigation of tensile creep plus additional batches.

VII.2.a. The Effect of Moisture Condition

The aim of this investigation is to study the effect of volume changes and moisture conditions on the short-term strength of concrete. The effect of wet and sealed curing on the strength of concrete was first investigated in Experiment 13. The effect of immersing and air drying on concrete specimens cured sealed, and the effect of immersing on concrete specimens cured sealed and then left to air dry for a period of time were then considered.

The reasons why the investigation had to include first the strength results of sealed and immersed concrete separately and then sealing and immersing consequently were;

1. Immersing a specimen after being sealed for a period of time involves differential swelling which consequently induces tensile stresses in its core. The effect of differential swelling continues until the water has penetrated into the whole section.

2. There was some uncertainty about whether the sealed and the immersed specimens would reach the same degree of hydration at the time of testing

although a water-cement ratio of 0.50 was employed. This ratio is believed to provide enough water inside the sealed concrete to keep the relative humidity above 80%, which is necessary for the continuation of hydration process.

The following mix details were used for experiments 13 to 16:

MIX DETAILS

Water-cement ratio : 0.50

Aggregate-cement ratio : 4.5

Grading curve No. 2.

Experiment 13

For this experiment one batch (C_5) of 20 specimens (all ungauged) and three cubes were cast. Half were unsealed at the age of 24 hours and immersed in water, while the other half left sealed in their jackets. All specimens were checked non-destructively and found to give similar readings. Strength tests were then carried out at the ages of 4, 7, 14, 21 and 31 days using 2 specimens for each curing condition. The tests were carried out using the loading rig described in Chapter III. The rate of loading was kept constant (about 100 lb/in^2 per minute) during testing. The cubes were used to check the crushing strength at the age of 28 days. The results of these tests are shown in Figure VII.1.

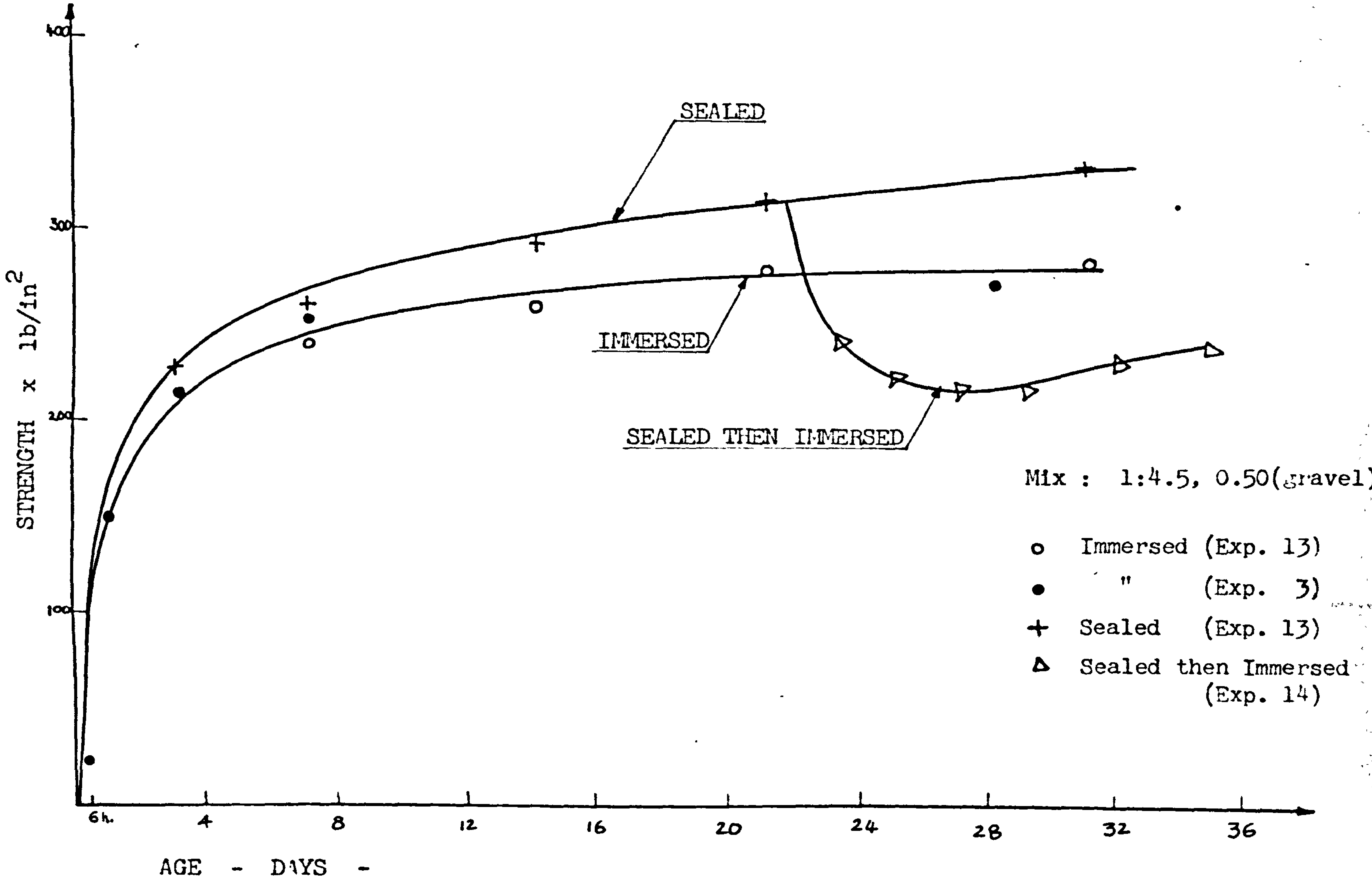


Fig. VII.1. Strength Development in Sealed & Immersed Concrete

Experiment 14

This experiment was made on two batches (C_6 & C_7) of 16 ungauged specimens and three cubes each. The two batches were cast at an interval of 2 days in order to ensure a similar curing temperature. The mould supports were dismantled at 24 hours and the specimens left to cure sealed in their jackets for 22 days. The exposed ends of the specimens were covered with wax to ensure proper sealing. At the age of 22 days and after checking all the specimens non-destructively, 24 specimens (12 from each batch) were unsealed, half were immersed in water while the other half left to air dry in the room atmosphere of about 65% relative humidity and 21°C temperature.

Two sealed, two immersed and two air drying specimens were then tested to failure following the same procedure as in Experiment 1, at the ages of 23, 25, 27 and 32 days. Another two immersed and two air drying specimens were tested at the ages of 29 and 34 days. The results of these tests are shown in Figures VII.2. & VII.3.

Experiment 15

For this experiment one batch (C_8) of 16 ungauged specimens was cast and left curing sealed for 22 days. They were then checked non-destructively, unsealed and left to air dry until the age of 35 days (thus a continuation of experiment 14). Ten specimens were then immersed in water and tested in groups of two at the ages of 35 days and

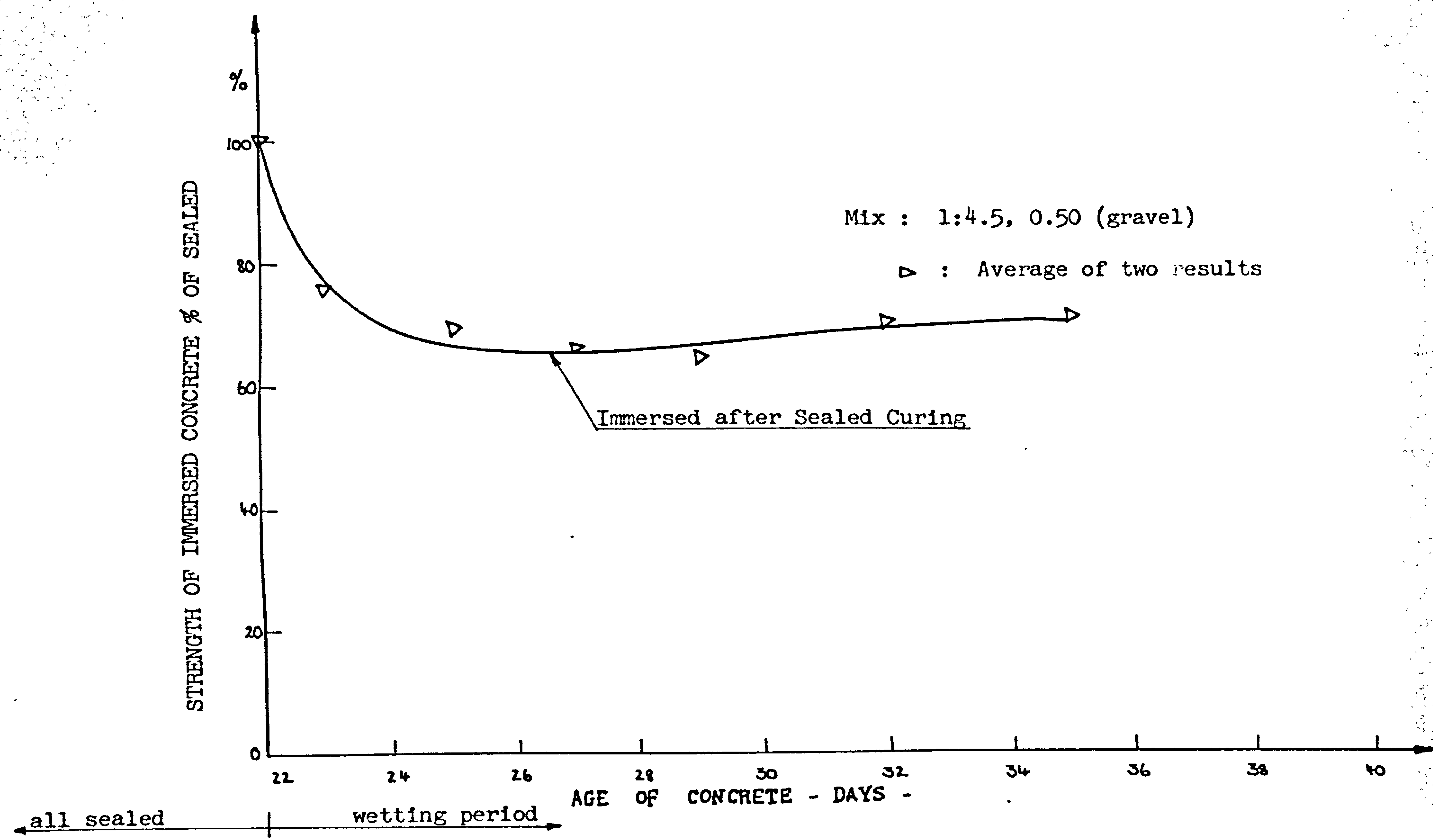


Fig. VII.2. The Effect of subsequent Immersion on the Strength of Concrete Sealed for first 22 Days.

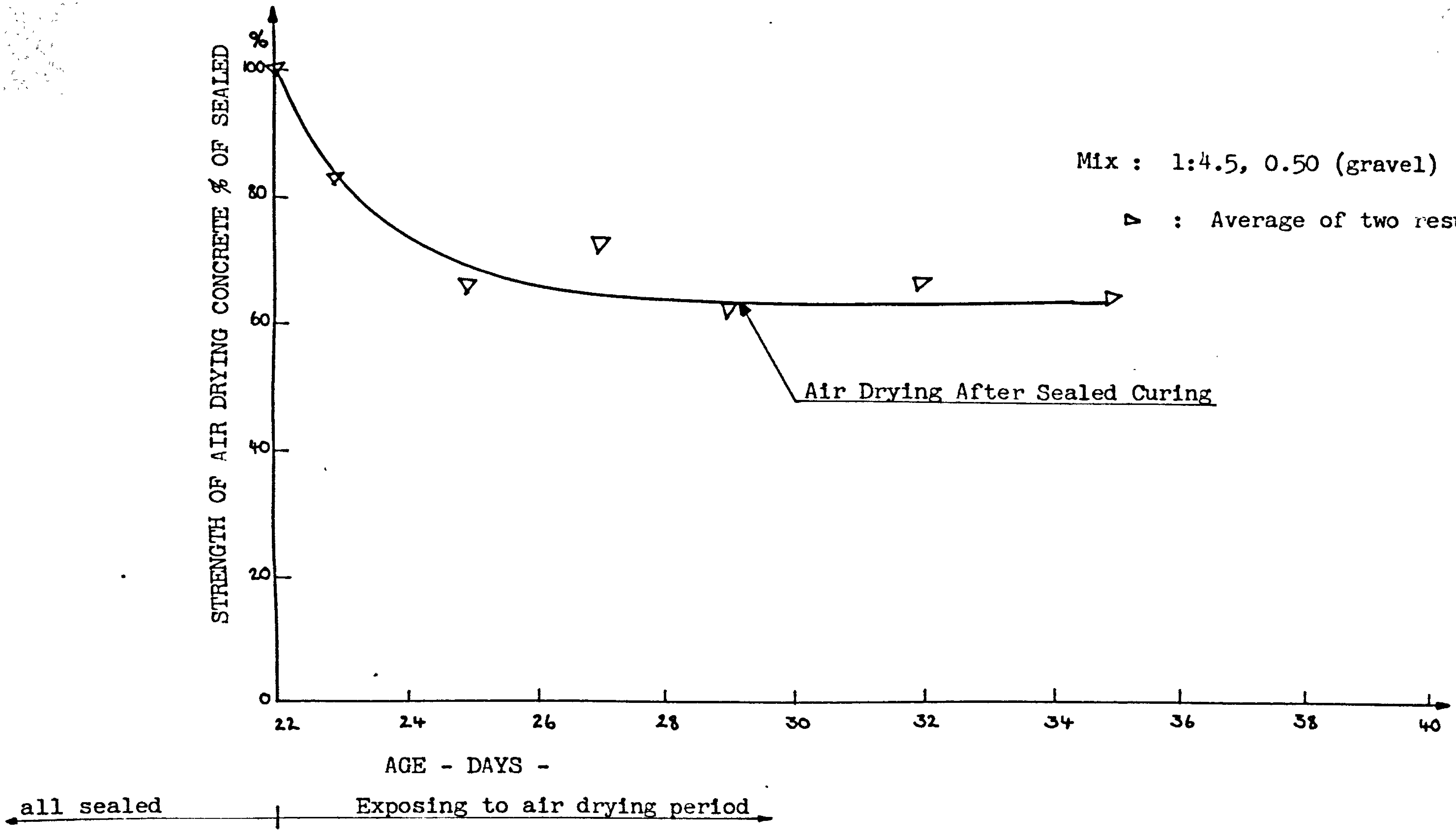


Fig. VII.3. The Effect of air drying on the Strength of Concrete Cured Sealed.

6 hours, 35 days and 12 hours, 36, 38 and 40 days. The air drying specimens were tested at the ages of 36 and 40 days. The results of this experiment are shown in Figure VII.4.

To check that the comparison with experiment 13 was valid, the 2 sealed specimens were first tested at 25 days.

VII.2.b. The Effect of Sustained Low-level Loading on the Strength

This investigation was designed to investigate the effect of low level sustained stress on the ultimate tensile strength of concrete. However, in order to find the possible combined effect of immersion plus sustained loading the same mix and procedure of experiment 14 was maintained. The effect of high-level sustained stresses is considered later.

Experiment 16

This included one batch (C_9) of 16 ungauged specimens and three cubes. The specimens were cast and left curing sealed for 22 days. All the specimens were checked non-destructively and found to vary little from those of experiment 14. They were then unsealed and immersed in water. Half (i.e., 8 specimens) were then loaded to 35% of the ultimate strength.

The specimens were unloaded and tested to failure thereafter in groups of two at the ages of 23, 25, 30 and 35 days.

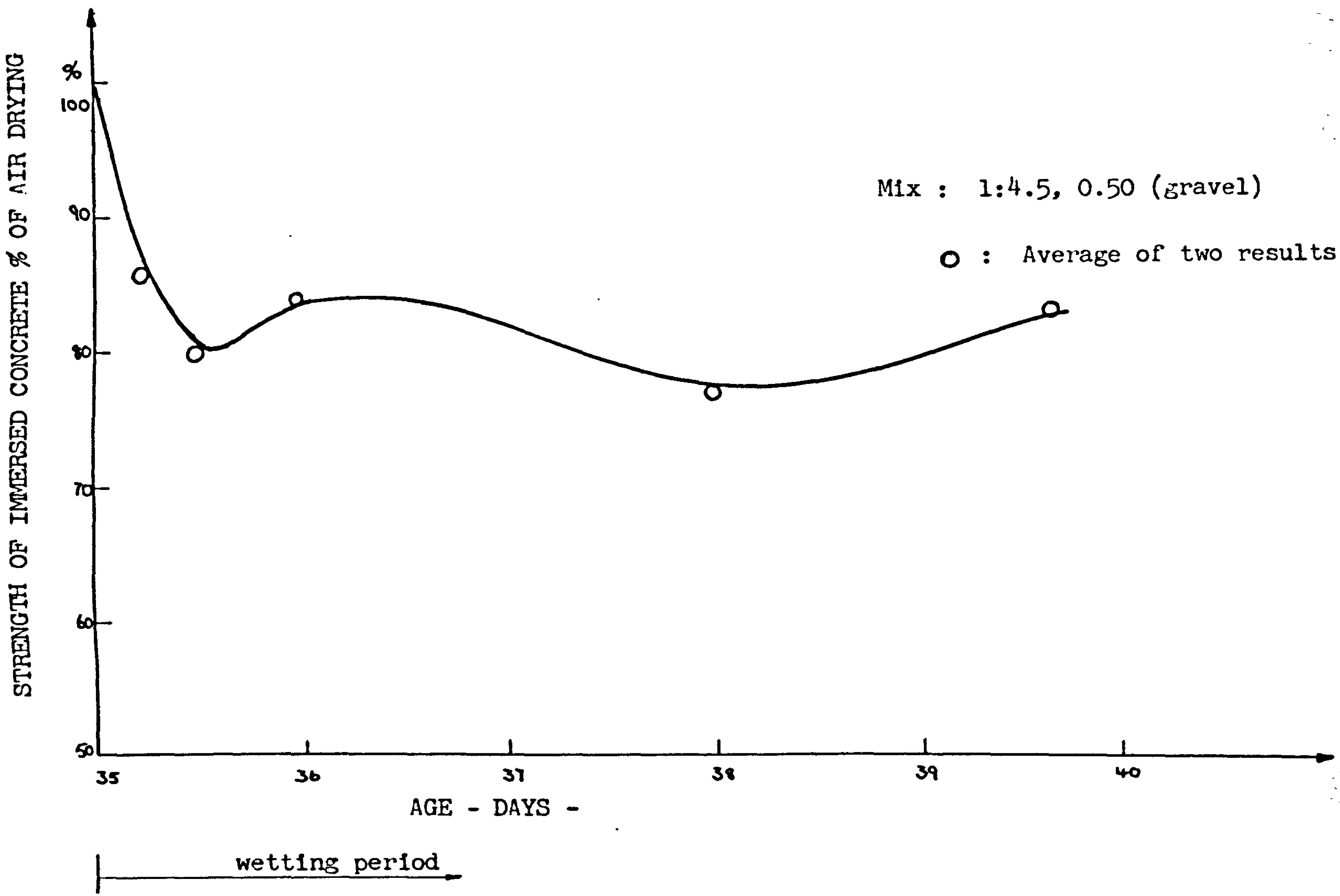


Fig. VII.4. The effect of Subsequent Immersion on the Strength of Concrete Sealed for 22 days and Air Dried for 13 Days.

VII.2.c. The Effect of Water-Cement Ratio, Aggregate-Cement Ratio and type of Aggregate

These included short-term tensile strength testing of the ungauged concrete specimens made when investigating the tensile creep where in every experiment, as can be seen in Chapter V, a number of ungauged specimens were cast together with the gauged ones. Some of the control gauged specimens were also used for testing the tensile strength. The rate of loading and procedure was the same as for Experiments 1, 2, 3 & 4.

The results include water-cement ratios of 0.42, 0.50, 0.60 and 0.70, aggregate-cement ratios of 4.5, 6 & 7 and two types of aggregate gravel and crushed granite (see IV.2 for Mix Details).

Another batch (C_{16}) of 20 specimens, some of which were gauged, made with granite aggregate and water-cement ratio of 0.50 was made and used for this investigation and that of the long-term strength experiment. Two specimens were cured sealed and the rest stored under water.

The specimens were generally tested at the ages of 3, 7 & 28 days. In some cases results of 1 day and 4 months were also obtained. In one case (Mix: 1:4.5, 0.50 gravel) two specimens were tested at the age of 6 hours. This was conducted in order to check the possibility of using this test method for early-age-concrete and no attempt was made to study the early tensile strength, (i.e. within the first 24 hours).

The results are shown in Figures VII.5 - VII.10. Every point on the curves represents an average of two specimens unless otherwise stated.

Additional Results

Most of the specimens originally employed for the creep experiments were finally tested to failure to find the effect which their sustained load had had on the ultimate strength of the concrete compared with that of the unloaded companion specimens. The results of these tests are not reported here but will be commented on in the discussion at the end of this Chapter.

VII.3. Long-Term Strength Investigation

The aim of this investigation was to find the true ultimate tensile strength of concrete under long-term loading, the maximum safe load (i.e. the maximum load that can be supported by the specimen for an indefinite time, and the period of time for which a specimen can support a certain load before failure occurs). The value of the ultimate strain that can be sustained by the concrete for these different cases was also sought.

It has been found that in compression a very slow rate of loading can decrease the ultimate short-term strength by 20-25%. It has also been found that in compression the ultimate strain increases with a decrease of the stress-strength ratio.

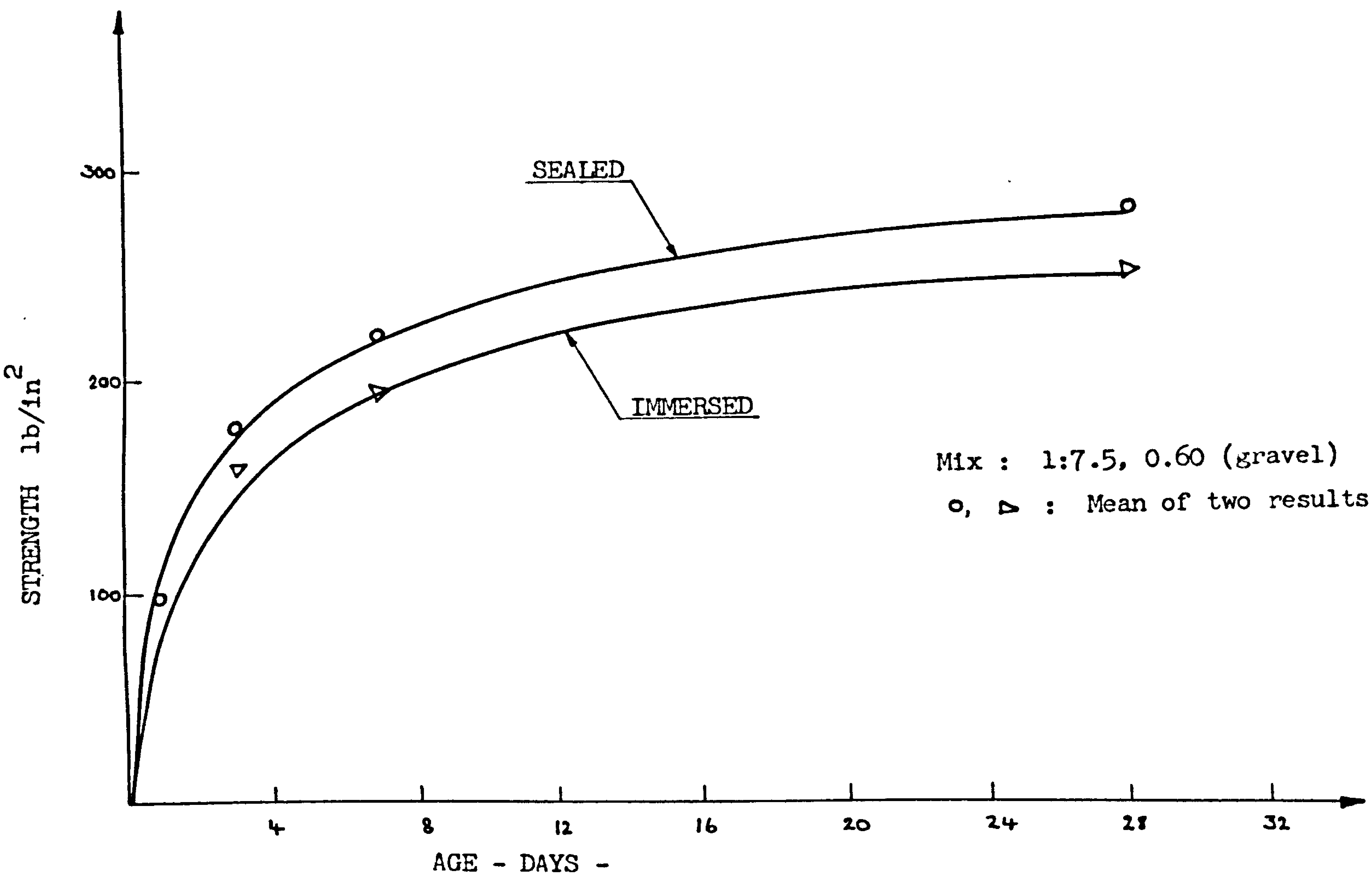


Fig. VII.5. Strength Development in Sealed and Immersed Concrete.

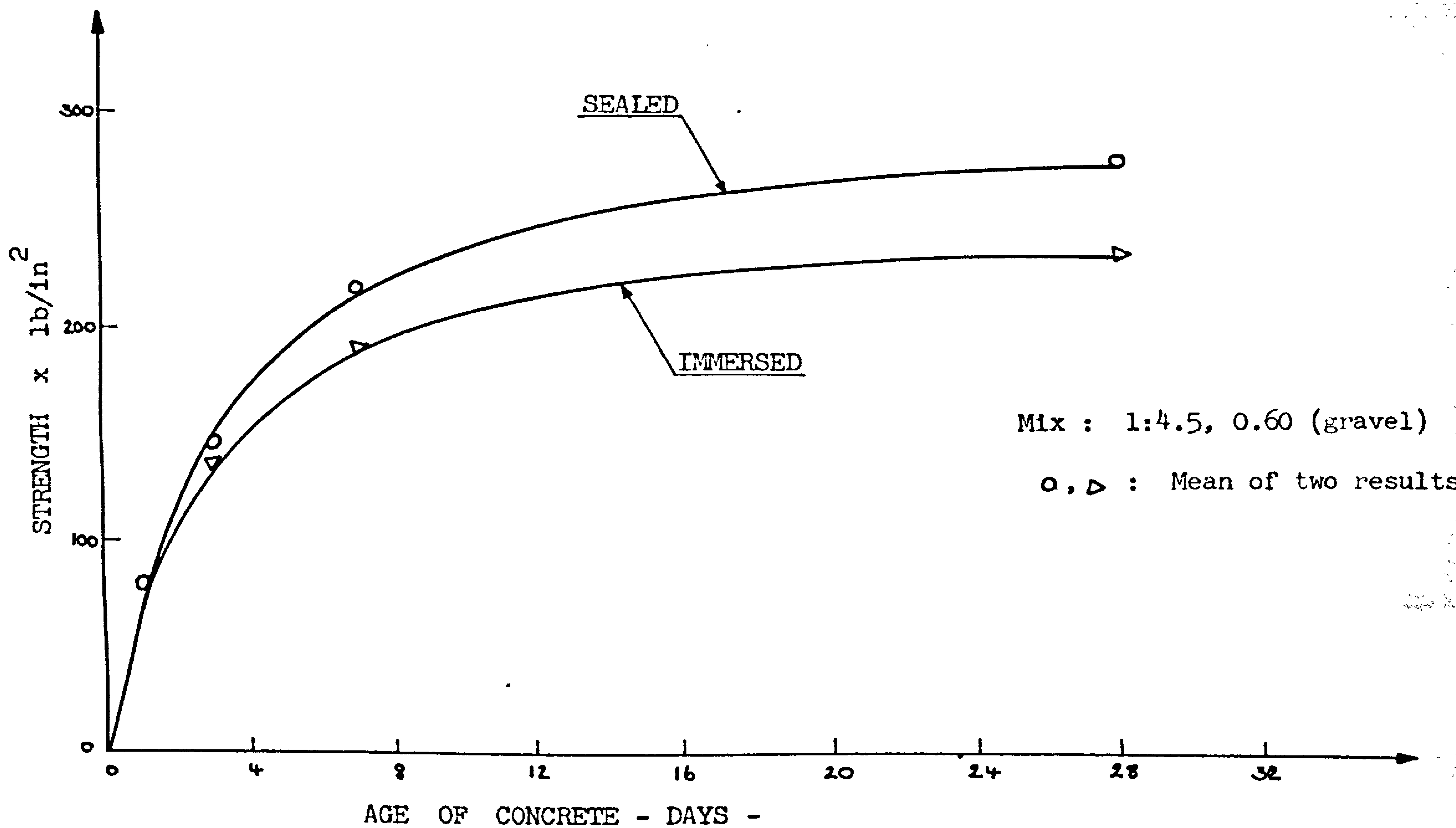


Fig. VII.6. Strength Development in Immersed and Sealed Concrete.

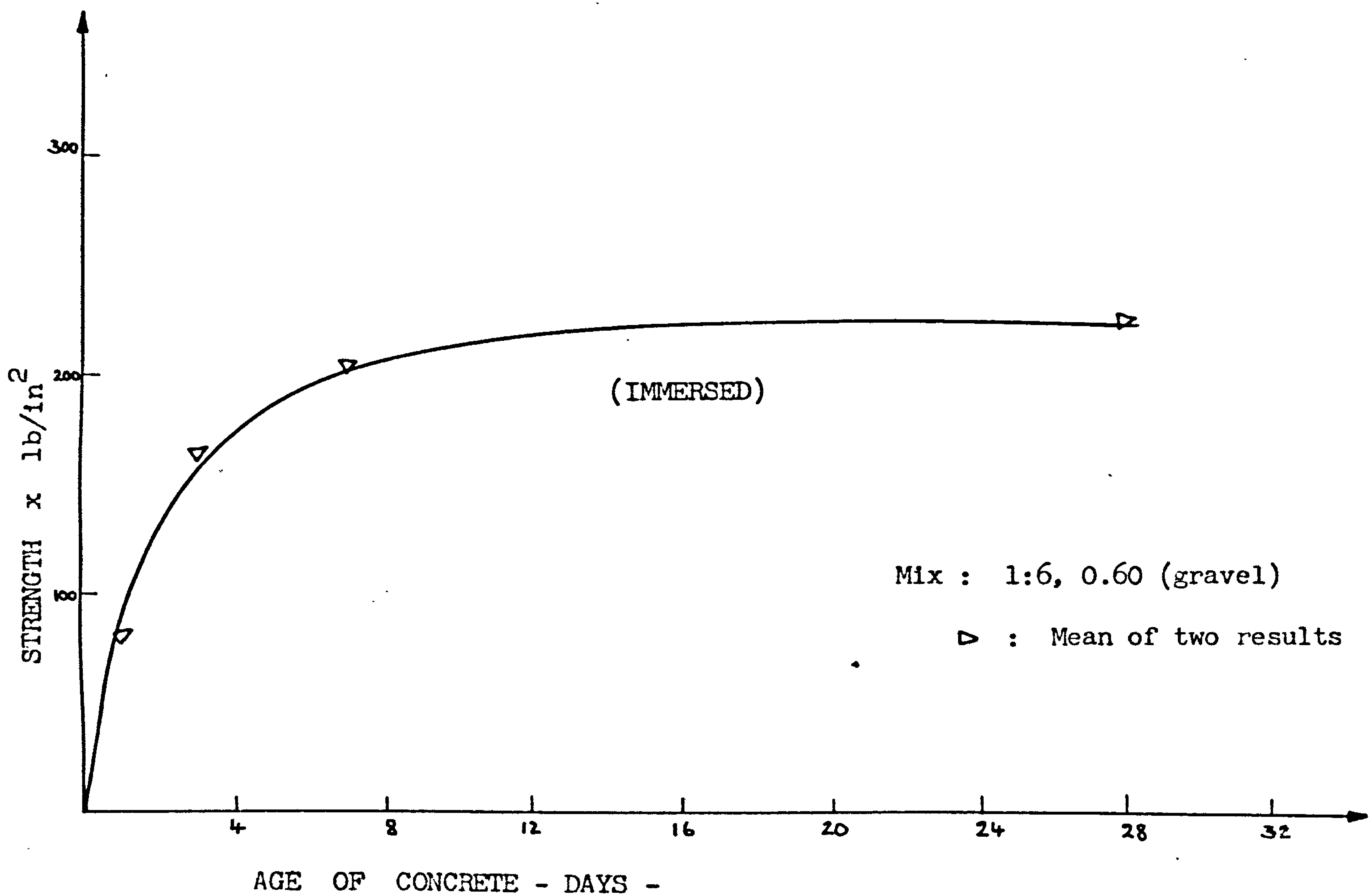


Fig. VII.7. Strength Development in Immersed Concrete.

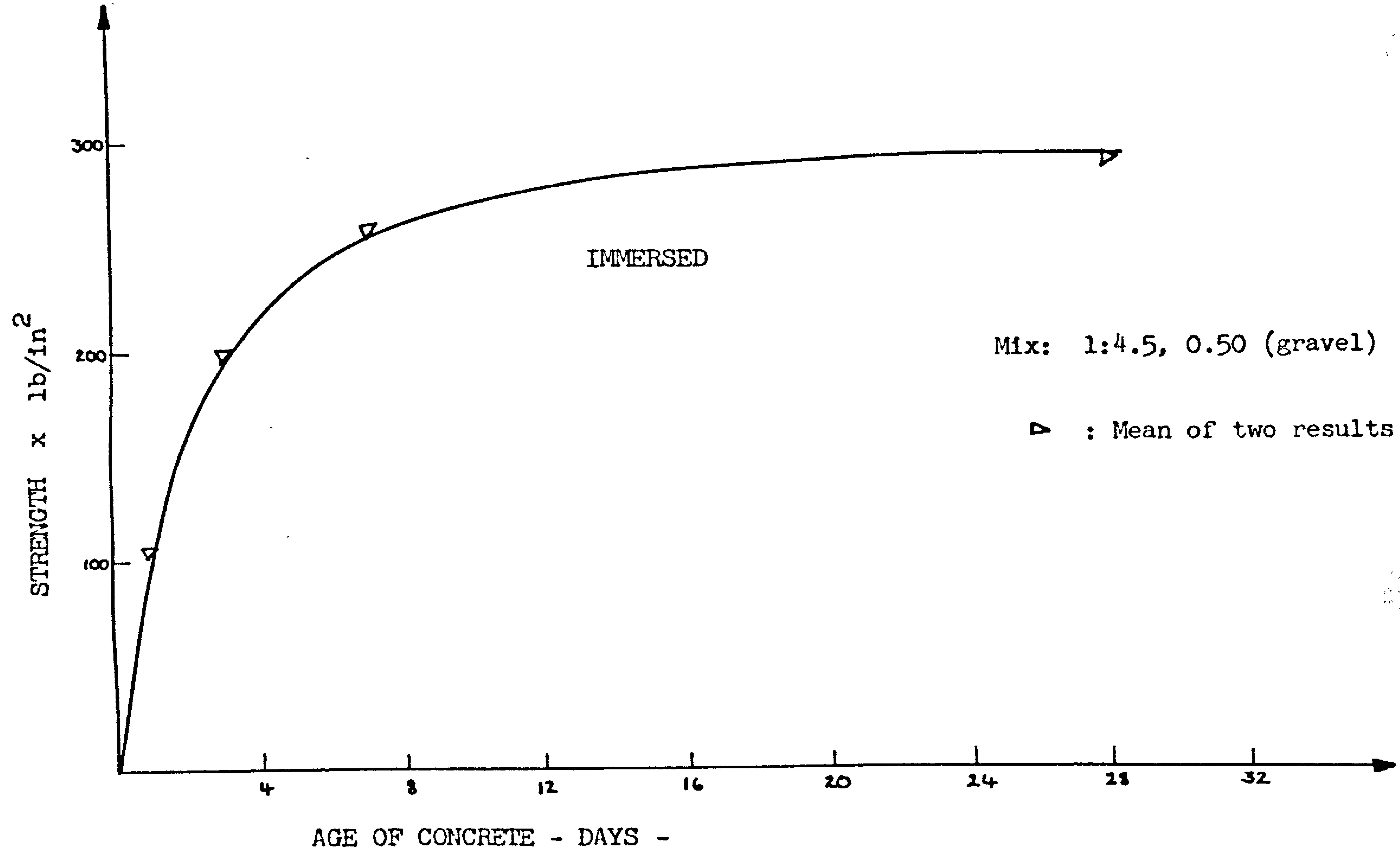


Fig: VII.8. Strength Development in Immersed Concrete.

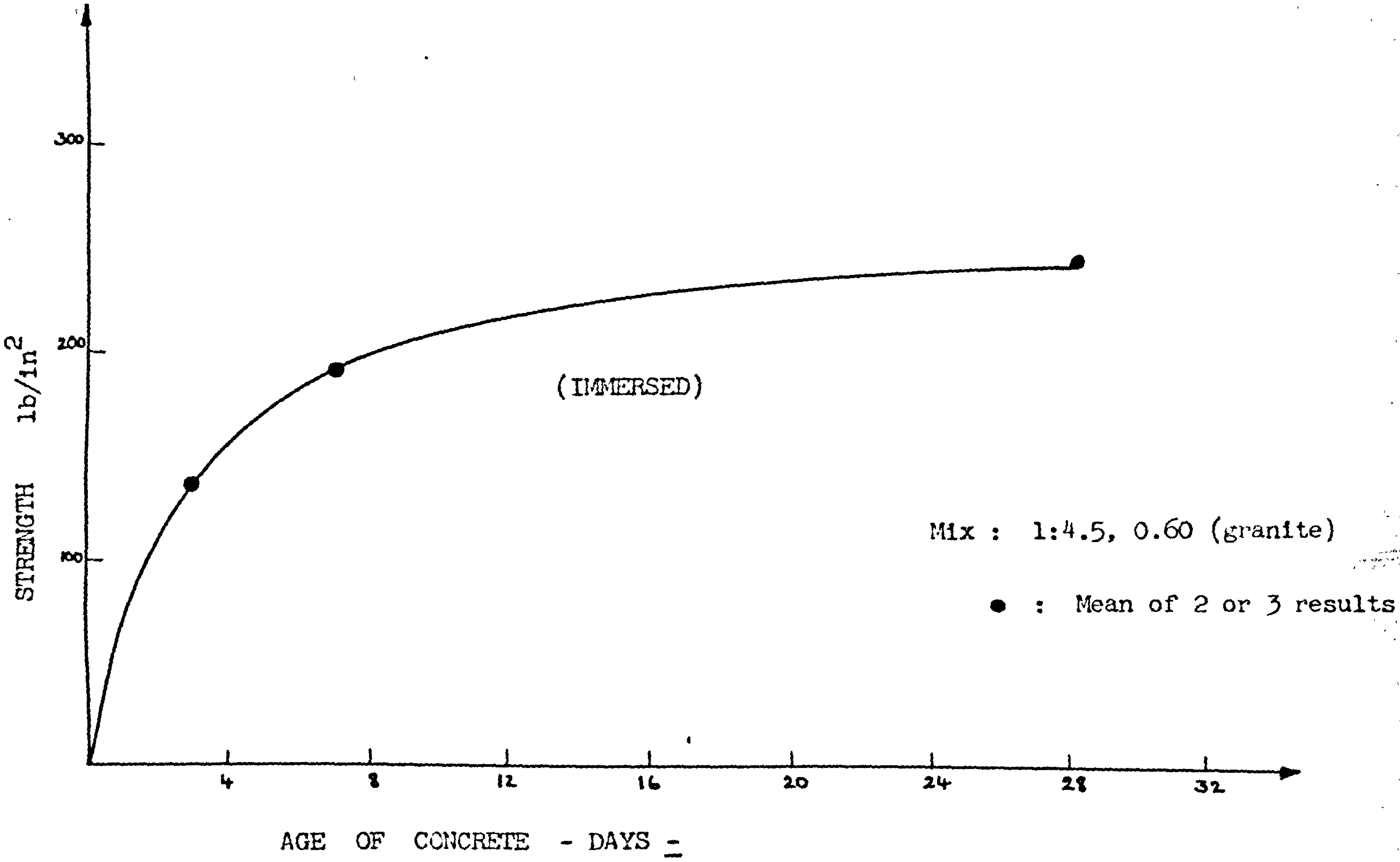


Fig. VII.9. Strength Development in Immersed Concrete.

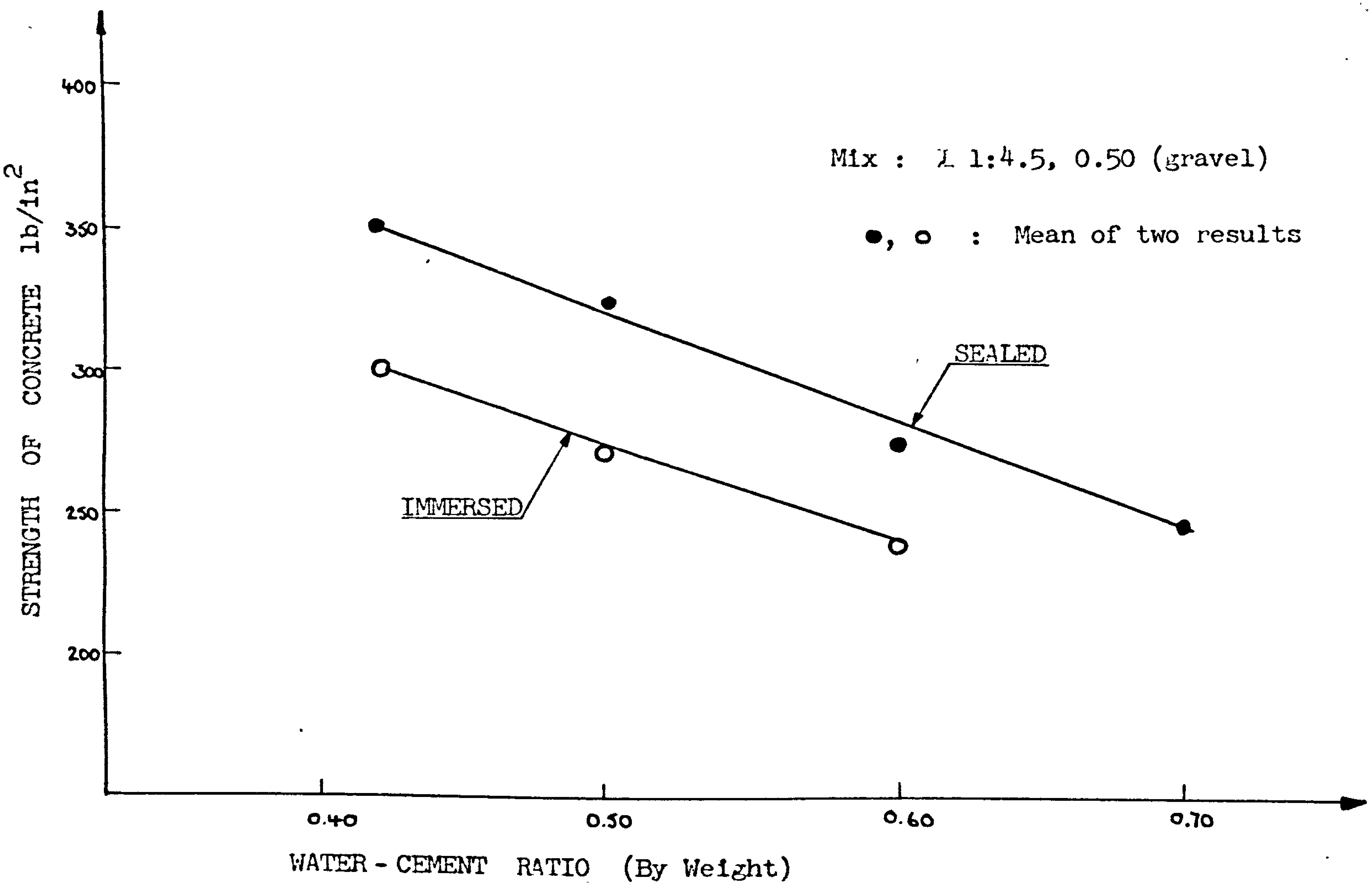


Fig. VII.10. The 28 Days Strength of Sealed and Immersed Concrete Made with Various Water-Cement Ratios

As previously mentioned when discussing the creep mechanism (see VI.2.) the ultimate strain of the immersed concrete of a certain mix is not expected to change much with the time under load or applied stress. The failure in this case is expected to start at the weak interface between the aggregate and cement paste. The bond stress in the direction perpendicular to the applied load should always be proportional to the strain in the aggregate. If the critical bond stress is the same for the different specimens of the mix the failure would be expected to occur when this stress is reached which correspond to a certain constant value of strain in the aggregate.

Mostly immersed concrete was employed for this investigation as it undergoes larger creep strain than the sealed concrete and the tensile stresses around the aggregate are expected to be at their highest level prior to loading, due to the swelling volume changes. The specimens were also tested at mature ages in order to reduce the increase in strength with time during the loading period and to allow for the volume changes due to swelling to stabalise.

A few sealed specimens were also tested to determine the different effects of the long-term loading on the sealed and immersed concrete.

Concrete specimens made with crushed granite were also used to check the effect on the long-term strength of the better bond characteristics that they possess between aggregate and cement paste.

MIX DETAILS

Aggregate-cement ratio : 4.5

Water-cement ratio : 0.50

Grading curve No.2.

The same Mix proportions were used for both immersed and sealed concrete made with gravel aggregate (Experiment 17) and for the immersed specimens made with crushed granite coarse aggregate (Experiment 18).

Experiment 17

This included one batch (C_{20}) of 16 specimens, 6 of which were gauged. All the specimens were unsealed at the age of 24 hours and immersed in water. Tests started at the age of 3 months. Three ungauged specimens, were first tested to failure to find the ultimate short-term strength. The rate of loading in this case was increased to 300 lb/in^2 per minute. The strength results were as follows:

Strength (lb/in^2)		
1	-	365
2	-	360
3	-	352

Ten specimens, five of which were gauged, were then loaded to 90%, 80%, 70%, 60% & 50% of the ultimate short-term strength in groups of two, one gauged and one ungauged, for each level of stress. The results of this experiment are shown in Figures VII.11 & VII.12.

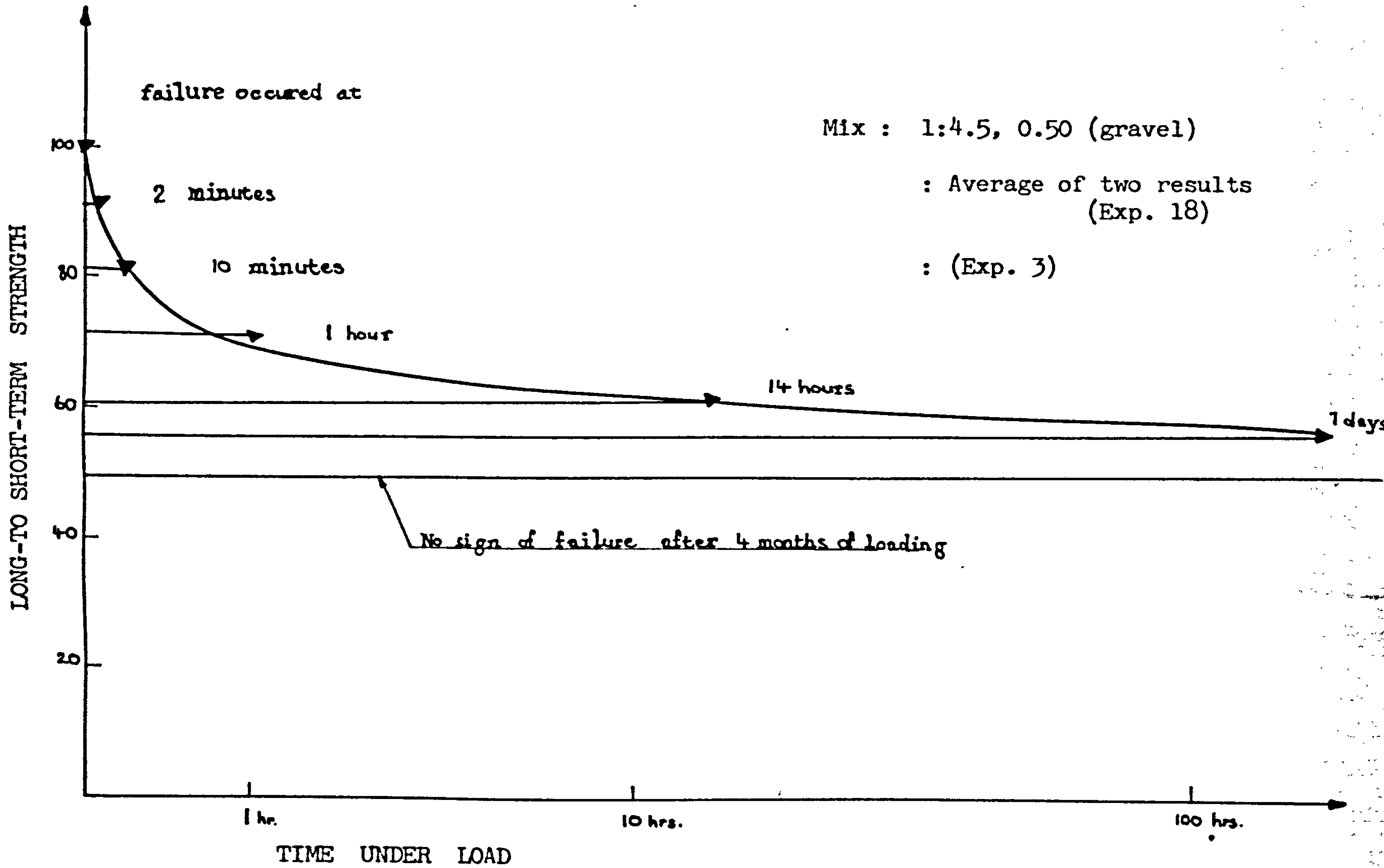


Fig. VII.11. The long-term Strength of Immersed Concrete Subjected to Different Sustained Loading.

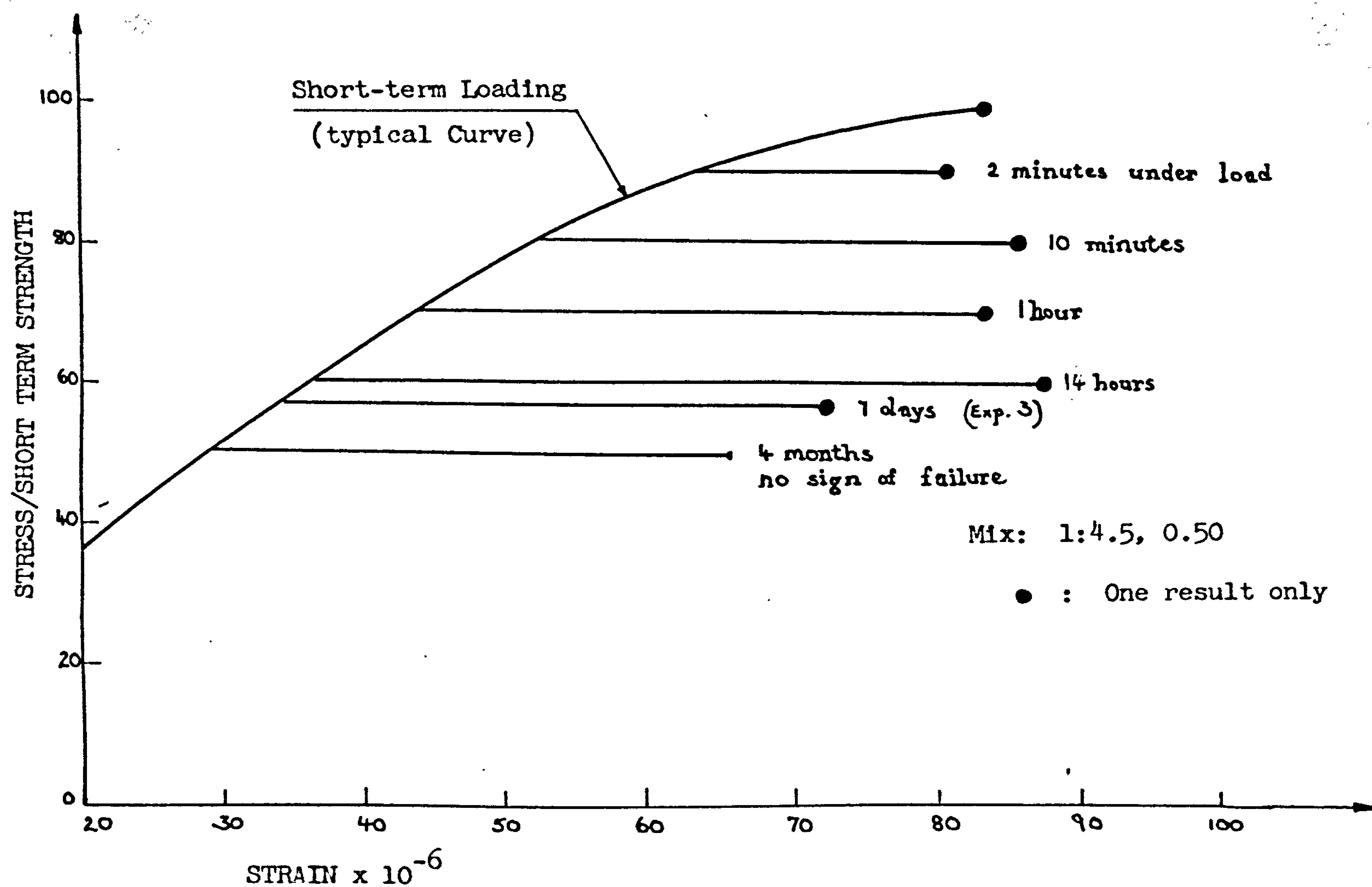


Fig. VII.12. The Ultimate Strain at Failure for Concrete loaded to Different Levels of Stress for Various Periods of Time.

Some unloaded concrete specimens of the same mix left from the creep experiments were also used for this test.

The sealed specimens used for the long-term tensile strength were some of the control specimens originally cast for the creep experiments. There were not, however, any systematic long-term strength tests on sealed specimens. The results of these tests will be discussed and compared with the other two experiments in the following pages.

Experiment 18

Ten specimens of batch (C_{16}), all ungauged, were employed for this experiment. The specimens were unsealed at 24 hours and immersed in water. Tests started at the age of 3 months. Three specimens were first tested to failure at a rate of loading of 300 lb/in^2 per minute. The results were as follows:

		Strength (lb/in^2)
1	-	390
2	-	375
3	-	370

Six specimens were then subjected to sustained loads of 90%, 80%, & 70% of the ultimate short-term strength in groups of two specimens for each level of stress.

The results of these tests are discussed later.

VII.4. Analysis of Results:

VII.4.a. Short-term Tensile Strength

Figure VII.1, shows the relationship between the direct tensile strength and age for concrete cured under sealed and immersed conditions.

It can be seen that the strength of sealed concrete increases with age at a rate faster than that of the immersed concrete and it reaches a value about 20% higher than the latter at the age of 31 days.

Storing under water is generally considered to be better for the continuation of hydration. Sealing a concrete made with a high enough water-cement ratio may, at best, keep the hydration process the same as that of the immersed concrete. Therefore, the difference observed could not be related to hydration particularly in view of the fact that both sealed and immersed concrete were stored in the same room and so had the same curing temperature.

The reason for the difference between the sealed and the immersed concrete is thought to be due to the different volume changes that take place under the two conditions and the internal stresses set up at the interfaces between the aggregate and cement paste and inside the cement gel.

The mechanism by which these stresses occur could be explained using the model presented in Chapter VI as follows:

In immersed concrete the surrounding water (of the main tank) keeps the swelling pressure at a maximum. The upper spring of assembly A is under compression balanced by tension in the other elements. When the tensile load is applied the tension in the elements B, C and D is increased and bond failure around the aggregate (represented in the model by element D) where the stresses are already mostly tensile due to the swelling pressure, is expected to initiate failure.

In the case of the sealed condition the model is assumed detached from the main tank, the self-desiccation within the concrete reduces the vapour pressure of the pores (represented by the small tank in the model) which would attract water from the viscous element of assembly A. This may either reduce the compression in the upper spring of assembly A (assumed to be in compression because of the existance of swelling pressure caused by the water of the mix) or reverse it to tension. In both cases the tensile stresses in the elements B, C and D will either be reduced or reversed to compression. When the tensile load is applied, failure may still initiate at the interface between the aggregate and cement paste but as the stresses around the aggregate are not as critical as in the case of immersed concrete, higher applied loads should be needed to cause failure.

On the other hand, failure may be initiated in some of the assemblies A if the stress caused by the self-desiccation and the applied load reaches a critical value over the region of the load-bearing water before bond failure occurs. This may depend on the water-cement ratio of the mix and the age at the time of testing.

Figures VII.2. shows that if the sealed concrete is unsealed and immersed in water, the strength is reduced by as much as 33%, but the difference reduces to about 27% after 10 days of immersion.

The greater difference between the strength of concrete after the early days of immersion may be due to the differential swelling stresses diminishing with time as the water penetrates into the whole section. However, this difference, after a relatively long period of immersion, was greater than that observed on concrete specimens cured under sealed and immersed conditions independently. The reason here may be related to either the occurrence of some cracks by the sudden volume changes on immersion or that the sealed concrete did not reach a degree of hydration equal to that cured under water.

Figure VII.3. shows the effect of air drying on sealed concrete. It shows that the strength decreased considerably after a few days of drying. The cause in this case may be due mostly to the differential shrinkage stresses. But another mechanism may also be at work:

Using the model of Chapter VI, the main tank is assumed to be at a lower level than that of the viscous element and the vapour pressure of the pores (small tank) is decreased. The water is attracted from the narrow spaces between the gel particles (viscous element) and the upper spring of assembly A is put in tension while the other elements are in compression. As shrinkage continues the tensile stresses are increased in the upper springs of assemblies A and failure may occur in some of them. When the tensile load is then applied, more tensile stresses are exhibited by the assemblies A and so failure is likely to initiate there.

Also, the shrinkage causes compressive and tensile stresses to be set up around the aggregate which may lead to some bond cracks.

On the other hand, shrinkage may increase the bond strength over the compressed regions.

Whenever failure is initiated whether at the aggregate-cement paste interface or inside the cement gel the crack will propagate and cause complete failure. Higher strength is thus thought to be achieved in concrete where the stresses around the aggregate and inside the cement gel reach their critical value at the same time. This is thought to be most likely in the sealed condition.

Figure VII.4. shows the effect of immersion on concrete that had been cured sealed and then left to air dry for 13 days before immersing. The result also indicates further decrease in strength which may be due to a combination of differential swelling and further occurrence of microcracks due to the volume changes.

Loading the specimens to 35% of their ultimate strength did not show any detectable effect on strength. However, the results of testing the specimens used for the creep experiments showed that the sustained loading could reduce the ultimate short-term strength. This reduction seemed to be larger in specimens that had undergone larger creep but no definite trend was obtained from testing the specimens of the creep experiments. Low sustained stress was not observed to effect the strength of the sealed specimens. These, however, underwent much smaller creep.

Figures VII.5-VII.10 show the relationship between strength and age of different concrete mixes under immersed and sealed conditions. The result confirmed those already obtained that sealed curing produces stronger concrete even for concrete made with water-cement ratio as low as 0.42.

This ratio is, however, considered insufficient to keep the hydration progressing in sealed concrete.

The two sealed specimens made with the granite aggregate (batch C_{16}) were tested to failure at the age of 28 days together with two immersed concrete specimens. The results were as follows:

	Sealed	Immersed
1.	270	286
2.	277	300

This indicates that the sealing in this case did not produce higher strength. However, the crushed granite aggregate that was used was noticed to have a lower modulus of elasticity than the gravel aggregate.

Larger strains can thus be exhibited by the crushed granite aggregate than the gravel aggregate for an equal applied stress. Adding to that the better bond characteristics expected from the crushed granite aggregate the failure may not initiate around the aggregate but inside the cement gel. Hence if the water-cured concrete reached a higher degree of hydration than that cured sealed, the expected result would be as obtained. However, the number of tested sealed concrete specimens made with granite aggregate does not justify any conclusion to be made.

VII.4.b. Long-term Tensile Strength

Figure VII.11 shows the relationship between the long to short-term tensile strength and time under load for immersed concrete loaded to various levels of sustained stress. It can be seen that the long-term strength could be as low as 50-60% of the ultimate short-term strength if the specimen is left under that load for a long period of time.

Figure VII.12 shows the relationship between the long-term strength and the ultimate strain at failure for concrete loaded for various periods of time. The results indicate that the ultimate strain at failure does not change much with the time under load or the stress applied. On the contrary, it seems to suggest that the concrete fails at nearly a constant value regardless of the initially applied stress.

The few tests conducted on sealed concrete did not indicate the same results and sealed specimens loaded to as much as 80% of their short-term strength did not show any sign of failure after 30 days of loading. It was also noticed that the creep in these specimens reached a limiting value after about that period of loading.

The long-term strength results of the immersed concrete made with crushed granite did not also show such a decrease in strength with time under load as for the immersed concrete made with gravel aggregate.

As previously discussed in VI.2 these results are expected. The immersed concrete undergoes larger creep strain than the sealed resulting in higher tensile stresses around the aggregate which would cause failure when the bond stress, which is proportional to the strain in the aggregate, reaches a critical value. As demonstrated earlier, the stress condition around the sealed concrete is not the same as in the immersed concrete and the creep is less, so that the strengths would not be expected to be the same.

For concrete made with granite aggregate, which had lower modulus of elasticity and better bond characteristics than the gravel aggregate the strain is expected to reach a higher value before failure occurs. The creep may reach a limiting value before the critical strain in the aggregate is reached and thus failure does not occur. In this case, higher initial stresses would be required to bring about failure. Failure under long-term loading in concrete made with crushed granite aggregate may also initiate inside the cement gel and not around the aggregate.

Conclusion

The following conclusions may be drawn from the experiments of the short-and long-term tensile strength of concrete:

Short-term strength:

1. The strength of concrete specimens cured sealed is higher than that of specimens cured under water or air drying. This was observed even on concrete specimens made with a water-cement ratio as low as 0.42 (a water-cement ratio of 0.50 or above is considered necessary for the continuation of hydration process in sealed concrete).

2. Immersing a concrete specimen for several days after having been cured sealed for a period of three

weeks decreases its strength to a value lower than that obtained from a companion specimen cured continuously under water.

3. Exposing a concrete specimen to air drying after it had been cured sealed, or immersing it after it had been air drying, could reduce its strength.

4. Subjecting an immersed concrete specimen to a low sustained load for a certain period of time may also result in a further reduction of its short-term strength.

The higher tensile strength of sealed concrete could be related to a better distribution of internal stresses in the sealed concrete than that occuring in immersed concrete, while the stresses caused by the volume changes and differential shrinkage on immersing or exposing to air drying, could be responsible for the consequent reduction in strength.

Long-term strength:

1. The long-term tensile strength of immersed concrete subjected to sustained loading could be as low as 50 to 60% of the short-term.

2. Sealed concrete and concrete made with crushed granite aggregate may have a long-term strength of 70-80% of the short-term. This may be related to the lower creep of sealed concrete and better bond characteristics of crushed granite aggregate.

3. The ultimate tensile strain of immersed mature concrete does not change much with the load or the period of loading which suggests a limiting positive strain for a certain mix. This may be explained as follows:

Failure is initiated by a bond breakdown at the interface between the aggregate and cement paste. This occurs when the stress, which is proportional to the strain in the aggregate (assumed elastic), reaches a certain value equal to that of the bond strength between the two materials. The stress over these areas is dominated not only by the applied load but also by the internal distribution changing with creep, shrinkage and swelling.

The observation on both short-and long-term strength results are thought to be well defined by the proposed mechanism and demonstrated by the model.

CHAPTER VIII

THE TENSILE STRESS-STRAIN RELATIONSHIP AND AN INVESTIGATION INTO THE FALLING BRANCH

INTRODUCTION

The present chapter describes some observations on the stress-strain characteristics of concrete which resulted from tests on specimens originally made for the creep experiments.

The first part includes tests on the short-term stress-strain relationship of concrete specimens (cured either sealed or wet) some of which had been loaded previously. The effect of the previous loading and curing conditions on strength and strain is discussed. The second part describes the test carried out in order to investigate the complete stress-strain relationship (including the falling branch).

VIII.1. THE STRESS-STRAIN RELATIONSHIP

These experiments include tests on concrete made with water-cement ratios of 0.50 and 0.60, aggregate-cement ratios of 4.5 and 7 and two types of aggregate, gravel and crushed granite.

All the specimens were tested to failure at the age of three months. Some of these were loaded to about 40-50% of their ultimate strength at the age of 30 days for a period of 30 days and then left to recover for another month before testing. Two specimens only were loaded at the age of 3 months to about 50% of their ultimate and then tested to failure without allowing recovery to take place in order to investigate this effect on the ultimate strength and strain.

The results of these tests are plotted and shown in figures VIII.1 to VIII.9.

Figure VIII.1. shows the stress-strain curves of previously loaded, immersed concrete specimens and the corresponding stress-strain curves for the immersed unloaded concrete specimens.

Figures VIII.2. and VIII.3. show the stress-strain and creep strain curves respectively of immersed concrete specimens of Mix 1 loaded to 50% of their ultimate strength for 15 days and then loaded to failure directly.

Figures VIII.4. and VIII.5. show the stress-strain curves of immersed concrete. Figures VIII.6. & VIII.7. show the corresponding curves for sealed concrete.

Figures VIII.8. and VIII.9. show the stress-strain relationship for concrete made with crushed granite aggregate.

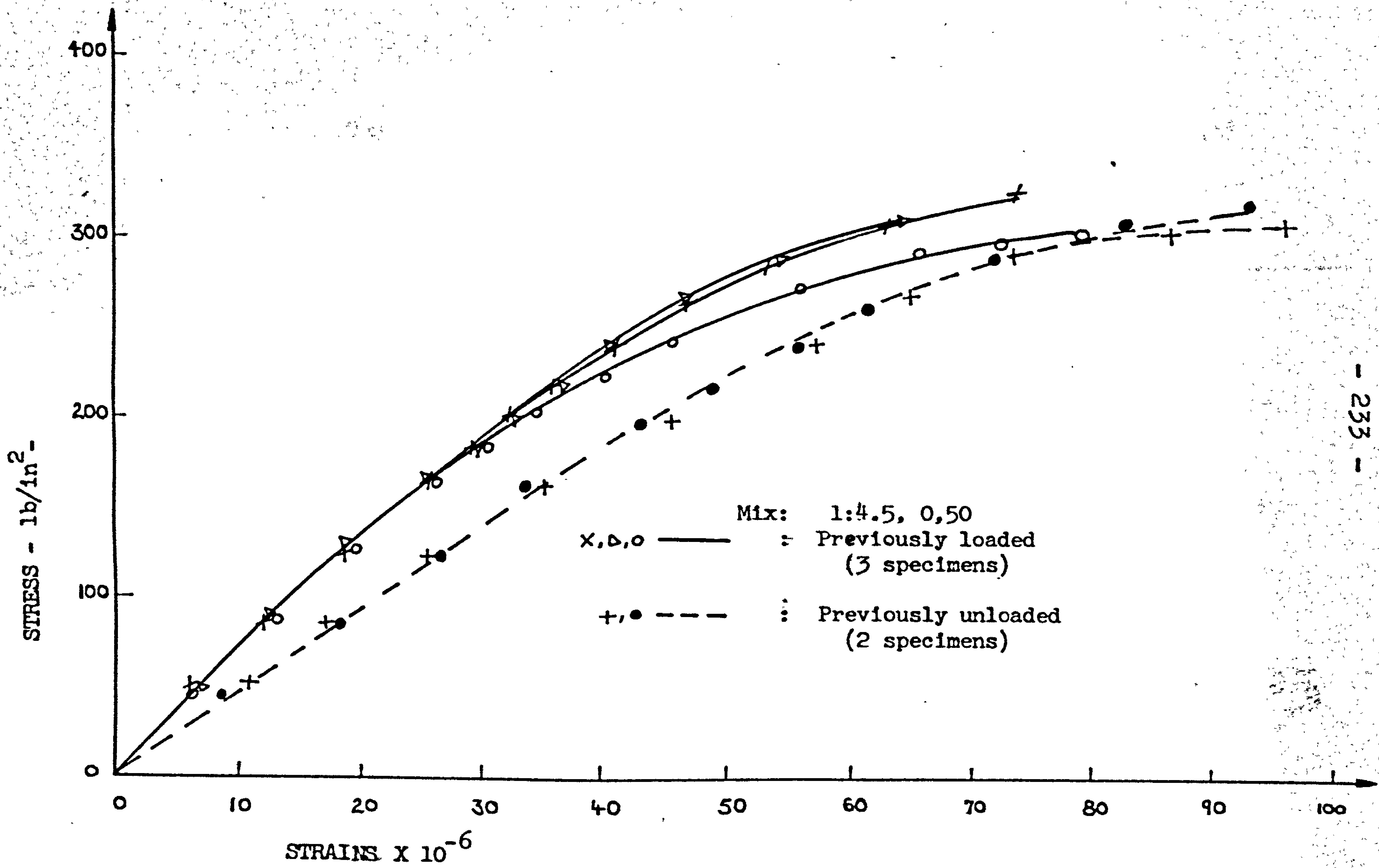


Fig. VIII.1. The effect of loading on the stress-strain relationship of immersed concrete.

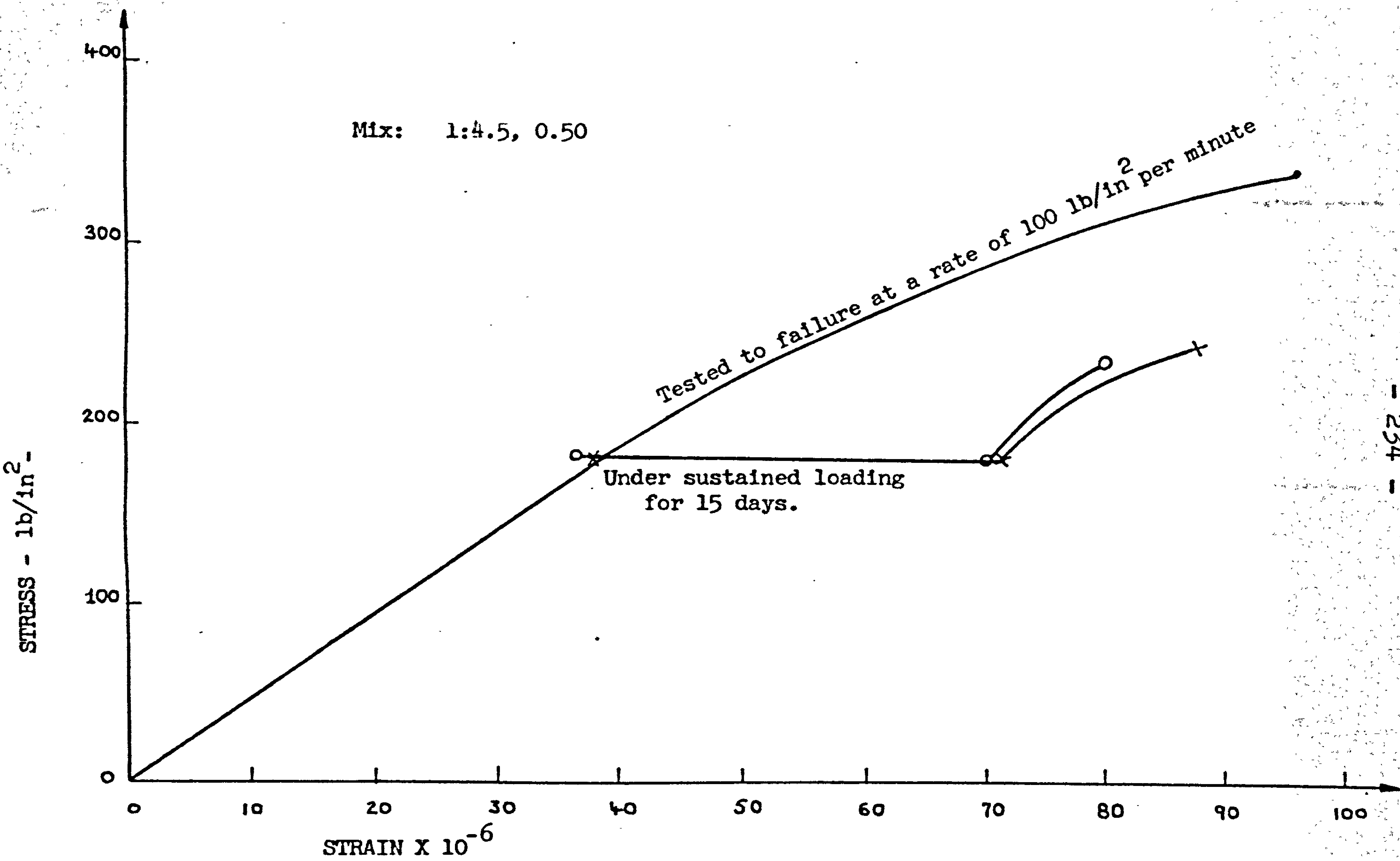


Fig. VIII.2. The Effect of Sustained Loading on the Ultimate Strain & Strength of Immersed Concrete.

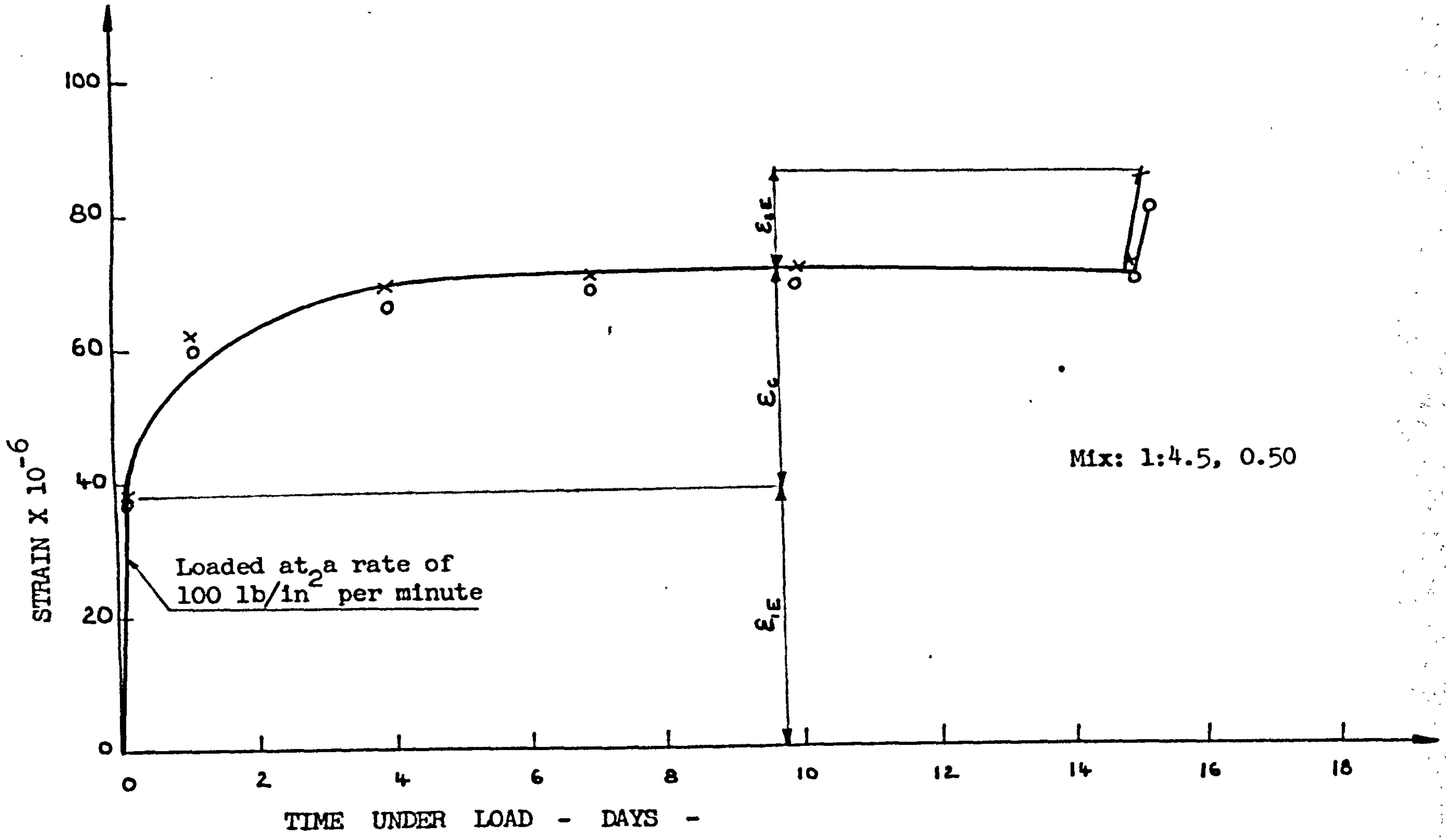


Fig. VIII.3. The Effect of Creep on the Ultimate Strength & Strain of Immersed Concrete.

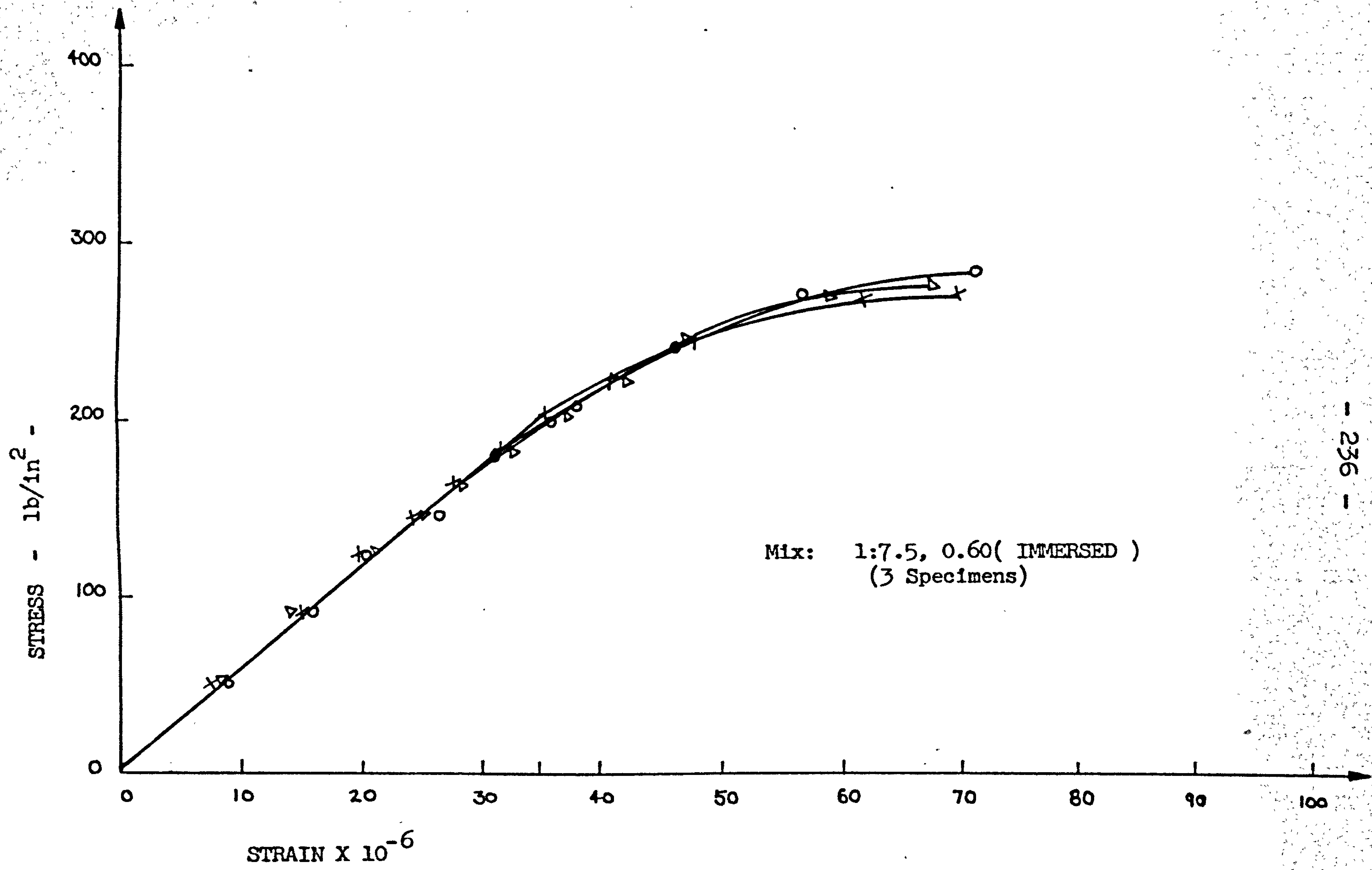


Fig. VIII.4. Stress - Strain Relashionship of Immersed Concrete.

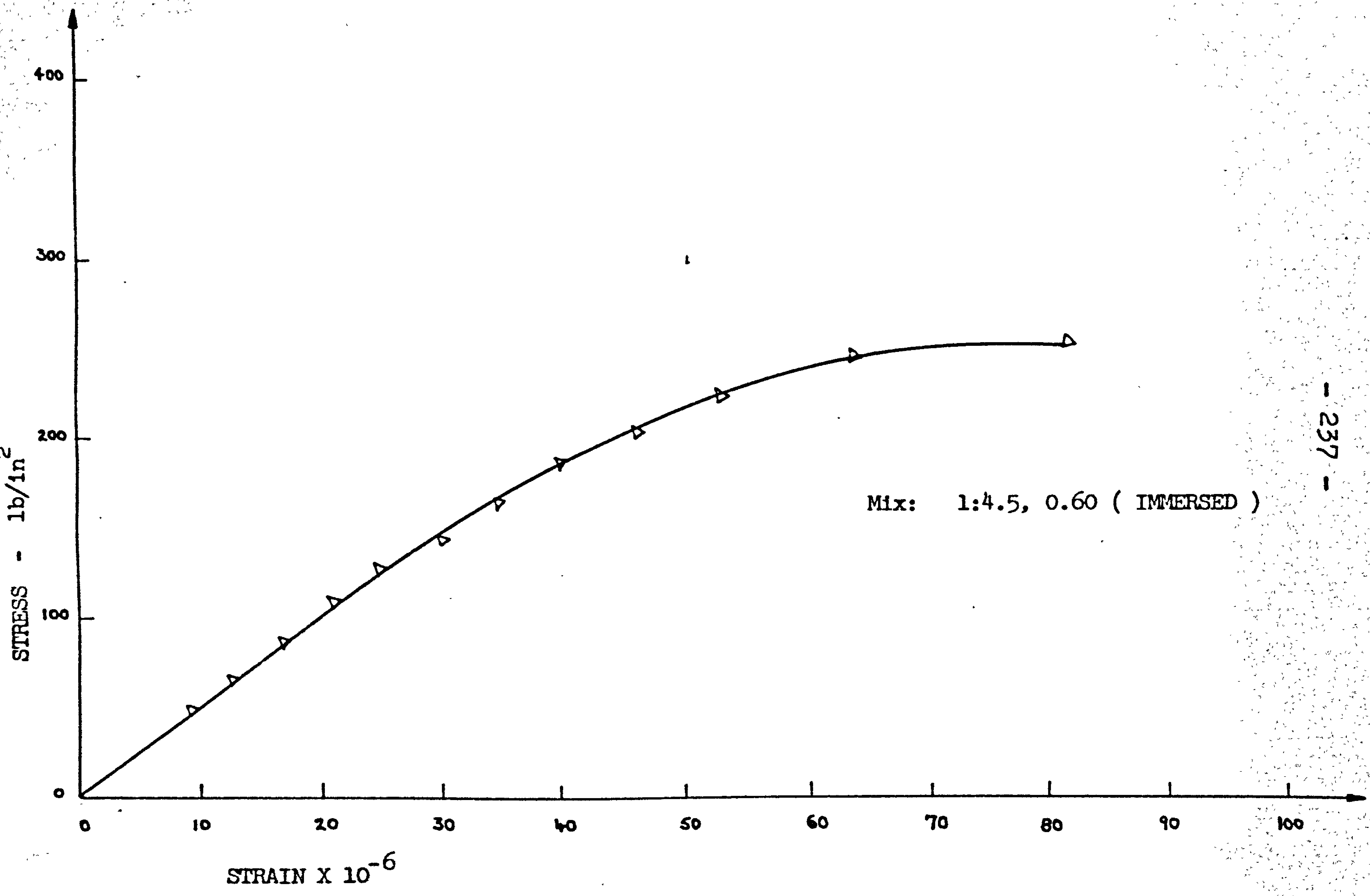


Fig. VIII.5. Stress - Strain Relationship of Immersed Concrete

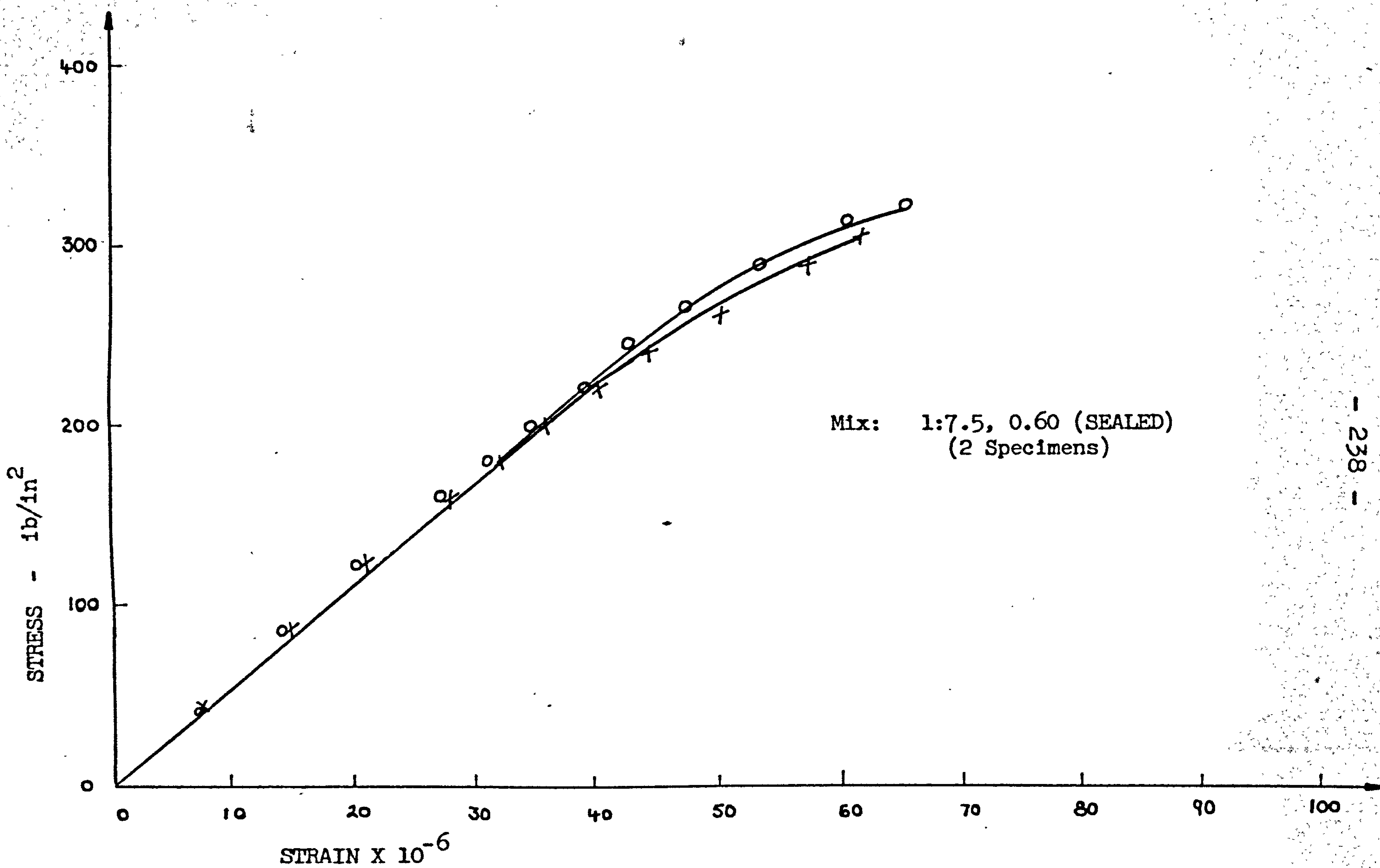


Fig. VIII.6. Stress - Strain Relationship of Sealed Concrete.

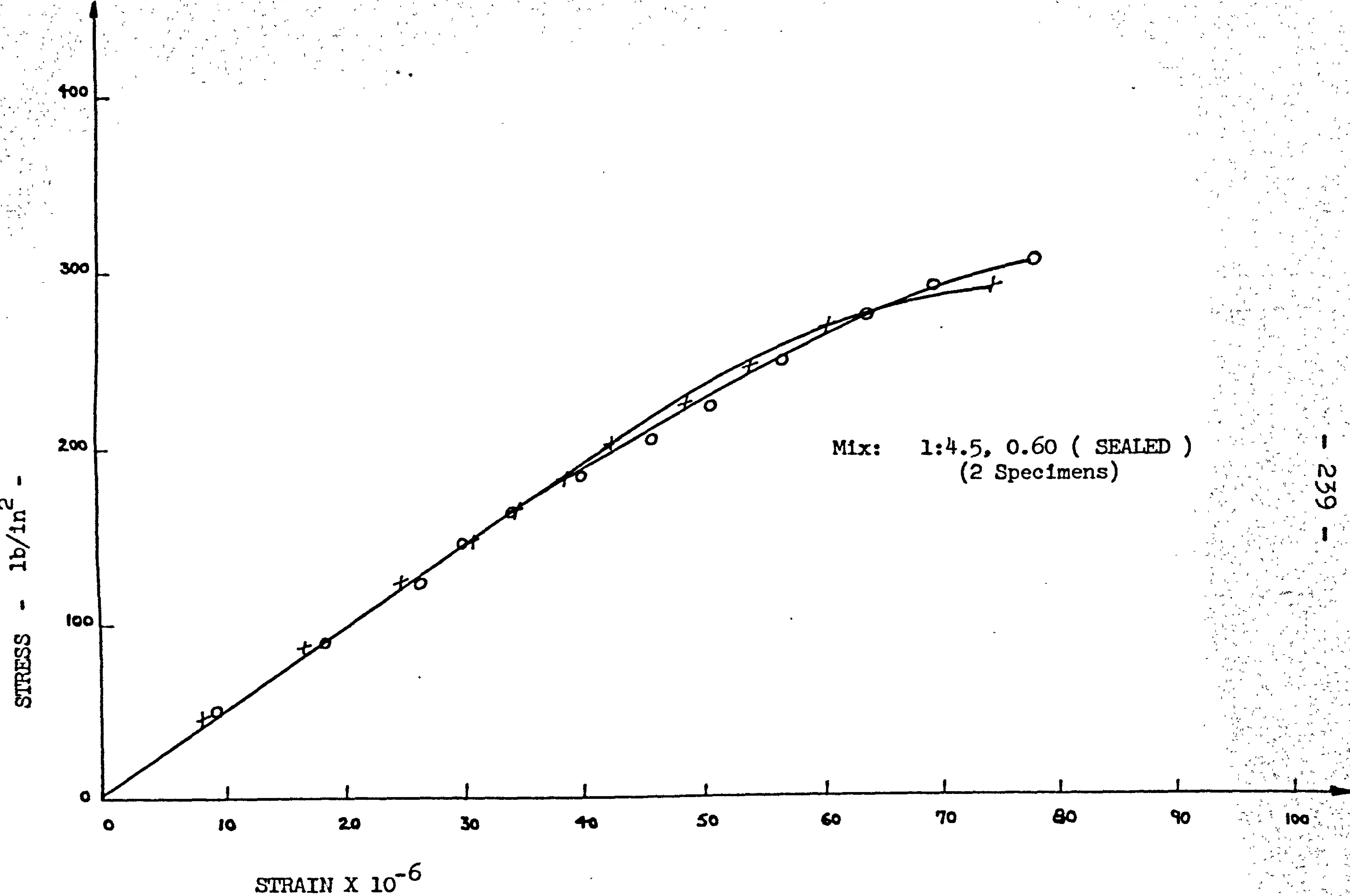


Fig. VIII.7. Stress - Strain Relationship of Sealed Concrete.

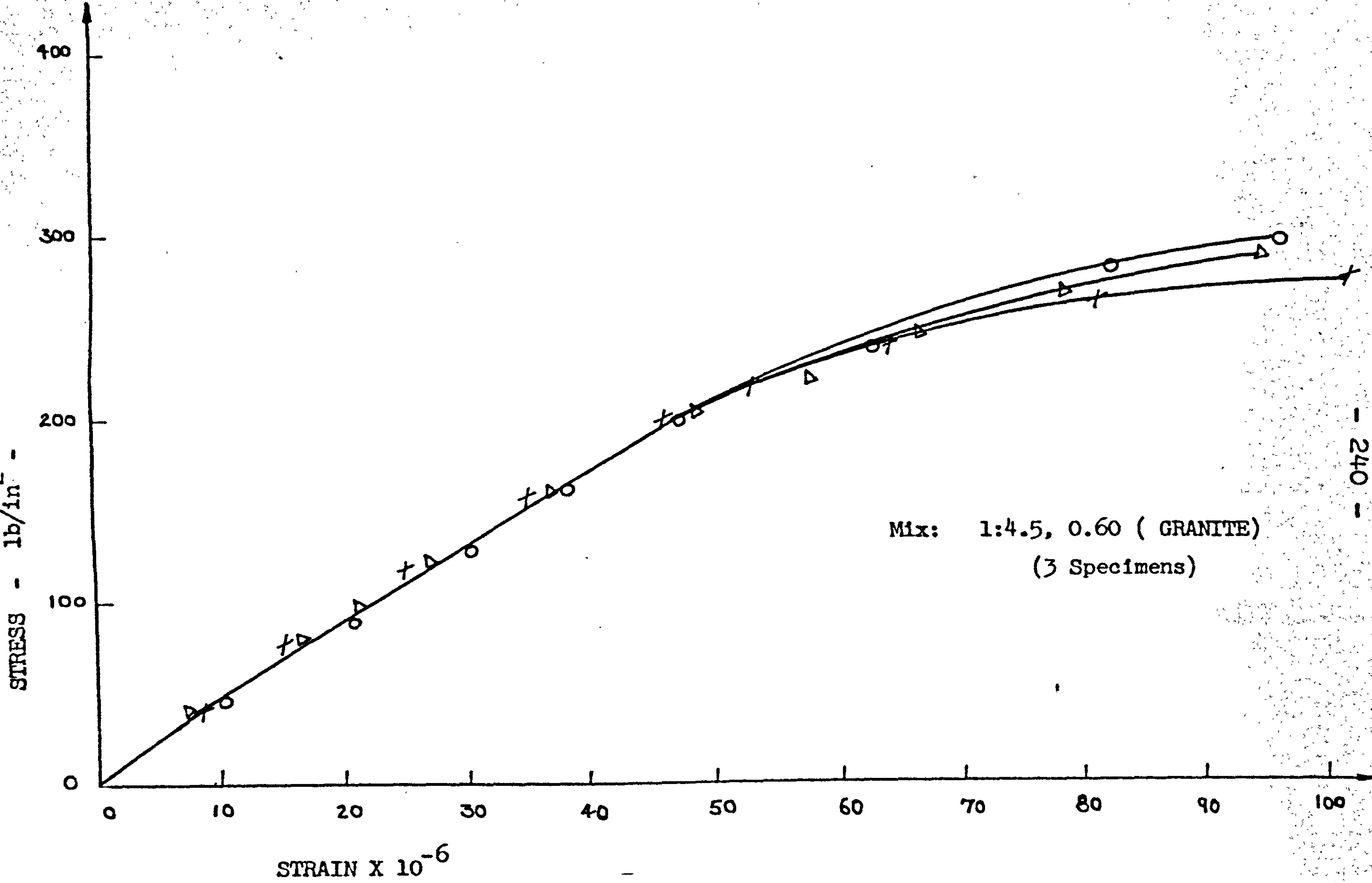


Fig. VIII.8. Stress - Strain Relationship of Immersed Concrete.

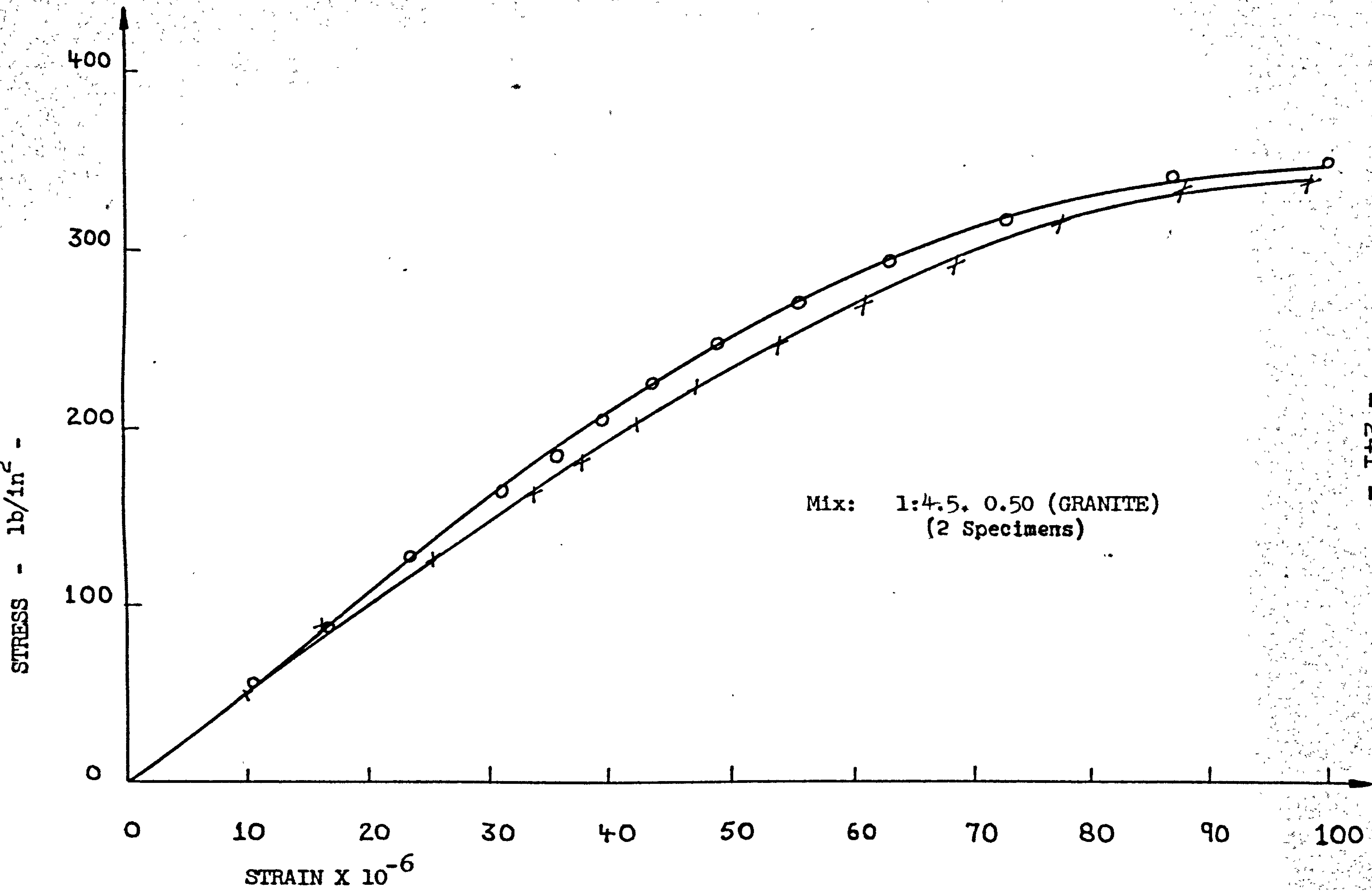


Fig. VIII.9. Stress - Strain Relationship of Immersed concrete

VIII.2. DISCUSSION OF RESULTS

It can be seen from Figure VIII.1. that the previously loaded specimens had higher moduli of elasticity and lower ultimate strains than the unloaded specimens.

This effect of increasing the modulus of elasticity was realised earlier when conducting the creep experiments. The reason for this increase may be related to an activation of hydration, and healing of some opened-up cracks, caused by the external loading.

The stresses around the aggregate in the previously loaded specimens were possibly in a more critical condition due to the existing tensile stresses released from the cement paste to the aggregate while under sustained loading. Bond failure at these points is thus expected to occur at a correspondingly lower strain in the aggregate than that of the unloaded specimens. This may explain why the previously loaded specimens which had higher modulus of elasticity did not have higher strength.

This view is further supported by the results of the tests on the two specimens that were loaded to 50% of their ultimate strength for a period of 15 days and then tested to failure without allowing the creep strain to recover. These specimens broke at a strain (including creep strain) equal to 80-90 $\times 10^{-6}$ (an elastic strain of 50 x 10^{-6} and a creep strain of 30-40 x 10^{-6}). The stresses at failure were 230 and 245 lb/in^2 for the two specimens corresponding to about 65 and 68% of the ultimate strength of similar concrete tested to failure at a rate of 100 lb/in^2 per minute (see figures VIII.2. and VIII.3.). In this case the stresses around the aggregate appeared to have almost reached their critical level although the average stress across the whole section was still relatively low. When the load was further increased the tensile stresses at the interfaces between the aggregate and cement possibly exceeded the bond strength between the two materials hence initiating failure.

The effect of sealed and immersed curing on the modulus of elasticity can be seen in figures VIII.4. to VIII.7. These results indicate that, under low stresses, the modulus of elasticity is the same for sealed and immersed concrete but the rate of strain in the immersed specimens seemed to increase faster at the higher stresses than in sealed specimens resulting in slightly higher ultimate strain and lower strength.

These results are confirmative to those obtained earlier in Chapter VII, i.e., sealed specimens had higher ultimate strength than the immersed specimens.

The greater increase of strain in immersed specimens at the elevated stresses may be due to the bond cracks being more likely to occur earlier in these specimens. This is caused by the higher tensile stresses already existing around the aggregate due to the swelling pressure and higher short-term creep.

The concrete specimens made with crushed granite aggregate had a lower modulus of elasticity and a higher ultimate strength and strain than those made with gravel aggregate, as can be seen in figures VIII.8. and VIII.9. The reasons may be related to the lower modulus of elasticity and better bond characteristics of the crushed granite.

Due to the lower modulus of elasticity the stresses around the aggregate caused by a certain load would be less than those occuring in corresponding specimens made with gravel aggregate. Adding to this that the bond strength is higher in the crushed granite aggregate, a premature bond failure would not therefore be expected. A large strain would thus be anticipated before the bond strength is reached at the interfaces between the aggregate and cement paste, and in this case failure would probably be initiated in the cement gel.

VIII.3. INVESTIGATION INTO THE COMPLETE STRESS-STRAIN CURVE AND THE FALLING BRANCH

It was observed during the investigation of the long-term strength that large strains were being exhibited by the specimens, prior to failure. These occured too rapidly to be measured at that time.

The following pages describe an attempt to measure these strains and a brief investigation into their cause.

Experimental Procedure

Three, 3-months old, immersed specimens of Mix 1 (1:4.5, 0.50) were used for this experiment.

As a stiff machine (strain controlled) was not available in the laboratory, the rig described in Chapter III was employed for this test. The immersed specimen was placed in the loading rig and the gauge wires were connected to the Maihak instrument. The specimen was loaded first to about 80-90% of its ultimate strength and the load was kept sustained while the strain was being continuously registered. The specimen was quickly unloaded when a fast increase in the rate of strain was observed on the gauge. It was then reloaded up to a stress at which the strain started to increase rapidly and then again unloaded. This procedure of loading and unloading was repeated until the specimen completely failed.

No visible crack could be seen on the surface of the specimen until the 9th cycle of loading at which time the stress at near failure was about 30% of the ultimate strength.

The measured strain at the point near failure was rising at an increasing rate with every cycle of loading.

The same test was repeated on the other two specimens the results of which were confirmative to the former.

The results of the test are plotted and shown in figure VIII.10.

VIII.4. DISCUSSION OF RESULTS

Figure VIII.10. can be explained in two ways:

1. By a progressive failure within the concrete during which time microcracks first form and, if these are prevented from developing, the section does not fail but weakens. During this time the rest of the material can still behave elastically, and withstands a load smaller than that prior to the occurence of the microcracks. More microcracks may form at the second cycle of loading and again, if prevented from spreading, the undamaged material in that section could still behave elastically while also supporting a lower load due to the decrease in its area

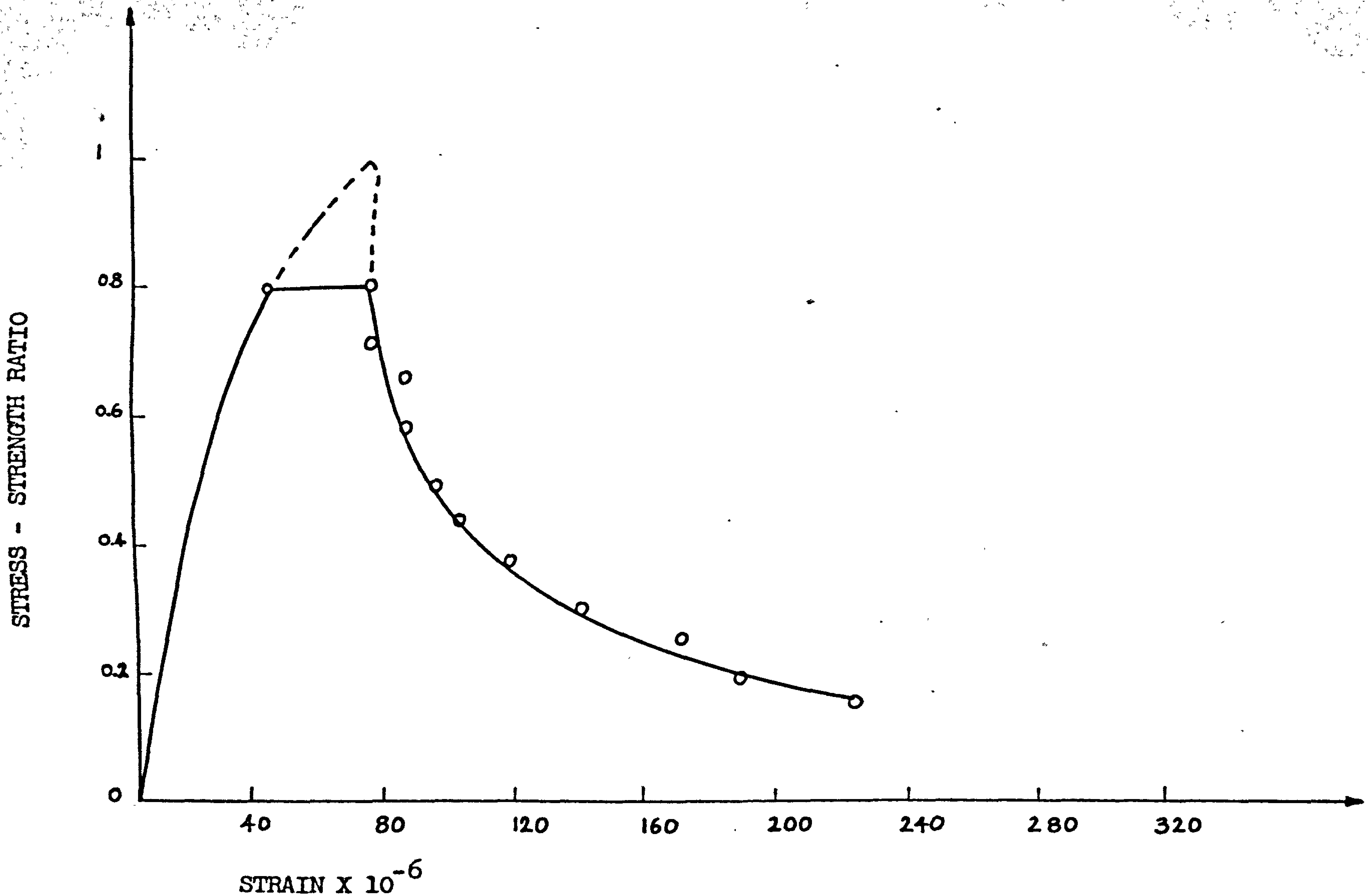


Fig. VIII.10. Stress - Strain Relationship (Including the Falling Branch).

relative to the whole section. However, this would imply that some of the undamaged material could exhibit very large strains before it ultimately ruptures.

2. A similar result may also be obtained from the strain measurements if a crack forms at the surface of the specimen and progresses inward gradually. The measured strain in this case will increase with every cycle due to the increasing eccentricity of the load with the progressing crack.

   A brief calculation of the increase in strain that would be expected if a crack formed at the surface and progressed inward is shown below:

Assumptions

The following calculation is intended to show the trend of the curve.

It is assumed that:

1. Linear stress-strain relationship up to near failure is exhibited by the specimen. No stress concentration occurs on the tip of the crack and the stresses are

assumed to be the same over a length of one inch above and below the uncracked region.

3. The plan section normal to the axis remains plane after cracking.

4. The maximum strain that could be exhibited by the material at any time is equal to that occuring just before the curve starts to descend, that is if we assme P_o to be the maximum load the maximum strain will be $= P_o/EA$.

Analysis of strains in circular section

Area of cracked region:

$$A_c = R^2(\varphi - \tfrac{1}{2}\cdot\sin 2\varphi)$$

Moment of this area about o-o:

$$= \frac{2R^3\sin^3\varphi}{3}$$

Distance from o-o to centre of gravity of uncracked region:

$$e = \frac{2R\sin^3\varphi}{3(\pi-\varphi+\frac{1}{2}\sin 2\varphi)}$$

Second moment of area of cracked region about o-o:

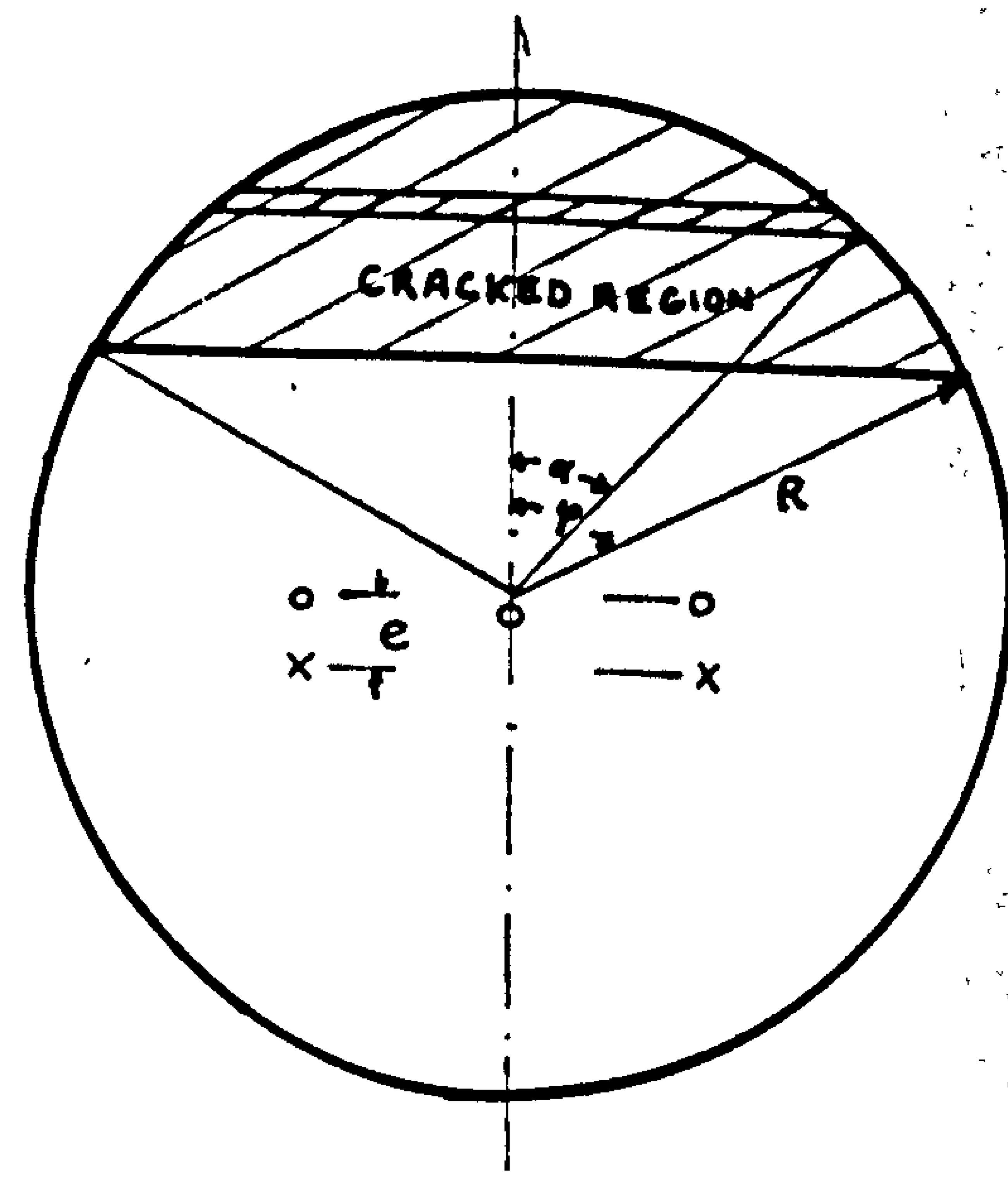
$$I^c_{oo} = \tfrac{1}{4}\cdot R^4(\varphi - \frac{\sin 4\varphi}{4})$$

Moment of inertia of uncracked area about centroid:

$$I_{xx}^{u} = \frac{1}{4}\cdot\pi R^4 + \pi R^2 e^2 - (Ae^2 + I_{oo}^{c} + 2e(\pi R^2 - A_c))$$

Maximum strain at first crack:

$$= \frac{P_o}{E\pi R^2}$$



Minimum strain in the section:

$$\varepsilon_{min} = \frac{P}{E}\left(\frac{1}{A_u} - \frac{1}{Z_1}\right)$$

Maximum strain in the section:

$$\varepsilon_{max} = \frac{P}{E}\left(\frac{1}{A_u} - \frac{1}{Z_2}\right)$$

$$= \frac{P_o}{E\pi R^2}$$

Hence,

$$\frac{P}{P_o} = \frac{1}{\pi R^2\left(\frac{1}{A_u} + \frac{e}{Z_2}\right)}$$

The strain at the surface of the crack:

$$\varepsilon_{crack} = (\varepsilon_{max} - \varepsilon_{min})\cdot\frac{2}{1 + \cos\varphi} + \varepsilon_{min}$$

A similar calculation was also made for a specimen of square section.

The strains are calculated for values of P equal to 0.8, 0.6, 0.4, and 0.2 x P_o, plotted and shown in figure VIII.11.

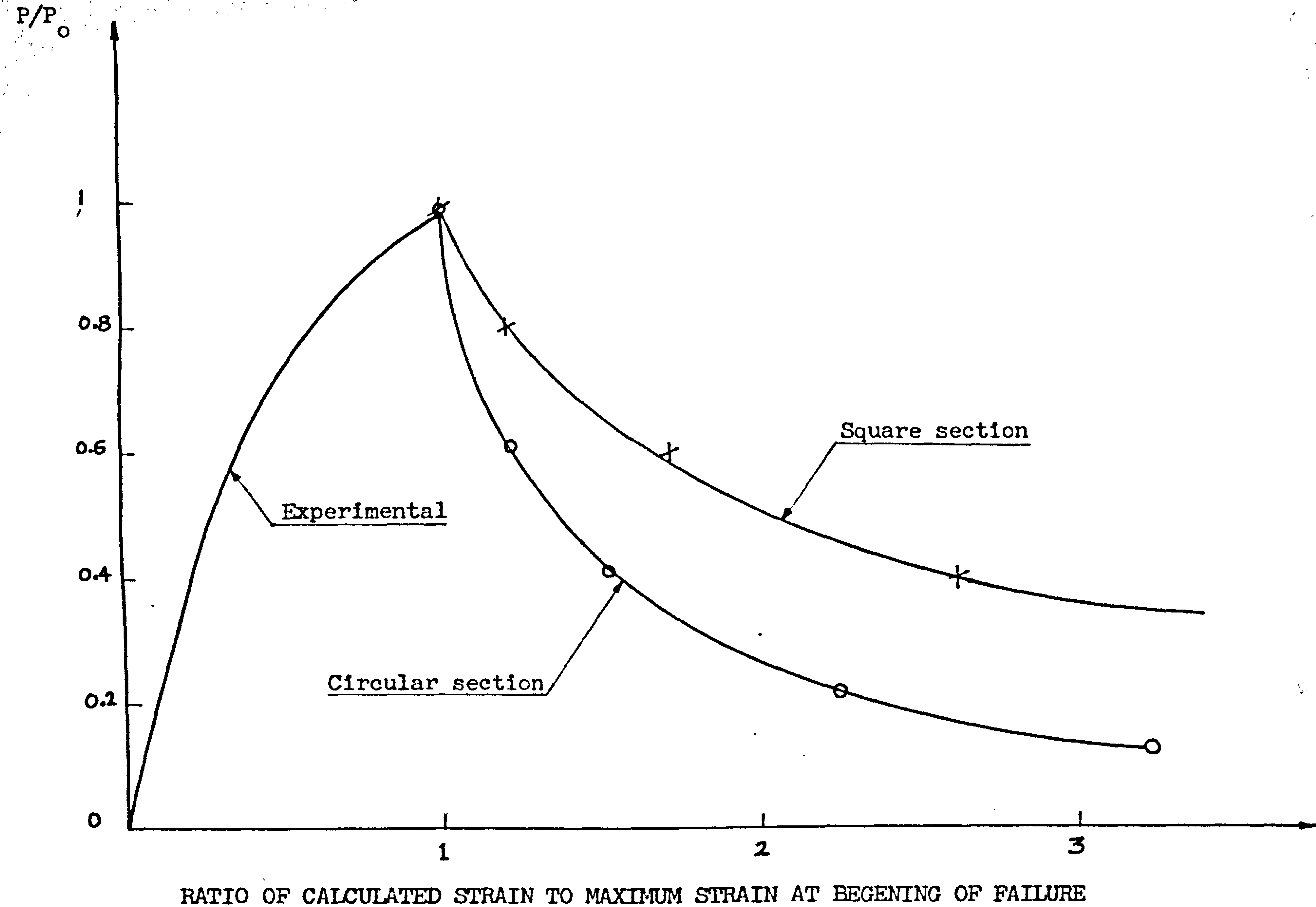


Fig. VIII.11. Calculated Strain at the Cracked Surface.

Experimental Verification of Cracking

In order to verify experimentally whether a crack was forming, two surface accoustic gauges were mounted at opposite sides of the specimen. The strains were measured for a stress of 20% of the ultimate strength before commencing and after every cycle of loading. The surface strains were measured at four points around the specimen.

Prior to loading of the specimen, the strains, measured at the four points, were almost identical but as soon as the first cycle was loaded and unloaded differences in strains registered by the surface gauges started to appear and there were very clear variations after few cycles of loading.

It is therefore thought that what appeared to be a falling branch in the tensile stress-strain curve could largely be due to a crack propagation and a shift in the axis of loading in respect to the centre of gravity of the uncracked region.

However, the possible existence of a small portion of a falling branch in the stress-strain curve and before a crack starts to form may narrow the differences between the calculated and experimental results.

Conclusion

The following conclusions may be drawn regarding the stress-strain relationship experiments:

1. A period of sustained loading followed by an equal period of recovery increased the modulus of elasticity of the concrete but did not seem to affect the strength. However, when the specimen was not allowed to recover, an appreciable decrease in strength was obtained.

2. The sealed specimens generally had the same modulus of elasticity as that of their immersed companions but the rate of strain in immersed specimens increased faster at the high level of stresses to yield a lower strength and perhaps larger strains.

3. The concrete specimens made with granite aggregate yielded considerably larger ultimate strains than those made with gravel aggregate. The crushed granite used was of a lower modulus of elasticity and had a rougher texture.

4. Deviation from linearity in the stress-strain curves were observed to occur at stresses ranging between 55 and 75% of the ultimate strength.

5. A falling branch in the stress-strain curve was obtained using a special loading procedure but it is considered that this could be due to a crack propagation and the consequent development of eccentricity in the loading system.

CHAPTER IX

CONCLUSIONS & SUGGESTIONS FOR FURTHER RESEARCH

IX.1. SUMMARY OF CONCLUSIONS

In order to successfully understand the behaviour of concrete under load, it is vitally important to have an explicit knowledge of its basic structure. This aspect has been dealt with briefly in the first chapter and was used as a reference in the proceeding chapters.

After surveying the existing methods for uniaxial testing of concrete specimens used by various investigators, it was concluded that the conflicting results of the little available literature on tensile creep may largely be due to the inadequate methods of testing and measurements of strains which had been employed.

A method has been developed and presented in chapter III which is thought to offer a solution for the long-term testing of concrete in uniaxial tension and has proved to be very satisfactory both for observing tensile creep in concrete and measuring its tensile strength.

A mechanism, based on a Seapage-Plastic theory, has been proposed in Chapter VI to explain the tensile creep and short-and long-term failure of concrete.

The experimental results are thought to be well defined by this mechanism.

The experimental work may be summarised under the following headings:

Tensile creep

Short-and long-term strength

Tensile stress-strain characteristics

IX.1.a. Tensile Creep

The available work on uniaxial tensile creep was first analysed in I.5. and no definite conclusion was thought possible at that time due to the small and conflicting informations available.

Chapter V. dealt exclusively with the experiments on tensile creep, while discussion of the results formed the first part of Chapter VI.

The effect on tensile creep of the following factors were investigated:

(a) Moisture condition of the surrounding medium.

(b) Level of stress

(c) Age at Loading

(d) Water-cement ratio

(e) Aggregate-cement ratio

(f) Type of aggregate.

The following conclusions have been drawn:

1. The creep of immersed concrete is nearly double that of sealed concrete.

2. The creep of air drying concrete is much larger than that of sealed or immersed concrete.

3. The creep of sealed and immersed concrete reached a limiting value after serveral weeks of loading while that of air drying was still continuing after one year.

4. The creep decreased with increasing age at loading.

5. A linear creep-stress relationship was generally obtained.

6. The creep increased with increasing water-cement ratio particularly at the higher ranges.

7. The creep decreased with increasing aggregate-cement ratio. However, this was not as influential as water-cement ratio.

IX.1.b. Short-and Long-term Strength

Short-term

The effect on the short-term strength of sealed and immersed curing, immersing or exposing to air drying after a period of sealed curing and immersing after a period of sealed curing followed by another period of air drying were investigated. The effect of sustained loading and most of the factors investigated in the creep experiments were also explored.

The following points have been concluded:

8. The strength of concrete specimens cured sealed is higher than that of specimens cured under water or air drying for mixes of various aggregate-cement and water-cement ratios.

9. Immersing a concrete specimen for several days after having been cured sealed for a period of three weeks decreases its strength to a value lower than that obtained from a companion specimen cured continuously under water.

10. Exposing a concrete specimen to air drying after it had been cured sealed, or immersing it after it had been air drying, could reduce its strength.

<u>Long-term</u>

The effect of sustained loading on the strength of immersed concrete made with gravel and crushed granite aggregate was investigated. The following points have been concluded:

11. The long-term tensile strength of immersed concrete subjected to sustained loading could be as low as 50 to 60% of the short-term.

12. The ratio of long-to short-term ultimate strength of sealed concrete made with gravel aggregate is much higher than that of immersed concrete. There is limited evidence to show that the long-term strength of sealed concrete can be 50% more than that of immersed concrete.

13. The ratio of long-to short-term ultimate strength of immersed concrete made with granite aggregate is not much lower than that of sealed gravel aggregate of the same mix proportions.

14. The ultimate tensile strain of immersed mature concrete does not change much with the load or the period of loading which suggestions a limiting positive strain for a certain mix.

IX.1.c. Stress-strain Characteristics

The tests carried out on previously loaded and unloaded specimens which were originally cast for the creep experiments resulted in the following conclusions:

15. Concrete previously subjected to sustained loading showed an increase in its modulus of elasticity.

16. A period of sustained loading, followed directly by testing to failure, appeared to reduce the ultimate strength of the concrete while the ultimate strain kept nearly constant with the range of about 70-90 microstrain.

17. Sealed and immersed cured concrete specimens had the same modulus of elasticity but the rate of strain in immersed specimens increased faster at the higher level of stresses to yield a lower strength.

18. Concrete specimens made with the crushed granite aggregate yielded larger ultimate strains than those made with gravel aggregate.

19. Deviations from linearity in the stress-strain curves were observed to occur at stresses ranging from 55 to 75% of the ultimate strength.

20. A falling branch in the stress-strain curve was obtained using a special loading procedure.

21. An explanation has been advanced to show that very high values of tensile strain measurement obtained at the surface of tensile specimens do not accurately reflect the true strain capacity of concrete.

IX.2. SUGGESTIONS FOR FURTHER RESEARCH

It is not for the author to suggest the type of further work in the field of chemistry and physics of cement gel, but it is obvious that a fuller knowledge of these will undoubtedly be of great use to the research engineer.

However, a few points have emerged as a result of the work presented in this thesis which are thought to be worthy of further investigation:

1. As has been demonstrated, sealed cured concrete has proved to be more stable with time than immersed or air drying concrete, i.e., it exhibits less volume changes under internal and external stresses, and thereby produces higher short-and long-term uniaxial tensile strength. Further investigation directed towards a study of more complex states of stresses should be undertaken to ascertain whether this effect is evident in a wider context.

2. Further investigation into the uniaxial long-term strength of concrete made with different mixes and particularly with various aggregate types, texture and stiffness, may provide the engineer with better information about the use of concrete in tension and may bring further corroboration of the validity of the mechanism suggested.

3. Creep experiments on specimens made with hardened cement paste, mortar and concrete with aggregate of practically zero stiffness could throw further light on the mechanism involved and may make a proper analytical solution possible.

Finally, if a practical sealing agent could be found and used economically in industry, a great deal of the uncertainty involved in the prediction of concrete behaviour resulting from the weathering effect on our structure might be eliminated.

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