A STUDY OF THE STRENGTH AND COMPRESSIBILITY CHARACTERISTICS OF COCHIN MARINE CLAYS

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BENNY MATHEWS ABRAHAM

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DECLARATION

I hereby declare that the work presented in this thesis is based on the original work done by me under the supervision of Dr. Babu T. Jose, Professor and Head, School of Engineering, Cochin University of Science and Technology, Cochin 682022, in School of Engineering. No part of this thesis has been presented for any other degree from any other institution.

Cochin 682022

November 23, 1993

BENNY MATHEWS ABRAHAM

CERTIFICATE

This is to certify that the thesis entitled "A Study of the Strength and Compressibility Characteristics of Cochin Marine Clays" is a report of the original work carried out by Shri. Benny Mathews Abraham, under my supervision and guidance in School of Engineering. No part of the work reported in this thesis has been presented for any other degree from any other University.

Cochin 682022

November 23, 1993

Dr. Babu T. Jose
(Supervising Teacher)
Professor and Head
School of Engineering
Cochin University of
Science and Technology

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Chapter I

INTRODUCTION

The Indian peninsula has a very long coastal line and almost the entire length of 6400 km from Gujarat on the west coast to West Bengal on the east is more or less completely covered by soft highly compressed marine clays. Since independence, the country has registered a steady progress in sea trade which marked the birth and development of a number of port cities - large and small - all along the coastal line.

With the advent of urbanisation and consequent search for more and more habitable land, it was imperative that the large tracts of marine clays, considered inhabitable earlier, had to be reclaimed and developed. These marine clays, wellknown for its high compressibility and poor shear strength, posed numerous problems to the builders and Cochin was no exception. It is only less than a decade since active research work was initiated on marine clays in general and Cochin marine clays in particular. Eventhough some systematic studies are available on compressibility characteristics, attempts to study the shear strength aspects and development of techniques to improve it have been very limited. This work is an investigation on the consolidation and shear strength characteristics of Cochin marine clays including methods to improve the same.

The contents of the various chapters are briefly described below.

Chapter II presents a review of investigations by earlier workers. The various theories regarding the evolution of the backwater region in Kerala and the distribution of marine clay deposits are discussed. Studies carried out on the physical properties of marine clays, compressibility and shear strength, improvement of these by treatment with additives are presented.

Chapter III gives a brief account of the materials used and methods adopted in this work. The distribution of sampling locations, collection and preservation of samples and preparation of samples for the individual tests are described. Properties of the additives used for stabilisation are presented. The chapter also includes an outline of the test procedures adopted especially in cases where there are deviations from standard procedures due to the peculiar nature of the marine clays and where standard procedures do not exist.

Chapter IV gives a detailed account of the studies on the compressibility characteristics of Cochin marine clays. A method has been evolved for the preparation of specimens consolidated at specified pressures. This made it possible to determine the accuracy and reliability of the Casagrande method for determination of p_c along with a few other techniques suggested by later workers. Having established the need for a more reliable procedure, two additional methods were tried and found superior. Through a series of consolidation tests on samples collected from various locations on the west coast, the superiority of the two suggested methods were established.

Preloading of clays to bring down the consolidation settlements has been in practice for quite some time. Improvement in compressibility characteristics with an additional surcharge load over and above the permanent load has attracted research workers. This technique known as precompression suffers from the nonavailability of a standard design procedure. The parameters influencing precompression and its effects have been identified through a series of well planned consolidation tests on marine clays from Cochin and Mangalore. A method has been suggested for the design or planning of a precompression project.

Chapter IV also presents the results on consolidation tests carried out on samples treated with lime and cement. After discussing the changes in physical properties, it discusses the various parameters which contribute to the improvement in compressibility characteristics. The influence of parameters like lime or cement content, curing period etc. on coefficient of consolidation and coefficient of secondary compression are discussed. The inferences drawn from the compression and rebound curves are also presented in this chapter.

Eventhough there have been studies on the compressibility characteristics of Cochin marine clays, not much work has been done on its shear strength aspects. Chapter V presents the details of investigations on the shear strength characteristics of these clays. Consolidated undrained tests with pore water pressure measurements, carried out on undisturbed samples helped to study the stress-strain behaviour of these clays. Variations in 'A' parameter, effective stress paths and dilation effects are also discussed. The details of the consolidated drained tests with volume change measurements and unconsolidated undrained tests along with the inferences drawn based on the test results are also presented. Most of the tests were repeated on marine clay samples collected from Mangalore to obtain a comparative study.

The profound influence that drying has, on marine clays are discussed in this chapter. The chapter also gives an outline of how the samples after air drying can be compacted and how the wetting and drying processes affect the shear strength parameters.

The shear strength characteristics of lime treated Cochin marine clays are discussed in great detail in chapter V. The chapter shows how the lime content and curing period improve the strength characteristics with the help of laboratory vane shear tests and unconfined compression strength tests on lime treated specimens. In order to study the improvement in shear strength during the initial periods of curing where strength is measured by laboratory vane shear tests and during long periods of curing where only UCC tests can be done, a correlation has been obtained between τ and c to establish continuity. An analysis of the stress-strain curves obtained from UCC tests and triaxial shear tests on lime treated clays is presented in this chapter.

Chapter VI gives a detailed account of a number of conclusions drawn based on the results of the studies carried out on Cochin marine clays.

Chapter II

REVIEW OF PAST WORK

2.1 Geology of Greater Cochin

Madagascar hugged the west coast of Kerala - or more probably the Western Ghats itself since it is mostly composed of the Archean rocks before the continents parted. After the continental drift and before the ice age, the waves of the Arabian sea washed the foot of the Western Ghats. Geologists have recognised the wave cuts on the western slopes. Thus the sea level is estimated to have been 50-100 m above the present one nearly a million years ago. While the macro level changes involving tens of kilometres in width of land takes place on a geological time scale as dictated by the major geological forces, smaller but significant changes take place as a result of land-sea interactions on a time scale which is of greater reference in normal human activities. It appears that such changes have been most pronounced on the coastal lands near Cochin (Erattupuzha and Padmanabhan, 1971).

King (1884) perhaps was the earliest to report on the geology of the area as part of his studies on mud banks off the coast of Cochin and Alleppey. According to him, 2500 years back, the sea washed upto the high ranges of Western Ghats. The land between the Western Ghats and the present coastal line was once well above the sea and was subsequently submerged. It was uplifted again by volcanic action and again partially covered by sea water. The uplifted area includes the coastal belt (10 to 30 km wide and about 150 km long) starting from Crangannore at the north to Quilon at the south. Most of Greater Cochin is situated in this belt.

Philip Lake (Memoirs of GSI, 1890) has stated that, before the lateritic period, Malabar (Kerala) stood nearly 150 m below the present sea level and the sea washed the Western Ghats. According to him, the land has slowly risen up from the ocean bed. It is probable that a combination of factors - retreat of sea due to glaciation and rise of land, also due to lowering of sea level have contributed to the formation of land now seen between the Western Ghats and the Arabian sea.

Historians of Kerala are generally unanimous in their opinion that the land mass west of the backwater areas from Crangannore to Quilon (Cochin is centrally located in this stretch) has come into existence only after the Christian Era started. The general physical features of the barrier land between the Arabian sea and the backwaters agrees with this view. However, it is to be confirmed whether the age of this barrier land is less than 2000 years only. It is possible that this barrier land was formed as a result of the action of the mud banks which is a peculiar phenomenon of our coast. Judging from the present day influence of mud banks on the coast and its performance, it can be stated that it traps all the material moved by the waves along the shore to grow wherever it is situated (Erattupuzha and Padmanabhan, 1971).

The barrier land beaches between the backwaters of Kerala and Arabian sea are of still more recent origin. Geologically the soil is made up of recent sediments. The age of this land from Crangannore to Quilon, is estimated to be less than 3000 years (Madras District Gazetteer, 1951).

2.2 Marine Clay Deposits

Marine deposits can be found all along the coastal belt of Indian Peninsula. Narasimha Rao and Kodandaramaswamy (1984) based on investigations on clays from Cochin and Madras, have drawn some useful conclusions on Indian marine clays. Indian marine clays are deposited at high water contents close to liquid limit giving rise to poor consistency and high void ratio. The soils have high colloidal activity and are low to medium sensitive.

Rao et al. (1990) made an extensive study on Cochin and Mangalore marine clays and compared their properties with a similar marine clay - the Ariake Clay from Japan. They have compared the chemical and engineering behaviour of Cochin and Mangalore clays with those of the smectite-rich Ariake clay. They found that although Cochin, Mangalore and Ariake clays contain comparable amounts of smectite (32-45%), Ariake clay exhibits lower consistency limits and much higher ranges of liquidity indices than the Indian marine clays, which is attributed to the absence of well developed, long range interparticle forces associated with the clay. They also found that the sensitivity of Ariake marine clay is much higher than the Cochin and Mangalore marine clays.

Noorany (1989) proposed a new classification system for marine sediments based on the principles of soil mechanics and geology. He is of the opinion that most of the marine sediments are derived from terrestrial sources, but vast areas of the sea floor are covered with sediments that are unique to the ocean environment.

Shimizu (1990) reported expressions for water content and void ratio for saturated marine sediments. He demonstrated that the water content, pore fluid content and void ratio would be underestimated by the order of 5% unless the salinity is taken into account.

Tan (1983) made an extensive study on the geotechnical properties of soft bluish grey rich in kaolinite

Singapore marine clay. These clays are relatively young in geological age (5000-11000 years) and are generally flocculated. The liquidity index of the Singapore marine clay varies between 0.66 and 1.05 and the sensitivity varies from 1.5 to 6, associated with low shear strength and very high compressibility.

2.3 Physical Properties of Marine Clays

Warkentin (1961) interpreted liquid limit as the distance between particles or units of particles at which the forces of interaction become sufficiently weak to allow easy movement of the particles or units relative to each other. Seed et al. (1964) suggested that the liquid limit depends on the interparticle forces and therefore on the surface characteristics of the soil. Sridharan et al. (1975) have investigated in detail the possible mechanisms governing the liquid limit of Kaolinite and montmorillonite type of clays.

The flow index obtained from the liquid limit test can be used with advantage of determining other properties such as plastic limit. The one point method has received wide attention as a simple procedure (Nagaraj and Jayadeva, 1981). The cone penetration method is, in some cases, especially those in which the pore fluid is other than water, very convenient compared to the percussion method. The validity of the cone penetration test has been studied in detail by Karlsson (1961); Clayton and Jukes (1978).

Jagadish Narain and Ramanathan (1967) were perhaps the earliest to observe the physical properties of marine clays of Kerala. According to them, the marine clays of this region undergo irreversible changes in plasticity characteristics. Air drying was found to cause formation of aggregates, which was considered responsible for the change in plasticity.

Frost (1967) attributed the phenomenon of reduction in plasticity on drying to dehydration of hydrated iron/aluminium sesquioxides and of the halloysite mineral. He brings out the significant changes during oven drying and indicates the importance of carrying out tests on soil in its natural state.

Narain and Ramanathan (1970), while discussing the geotechnical properties of the marine clays from Kuttanad area, focusses attention on the peculiarity of the soil where there is a variation in properties caused by air drying. They showed that the mechanical properties are dependent on soil structure which is a function of interparticle forces as well as particle size and arrangement.

Jose et al. (1987, 1988a, 1988b) and Sridharan et al. (1991) discusses at length the physical properties of Cochin marine clays and the effect of air and oven drying of these clays on their physical properties. It was found that the liquid limit and free swell index decreases drastically. While there is a marginal decrease in plastic limit, the shrinkage limit increases by reasonably high percentage. The results of grain size distribution revealed that the silt size fraction increases significantly due to air or oven drying and the dispersing agent plays a decisive role in obtaining grain size distribution curve.

Organic matter and carbonates are recognised as major cementing agents contributing to particle aggregation and reduction in Atterberg limits upon drying (Rao et al. 1989). They indicate the dominance of calcium and magnesium ions and presence of a high salt concentration facilitates strong interparticle attraction and a close spacing of the particles. According to them, the aggregation of particles and variations in plasticity characteristics is due to cohesion from surface tension forces arising from capillary menisci formed between the particles at the boundaries of the soil specimen during drying.

The X-ray diffraction pattern of the treated clay fraction of Cochin marine clay showed strong reflection around 17.65Å which is characteristic of smectite group of clay and relatively weaker reflection around 7.25Å, typical of kaolinite cl (Jose, 1989). Additional reflections were also noted around 3.58Å and 3.35Å, attributed to kaolinite and smectite group of clay respectively. the untreated sand and silt fractions were also subjected separately to X-ray diffraction analysis and was observed to comprise of felspar, mica, quartz, calcite and haematite.

Ramanatha Ayyar et al. (1990) are of the opinion that organic matter and other salts present in the pore water system may cause the formation of new cementing compounds on air drying of the clay. According to them, the clay minerals in young sedimented organic clays, especially in a marine environment, can get significantly altered by pre-treatment and the X-ray diffraction patterns obtained after treatment and air drying may not represent the true nature of the clay minerals in their original state. On air drying, it is possible that new compounds of organic aluminium complexes may be formed causing aggregation of the clay particles.

Pandian et al. (1991) discusses the effects of drying on the engineering behaviour of Cochin marine clays. They found that drying has significant effect on the index properties and on the compressibility and shear strength behaviour of these clays. They are of the opinion that normalising the soil data with respect to the liquid limit helps to provide a framework for examining the consolidation and undrained shear strength behaviour of the soil.

2.4 Estimation of Preconsolidation Pressure

The preconsolidation pressure may be defined as the pressure under which a soil deposit has been fully consolidated. A soil may show a preconsolidation pressure caused by a variety of factors including overburden pressure that may or may not have been reduced because of erosion, desication, temporary overloading, sustained seepage force etc. (Leonards, 1962).

Ladd (1991) defined the preconsolidation pressure as the yield stress that separates small strain elastic behaviour from large strains accompanied by plastic (irrecoverable) deformation during one dimensional compression.

Casegrande (1936) proposed a method for determination of the preconsolidation pressure from the e-log p curve. The method involves selecting a point corresponding to the minimum radius of curvature on the reloading curve, drawing horizontal and tangent lines at this point and bisecting the angle between them, then projecting back the straight line portion of the curve to intersect the bisector of the angle, the point of intersection being the preconsolidation pressure.

Burmister (1951) proposed another method for determination of preconsolidation pressure from the results of consolidation test. In this method, the sample is rebounded as soon as the compression curve reaches the straight line portion and is then reloaded. The characteristic triangle thus formed is then transferred to the first loading curve by trial and error 'giving more weightage to the vertical leg than the horizontal leg'. The position of the vertical leg then corresponds to p_c.

In the method suggested by Schmertmann (1955) the laboratory loading, rebound and reloading procedure is the same as Burmister's method. An assumed value of p_c is chosen on a line drawn parallel to the mean slope of the rebound curve through the point (e_o, p_o) which are in situ void ratio and overburden pressure. This p_c point joined to the point corresponding to 0.42 e_o on the compression curve is the assumed virgin curve. The reduction in void ratio Δe, between the assumed virgin curve and the laboratory curve is then plotted against log p. The procedure is repeated for different assumed p_c points and virgin curves and the most symmetrical void ratio reduction pattern is chosen for p_c.

Sallfors (1975) made an extensive study on various proposals of determining preconsolidation pressure of clays. He recommends a constant rate of strain test carried out at a strain rate less than 0.0024 mm/min. for this purpose. On plotting the compressive strain of the specimen against effective vertical stress (both to arithmetic scales), two linear parts of the curve before and after the location of the anticipated preconsolida' on pressure are obtained, which are made to intersect. An is celes triangle is constructed with its sides resting c the two straight portions and its base touching the curve tangentially. The left hand vertex of the triangle corresponds to the preconsolidation pressure.

Holtz and Kovacs (1981) suggested the extension of the two straight line portions of the laboratory e-log p curve to get the intersection point, which is the most probable value for preconsolidation pressure.

Hamilton and Crawford (1959) have investigated the marine clay of eastern Canada, called Leda Clay. One of the notable features of this marine clay was an unusually sharp change in compressibility at the preconsolidation load, which results in extremely high compression indices.

Butterfield (1979) reported that the log f vs. log p relations could be represented by a straight line in the range of normal consolidation in case of soft soils, where f is the specific volume defined as f = 1 + e. Oikawa (1987) extended these studies through investigations on a wide range of soils with natural water content varying from 25-941% and atural void ratio ranging over 0.7 to 17. The results showed that the log f - log p curves are essentially composed of two segments of straight lines. The stress at which the two line segment of log f - log p curves intersect agrees well with the consolidation yield stress which is obtained from the graphical method proposed by Casagrande.

Karunaratne et al. (1983) reported that the preconsolidation pressure can be determined through the measurement of the permeability of the soil sample. A plot between void ratio (arithmetic scale) and coefficient of permeability (logarithmic scale) can be made, resulting in two distinct straight lines. The intersection of the two linear segments will give the void ratio corresponding to the preconsolidation pressure.

Koerner et al. (1984) reported the development of Acoustic Emission method to directly determine the preconsolidation both in situ and in the laboratory. The results of their investigation indicate that the preconsolidation pressure of soils at water contents representative of the different consistency states of in situ fine grained soils can be determined using AE technology to within approximately ±10% of the actual value. This seems to indicate that it should be feasible to use an in situ test procedure such as pressuremeter test along with acoustic emission monitoring to determine the p_c value in situ.

A procedure to estimate the preconsolidation pressure of over consolidated soils was suggested by Nagaraj and Srinivasa Murthy (1985). Their procedure is based on the generalised compressibility equation which require the knowledge of in situ void ratio, liquid limit of the soil, in situ overburden pressure and specific gravity of the soil solids. The advantage of this procedure is that the preconsolidation pressure can be obtained without carrying out the consolidation test.

Jamiolokowski et al. (1985) discuss at length the mechanism of preconsolidation pressure and are of the opinion that the most popular method for determining the p_c still is Casagrande's empirical construction technique. According to them, the mechanisms that can cause overconsolidation can be classified as (i) Mechanical - due to the overburden removal or lower water table, (ii) Desiccation - due to evaporation or freezing, (iii) Aging (secondary compression) - due to drained creep, (iv) Physico-chemical - due to natural cementation and related phenomena. They have identified the major variables that can influence the measured value of preconsolidation pressure as (i) sample disturbance, (ii) the test equipments and procedures used to obtain the one dimensional compression curve, (iii) the interpretative techniques used to estimate the value, and (iv) environmental factors such as pore fluid composition and temperature.

A method of interpreting conventional oedometer test data using work per unit volume as a criterion for determining the yield stress in clay was presented by Becker et al. (1987). A plot of the work per unit volume and effective stress using arithmetic scales, results in two linear segments. They found that the intersection of initial fitted line and the linear segment at higher stresses can give the yield (preconsolidation) stress.

Murakami (1992) reported that there exists some difference between the observed and the estimated values of quasi-preconsolidation pressure. He found that the degree of difference is unchanged by consolidation pressure, but is influenced by the duration of sustained loading at a previous stage and has some relation with the properties of a clay.

2.5 Acceleration of Consolidation Process Through Preloading

Construction on sites underlain by thick strata of soft cohesive soils requires some ground improvement techniques to prevent bearing capacity failure and/or to avoid excessive total and differential settlements. Preloading can be used as an effective tool to serve this purpose.

The principle of precompression and the various aspects of preloading are dealt with in detail by Johnson (1970). He has discussed the various methods of preloading, the economy than can be effected and the various factors to be considered for adopting the preloading technique in the field. He has given expressions to compute the amount of surcharge and the time required for surcharge removal to arrest the settlements occurring due to primary consolidation in full and a portion of the secondary consolidation, once the final load (structural load) is applied.

The improvement of alluvial soil deposits of normally consolidated soft clays in the lower part of the 'Po valley' (towards the Adriatic sea) by using preloading fills is reported by Colleselli et al. (1981). The results of static cone penetration tests and uncofined compression tests showed tremendous improvement of these soils on preloading.

Tsai et al. (1981) investigated the effectiveness of sand drains with preloading in improving a reclaimed tidal marsh land. They found that the bearing capacity of the marsh land could be improved to such an extent, as it can carry up to 350 kPa of heavy storage material loading, with proper design and installation of sand drains with preloading.

One of the major field trials ever undertaken in India to stabilise marine clay was at the outer harbour in Visakhapatnam (Natarajan et al. 1982). The site of the ore handling yard underlain by 12 metres of soft marine clay of very low shear strength and high compressibility, was stabilised using sand drains and sankwicks preloaded with embankments. But the reasonable period of preloading in stages to develop strength to store 9 metres of iron ore was as long as 4 to 5 years.

Gatti and Goida (1983) reported the applicability of the finite element method to the determination of the degree of consolidation - time curve during a preloading process. The theoretical results were in very good agreement with in situ measurements.

Dembioki et al. (1983) investigated the preloading technique as means of improvement of weak marine sub soil. They found that by repeated loading and unloading of weak sand mud mix, the bearing capacity could be increased many fold and the settlements could be drastically reduced.

Veder and Prinzl (1983) through their laborator and field studies on normally consolidated clays conclude that overloading is a very useful method to avoid tl secondary settlements.

Jamilokowski et al. (1983) discusses the various aspects of preloading or precompression. The essential idea behind the concept of precompression of soft cohesive strata in subjecting the foundation to a load, the intensity and geometry of which is suitable to anticipate partially or entirely the settlements of the foundation under the weight of the structure which has to be built. Precompression is used with the aim of minimising the initial compression, anticipating partially or completely the consolidation settlement and reducing the rate of secondary compression.

2.6 Compressibility Behaviour of Stabilised Clays

In the last few decades, there has been a growing demand for construction on sites underlain by thick strata of soft cohesive soils. In such circumstances, a method of foundation improvement is generally required to prevent bearing capacity failures and/or to avoid excessive total and differential settlements.

Yamanouchi (1965) found that strong soil-cement forms a good layer when placed deep in a pavement structure. The layers nearer the surface will first form a strong material, then broken up by heavy rolling within one or two days after setting.

Croft (1967) reported that the suitability of a clay soil for cement stabilisation is controlled by its texture and chemical and mineralogical composition. From the results of several mineralogical examinations of clay-cement mixtures, he concluded that certain of the clay minerals interfere with the stabilising action of cement. For example, expansive clay minerals have a profound influence on the hardening of cement, notably montmorillonite, whose affinity for lime reduce the pH of the aqueous phase. The cementitious products developed during the curing of montmorillonite clay-cement mixtures are inferior in their degrees of crystallinity to those of non-expansive kaolinitic or illitic clays. Hence the developed strengths are lower unless cement addition is large enough to supply enough free lime.

Lime stabilisation produce immediate changes in plasticity, workability and swelling properties in addition to bringing in gains in uncured strength and load-deformation properties (Thomson, 1968).

An indication of whether or not a clay is amenable to stabilisation by cement is provided by its Atterberg limits (Croft, 1968). Generally clays with plastic limits greater than 20 or liquid limits in excess of about 45 to 50 are considered uneconomic, as such clays are difficult to mix and requires a high addition of cement. But addition of a small amount of lime makes the soil more workable and the cement stabilisation can be employed.

Cement stabilisation studies by Ingles and Metcalf (1972) showed that the cement content should be sufficient to reduce soil swellin below 2% and to resist the effects of freeze-thaw or wet-dry cycles without excessive degradation.

Organic matter and excess salt content, especially sulphates can retard or prevent the proper hydration of cement in soil-cement mixtures (Bell, 1976). One reason why organic matter retards the hydration of cement is because it preferentially absorbs calcium ions. Sulphates have proved to be deleterious to soil-cement mixtures. Sulphates interferes with cement hydrations and the resulting crystallisation and expansion causes disruption of the soil-cement. Sherwood (1962) through his experiments demonstrated that the adverse effect of sulphates is due to the reaction with the clay content.

Bell (1978) reports the chemistry of cement stabilisation. According to him, when hydration of cement takes place in the cement stabilisation process, it results in the formation of calcium hydroxide and the pH of the aqueous phase is raised to approximately 12.2. The products formed after short periods of aging are largely gelatinous and amorphous, but with further curing, poorly ordered varieties of hydrated calcium silicate and hydrated calcium aluminate develop. During the early stages of hydration, gelatinous products form around anhydrous cement cores and the precipitated from solution in the interstices between particles. Hardening is due to the gradual desiccation of these gelatinous products and the crystallisation of new minerals. As the cement hydrates, the strength of the stabilised soil improves and it becomes water resistant. The hydrated cement coatings form a skeletal structure of considerable strength, the actual strength depending on the lump size and the amount of cement used. The addition of as little as 2% cement to clay soil will generally modify the soil properties, the greater the proportion added, the greater the change.

Kujala and Neeminen (1983) through laboratory experiments found that the use of gypsum along with lime for stabilising clays will give better results when compared with the stabilisation using lime alone.

Compaction of lime stabilised soils is more tolerant than those stabilised with cement. Lime treatment flattens the compaction curves which ensures a given percentage of prescribed density over a much wider range of moisture content. The addition of lime increases the optimum moisture content and reduces maximum dry density for the same compactive effort (B. l, 1988).

The effect of lime on the strength and deformation characteristics of soft Bangkok clay has been investigated by Balasubramaniam et al. (1989). They have reported an increase of ten fold in the undrained strength on treatment with 5% lime. Significant effect was noticed in the preconsolidation pressure, thus improving the compressibility characteristics. Results of undrained triaxial tests showed an increase in both the cohesion and the angle of shearing resistance, on treatment with lime.

2.7 Shear Strength of Clayey Soils

Bjerrum (1954) reported the loss of remoulded strength of leached clay samples deposited under marine conditions.

Seed and Chan (1959) from their experiments concluded that flocculated samples have steep stress-strain curves and develop peak strength at low strains. Also dispersed samples have flat stress-strain curves and continue to increase in strength even at high strains. They found that flocculated structure develops less pore water pressure on shearing than dispersed structure.

Hvorslev (1960) reported that there is no consistent trend of the influence of particle orientation on the unconfined compressive strength of clay, though the difference between maximum and minimum is substantial.

Experiments by Seed, Mitchell and Chan (1960) on saturated samples of compacted clay and consolidated clay showed the same value of $\emptyset_e$ inspite of the bigger particle size of the clay in the compacted sample. Kirkpatrick (1965) on studying the effect of grain size on shear strength parameters, found that angle of internal friction $\emptyset'$ increases with decreasing grain size for equal void ratio for sands.

Moum and Rosenqvist (1961) produced evidence of the change of undrained shear strength of samples in which ionic replacement is effected without disturbing the sample.

Olson (1962) reported that the value of A_f for the sedimented sample (flocculated structure) was more than that of the remoulded sample (dispersed structure).

Sridharan and Madhav (1964) studied the effects of the rate of rotation on the shear strength properties and on the sensitivity of a soft silty clay through laboratory vane shear tests. They found that the strength is effected considerably by the rotation rate, especially for the undisturbed soils. The shear strength of these soils were found to increase upto 60 per cent when the rate or rotation was increased from 1.2 to 30 deg. per minute. They also observed an increase in sensiti ity as the rate of rotation increased.

Skempton and Bjerrum have found that there is a well de ined correlation between C_u/p and plasticity index and any deviation from this relation can be considered as a test for sampling disturbances, in case of normally consolidated marine clays (Bishop & Henkel, 1962).

Sridharan and Narasimha Rao (1973) critically examined the well-known linear correlation between linearly increasing Su/p ratio and plasticity index in normally consolidated, undisturbed clays. Their results show that Su/p tends to decrease as Ip increases and the significant incluence of A_f on the value of Su/p. They have concluded that any attempt to relate Su/p with Ip should give a band rather a unique relationship, since many factors such as soil structure, stress level and type of testing govern the undrained strength behaviour of normally consolidated clays.

Skempton (1964) suggested that the residual angle of shearing r sistance would be comparable to the angle of friction of the clay minerals themselves.

Sridharan et al. (1971) reported the effect of soil structure as brought out by a change in the initial moulding water content, stress history and chemical environment on the shear strength characteristics of saturated clays. Many factors such as type of clay mineral, water content, nature of pore water, stress history, drainage during shear, stress path, rate of loading, temperature and soil structure govern the strength behaviour of a clay. They have stressed the need for quantitative evaluation of the modified effective stress, such that realistic strength parameters could be determined.

Investigations by Sridharan and Narasimha Rao (1973) revealed that Hvorslev parameters are not unique and can have a wide range of values depending upon the method by which they are determined, the initial conditions (stress history and water content) of the sample, and the stress level. They found that the Hvorslev parameters are largely dependent on soil fabric.

Sridharan and Venkatappa Rao (1979) discusses the mechanisms controlling the shear strength behaviour of saturated kaolinite and montmorillonite clays, based on the modified effective stress concept, which takes into consideration the interparticle electrical attractive and repulsive forces. They have found that the soil strength is significantly influenced by the dielectric properties of the pore medium.

The influence of aging on the shear strength behaviour of fine grained soils was reported by Allam and Sridharan (1979). It was found that apart from an increase in shear strength and brittleness, the soils exhibited an increase in their angles of shearing resistance when aged for a month. They have concluded that the quantitative influence of aging under sustained loads, will depend on their clay mineral composition and the amount of secondary time effects displayed by them.

The causes and effects of aging in Champlain quick clays were discussed by Lessard and Mitchell (1985). They have found an increase in the remoulded strength and the liquid limit and a decrease in pH, sensitivity and liquidity index. The aging phenomenon was attributed to the oxidation of iron sulfide, which results in the formation of iron hydroxide and sulfuric acid.

Krishna Murthy et al. (1980a) reported the effects of stress path on the strength behaviour of a layered soil. They have found that the stress-strain behaviour of the specimens with both orientations (vertical and horizontal) is apparently same, although the volume change behaviour was different, either in compression or in extension. The effective stress parameters were found to be independent of the stress path.

Krishna Murthy et al. (1980b) studied the strength anisotropy of layered soil system, through a series of triaxial compression and extension tests on an artificially prepared layered soil. They have shown that the variation in strength is not only a function of the degree of orientation of the bedding planes to the direction of the principal stresses, but also on the mode of deformation.

Krishna Murthy et al. (1981) presented the results of investigations on the shear strength behaviour of overconsolidated clays. Their results show that the effective stress Mohr-Coulomb failure envelopes are independent of over consolidation ratio, past maximum pressure and the failure conditions such as the peak deviator stress, peak effective principal stress ratio and the peak pore water pressure.

Allam and Sridharan (1984), based on their investigations on desiccated soils concluded that the fabric changes produced by climatic changes play a larger role in determining the stiffness characteristics of desiccated soil while the cementat n effects predominate in the strength mobilisation. They observed that desiccated soils possess a stiffer stress-strain response and brittleness in the undisturbed state which is largely due to fabric effect and partly due to the presence of desiccation bond.

Nambiar et al. (1985) made an extensive study on the eng neering behaviour of fine grained carbonate soil from off the west coast of India. From the stress-strain characteristics, A-factor at failure and ratio of undrained shear strength to effective overburden pressure, they concluded that the soil is normally consolidated.

Yoshitada et al. (1991) studied the effects of saturation on the strength parameters of partly saturated soils. From a series of triaxial compression tests, they concluded that both the cohesion and angle of internal friction tend to decrease significantly with increasing saturation ratio.

2.8 Shear Strength of Treated Clays

The effect of additives on soil, like lime, dispersing agents and flocculating agents have been studied by Lambe (1960), Nagaraj (1964), Ranganathan (1965). It has been found in gener l ... lime and flocculating agents increase the undrained shear strength whereas the dispersing agents reduce the strength. Lambe (1960) showed the effect of additives like lime on the effective stress parameters viz., the cohesion C' and angle of internal friction Ø'. He found that the addition of lime causes aggregation thereby increasing the angle of internal friction.

Schmertmann and Osterberg (1960) presented results on the shear strength characteristics of Edgar Kaolinite and Boston Blue Clay with and without treatment. Their results did not show consistent trend on the influence of the dispersing agent.

Ranganathan (1965) through his studies on the effect of different chemical environment on compacted samples of kaolinite showed that the angle of internal friction, $\text{Ø}_d'$ increases on treatment with lime and flocculating agents and decreases on treatment with dispersing agents.

Wissa et al. (1965) showed that with lime treatment cohesion C' is a function of the curing time while there is no effect on Ø'. The value of Ø (ultimate) increases which may be due to the lime cementing smaller particles into larger ones.

Ingles and Metcalf (1972) on experiments with lime treated clays found that the addition of upto 3% lime would modify silty clays, heavy clays and very heavy clays. They have reported that the strength of lime soil mixture is influenced by several factors such as soil type, type of lime and amount added, curing time, moisture content and time elapsed between mixing and compaction.

The reactivity of Japanese Marine soils to lime treatment is quite high as evaluated by unconfined compressive strength (Terashi et al. 1977). The residual strength is very low and the strain at failure is small, indicating the brittle nature of the treated Kawasaki marine clay. The treated marine clay behaves like an oversolidated soil due to the development of shear strength by bonding.

The shear strength of lime treated clays is related to their moisture contents, decreasing with increasing moisture contents (Holm, 1979). He found that the value of Young's modulus is increased by 15 times on 3 weeks curing and around 35 times on 16 months curing.

Soil mixed with low lime content attains maximum strength in less time than that to which a higher content of lime has been added, strength does not increase linearly with lime content and in fact, excessive addition of lime may reduce strength (Bell, 1988). The optimum lime content tends to range from 4.5-8%, higher values occuring in soils with higher clay fractions.

2.9 Stabilisation of Clays - Field Applications

The technique called as dynamic compaction was introduced by Louis Menard in 1970. Initially, its field of application covered sandy gravel soils, it was soon found possible to extend it to saturated clays. From then onwards, the technique has taken the name 'dynamic consolidation'.

Theoretical and practical aspects of dynamic consolidation are presented by Menard and Broise (1975). It was observed that clay soils settled instantaneously several tens of centimeters due to heavy tamping and then discovered that they contained micro-bubbles of gas rendering them compressible under the effect of dynamic forces.

Wright (1973) described the lime slurry pressure injection system suitable for stabilisation of foundation beds. The method involves pumping hydrated lime slurry under pressure into soil.

The use of lime and lime-fly ash slurry injection technique, for deep in-situ stabilisation of fine grained soils is described by Joshi et al. (1981). Laboratory tests showed improvements in shear strength of soft soils on lime slurry injection. They have also presented a case history on pressure injection of subgrade soils in Illionis, U.S., to a depth of 13 m, with lime and lime-fly ash slurry.

Ruenkrairergsa et al. (1982) studied the effect of lime migration from bore holes. Holes of 15 cm dia were drilled at a spacing of 3 m. The lime powder and water were poured simultaneously into the hole. As the lime migrates, the quantity of lime in the hole will be decreased and additional lime water will be added. Natural migration plays an important role in strength development of surrounding soil.

Bredenberg (1983) reported the installation of lime columns for improvement of soft clay at new cargo terminal in Stockholm. About 47,000 m lime columns were installed which reduced the long term settlements and improved the stability during construction stage.

The strength of the lime columns can be improved by the addition of gypsum with lime (Holm et al. 1983a). They found that by using 25% gypsum along with lime, the shear strength increased 3 times as compared to clay stabilised by lime alone, after one month curing.

The performance of lime columns were compared with that of sand drains by Holm et al. (1983b). They have conducted full scale tests under embankments using both lime columns and sand drains. A comparison showed a great shear strength increase thereby increasing the stability of the embankment was achieved through lime columns. The reduction in settlement was about 60 per cent as compared to sand drains.

Samayazulu et al. (1984) carried out detailed model studies on lime columns in test tanks with piles of 50 mm diameter and 300 mm length, in soft clay at a moisture content close to liquid limit. The results indicated that lime column foundations can tackle several problems in clays.

Lime piles offer an effective and inexpensive method of soil stabilisation (Bell, 1988). Lime piles can be installed in saturated soils by means of a special metal tube with a closed tip. The tube is then withdrawn from the soil and the hole is filled with quick lime and is packed by tamping.

Many mechanical machines are developed in developed countries, capable of spreading and mixing to a depth of around 50 cm. These are suitable for subgrade and sub-base stabilisation, stabilising embankments etc. (Bell, 1988).

Yamanouchi et al. (1978) have described the technique of lime stabilisation of soft clays adopted by a machine developed in Japan. This machine is capable of mixing soft clay ground surface upto 1.3 m depth while travelling on the surface at a speed of 50 to 200 m per hour. In order to facilitate travel on soft ground, the machine is given a caterpillar made of wooden planks for lowering the contact pressure to 0.1 kg/cm^2.

Kawasaki et al. (1981) reported a deep mixing method with cement slurry as hardening agent using a deep mixing machine, specially developed for this purpose. There was considerable improvement in shear strength and reduction in compressibility characteristics after the treatment.

Chapter III

MATERIALS AND METHODS

The aim of the present study has been to investigate the strength and compressibility behaviour of untreated and treated Cochin Marine Clays. For this purpose, soil samples--both undisturbed and remoulded were collected from different locations spread over the Greater Cochin area. Marine clay samples were also collected from Karwar, Mangalore and Kuttanad for some selected tests to have a comparison of their behaviour with that of the Cochin Marine Clays.

3.1 Locations from where samples were collected

Marine clay samples were collected from different locations spread over the Greater Cochin area, most of them from the backwaters or very close to it. The various locations are:

1. Parur
2. Maradu
3. Nettoor
4. Willingdon Island

While Parur is located near the northern boundary of the Greater Cochin area (about 35 km from Cochin air port), Maradu and Nettoor are about 12 km and 10 km east of the Cochin air port, respectively. Samples were also collected from the backwaters (close to Willingdon Island) which is hardly 1 km from Cochin air port.

Marine clay samples from sea bed were collected from Mangalore about 1.5 km off the sea shore. The site is near Padubidri, at a distance of 30 km from Mangalore railway station. Samples of Kuttanad clay were collected from Kidengara and that of Karwar were collected from a site near Kalinadi bridge.

To compare certain behaviour of marine clays with other clays, tests were also carried out on black cotton soil and kaolinite. The black cotton soil was obtained from Hubli district of Karnataka and the Kaolinite used was a commercially available one, also from Karnataka.

The physical properties of the various soils used in this investigation are given in Table 3.1.

3.2 Collection and preservation of soil samples

Considerable quantities of samples were required for the numerous tests carried out on soil samples collected from the above mentioned locations. The procedures adopted for collection of samples is described in detail below.

Table 3.1

Typical physical properties of the soils used

Sl.No.	Type of Clay	Location	Depth (m)	Natural water content (%)	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)	Shrinkage limit (%)	Free swell index (cc/g)	Grain size distribution		
										Clay size (%)	Silt size (%)	Sand size (%)
1.	Cochin Marine Clay	Parur	6.0	96	129	53.5	75.5	20.0	5.10	47	38	15
2.	-do-	Nettoor	2.0	124	138	48.5	89.5	17.8	5.20	48	31	21
3.	-do-	Maradu	9.0	77	137	55.0	82.0	21.8	6.50	50	40	10
4.	-do-	Wellingdon Island	11.0	108	145	54.0	91.0	20.5	5.35	50	32	18
5.	Kuttanad Marine Clay	Kidengara	6.0	103	151	59.0	92.0	21.7	9.10	56	35	9
6.	Mangalore Marine Clay	Padubidri	4.5 m (below sea level)	89	104	41.0	63.0	24.4	6.30	55	38	7
7.	Karwar Clay	Near Kalinadi bridge	9.0	63	91	40.0	51.0	23.5	4.25	37	43	20
8.	Black Cotton Soil	Hubli, Karnataka	—	—	93	39.2	53.8	9.0	2.90	62	30	8
9.	Kaolinite	Karnataka	—	—	47	34.8	12.2	30.2	1.15	9	62	29

In the case of Cochin marine clays, the top layers in almost all locations except Nettoor consisted of sandy soils and thick layers of marine clay deposits were available only at depths varying from 3 to 9 m. At Parur and Maradu, bore holes were taken to the clay layers for collection of samples. Bore holes having a diameter of 150 mm were advanced using shell and auger method. Casing pipe with an internal diameter of 150 mm were introduced simultaneously with boring operations so that the sides did not cave in and the loose soil could be taken out with auger or shell before a standard penetration test was conducted or an undisturbed sample was collected. Thus an accurate logging of soil profile could be done. Boring operations were carried out as per the direction given in IS: 1892-1979: Code of Practice for subsurface investigation for Foundations.

Representative samples were collected from the auger and immediately transferred to polythene bags without permitting any loss of fines or pore water. The polythene bags were sealed immediately and transferred to the laboratory on the same day where they were preserved under humid conditions.

Undisturbed samples were collected in sampling tubes of 100 mm internal diameter provided with a cutting edge. The inside diameter of the cutting edge was kept 2%

less than the inside diameter of the sampling tubes so that the frictional drag on the sample from the wall of the tube is reduced. An area ratio of 7.5% was adopted for the cutting edge, which is sufficiently less than the maximum value of 10% permitted for soft and sensitive clays.

The samples collected in sampling tubes were trimmed on both ends and sealed with molten wax immediately. They were kept in sample boxes and transferred to laboratory on the same day and kept intact under humid conditions.

Incase of the site at Nettoor, thick deposits of marine clay were available at shallow depths. Therefore samples were collected by open excavation from a depth of 1 to 2 m. The samples collected were preserved in the same manner as described above.

For collection of samples from backwaters close to Willingdon Island, a specially designed barge was fabricated. A platform on wooden cross beams running across two large parallel country boats was made using wooden planks. The boring plant was erected on this platform. Sufficient number of heavy anchors were provided on all sides so that the lateral movement of the barge was completely arrested, while vertical movement due to tidal variations was permitted.

Since the top layers contained shells and sandy layers along with clay, samples were collected only between depths of 8 and 11 m for the purpose of this investigation. The samples collected were preserved in the same way as already discussed.

Samples of Kuttanad and Karwar clays were also collected by the auger and shell method from depths of 5 to 7 m and 8 to 11 m respectively, and preserved in the same manner.

Mangalore marine clay samples were obtained by using boring plant erected on a jack-up platform supported on four vertical legs with their feet firmly embedded in the sea bed. The jack-up platform had the facility to float on two large pontoons. Boring was carried out using the shell and muck barrel method. Casing pipes were introduced to protect the sides from caving in and getting mixed up with soil retrieved from bottom. The soil samples used for this investigation was collected from a depth of 4 to 6 m below the sea bed.

3.3 Additives used for stabilisation

One of the main objectives of the study was improvement of shear strength and compressibility characteristics of Cochin marine clays through stabilisation with additives. Among the twenty additives tried (Jose, 1989) for Cochin marine clays, lime and cement were found to be more effective. Hence investigations were limited to study the behaviour of these clays stabilised with lime and cement.

Specially selected uniform shells were used for preparation of lime for stabilisation. The shells were burnt to remove CO_2 completely when they change to brittle white shells of calcium oxide which were preserved in airtight multilayer polythene bags. Just sufficient water was sprinkled over the lone shells taken from these bags on each day of lime treated samples, till all the shells crumble to fine powder which was then sieved through IS 425 micron sieve. This method of preparation of lime was used because of its simplicity and ease with which it can be prepared for field application.

Ordinary Portland cement of the brand 'Malabar Cement' conforming to IS 269-1976 was used for the preparation of specimens stabilised with cement. The cement bag was kept in an air tight bin to avoid any change in properties with time of storage. The properties of the cement used in this study are given in Table 3.2.

Table 3.2

Properties of the cement used

1. Standard consistency	:	29.5%
2. Initial setting time	:	160 minutes
3. Final setting time	:	220 minutes
4. 3 day compressive strength of mortar cube	:	175 kg/cm^2
5. 7 day compressive strength of mortar cube	:	245 kg/cm^2

3.4 Preparation of soil samples

3.4.1 Untreated soil samples

It has been proved conclusively that drying of Cochin marine clay significantly affects its physical and engineering properties (Jose, 1989). Tests on air dried and overdried soils of Mangalore marine clay also gave similar results. Typical results on the effects of drying on the physical properties of Cochin and Mangalore clays are presented in Table 3.3. Hence care was taken to use the marine clays in their moist condition itself for all the tests.

The main problem faced in this case is the lack of uniformity even among the samples collected from the same depth in different bore holes. Lack of uniformity in soil

Table 3.3

Effect of drying on the physical properties of Cochin and Mangalore clays

Sl. No.	Description of the soil	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)	Shrinkage limit (%)	Free swell index (cc/g)	Grain size distribution		
							Clay size (%)	Silt size (%)	Sand size (%)
1.	Cochin Marine Clay								
	(a) moist soil	129	53.5	75.5	20.0	5.10	47	38	15
	(b) air dried	93	36.2	56.8	20.5	2.90	33	48	19
	(c) oven dried	76	33.5	42.5	21.4	1.80	24	49	27
2.	Mangalore Marine Clay								
	(a) moist soil	104	41.0	63.0	24.4	6.30	55	38	7
	(b) air dried	88	35.3	52.7	25.3	5.50	48	35	17
	(c) oven dried	73	32.5	40.5	25.0	4.0	27	42	31

samples tested may seriously affect the correlation between index and engineering properties. To overcome this difficulty, representative samples collected from same depth, but from different bore holes of the same site were pooled together and mixed thoroughly into a uniform mass and repacked in polythene bags. This soil in the moist condition was used for all the tests done with the representative samples.

For shear strength and consolidation tests on undisturbed soil, specimens were prepared by extracting the clay from the stainless steel sampling tubes of 100 mm diameter. The first 3 to 4 cm length of sample was cut off and discarded as this might have been disturbed during sealing with molten wax or during preservation, by loss of moisture content. For consolidation test specimens, the bevelled edge of the consolidation ring was then pushed into the samples being extracted from the sampling tube such that soil projects on either side of the consolidation ring. The sample was then trimmed in level with the edges of the oedometer ring. The initial water content of soil sample was determined from these trimmings. For shear strength test specimens, 3 nos. of 38 mm dia. stainless steel tubes were pushed into the soil being extracted from the 100 mm dia. sampling tubes. The soil sample from the 38 mm dia. tube is then extracted and cut to the required length (normally 75 mm), and is used for shear strength tests.

3.4.2 Treated soil samples

For consolidation tests, a portion of the thoroughly mixed moist representative sample was taken and its moisture content determined. The estimated amount of additive (cement or lime as a percentage of the dry weight of the soil) was then mixed with the moist soil. Water was added and mixed thoroughly till the liquid limit of the soil-lime or soil-cement mixture was reached. It was then moulded in PVC rings of 150 mm dia. and 50 mm height taking care that no air is entrapped during moulding. These PVC rings were then covered with polythene sheets. GI circular plates were placed at the top and bottom of the PVC rings, tied with rubber bands and again the assembly was covered with polythene sheets and kept in humid conditions for curing. At the end of the specified curing period, the assembly was taken out and the polythene sheets and the GI plates were removed. Treated soil specimen was extracted into the oedometer ring in the same method as described for the case of undisturbed samples.

The treated samples for shear strength tests were also prepared in the same manner. The estimated amount of additive (cement or lime) as a percentage of the dry weight of the soil, was added to the moist representative sample. Water was added and the additive was mixed with the soil thoroughly till the water content of the soil-cement or soil-lime mixture was equal to the initial water content of the soil. Specimens for vane shear tests were prepared by filling this mixture into the vane shear cups of 38 mm dia. and 75 mm height, taking care that no air was entrapped in between. These vane shear cups were then covered with polythene sheets and kept in humid conditions for curing. Samples for unconfined compression and triaxial shear tests were prepared by moulding the soil-cement or soil-lime mixture into split mould of dia 38 mm and height 175 mm. The samples were taken out immediately after moulding, covered with polythene sheets and kept humid conditions for curing.

3.4.3 Compacted soil samples

The significant improvement in compressibility characteristics of the Cochin marine clays on air drying has already been brought out (Jose, 1989). The effects of drying on the shear strength behaviour of these clays has been studied through a series of triaxial shear tests. Compaction tests on air dried samples with compactive efforts corresponding to Standard Proctor test and Modified Proctor test gave the values of maximum dry density and optimum moisture content for the samples prepared for triaxial shear tests.

Samples were prepared by two methods--ie., the wetting process and by the drying process. In the wetting process, the moist marine clay was allowed to air dry completely. It was powdered well and sufficient water was added to reach the optimum moisture content. Then it was compacted in layers to achieve the dry density corresponding to that obtained from the Proctor test. In the drying process, the moist marine clay was allowed to dry only upto a moisture content corresponding to the optimum moisture content obtained from the Proctor test. This partially dried sample is then compacted in layers to achieve the same dry density as in the wetting process.

3.5 Test Procedures adopted for determination of various parameters

3.5.1 Atterberg limits

The liquid limit and plastic limit were determined as per IS 2720 (Part 5) - 1985. Since drying significantly reduced the Atterberg limits of marine clays (Table 3.3), tests for liquid limit and plastic limit were done on the moist soil itself. The liquid limit tests were conducted using Casagrande's apparatus, starting from a water content which required around 10 blows only for the groove to close. The paste was then spread over glass plate to allow evaporation. This was then mixed thoroughly for the next test. Thus all tests were started on the wetter side of liquid limit.

3.5.2 Grain size distribution

Grain size distribution is the most obviously affected index property of marine clays during drying. Due to aggregation, a portion of the clay fraction is changed to silt size and a portion of the silt size becomes sand size (Figures 3.1 and 3.2). Hence grain size distribution tests on dried samples give anomalous results. Eventhough IS 2720 (Part 4) - 1985 recommends the use of soil oven dried at 105-110°C for sedimentation analysis, because of the reason mentioned above, only moist samples were used for the sedimentation analysis in this study.

The sedimentation analysis was done using hydrometer. Compared to the normally encountered clayey soils, the need of a deflocculating agent was more keenly felt for marine soils (Jose, 1989; Sridharan et al. 1991). Hence the standard dispersing agent recommended by the IS code (sodium hexametaphosphate + sodium carbonate) and proved to be the most ideal one for Cochin marine clays, was used for the hydrometer analysis.

3.5.3 Free swell index test

IS 2720 (Part 40): 1977 defines the free swell index as the increase in the volume of a soil, without any external constraints, on submergence in water. As per the code, the free swell index is calculated by,

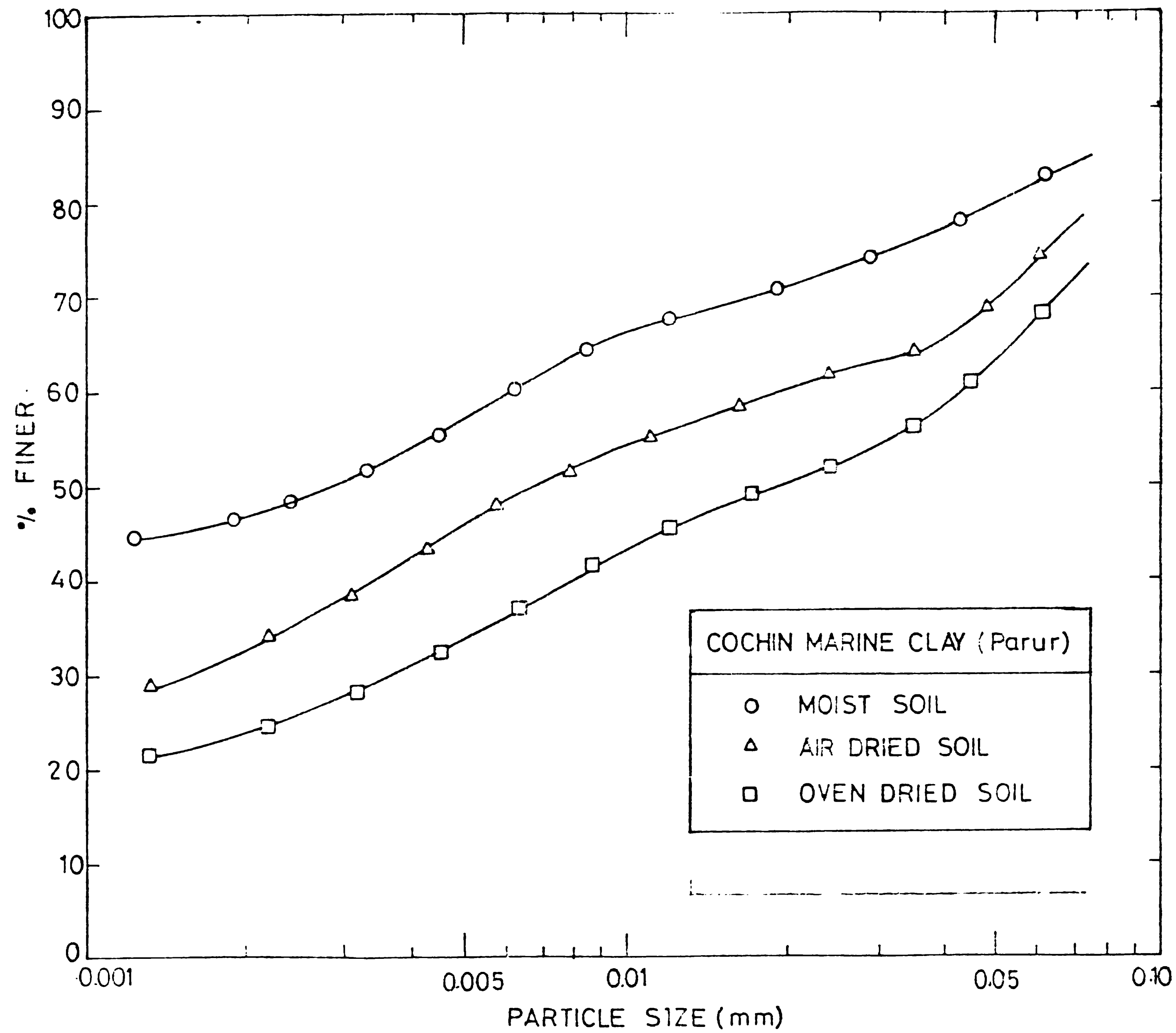

Fig. 3.1 TYPICAL GRAIN SIZE DISTRIBUTION CURVES - COCHIN CLAY

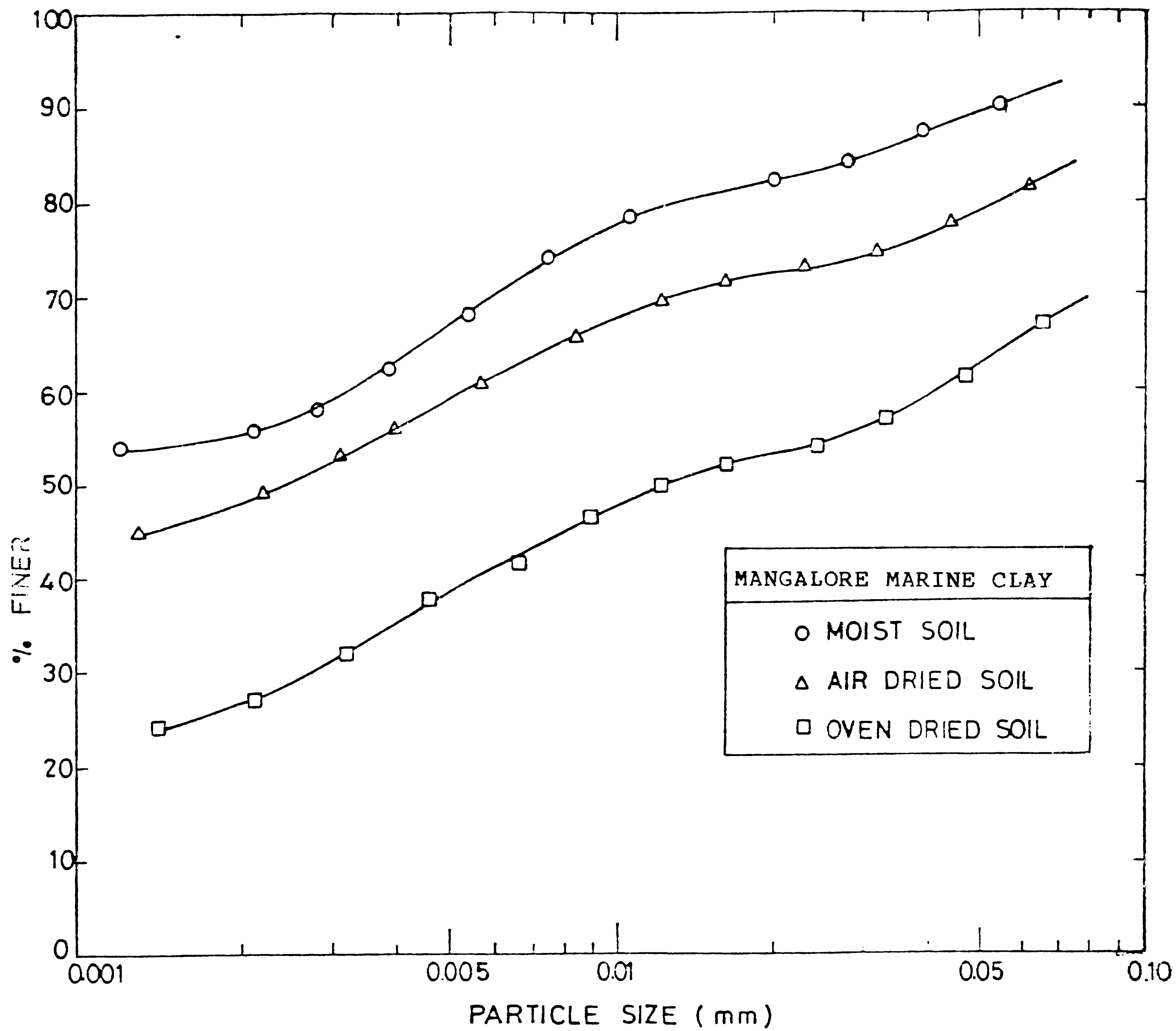

Fig.3.2 TYPICAL GRAIN SIZE DISTRIBUTION CURVES - MANGALORE CLAY

$$\text{Free swell index} = \frac{V_d - V_k}{V_k} \times 100$$

where V_d - volume of 10 g of oven dry soil passing through 425 micron sieve, in distilled water

V_k - volume of 10 g of oven dry soil passing through 425 micron sieve, in kerosene.

This method of determination of free swell index has some inherent limitations as pointed out by Sridharan et al. (1985). They proved that pure kaolinite mineral occupies a higher sediment volume in a non-polar solvent like kerosene than in water thus resulting in negative values of free swell index. Hence the method proposed by Sridharan et al. was used for the determination of free swell index, in this study. According to them,

$$\text{Free swell index} = \frac{V_d}{10} \text{ cc/g}$$

where V_d - volume of 10 g of soil specimen read from the 100 ml graduated cylinder containing distilled water

This equation defines free swell index as the volume occupied by a unit weight of soil in water without any external constraint.

Since drying is inevitably accompanied by aggregation marine clays, use of dry soil specimen as stipulated by IS code will lead to erroneous results. Hence for the determination of free swell index of marine clays, only a known quantity of moist soil can be transferred to the jar. The dry weight of the soil specimen can later be defined by finding out the moisture content of the moist sample used for the test.

3.5.4 Laboratory vane shear test

To study the improvement in shear strength of Cochin marine clays, when stabilised with lime, laboratory vane shear tests were the only solution, during initial periods of curing. These were done as per IS 2720 (Part 30): 1980. The vane used was having an overall diameter of 12 mm and a height of 24 mm. The vanes were gradually lowered into the cup containing the soil, without disturbing the soil specimen, so that the top of the vane is at least 10 mm below the top of the specimen. The vane was kept in that position for a few seconds, in order to allow the dissipation of pore pressure developed, if any, on insertion of the vane. Then torque was applied at a constant rate of 0.1°/sec till the specimen failed.

3.5.5 Triaxial shear tests

Triaxial shear tests were conducted on both untreated samples (undisturbed and remoulded) and on samples treated with lime and cured for different durations. The specimens were saturated by applying back pressure before the commencement of the test. Tests with all the three drainage conditions were performed.

(i) Unconsolidated undrained (UU) test - done at a strain rate of 0.24 mm/min.

(ii) Consolidated undrained test with measurement of pore water pressure ($\overline{CU}$) - done at a strain rate of 0.048 mm/min.

(iii) Consolidated drained test with measurement of volume change (CD) - done at a strain rate of 0.0096 mm/min.

The chamber pressure was applied through the constant pressure mercury compensating system and the pore pressure measurements were done with the help of a transducer and digital indiçator.

3.5.6 Consolidation tests

Several series of consolidation tests were carried out on undisturbed, remoulded and treated marine clays. The sample was pushed into the oedometer ring of 60 mm dia and 20 mm height, and trimmed in level with the edges of the oedometer ring as described in the earlier section. The ring is then introduced into the consolidation test assembly with porous stones on either side. Filter papers were placed between the porous stones and the soil specimen to prevent the soil from being forced into the pores of the stones. The consolidation assembly was then positioned in the loading frame and the specimen was loaded with a seating load of 0.625 kg/cm^2. The sample was then inundated with water from a reservoir with a head of 50 cm.

The load increment ratio for the routine tests was kept as one. It was found that about two days were required for a complete dissipation of pore pressure and for reaching an equilibrium void ratio for a particular loading stage. Hence the duration for each load increment was kept at two days. A loading sequence of 0.0625, 0.125, 0.25, 0.50, 1.0, 2.0 and 4.0 kg/cm^2 was adopted for the numerous consolidation tests performed on untreated and treated specimens of marine clays.

For each loading, dial gauge readings were taken at 0, ¼, 1, 2, 3, 4, 5, 8, 11, 14, 17, 20, 30, 40, 50, 60, 80, 100, 120, 140, 160, 180, 210, 240, 270, 300, 330, 360, 420, 480, 1440, 2880 minutes from the time of loading. Normally these values are chosen such that they give a uniform spacing of points for $\sqrt{t}$ curves. But the Taylor's $\sqrt{t}$

method and Casagromde's log t method sometimes fail to estimate values of t_{90} and t_{100} respectively. Hence the Rectangular Hyperbola method proposed by Sridharan et al. (1987) which is devoid of such estimations, was used for the determination of the coefficient of consolidation.

Chapter IV

COMPRESSIBILITY CHARACTERISTICS OF COCHIN MARINE CLAYS

4.1 Introduction

Man, the builder has been facing the settlement problems associated with soft clays from the day he began to construct. The foundations of the Sumerian temples constructed over 6000 years ago on the embankments of soft soils of Euphrates valley has settled by more than 12 m. Experience taught him to choose ideal sites for his houses. But with the rapid industrialisation along with the urbanisation and the tremendous growth in the transportation network by roads and railways, he was left with no choice, but to build on the soft fine grained soils. Studies on the compressibility behaviour of soft soils and improvement of their strength characteristics thus received the attention of engineers especially from the last century.

According to Skempton (1960) the first practicing engineer who was aware of the consolidation process was Telford who in 1809 preloaded a 17 m thick soft clay bed before constructing a sea lock. English Geologist Lyell (1871) found that water was squeezed out of clay under load causing compaction. Smith (1892) discovered for buildings founded on Chicago clay that 'slow progressive settlements result from squeezing out of water from earth'. Perhaps the first field data on consolidation was published by Shankland (1896) who published time-settlement curves for the Masonic temple in Chicago for a period of $4\frac{1}{2}$ years.

Attention was first drawn to the problem of long term consolidation of clay by Terzaghi (1925) with the publication of 'Erdbaumechanik'. He also designed the first consolidation apparatus which he named as an oedometer from the Greek word Oidema meaning swelling. Eversince consolidation has received the best attention from research workers.

An attempt has been made here to study certain aspects of the consolidation characteristics of Cochin marine clays. A comparative study of the available methods to determine the preconsolidation pressure has been undertaken. In addition, a new method which is simple and fast has been proposed. How the poor strength characteristics of soft Cochin marine clays can be improved by precompression has been discussed in the subsequent section. The improvement of compressibility characteristics on treatment with lime and cement has also been taken up in the present investigation.

4.2 Estimation of preconsolidation pressure

The simplest definition of preconsolidation pressure, perhaps, is the pressure at which a clay deposit has been fully consolidated (Casagrande, 1936). But there are innumerable parameters as discussed earlier in section 2.4, influencing the p_c value and making the definition much more involved. For example, according to Ladd (1991), the preconsolidation pressure p_c is the yield stress that separates small strain elastic behaviour from large strains accompanied by plastic deformation during one dimensional consolidation.

Studies on consolidation perhaps marked the birth of the art and sciences of soil mechanics. Still, inspite of extensive investigations, a unique definition for the value of p_c has remained elusive for the research workers. Similarly, eventhough Casagrande's method has been the most widely used technique for determination of p_c, its reliability and acceptability have been questioned by many workers (eg. Jose et al. 1989). Eventhough several methods have been proposed for estimation of p_c, over the last two or three decades, (Burmister, 1951; Schmertmann, 1955; Sallfors, 1975; Butterfield, 1979; Holts and Kovacs, 1981; Karunaratu et al. 1983; Koerner et al. 1984; Nagaraj and Srinivasa Murthy, 1985; Becker et al. 1987 etc.) the superiority of one over the other or selection of the most accurate and reliable method has remained an unattainable aim as the exact value of p_c was not readily available for purposes of comparison.

Attempts have been made to develop methods for determination of preconsolidation pressure making use of the data generated from a large number of consolidation tests. A unique feature of the work presented below is the preparation of the samples consolidated at predetermined pressures and comparison of these, with results from the new techniques proposed.

A notable feature of the present work has been, preparation of clay specimens for consolidation tests for which we had a preknowledge of the preconsolidation pressure. Samples were consolidated in oedometer rings to predetermined pressures in consolidation test apparatus. The load was then released to a low seating load. On these preconsolidated samples, consolidation tests were carried out and p_c values were estimated and compared with the predetermined consolidation pressures. Since a comparison with the actual p_c values was possible, the usefulness of the different methods already available in literature could be studied critically.

4.2.1 Current methods for determination of p_c

The chapter on review of past work (section 2.4) gives a description of the important methods suggested by various workers. Among these, those which were found to be popular among recent investigators in this area are the methods suggested by Burmister (1951), Schmertmann (1955), Becker et al. (1987) and Jose et al. (1989) in addition to the time tested Casagrande method.

While in the Casagrande method, one has to select the point at which the radius of curvature is minimum and make a geometric construction on the e - log p curve, the method suggested by Burmister is a trial and error procedure using a characteristic triangle.

Schmertmann also employs the same method of rebounding and reloading. A probable value of p_c is assumed and the procedure again involves a trial and error method for determination of p_c.

Compared to the above two methods, the procedure suggested by Becker has greater clarity at least procedure-wise. Instead of e - log p curve, Becker suggests a plot between work done per unit volume and effective stress, both on arithmetic scales. According to him, the plot results in two linear segments, the intersection of which gives the value for p_c.

Jose et al. (1989) proposed a new method called log-log method which consists of plotting both the pressure, p and the corresponding void ratio, e in logarithmic scale. The initial and final sets of points on the log e - log p plots result in two distinct straight lines, the intersection of which gives the p_c value. The validity and accuracy of the method suggested could be ascertained through a series of consolidation tests on clay specimens for which a preknowledge of the p_c value was available. In order to have a preknowledge of the preconsolidation pressure, specimens were loaded to certain selected sustained pressures, unloaded to seating pressure and reloaded to the full extent. A comparative study of the results showed that while the results by the Casagrande method were higher than the actual values by 15-50% the log-log method gave results where the variation ranged over 2-20%.

4.2.2 Reliability of the experimental procedure

A discussion on the merits and demerits of the various methods and suggestions for selection of the most suitable one becomes relevant only due to the availability of a preknowledge of the preconsolidation pressure for comparison. Jose et al. (1989) had discussed the procedure for preparing samples with known p_c values. It was again verified by running a series of consolidation tests on different types of soils, consolidated at different pressures.

Fig.4.2.1 shows the e - log p curves for three soils viz., Black cotton soil, Cochin marine clay from Parur site and Kaolinite, consolidated at a pressure of 100 k Pa. The samples were then unloaded to a seating load of 6.25 k Pa and then a full scale consolidation test was carried out. Immediately before and after the sustained pressure of 100 k Pa, the load increments were kept low to obtain a large number of points around p_c and a more reliable e - log p curve is available for determination of p_c.

Similarly Fig.4.2.2 shows the e - log p curves for Cochin marine clay (Nettoor site) and black cotton soil, consolidated at 200 k Pa for a period of 15 days. Fig.4.2.3 shows e - log p curves for a pressure of 100 k Pa sustained for 2 and 7 days. In all the above cases, it is obvious from the plots that the preconsolidation pressure values are either same or very close to the sustained pressures.

Fig.4.2.4 shows the e - log p curves for samples consolidated at 100 k Pa for periods of 30 days and 90 days. The reloaded portion and the subsequent virgin curve indicates that the p_c values obtained from these curves will be higher than the sustained pressure of 100 k Pa. This is due to the development of quasi preconsolidation pressure developed during the longer periods allowed for consolidation

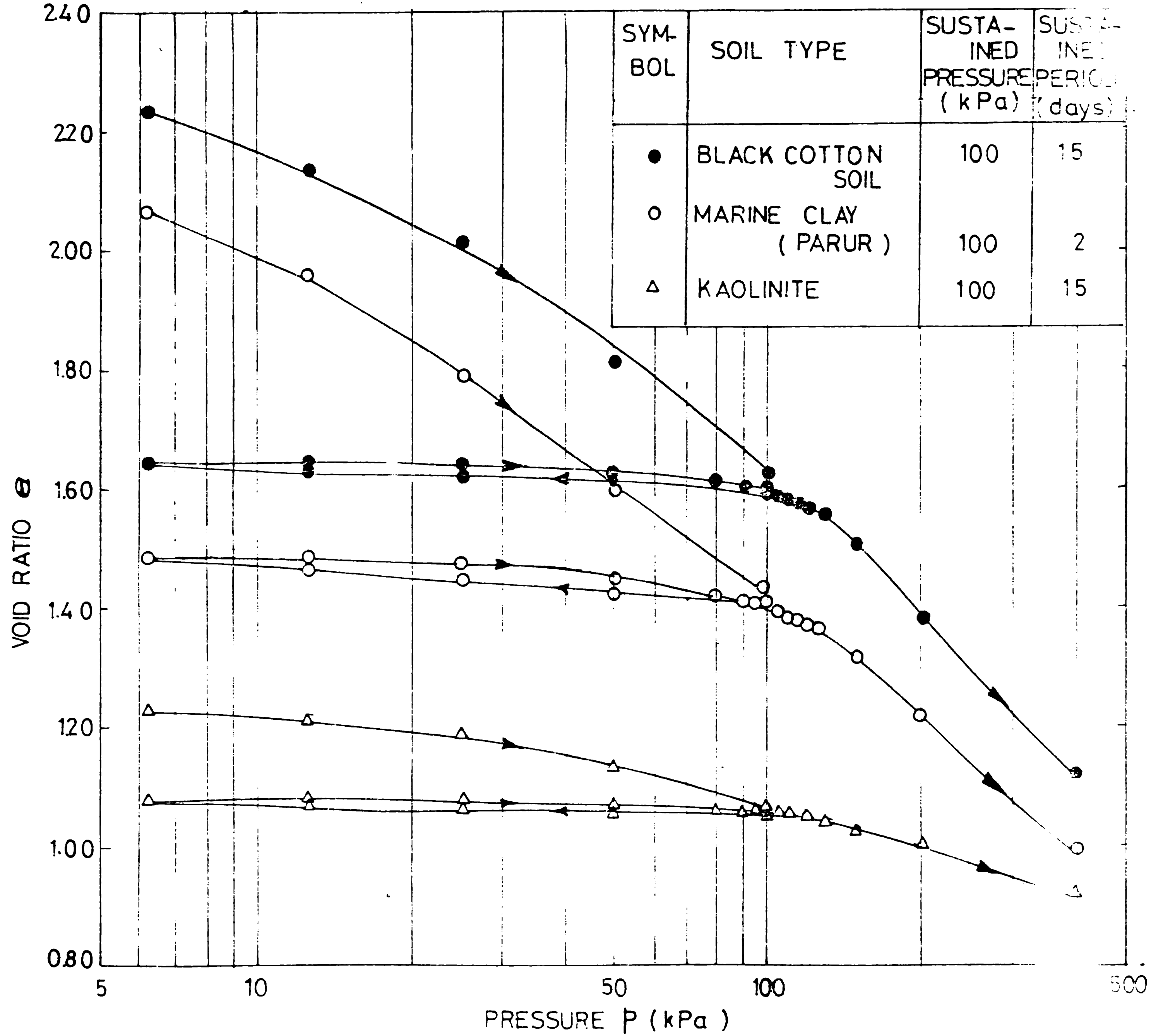

Fig.4.2.1 TYPICAL e-log p CURVES

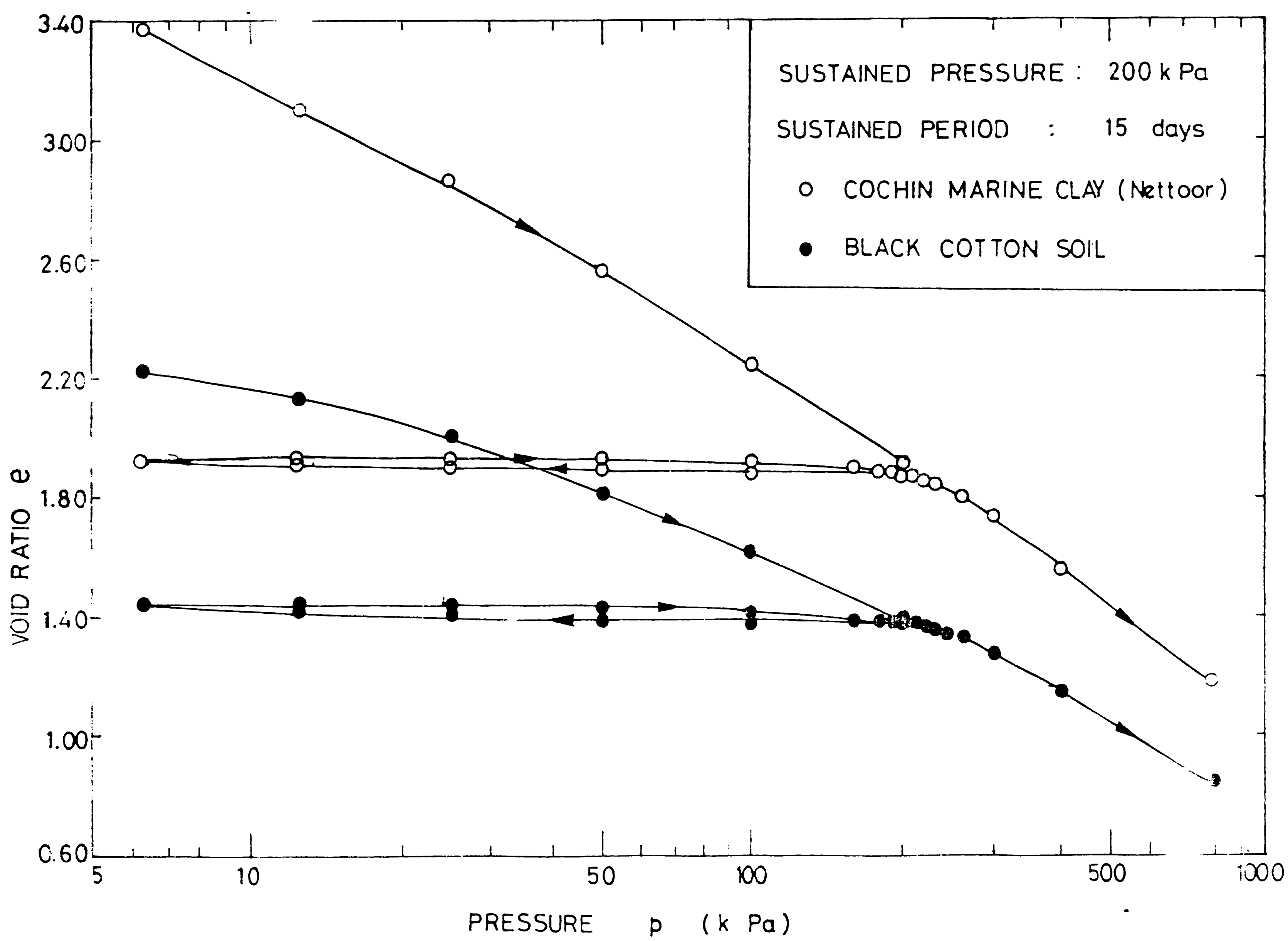

Fig.4.2.2 e-log p CURVES FOR A SUSTAINED PRESSURE OF 200 k Pa

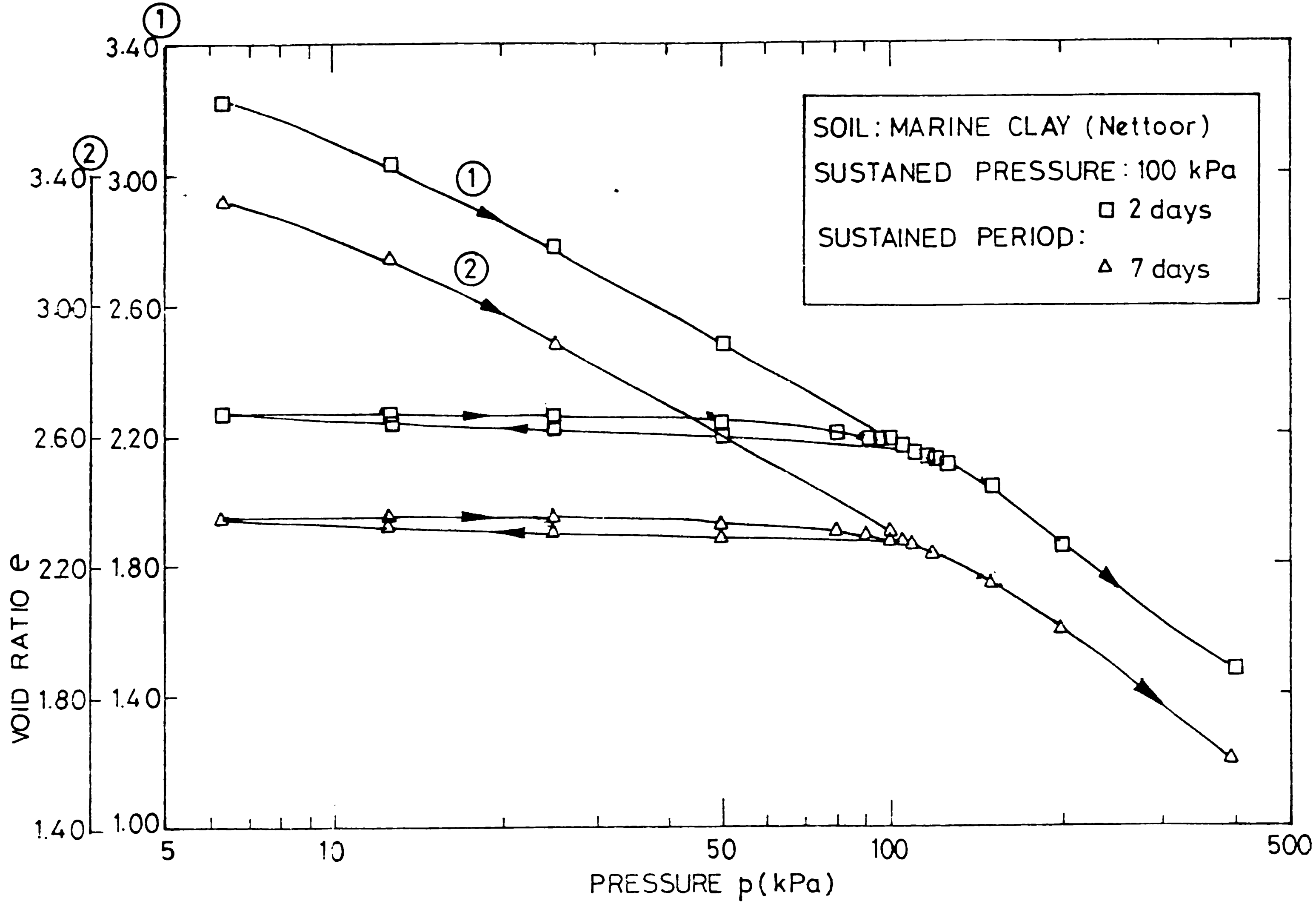

Fig.4.2.3 e-log p CURVES FOR MARINE CLAY PRECONSOLIDATED AT 100 k Pa

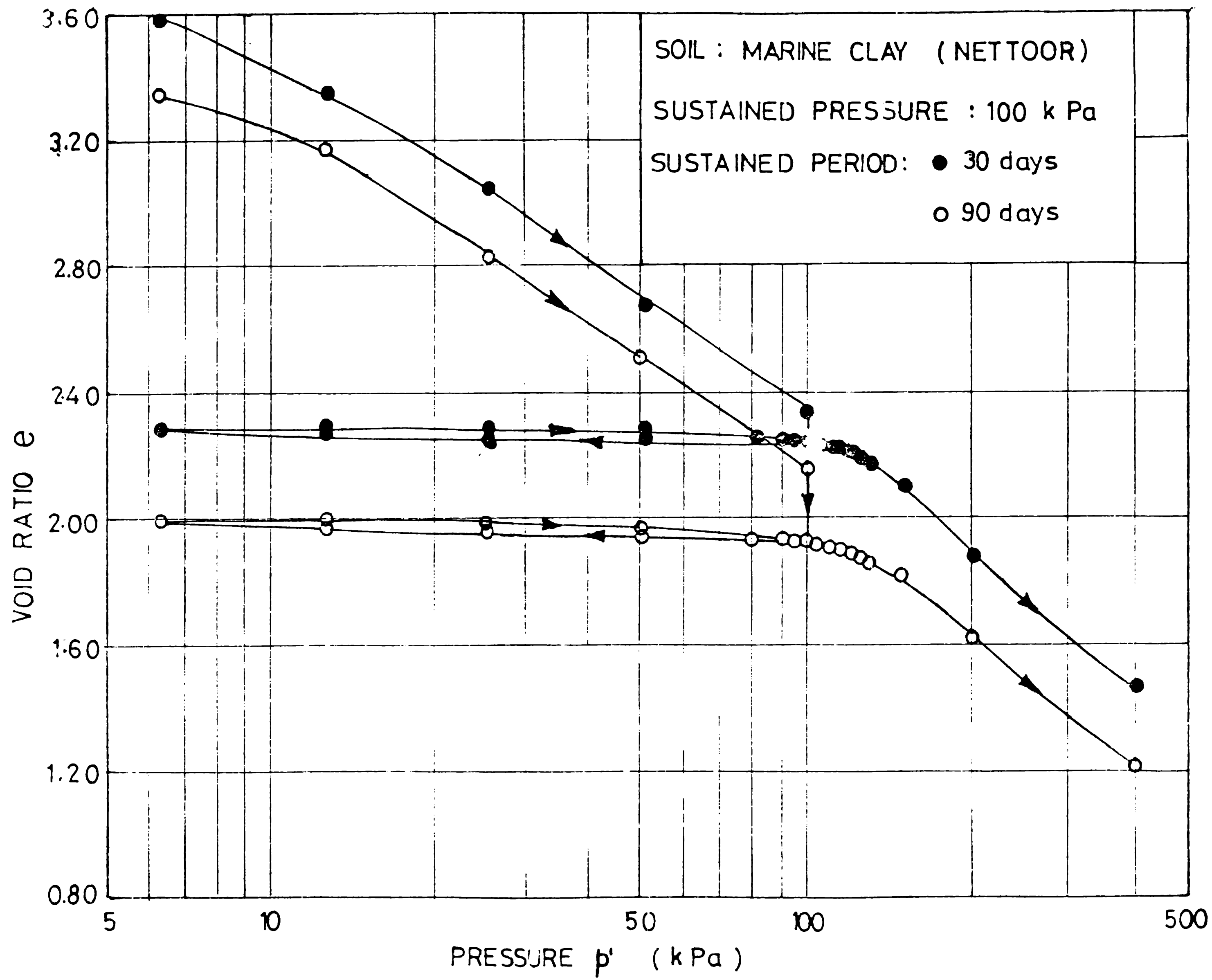

Fig.4.2.4 e-log p CURVES FOR MARINE CLAY WITH SUSTAINED LOAD FOR LONGER DURATIONS

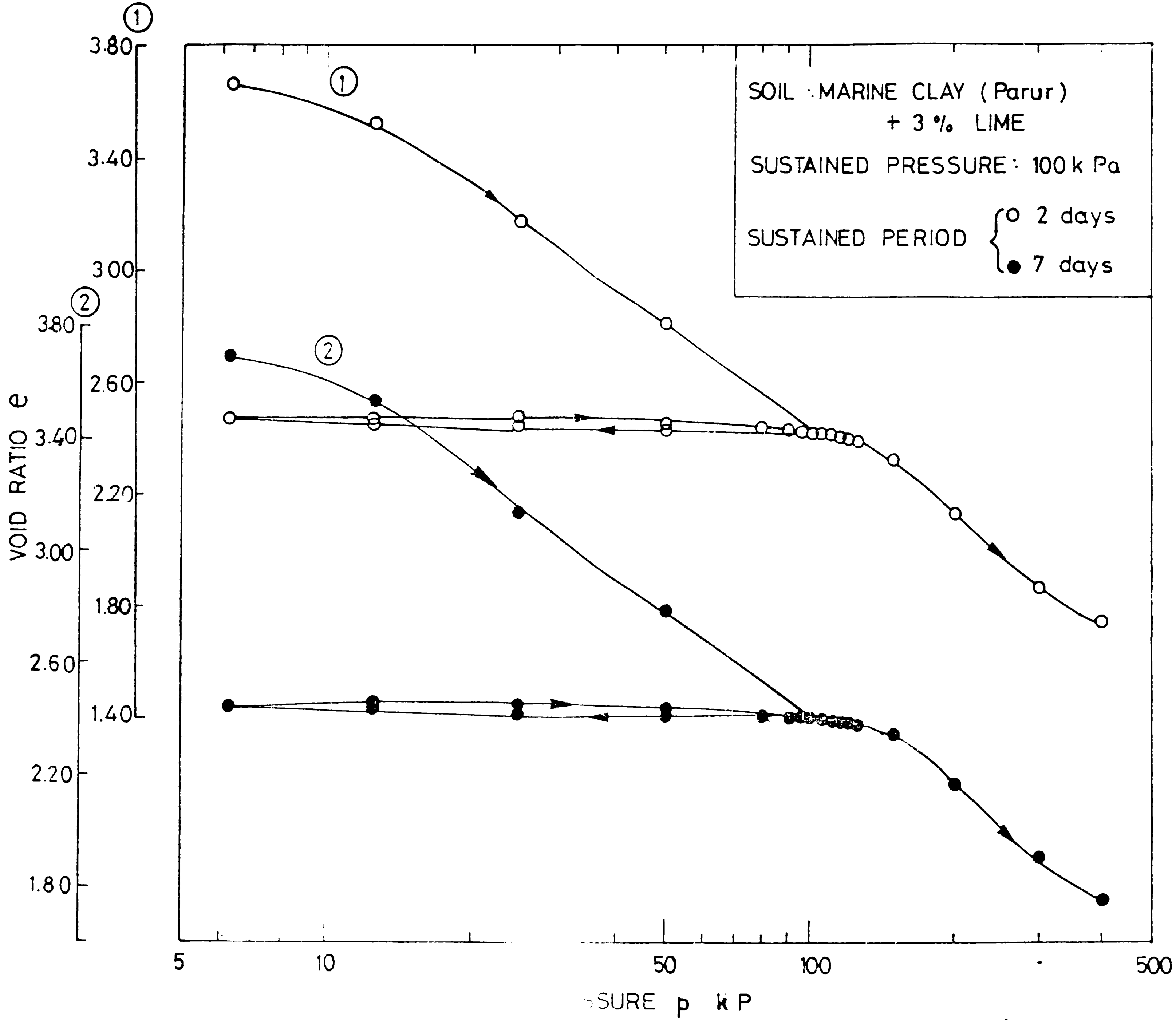

Fig.4.2.5 TYPICAL e-log RVES FOR LIME TREATED MARINE CLAY

under 100 k Pa (Leonards and Ramiah, 1959; Bjerrum, 1967). The gradual development of quasi preconsolidation pressure is clearly brought out by a comparative study of the four e - log p curves in figures 4.2.3 and 4.2.4 wherein the consolidation period increases from 2 to 90 days.

Fig.4.2.5 shows typical e - log p curves for Cochin marine clay (Parur site) which was treated with 3% lime. No curing period was allowed for the oedometer specimens as the tests were carried out on samples immediately after mixing the lime. The additional resistance to compression developed due to lime stabilisation and consequent development of bonds is clearly broughtout by the e - log p curves.

Thus the figures 4.2.1 to 4.2.5 clearly show that the method developed for preparation of samples is quite reliable as the e - log p curves are able to bringout all the aspects of the compressibility characteristics of preconsolidated fine grained soils.

4.2.3 Application of the new experimental procedure to existing methods

In the review of literature and in section 4.2.1, a brief note on the popular methods was given. They are:

1. Casagrande method (1936)
2. Burmister method (1951)

3. Schmertmann method (1955)
4. Becker method (1987)
5. log-log method (1989)

Consolidation tests were conducted on samples of Cochin marine clay (Nettoor site) which were preconsolidated at 100 k Pa sustained for periods of 2, 7, 30 and 90 days. The results of the routine consolidation tests performed on these specimens are shown in Figs.4.2.6 and 4.2.7. The Casagrande geometric construction gave p_c values of 130 k Pa eventhough the samples were consolidated at 100 k Pa for periods ranging over 2 to 90 days. This indicates that the Casagrande method has failed to bringout the development of quasi-preconsolidation pressure as indicated by Leonards and Ramiah, 1959; Bjerrum, 1967 etc.)

The methods suggested by Burmister and Schmertmann are rarely used by research workers or practising engineers due to many factors affecting the efficacy of the procedures as explained in section 4.2.1. Hence they have not been taken up for detailed investigation.

The work per unit volume method suggested by Becker et al. has the advantage that the results are plotted in linear scales for pressure and work per unit volume. The results of the same series of four consolidation tests are

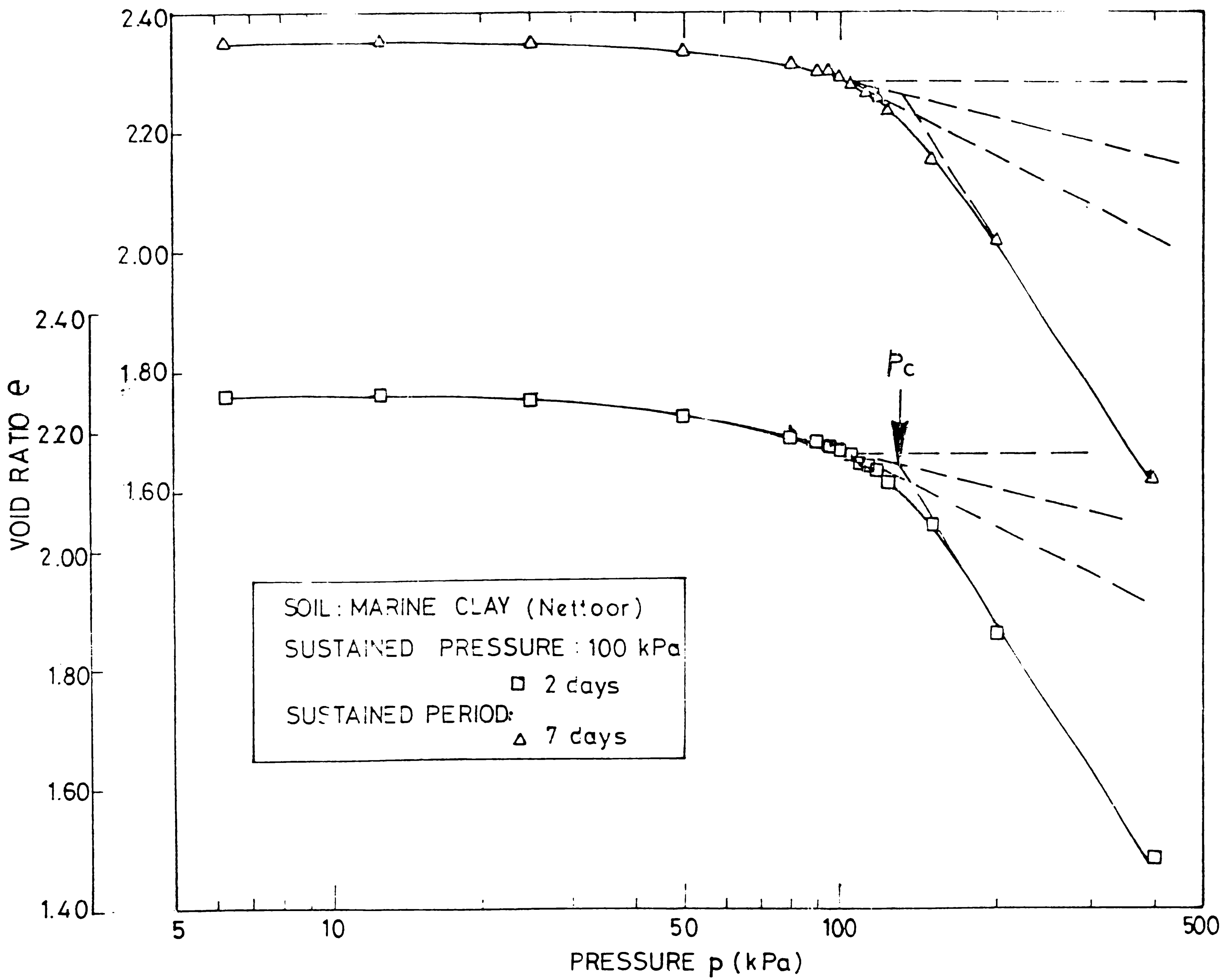

Fig.4.2.6 PRECONSOLIDATION PRESSURE FROM CASAGRANDE METHOD

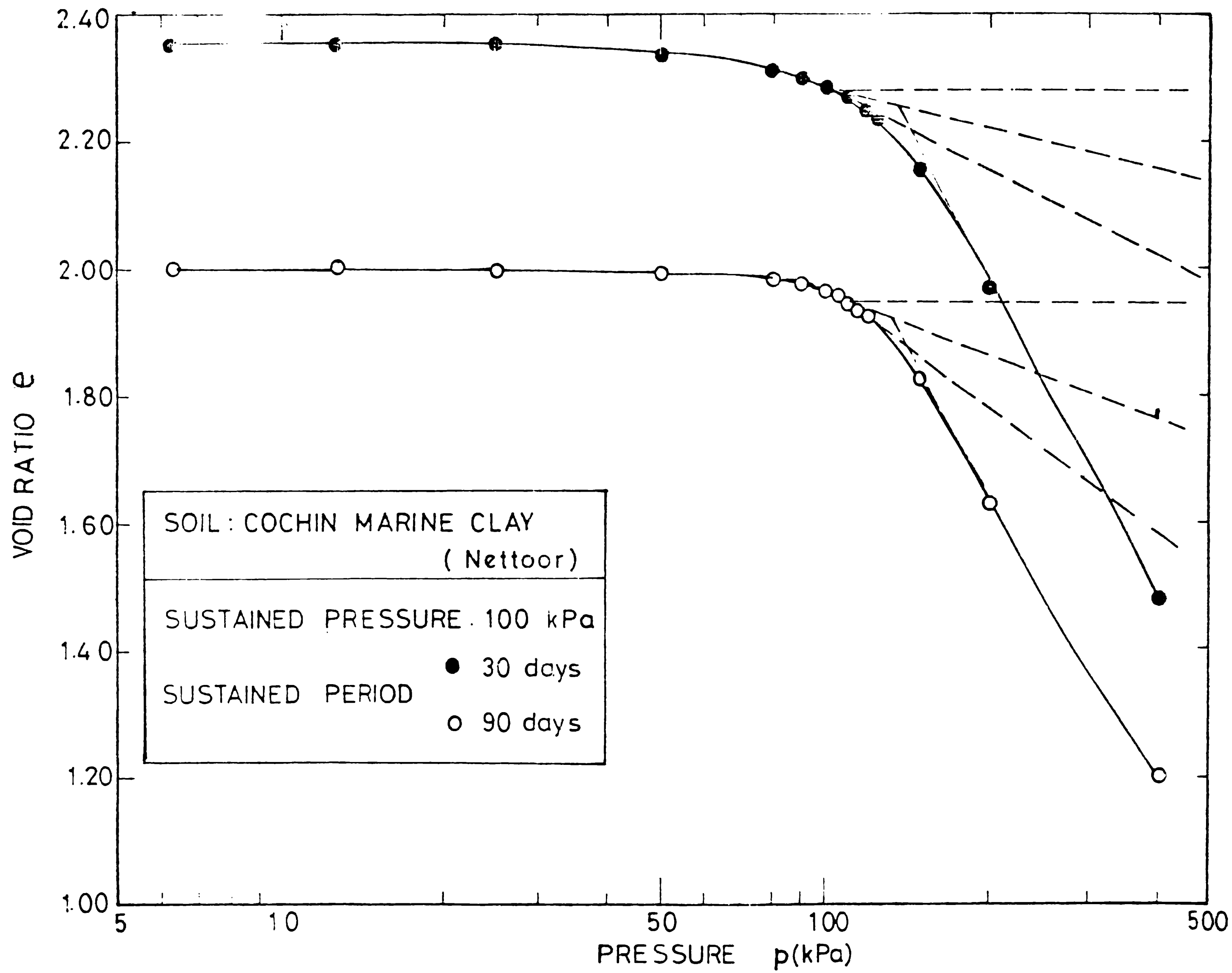

Fig.4.2.7 PRECONSOLIDATION PRESSURE FROM CASAGRANDE METHOD

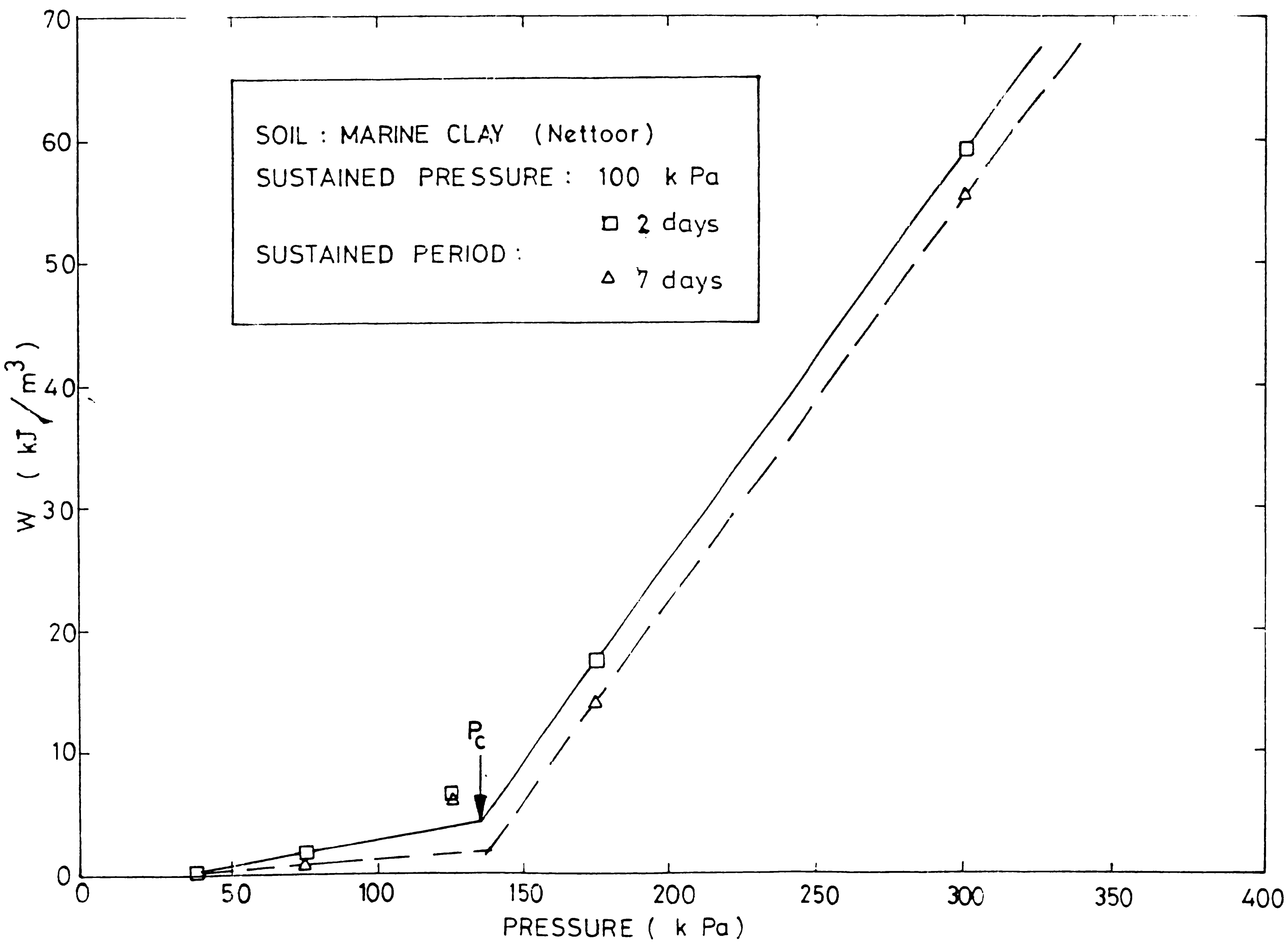

Fig.4.2.8 PRECONSOLIDATION PRESSURE FROM WORK PER UNIT VOLUME INTERPRETATION

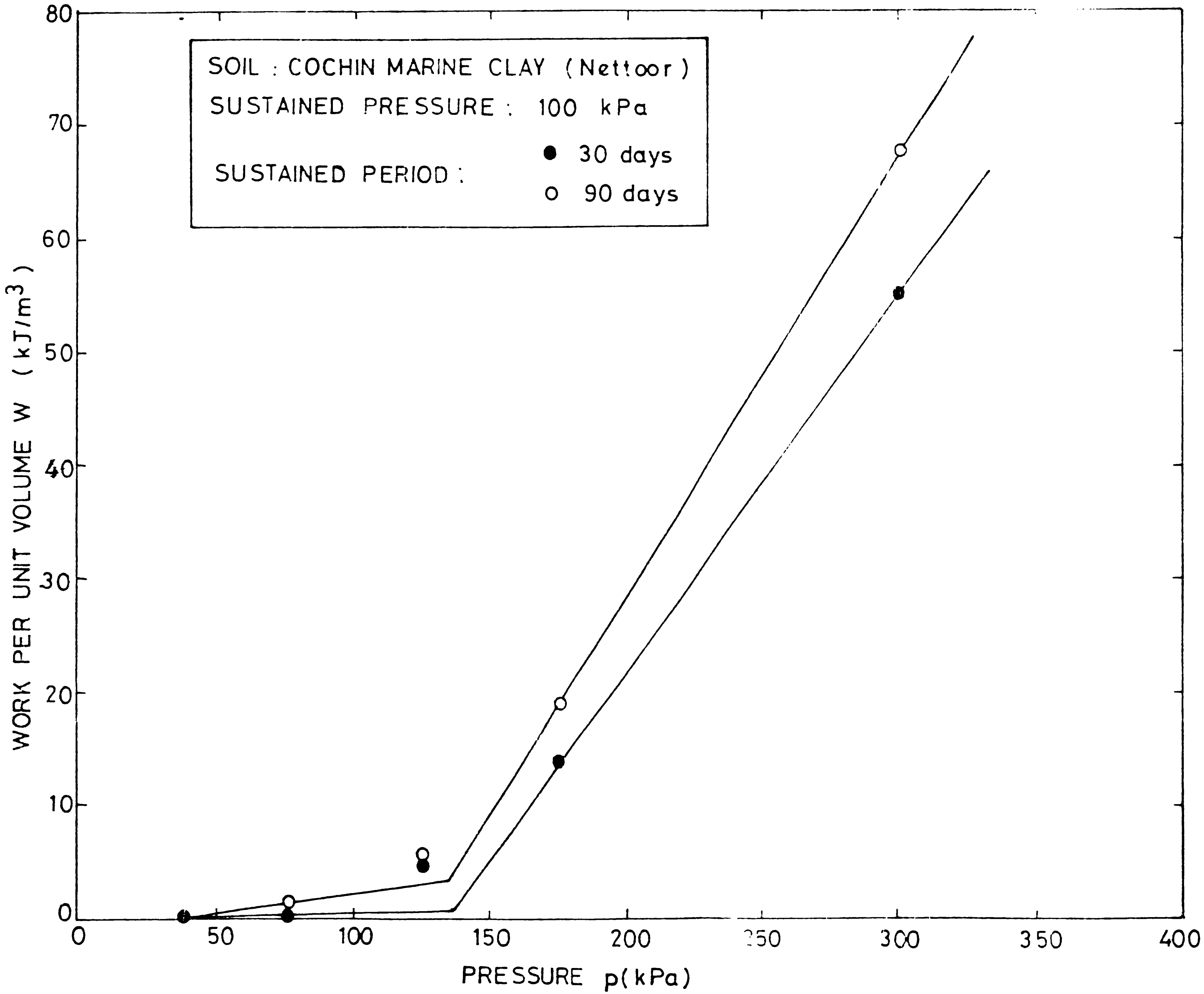

Fig.4.2.9 PRECONSOLIDATION PRESSURE FROM WORK PER UNIT VOLUME INTERPRETATION

presented in Figs.4.2.8 and 4.2.9. The intersection of lines fitted to the initial and final portions of the w-p plots gave p_c values of 135, 138, 138 and 136 k Pa respectively for sustaining periods of 2, 7, 30 and 90 days.

Eventhough the samples were consolidated at 100 k Pa, the p_c values given by Becker's method are varying by 35 to 38% from the actual p_c value. The development of quasi-preconsolidation pressure is also not reflected in the results. Further, the calculation of work per unit volume is involved. Thus the Becker's method cannot claim superiority over the Casagrande method.

The log-log method proposed by Jose et al. (1989) claims several advantages over the existing procedures. In this method, both the void ratio and consolidation pressure and plotted to logarithmic scale. Straight lines fitted to the initial and final portions of the log e - log p curves intersect at the p_c values.

Fig.4.2.10 and 4.2.11 show results of the earlier four consolidation tests - Cochin marine clay (Nettoor site) consolidated at 100 k Pa for 2, 7, 30 and 90 days - plotted as log e - log p curves. Straight lines fitted to the initial and final sets of points intersect at p_c values of

108, 115, 119 and 122 k Pa, for sustaining periods of 2, 7, 30 and 90 days respectively. Compared to the results from earlier methods, these values are consistently closer to the preconsolidation pressure applied. In addition, the p_c values steadily increases along with the sustaining periods, bringing out the effect of quasi-preconsolidation pressure also.

It may be noted that the Casagrande method gave higher values than the values obtained from the log - log method. The determination of p_c is important for the prediction of settlement of structures. Since log - log method gives values closer to the actual preconsolidation pressure and lower than the values given by Casagrande procedure, this technique is safer and more conservative. Not only it is simpler than Casagrande method, but less time consuming. The need for the selection of the point of maximum curvature with the attendent errors from personal judgement is also avoided.

Since the method was found more useful, the results of consolidation tests from some more tests were plotted on log - log scale and they are presented in Fig.4.2.12 and 4.2.13. Here also it can be seen from the plots that the log - log method give p_c values close to the preconsolidation pressures irrespective of the type of soil, period or consolidation pressure.

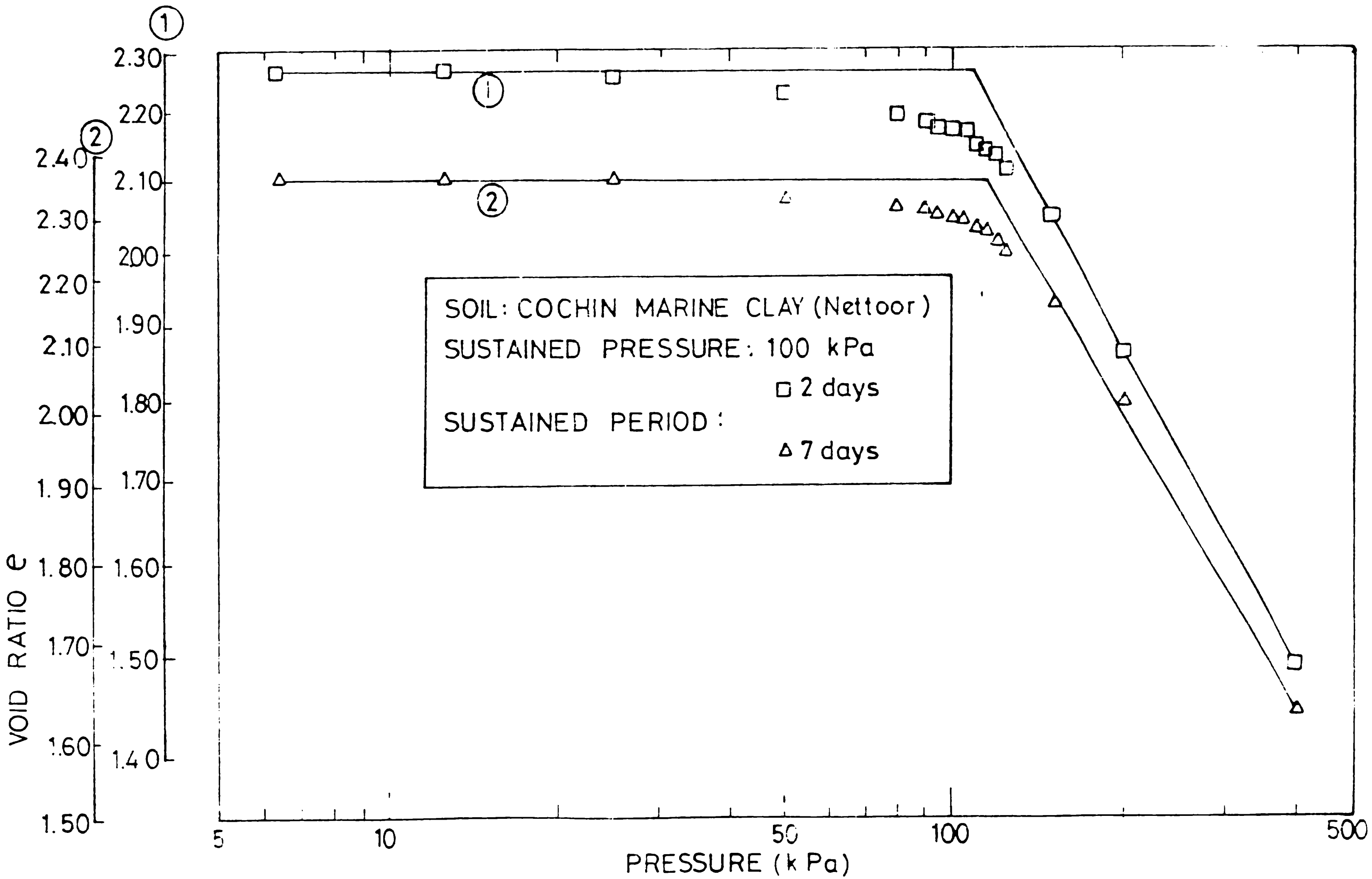

Fig.4.2.10 log e - log p CURVES FOR SUSTAINED PRESSURE OF 100 k Pa

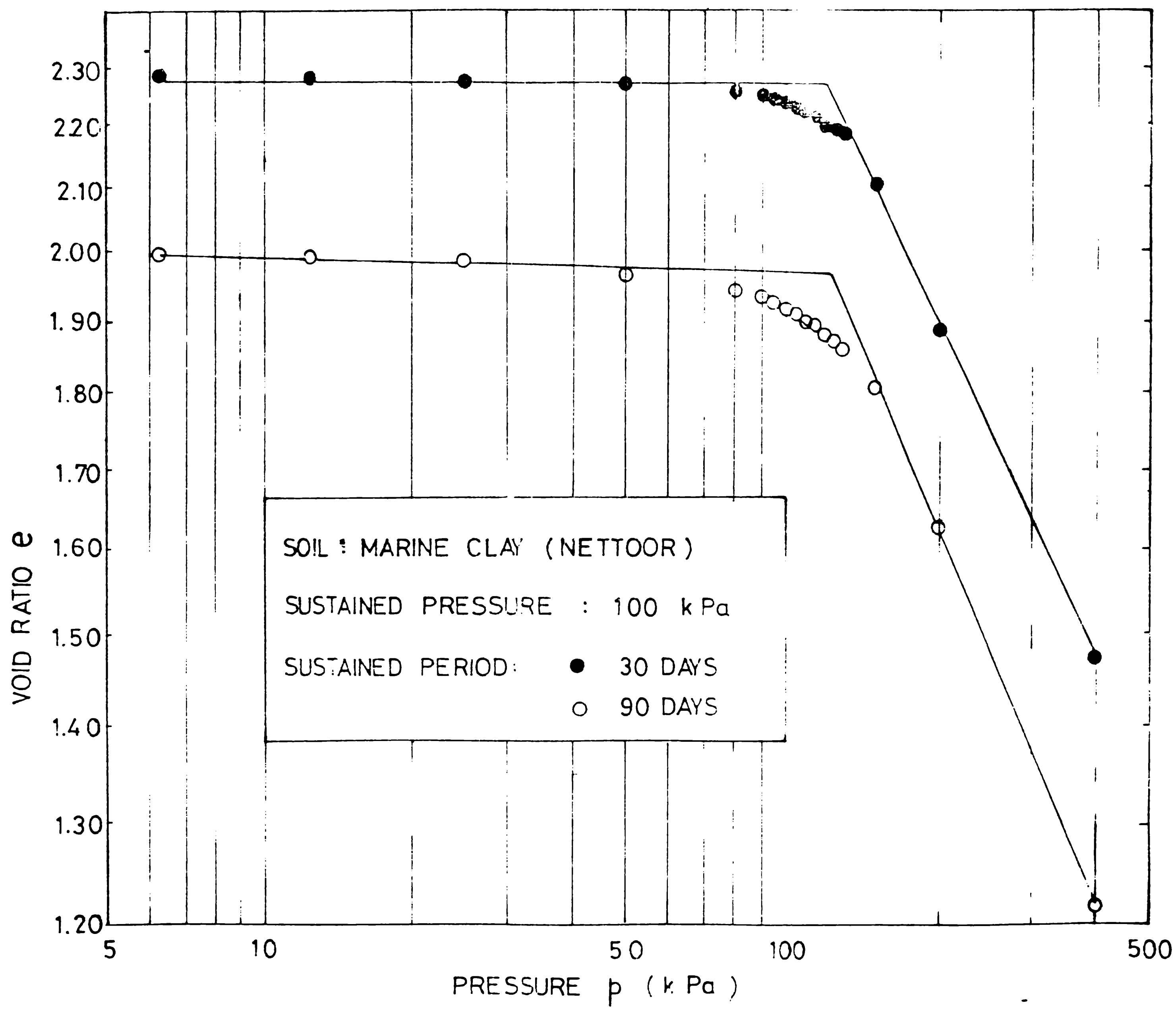

Fig.4.2.11 log e - log p CURVES FOR MARINE CLAY WITH SUSTAINED LOAD FOR LONGER DURATIONS

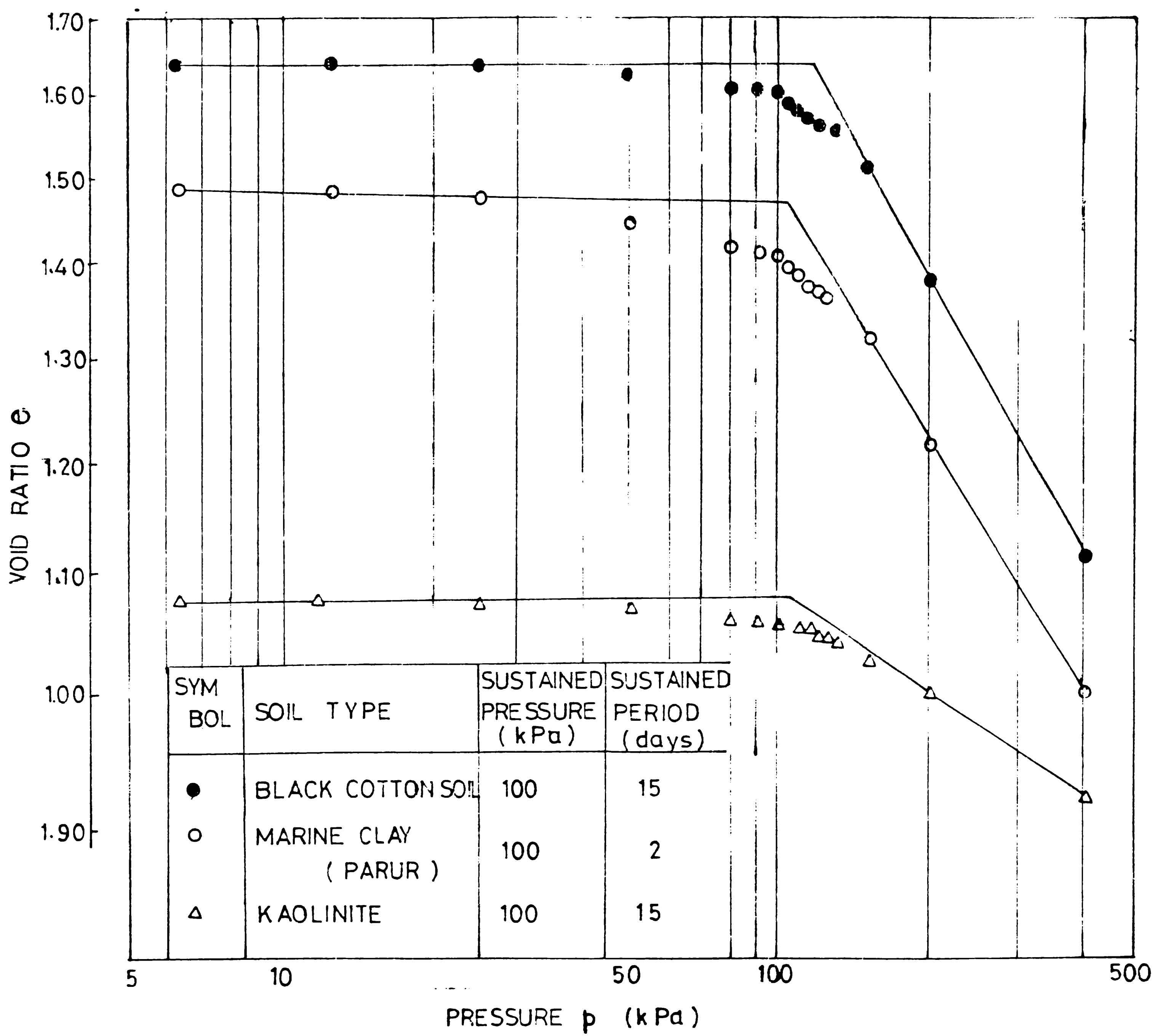

Fig.4.2.12 TYPICAL log e - log p CURVES

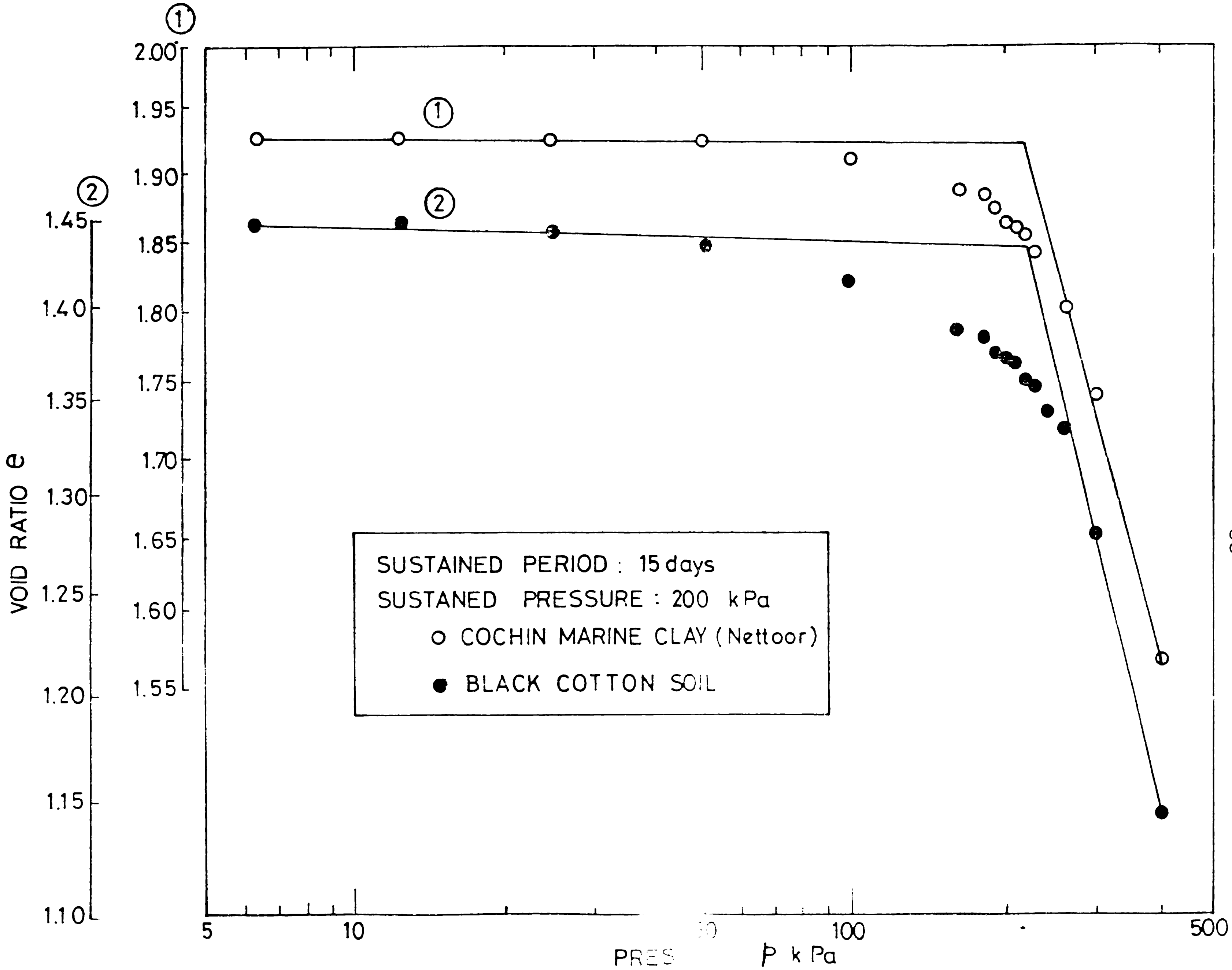

Fig. 4.2.13 log e - log p CURVES FOR [illegible]AINED PRESSURE OF 200 k Pa

In order to obtain a comparative study of the results of the various methods for determination of p_c, the values obtained from a series of consolidation tests, analysed by different methods are presented in Table 4.2.1.

4.2.4 Preconsolidation pressure from log H - log p plots

In the earlier method making use of log e, variation in void ratio was plotted against consolidation pressure to obtain the p_c value. Since variation in void ratio in the oedometer is synonymous with variation in H, it was felt that log H vs. log p curves, which does not involve any arithmetical computations, can also give values of p_c of the clay specimens.

In order to explore this possibility, results of consolidation tests on marine clay samples from Nettoor consolidated at 100 k Pa for identical sustained periods as in earlier cases were plotted as log H vs. log p. Fig.4.2.14 shows log H - log p curves for specimens consolidated for 2 and 7 days and Fig.4.2.15 gives the relation for 30 and 90 days. The results tabulated in Table 4.2.1 show that the p_c values by this method are closer to preconsolidation pressure than those even by Casagrande method. But compared to log e - log p plots, p_c values do not consistently increase with period of preconsolidation.

Table 4.2.1

Preconsolidation pressure by different methods

Sl.	Soil type	Sustained period (days)	Sustained pressure (k Pa)	Preconsolidation pressure, p_c (k Pa)				
				From Casagrande method	From log e-log p plot	From log H-log p plot	From δ-log p plot	From linear intersections of e-log p plots
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1.	Marine clay (Nettoor)	2	100	130	108	110	104	104
2.	-do-	7	100	130	115	110	106	110
3.	-do-	15	100	135	119	113	110	110
4.	-do-	30	100	130	119	118	113	111
5.	-do-	90	100	130	122	118	115	110
6.	Marine clay (Parur)	2	100	125	105	113	102	94
7.	Black cotton soil	2	100	130	114	112	108	105
8.	Red earth	2	100	150	120	135	137	130

(contd...)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
9.	Kaolinite	2	100	125	102	106	105	100
10.	Marine clay (Nettoor)	15	200	245	220	250	220	215
11.	Black cotton soil	15	200	255	222	230	215	215
12.	-do-	15	100	130	118	114	110	105
13.	Kaolinite	15	100	145	120	126	123	98
14.	Marine clay +3% lime	2	100	145	120	126	123	122
15.	-do-	7	100	150	125	132	130	131
16.	-do-	15	100	150	135	134	130	126
17.	-do-	60	100	160	160	150	155	161

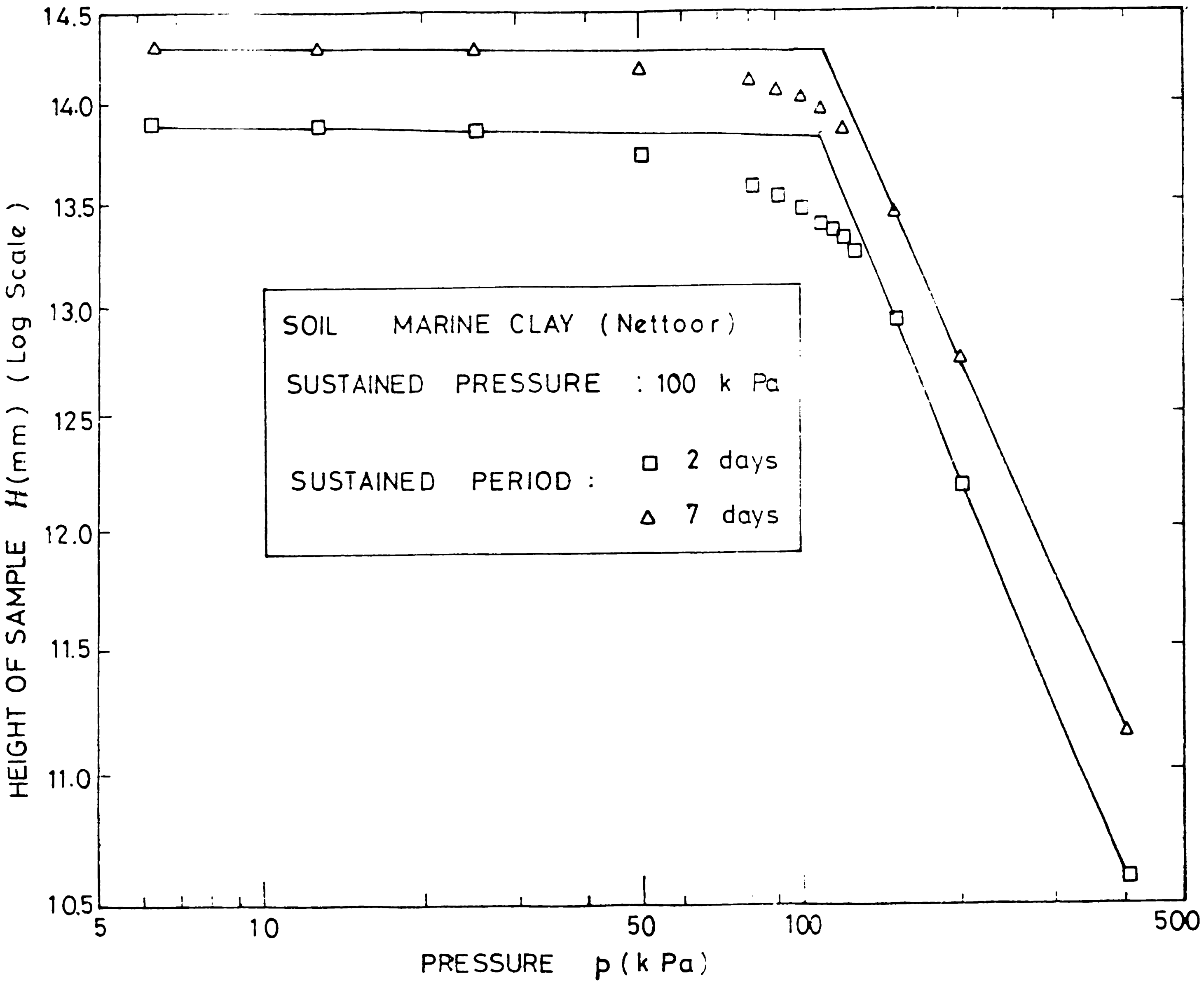

Fig.4.2.14 log H - log p PLOTS FOR SUSTAINED PRESSURE OF 100 k Pa.

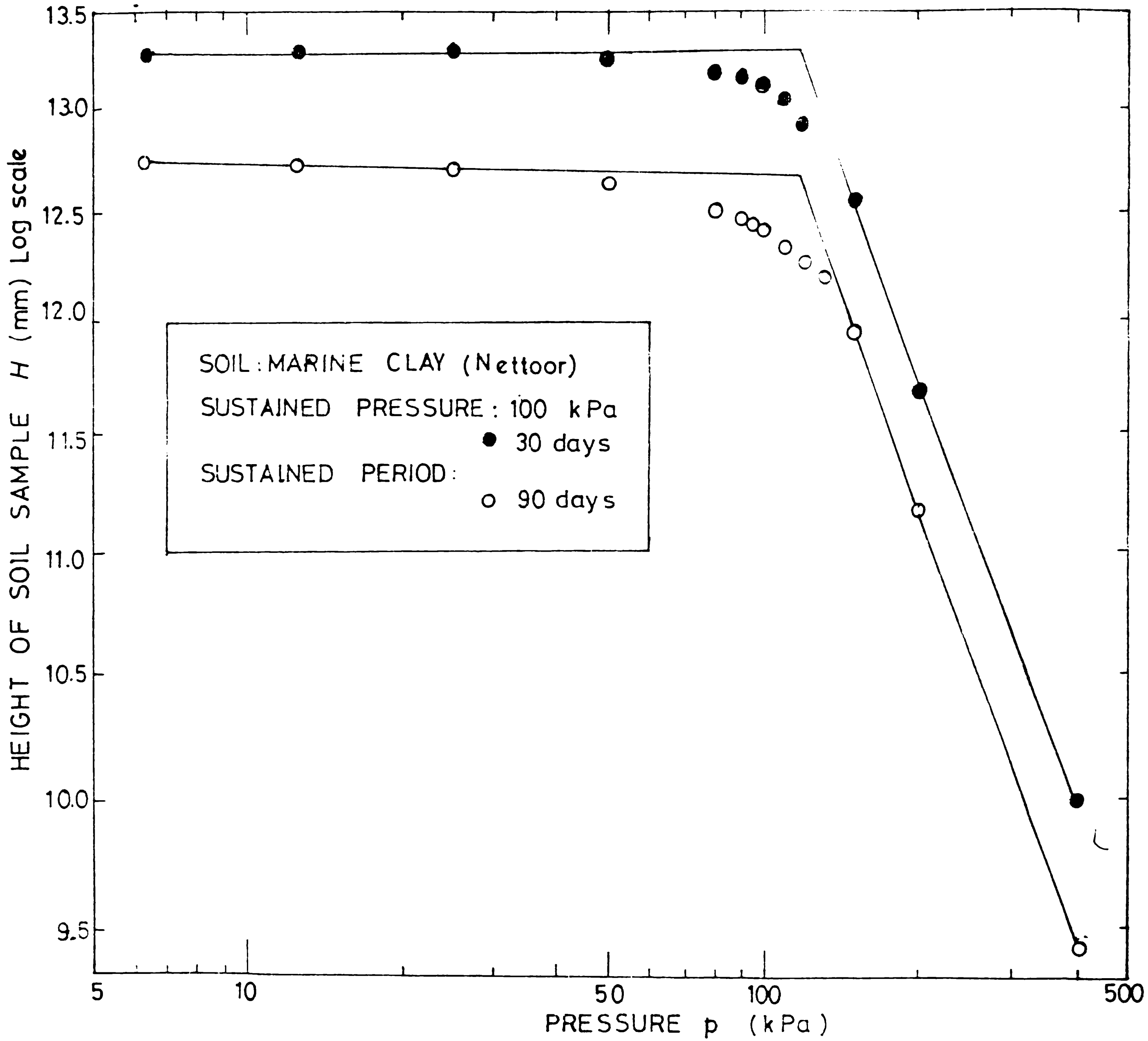

Fig.4.2.15 log H - log p PLOTS FOR MARINE CLAY WITH SUSTAINED LOAD FOR LONGER DURATIONS

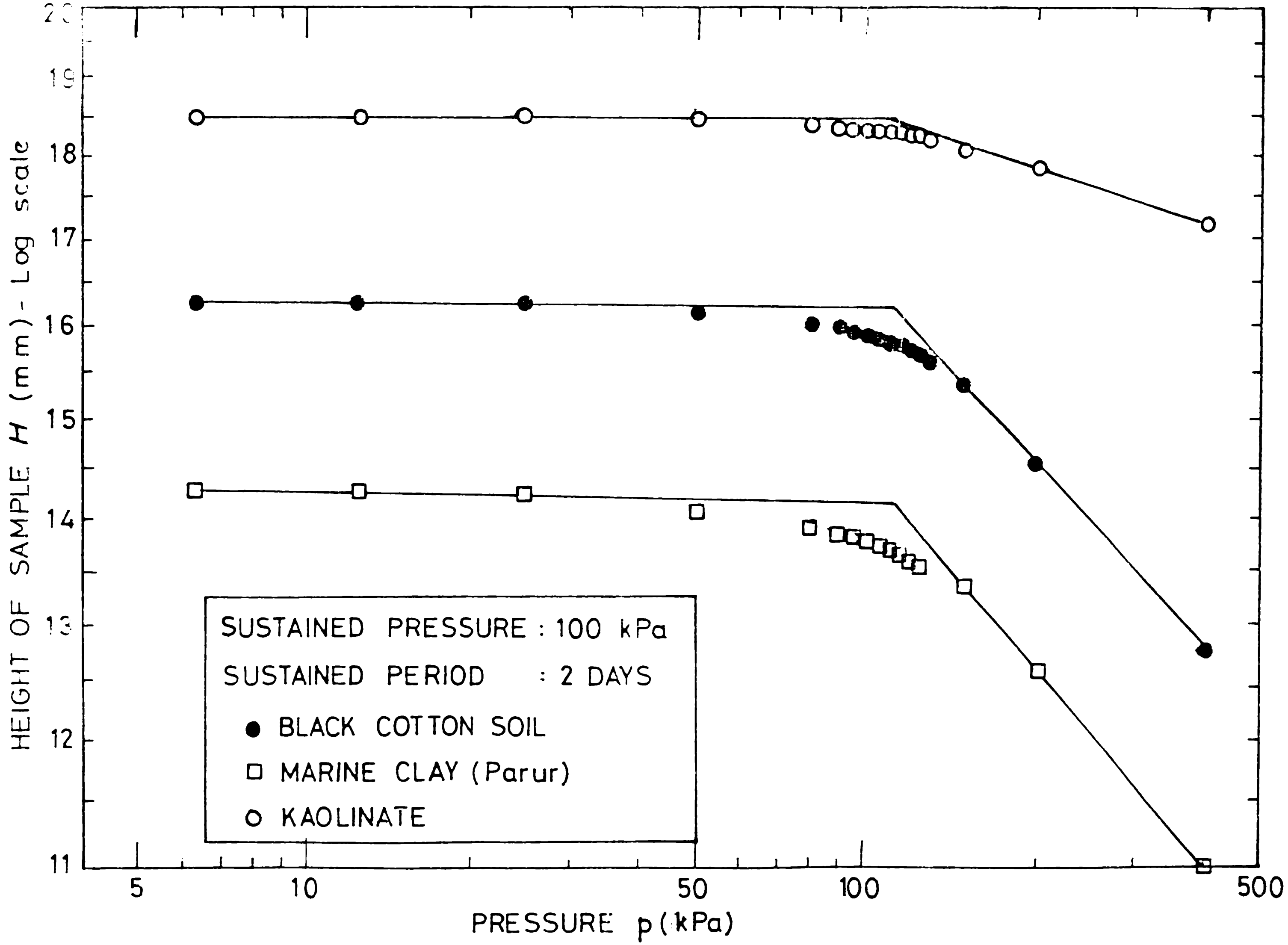

Fig.4.2.16 log H - log p PLOTS FOR DIFFERENT SOILS

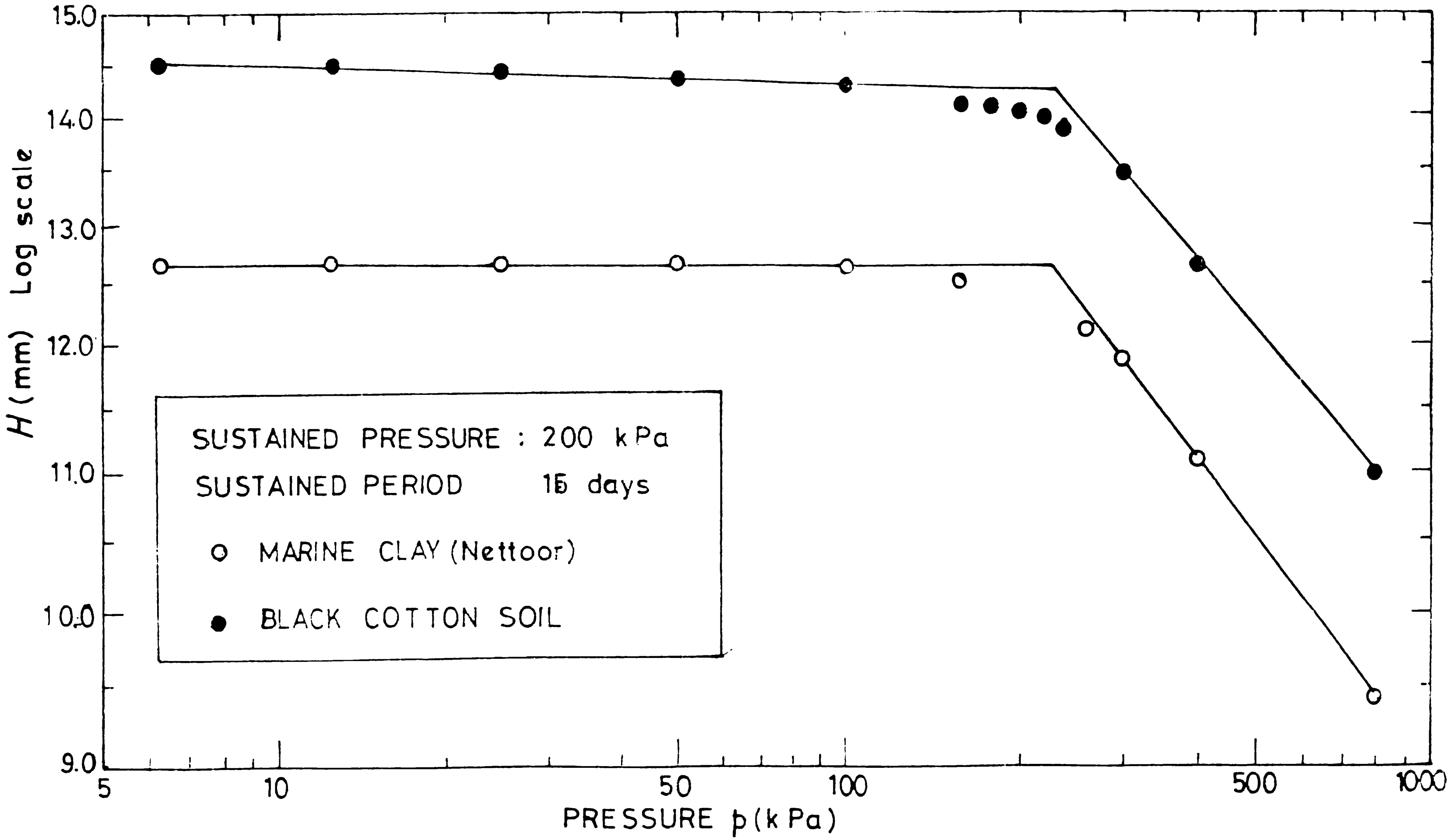

Fig.4.2.17 log H – log p PLOTS FOR SUSTAINED PRESSURE OF 200 k Pa

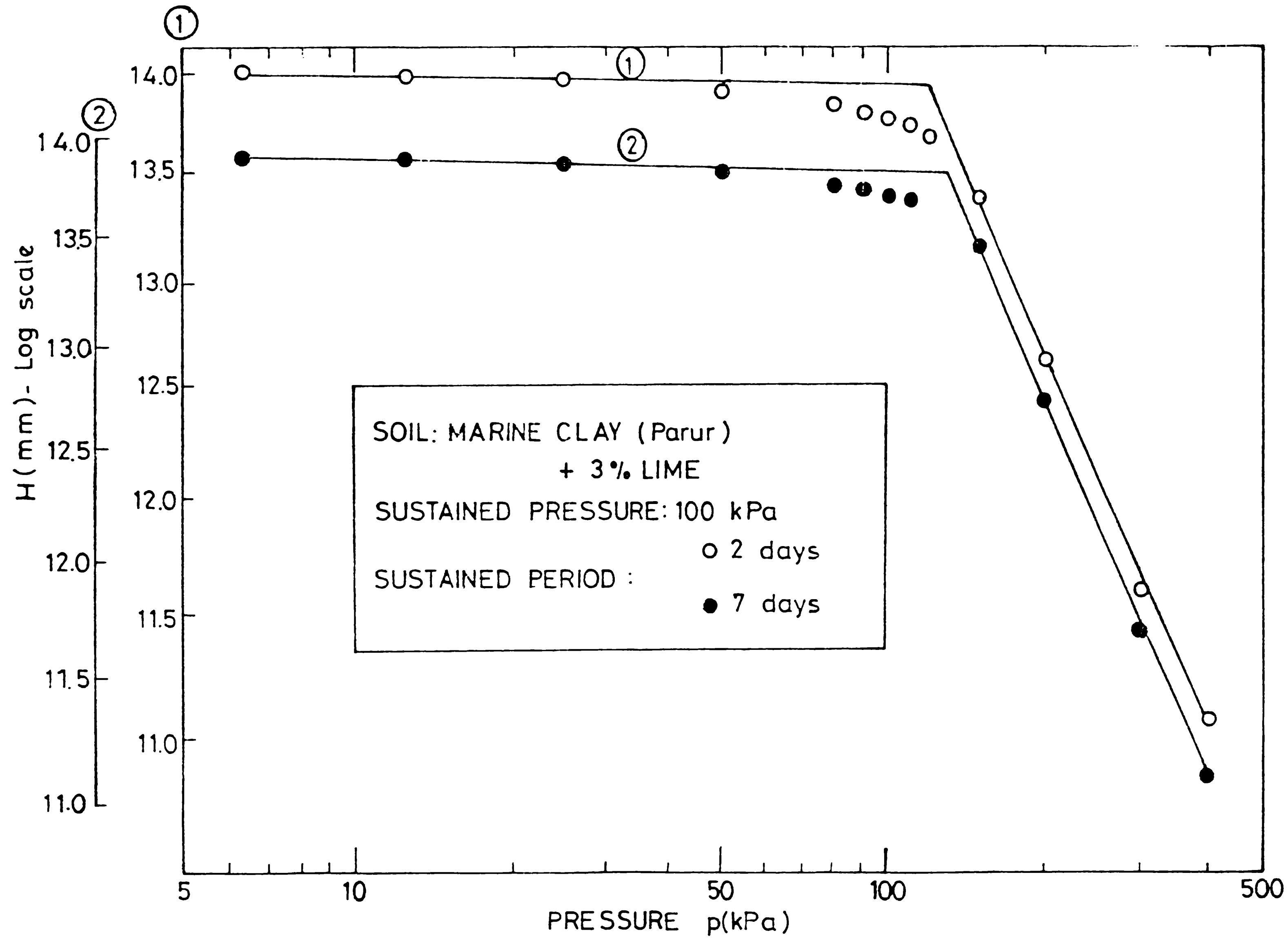

Fig.4.2.18 TYPICAL log H - log p PLOTS FOR LIME TREATED MARINE CLAY

Fig.4.2.16 shows the relationship for three other soils viz. - Black cotton soil, Marine clay from Parur and Kaolinite. Log H - log p plots for two specimens - marine clay from Nettoor and Black cotton soil are presented in Fig.4.2.17 from results of consolidation tests on samples preconsolidated at 200 k Pa for 15 days.

The development of quasi-preconsolidation pressure and bond strength developed by stabilisation with lime are presented in Fig.4.2.18.

A comparison of the two methods viz. log e - log p and log H - log p shows that both the methods give more or less identical results, but log e - log p curves are more consistent when the parameters are varied. But log H - log p has the advantage that no arithmetical computations are involved.

4.2.5 Preconsolidation pressure from δ - log p plots

According to Ladd (1991), preconsolidation pressure is the maximum past pressure representing the yield stress that separates small strain elastic behaviour from large strains accompanied by plastic (irrecoverable) deformation during one dimensional compression. An e - log p curve for normally consolidated clay, which describes the compressibility characteristics, consists of two distinct portions - a recompression portion and a virgin curve. The point separating these two portions gives the preconsolidation pressure. Obviously, when the clay specimen is loaded within the recompression range, the deformations will be minimal as the soil has been fully consolidated and the soil fabric has already developed the required resistance to withstand such loads. But once p_c is exceeded, the specimen is being subjected to these higher pressures for the first time. Obviously the amount and rate of deformation will be considerably higher, once the pressures exceed p_c. Hence p_c could also be defined as the pressure above which the compression takes place at a much faster rate.

Making use of the original suggestion by Ladd, and the more practical aspects of consolidation tests, it was felt that relations between δ (compression in divisions) and log p should also give a clue for the preconsolidation pressure. If the attempt is successful, it will have the added advantage that arithmetical computations are totally eliminated and p_c can be obtained as soon as or even before a routine consolidation test is completed.

In order to examine the possibility of using δ-log p plots to evaluate p_c, the results of the consolidation tests reported earlier were made use of. Fig.4.2.19 and

4.2.20, show the relation between the change in thickness of the clay specimens (expressed as number of divisions) and log p for samples preconsolidated at 100 k Pa for 2, 7, 30 and 90 days. Straight lines were fitted along the initial and final points, the intersection of which gives the preconsolidation pressure p_c. The values are listed in Table .2.1. It can be seen that the values given by this method are closest to the preconsolidation pressure of 100 k Pa.

Fig.4.2.21 show δ-log p curves for a marine clay specimen and a Black cotton soil preconsolidated at 200 k Pa for a period of 15 days. The p_c values obtained using this procedure are 220 and 215 respectively which again are closer to the preconsolidation pressure of 200 k Pa, compared to the results given by other methods as brought out in the Table 4.2.1.

Fig.4.2.22 shows δ - log p curves for three different soils - Black cotton soil, Marine clay (Parur) and Kaolinite preconsolidated at 100 k Pa. The p_c values obtained are again closer to 100 k Pa.

Since the consolidation tests on different clays gave satisfactory results, δ - log p curves were plotted for Cochin Marine clay samples treated with 3% lime and consolidated at 100 k Pa for periods of 2 and 7 days

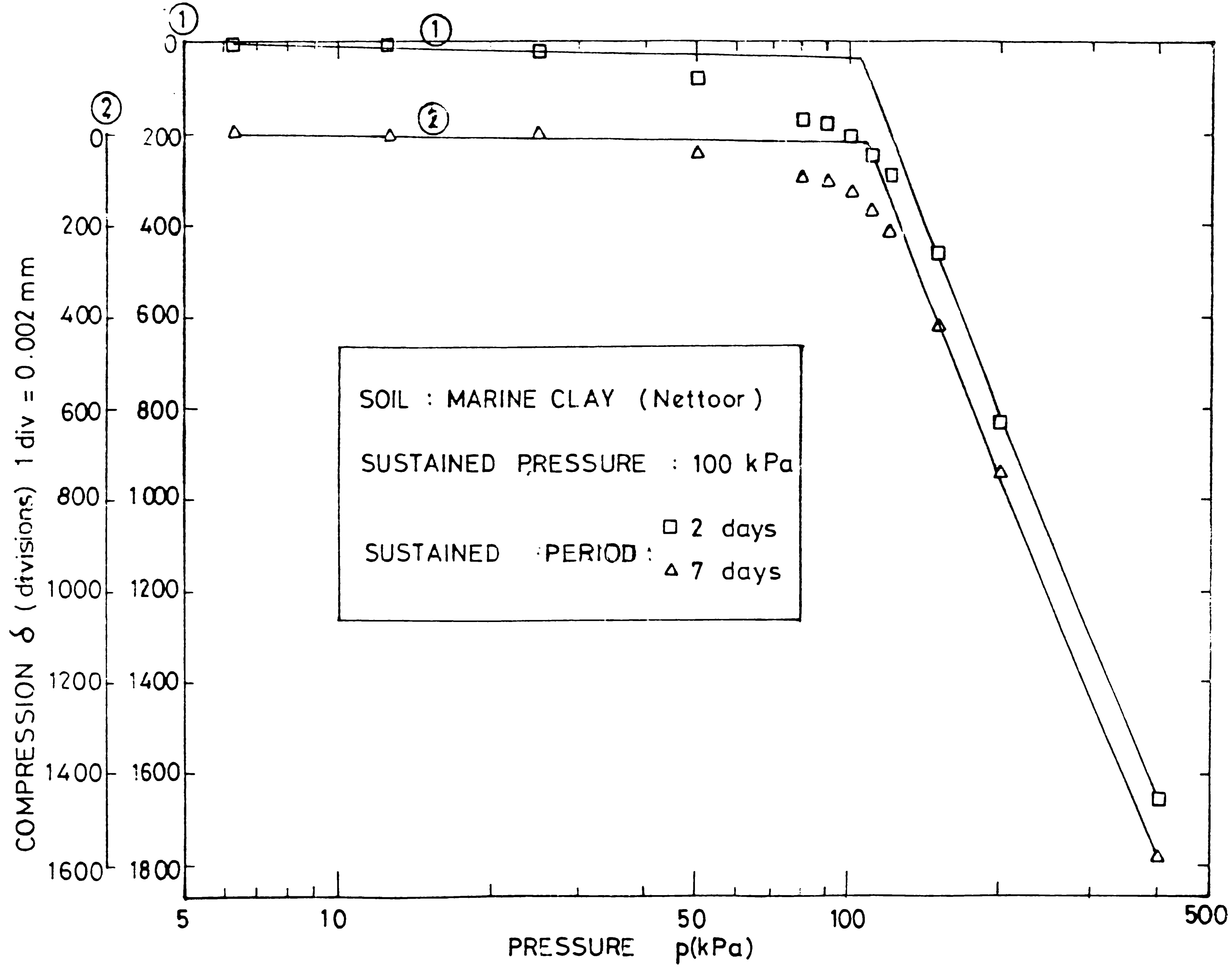

Fig.4.2.19 δ - log p PLOTS FOR A SUSTAINED PRESSURE OF 100 k Pa

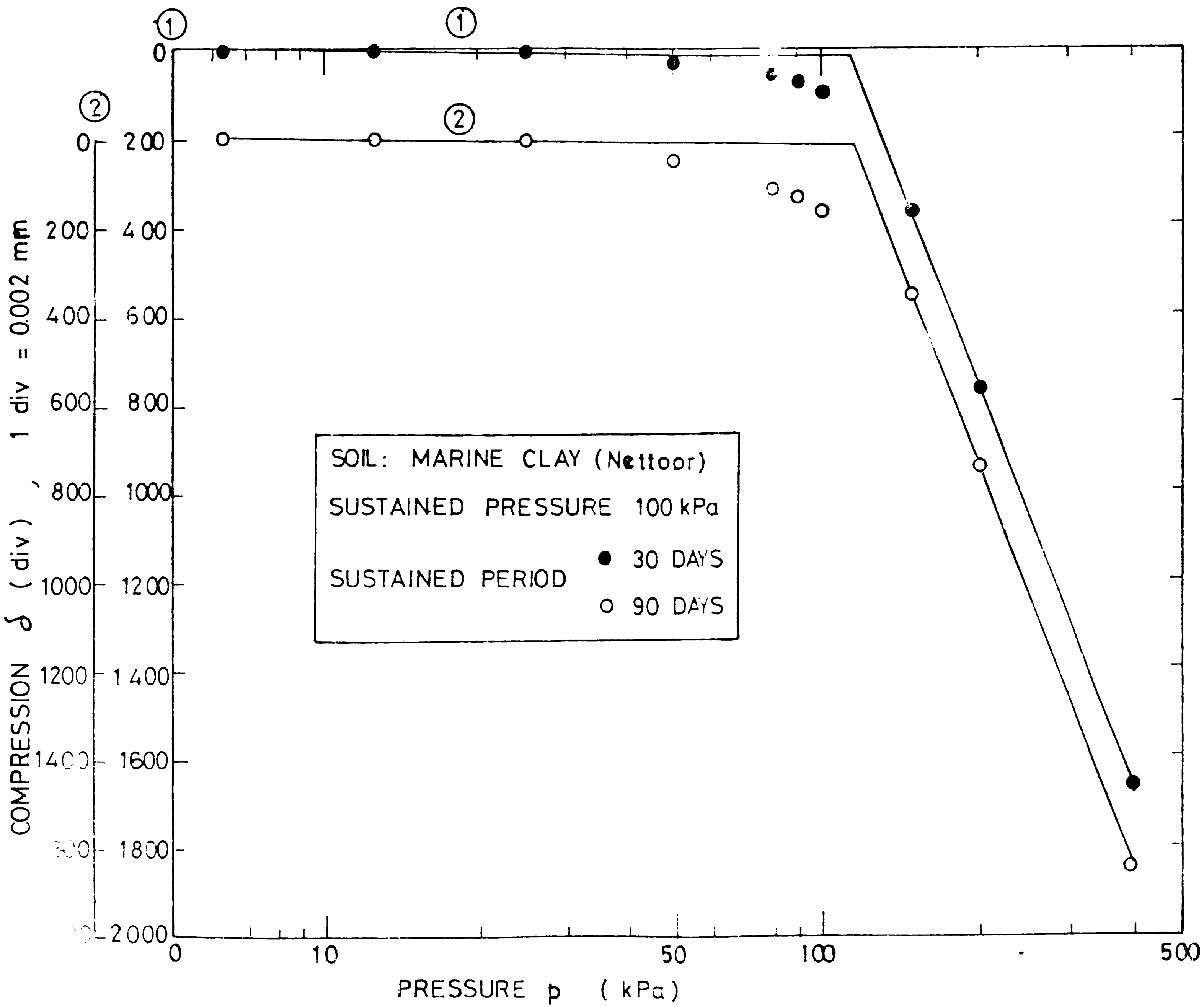

Fig.4. δ -log p CURVES FOR MARINE CLAY WITH SUSTAINED LOAD FOR LONGER DURATIONS

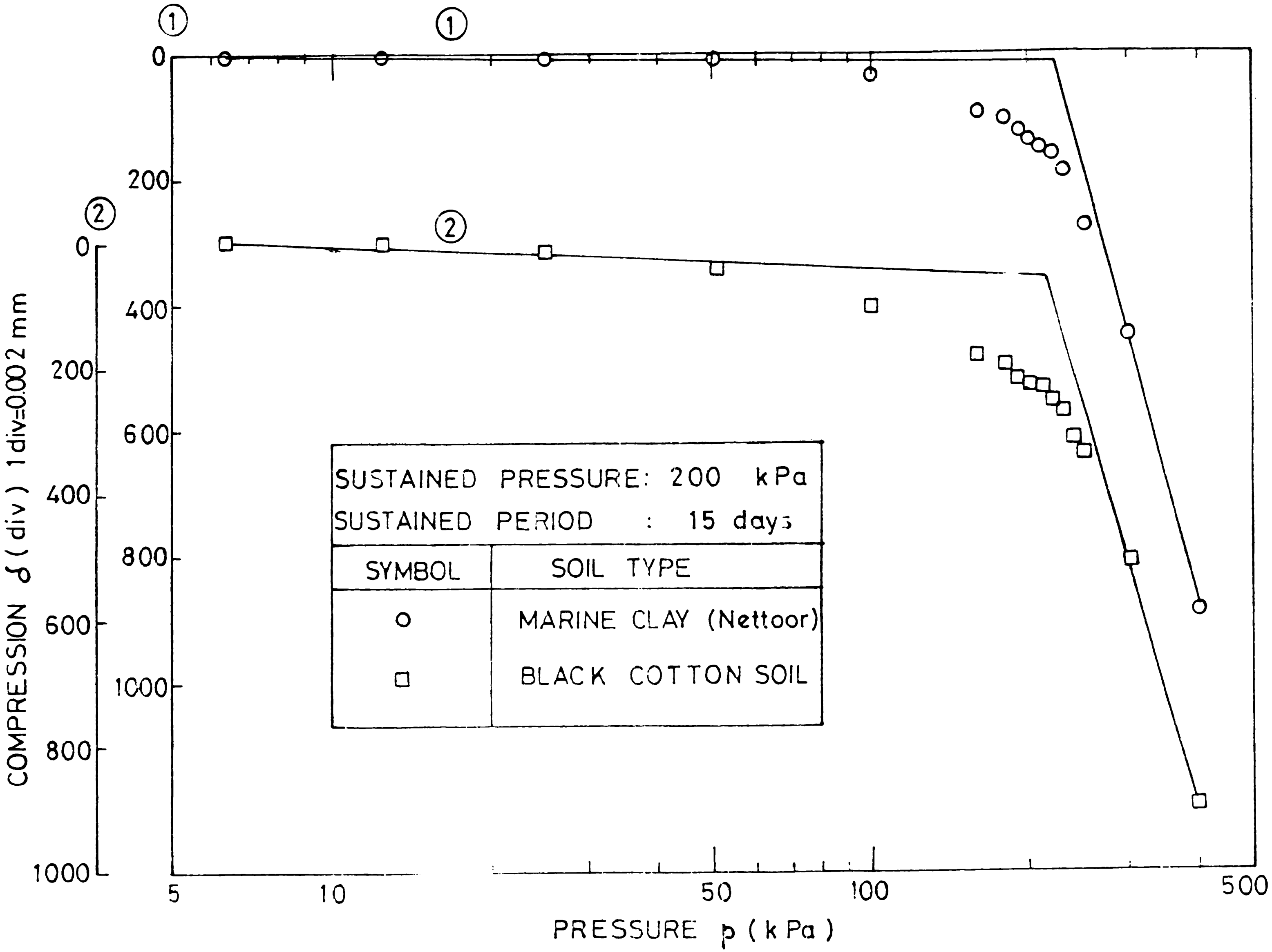

Fig.4.2.21 δ - 1 PLOTS FOR A SUSTAINED PRESSURE OF 200 k Pa

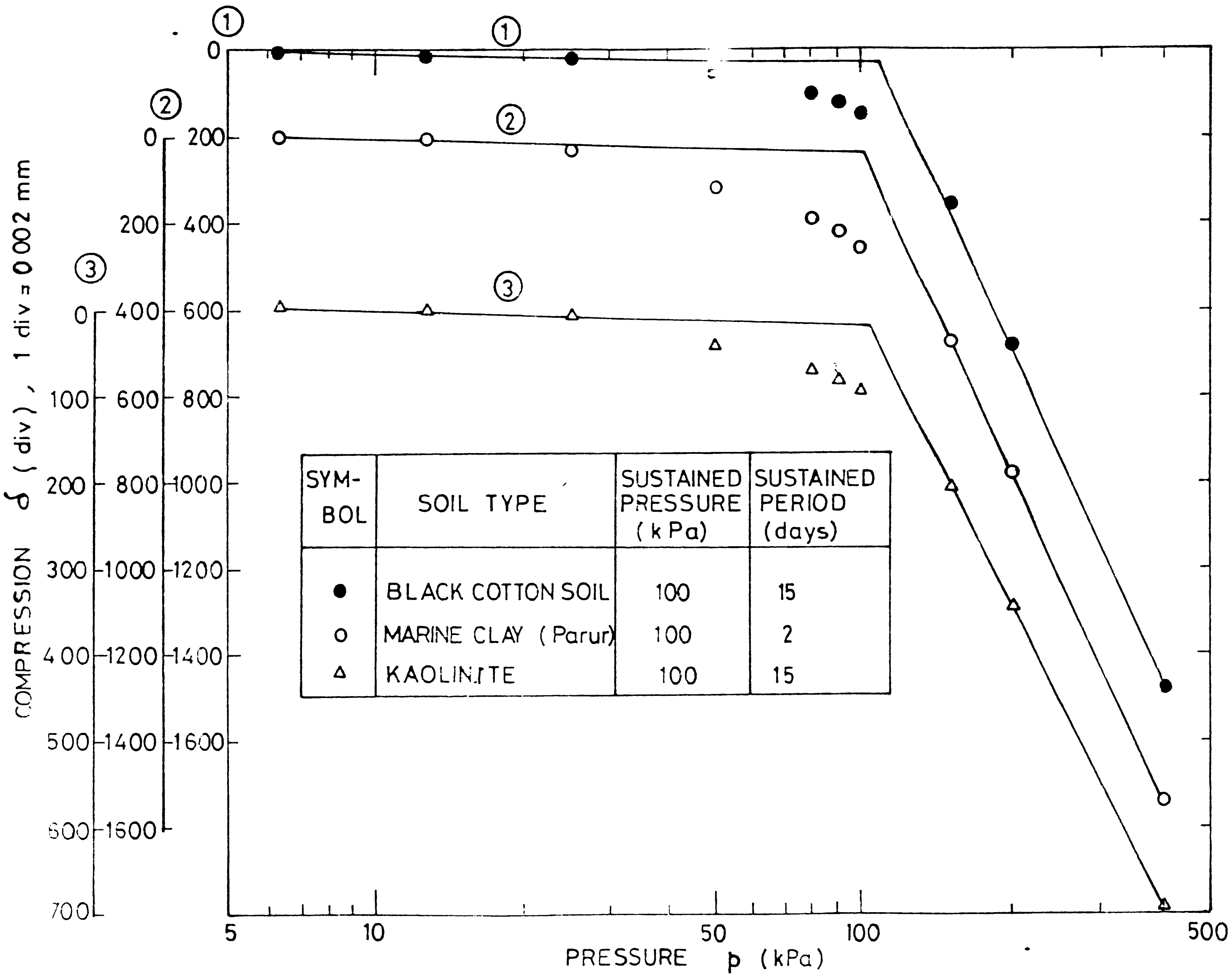

Fig.4.2.22 δ - log p PLOTS FOR DIFFERENT SOILS

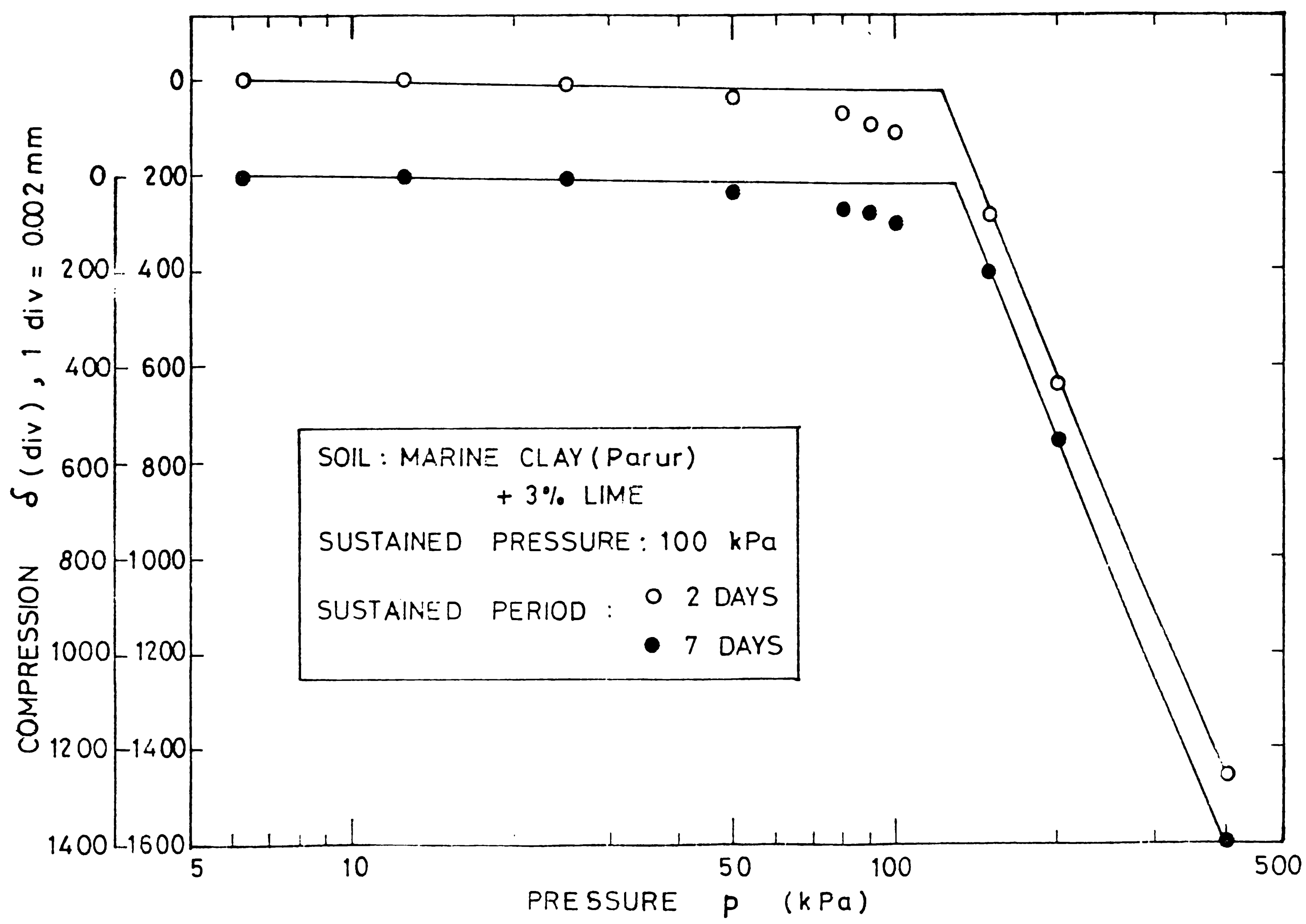

Fig.4.2.23 δ -log ..OTS FOR LIME TREATED COCHIN MARINE CLAY .

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(Fig.4.2.23). The p_c values obtained from these plots are also presented in Table 4.2.1. A comparison of these values show that they are much more realistic and reliable compared to results from other techniques. Further, the values are on the conservative side making the designs safer. Thus the δ - log p plots is perhaps the most acceptable technique for determination of preconsolidation pressure in view of the several advantages discussed earlier.

4.2.6 P_c from linear intersections of e - log p plots

According to Holtz and Kovacs (1981), the intersection of the two straight line portions of the laboratory e - log curve is the most probable value for preconsolidation pressure. To check the varacity of this statement, the results of the consolidation tests described earlier were plotted as void ratio vs. consolidation pressure p. Fig.4.2.24 shows the e - log p plots for marine clay specimens preconsolidated at 100 k Pa for periods of 2, 7 and 90 days. The linear intersections give values of 104, and 110 k Pa. The results, though close to p_c value failed to bring out the increase due to quasi-preconsolidation pressure. Fig.4.2.25, where e - log p plots are presented for three different soils show that p_c obtained from the linear intersections are consistently lower than results obtained from other methods presented in Table 4.2.1.

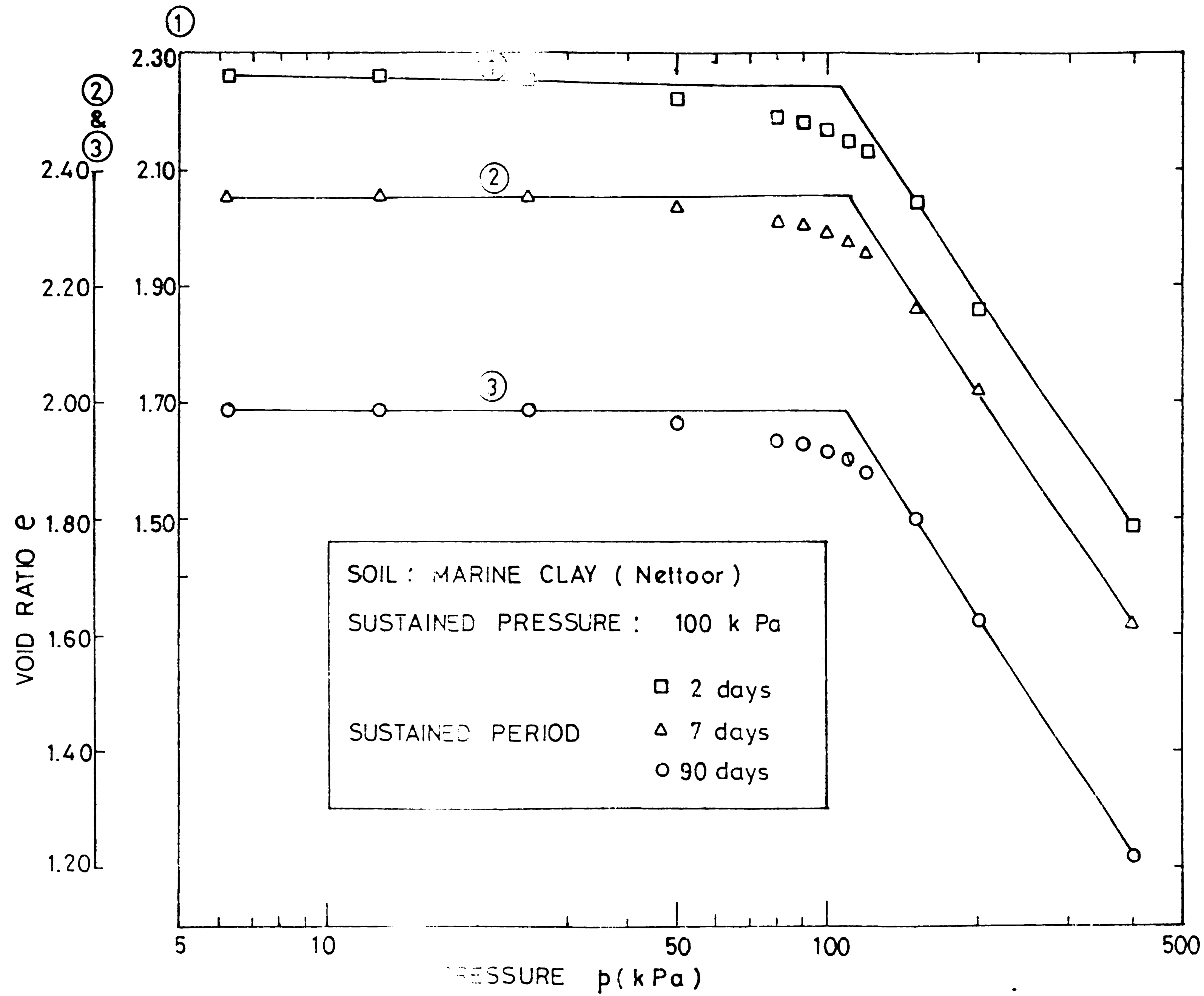
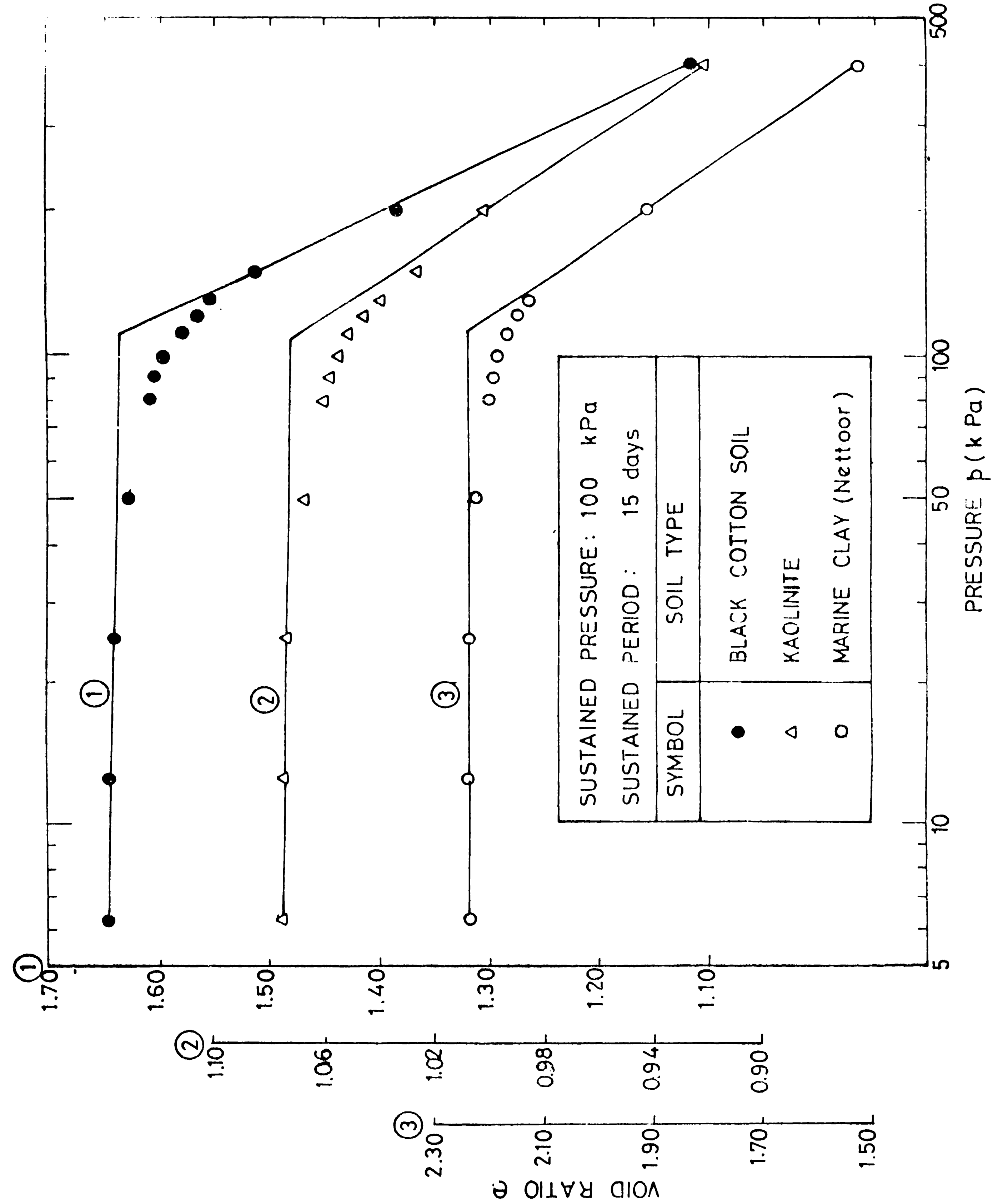


Fig. 4.2.24 e-log p ... FOR MARINE CLAY WITH SUSTAINED LOAD FOR DIFFERENT ... IONS


SUSTAINED PRESSURE : 100 kPa
SUSTAINED PERIOD : 15 days
SYMBOL
SOIL TYPE
BLACK COTTON SOIL
KAOLINITE
MARINE CLAY (Nettoor)
PRESSURE p (k Pa)
VOID RATIO e
①
②
③
5
10
50
100
500
1.70
1.60
1.50
1.40
1.30
1.20
1.10
1.06
1.02
0.98
0.94
0.90
2.30
2.10
1.90
1.70
1.50

The method using the results of marine clay (Parur site) preconsolidated at 100 k Pa gave a p_c value of only 94 k Pa, which is on the lower side. A comparative study of the results listed in Table 4.2.1 shows that this method fails to give values of p_c when the results from various soils preconsolidated at different pressures for different periods do not give consistent results.

4.2.7 Selection of an acceptable procedure for determination of p_c

The Casagrande method proposed in the thirties has been the only method in practice for the past half a century inspite of certain inherent drawbacks. Eventhough there have been some sporadic attempts to develop new methods, these procedures could not earn wider acceptability as they were time consuming and cumbersome. They also lacked precision and clarity. The validity of several methods proposed in literature could not be verified as the true preconsolidation pressure values were not available to compare with. In this context, any attempt to evolve simpler, faster and yet accurate procedures to determine p_c merits consideration.

The present study attempted to examine the accuracy and reliability of as may as eight procedures. The experimental technique developed by Jose et al. helped to identify the method which gave results for preconsolidation pressure closest to the actual p_c value. Consolidation tests were carried out on various soils preconsolidated at different pressures for varying durations. The series of tests attempted to evolve a procedure which will be applicable to all types of fine grained soils. The methods suggested by Burmister and Schmertmann were not taken up for detailed study as the present investigations aimed at the development of a simpler and faster method, wherein trial and error procedures will not be preferred.

Results from Casagrande method presented in Table 4.2.1 show that this method consistently give the highest values of p_c, irrespective of type of soil, consolidation pressure or its duration. Since these values are higher than the actual values, results obtained using p_c from Casagrande method can be on the unsafe side.

The method suggested by Becker using work per unit volume - pressure plots gave values slightly above those obtained by Casagrande method. Hence it was not pursued further.

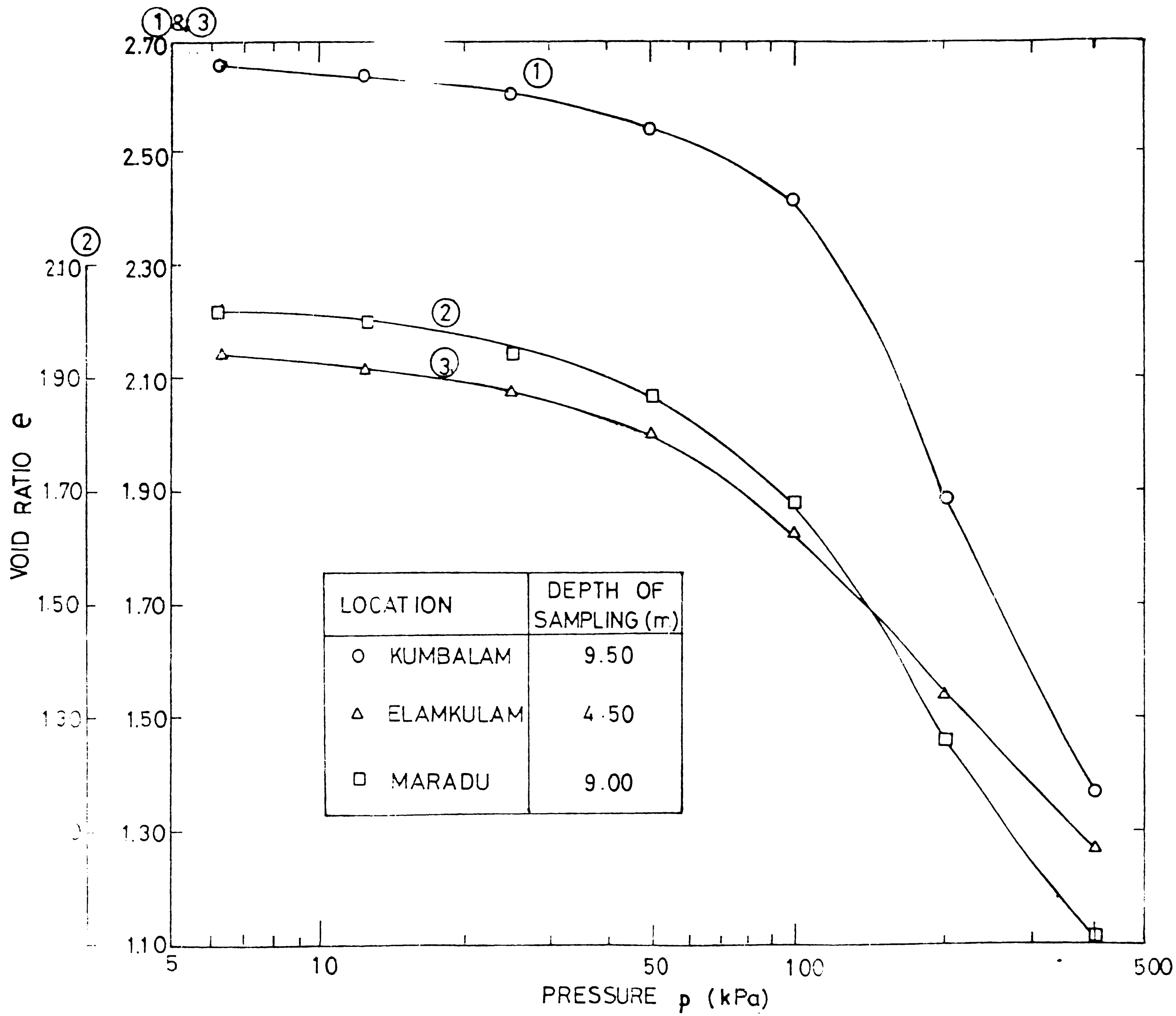
The procedure suggested by Jose et al. wherein log e - log p plots are used for determination of p_c gave consistent and reliable results for a wide range of consolidation tests. Still other methods such as log H vs. log p and δ - log p were also tried in the search for a more accurate procedure for determination of p_c. The results presented in Table 4.2.1 show that all the three yield more or less identical values for p_c, making it difficult to name the most superior method.

For a field engineer who is more concerned about the settlements due to consolidation, will appreciate the development of a method wherein p_c is defined as the pressure which separates small strain elastic behaviour and large scale irrecoverable deformations. In this context, the evolution of the δ - log p method can claim wider acceptability. In addition to giving results closest to the preconsolidation pressure, it has shown consistency over a wide range of consolidation tests. Elimination of arithmetical calculations and the fact that p_c is obtained during the consolidation test is in progress are added advantages for this procedure. Hence this method can be singled out as the most acceptable among all the procedures tried. The results of a few consolidation tests carried out on undisturbed samples collected from field along the western coast spread over about 800 km from Karwar to Kayamkulam are presented in the following sections.

Fig.4.2.26 shows the e - log p curves from consolidation tests carried out on undisturbed samples collected from three sites in Cochin viz. Kumbalam, Elamkulam and Maradu. It can be seen from the curves that the point of maximum curvature is clear and obvious in the e - log p curve for Kumbalam sample. Selection of such a point is not easy in case of compression curves for Elamkulam and Maradu. Hence there is a chance that personal errors may creep in 'e p_c values are determined.

Fig.4.2.27 shows the log e - log p curves plotted for the same consolidation test data. It can be seen that the intersection of straight lines fitted to the initial and final sets of points gives p_c values without any ambiguity. similarly figure 4.2.28 where log H - log p plots are shown, give almost identical results with sufficient accuracy. The plot between δ (the deformation in number of divisions) and log p are plotted in Fig.4.2.29. This method, which requires least computational effort also yields a set of reliable p_c values consistent with the above methods. Thus all the three methods are equally applicable to field samples as well as preconsolidated laboratory specimens.

Quite often, the e - log p curves obtained from consolidation tests on field ımples do not give rise to the typical standard e - log p curve, with distinct portions for recompression and virgin compression. Fig.4.2.30 shows e - log p curves obtained for two undisturbed samples collected



4.2.26 TYPICAL e-log p CURVES - UNDISTURBED COCHI MARINE CLAYS

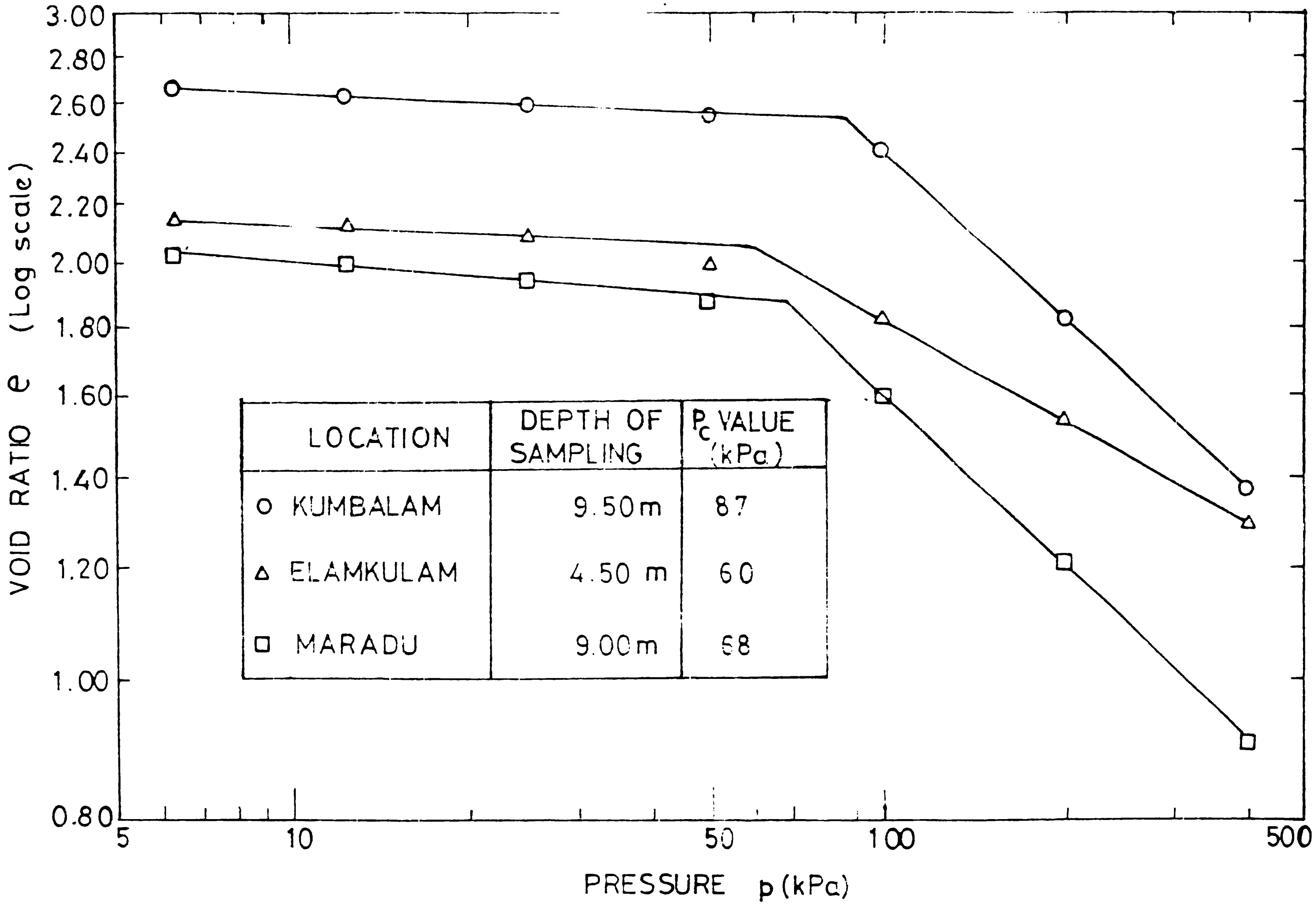


Fig.4.2.27 log e - log p PLOTS - UNDISTURBED COCHIN MARINE CLAYS

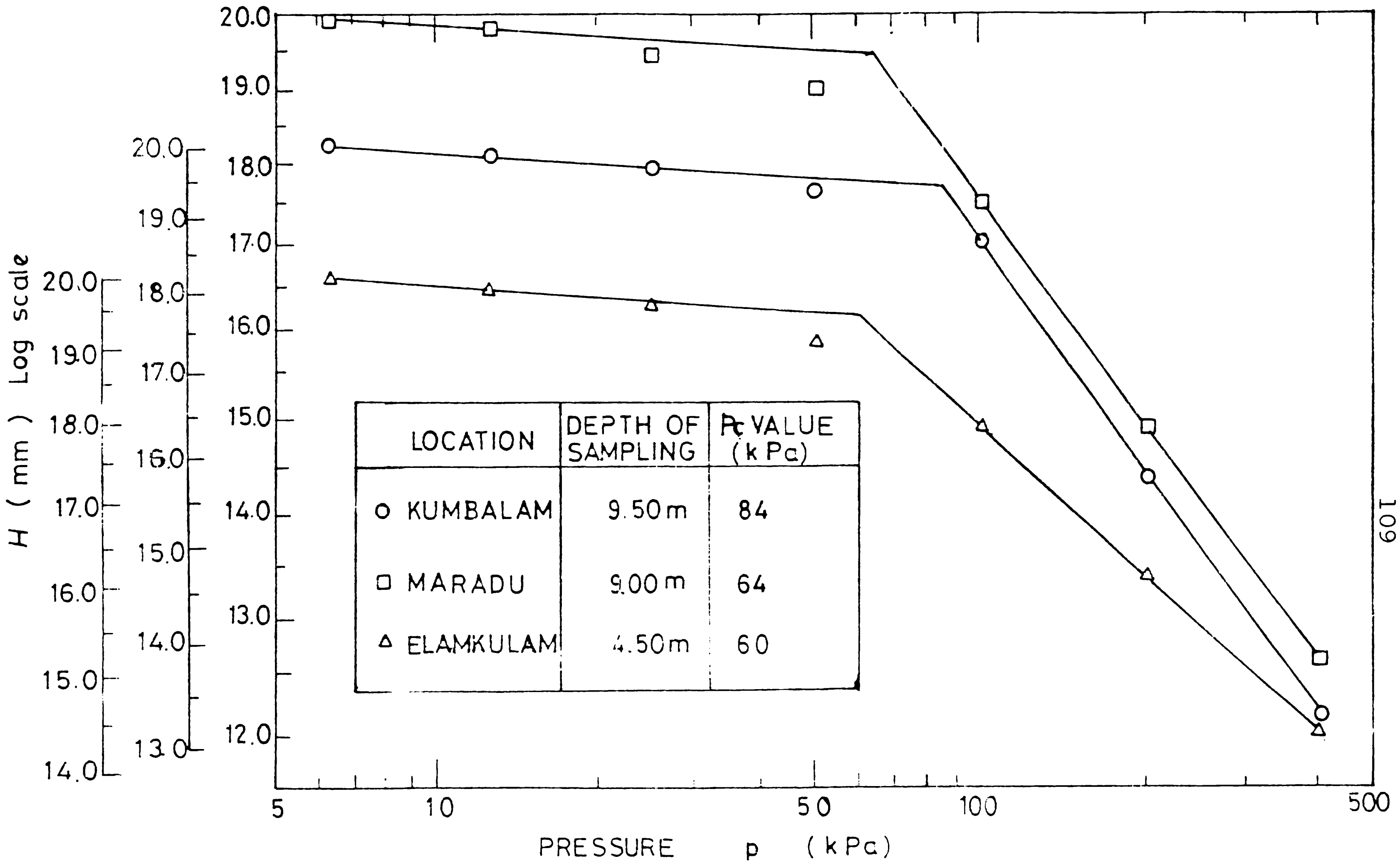


Fig.4.2.28 log H - log p PLOTS - UNDISTURBED COCHIN MARINE CLAYS

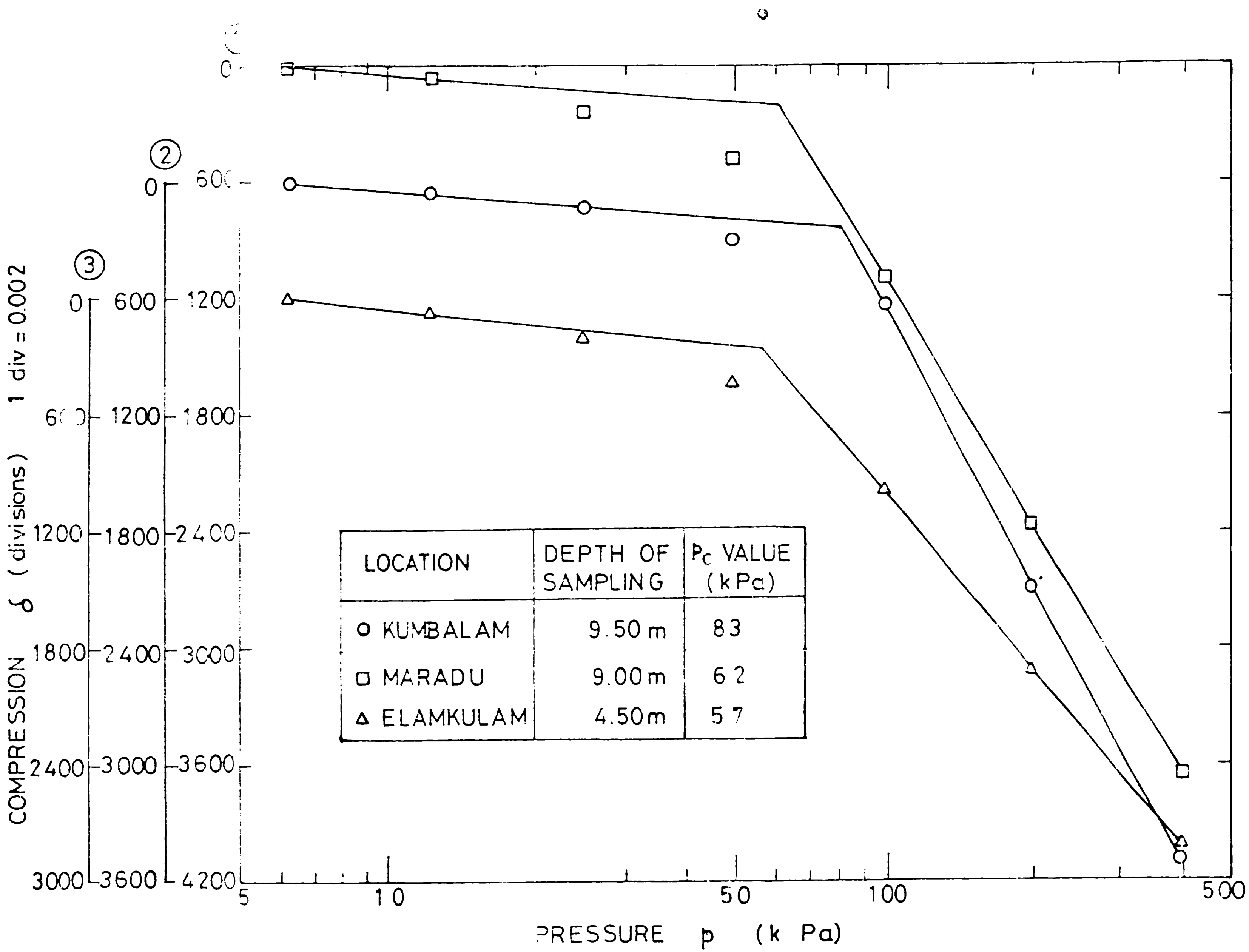


Fig.4.2.29 δ -log p CURVES - UNDISTURBED COCHIN MARINE CLAYS FROM THREE SITES

from two sites in Cochin. The transition from the recompression to virgin portion is smooth and gradual that selection of the point of maximum curvature becomes a difficult proposition. This inevitably causes errors due to personal judgement.

The advantages of δ - log p curve discussed in detail earlier are clearly broughtout in the preparation of curves shown in Fig.4.2.31. Short straight lines can be easily fitted to the initial and final sets of the points and they intersect at p_c values, without the errors that might have influenced the values of p_c obtained from the Casagrande procedure. Thus the new method has a special advantage in the case of most of e - log p curves obtained from the routine consolidation tests conducted on undisturbed field samples.

Consolidation test data obtained from tests carried out on undisturbed samples collected from Kali river mouth at Karwar and 5 m below sea bed at Padubidri, near Mangalore are shown in Fig.4.2.32. While e - log p curve for Karwar clay shows no obvious point of maximum curvature, that of Mangalore clay is less ambiguous. The δ - log p plot in .2.33 give distinct straight lines whose intersections give the values for p_c.

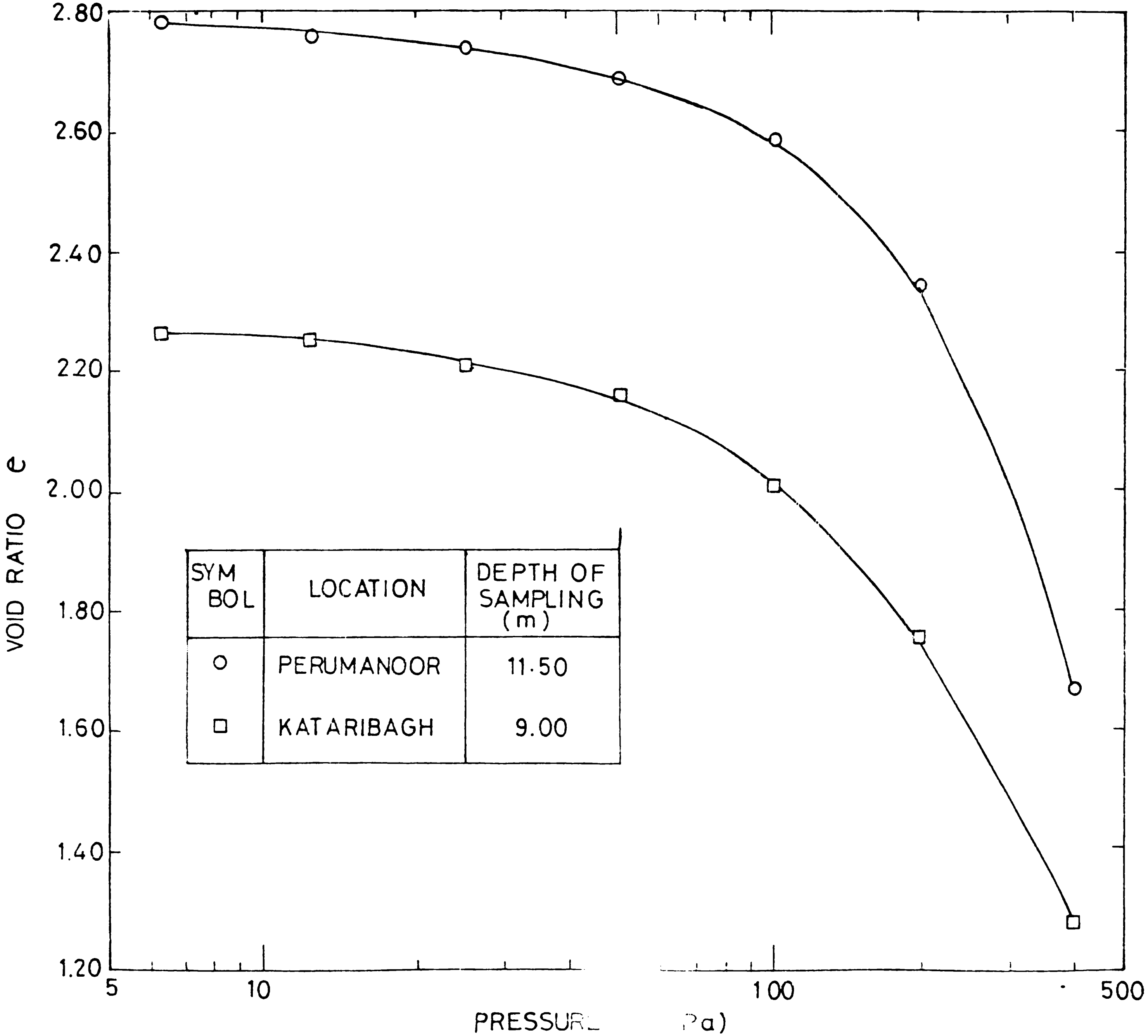


Fig.4.2.30 e-log p CURVES FOR COC RINE CLAY FROM TWO SITES

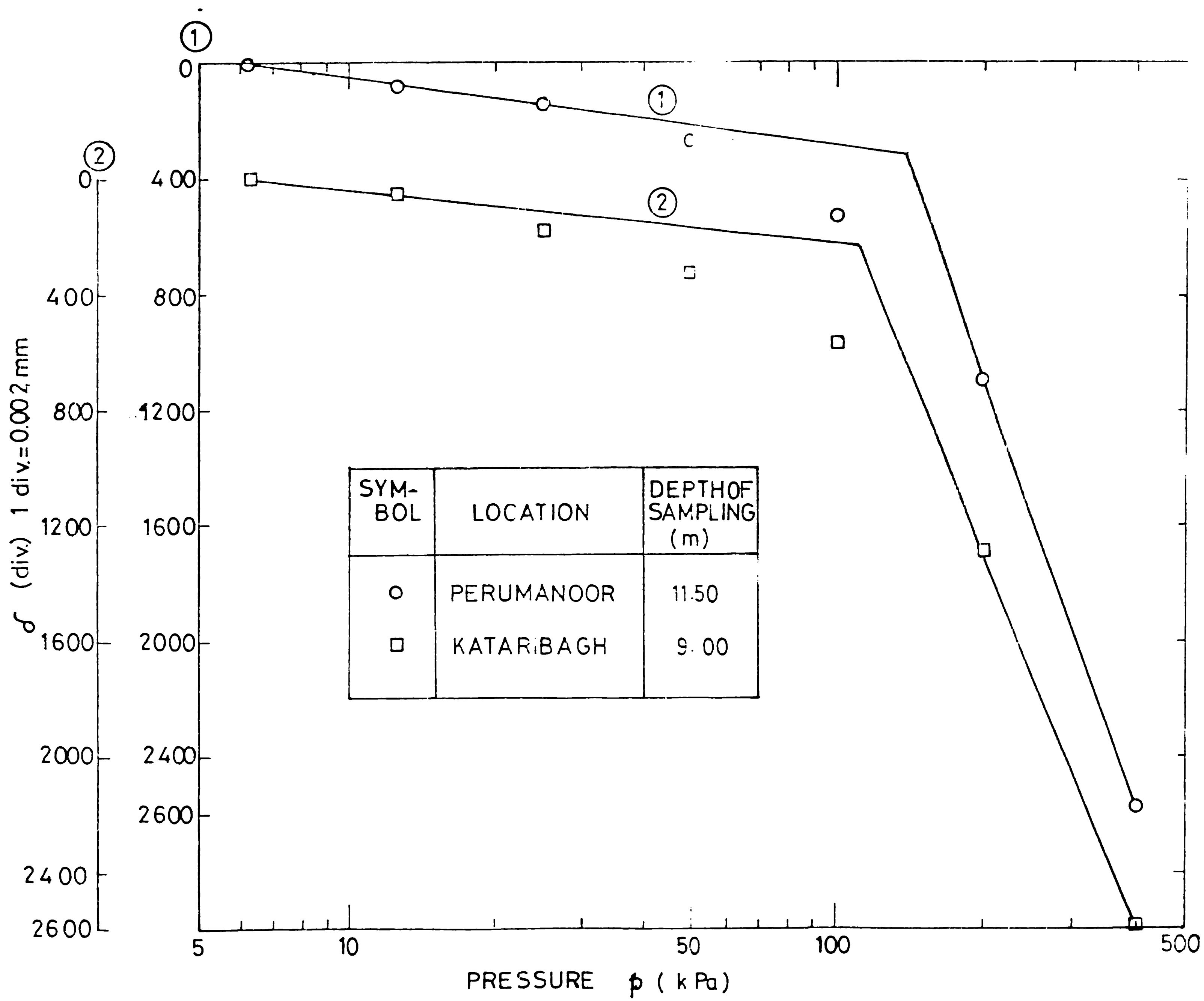


Fig.4.2.31 δ -log p PLOTS - UNDISTURBED COCHIN MARINE CLAYS

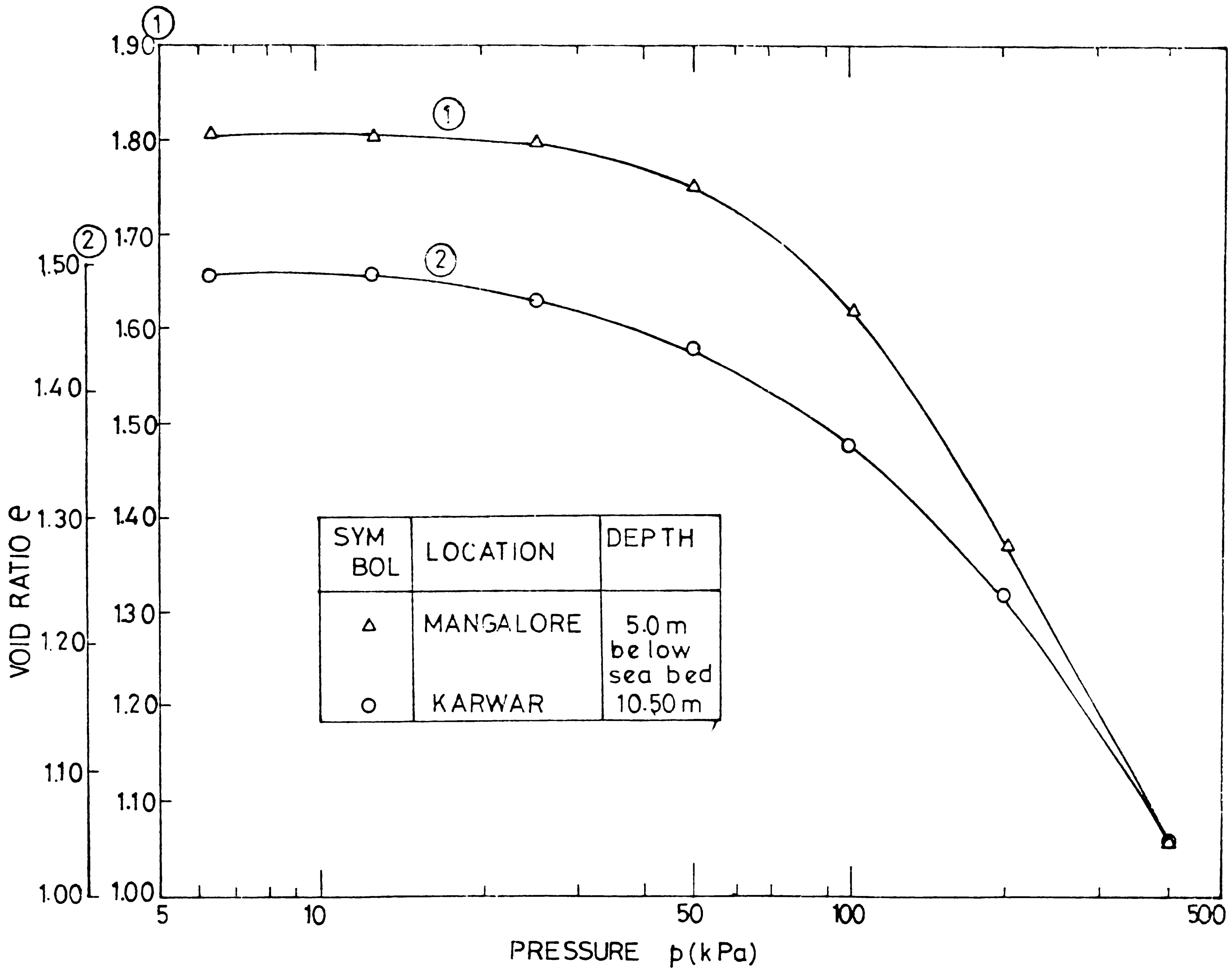


Fig.4.2.32 e-log p CURVES - CLAYS FROM MANGALORE AND KARWAR

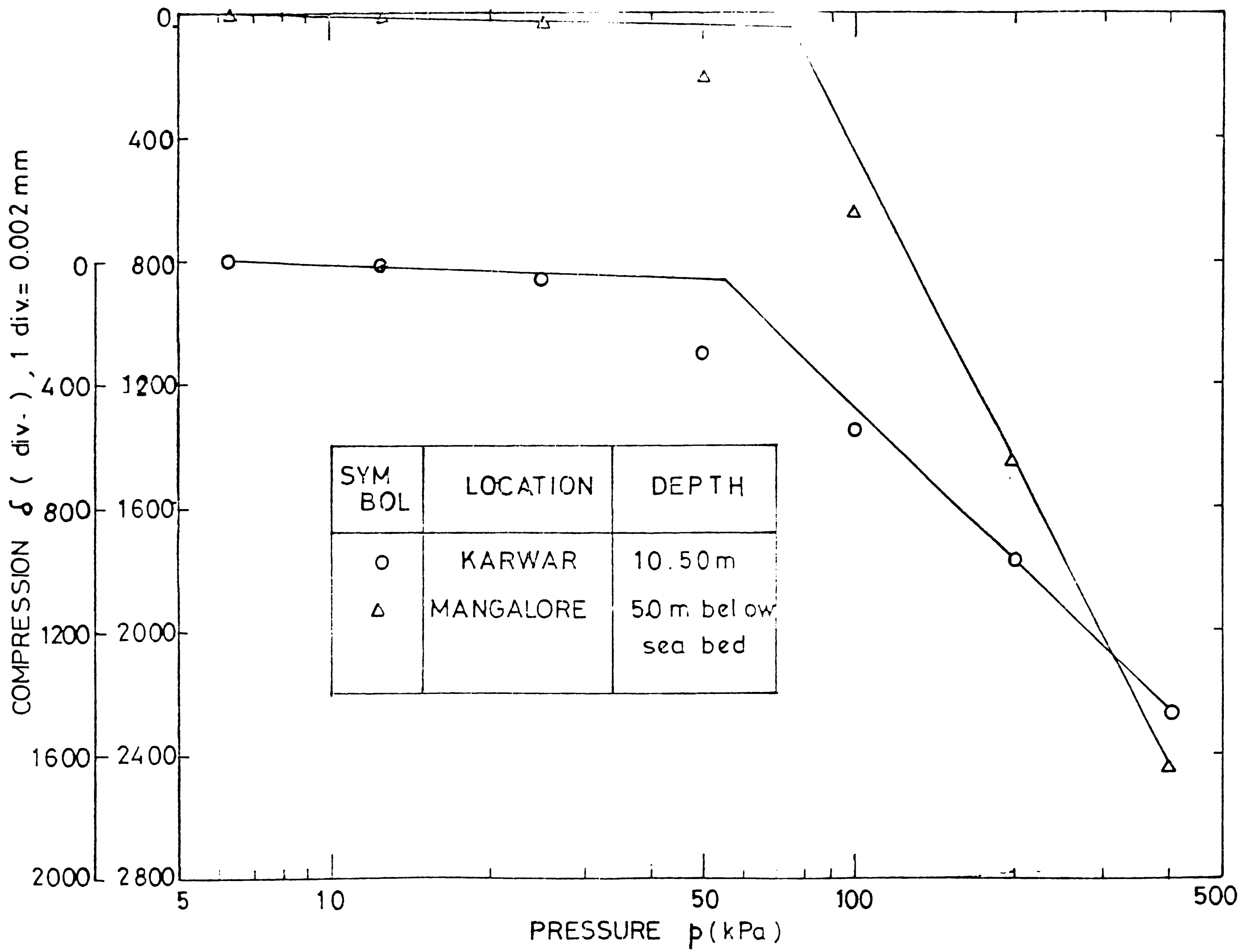


Fig.4.2.33 δ -log p PLOTS - CLAYS FROM MANGALORE AND KARWAR

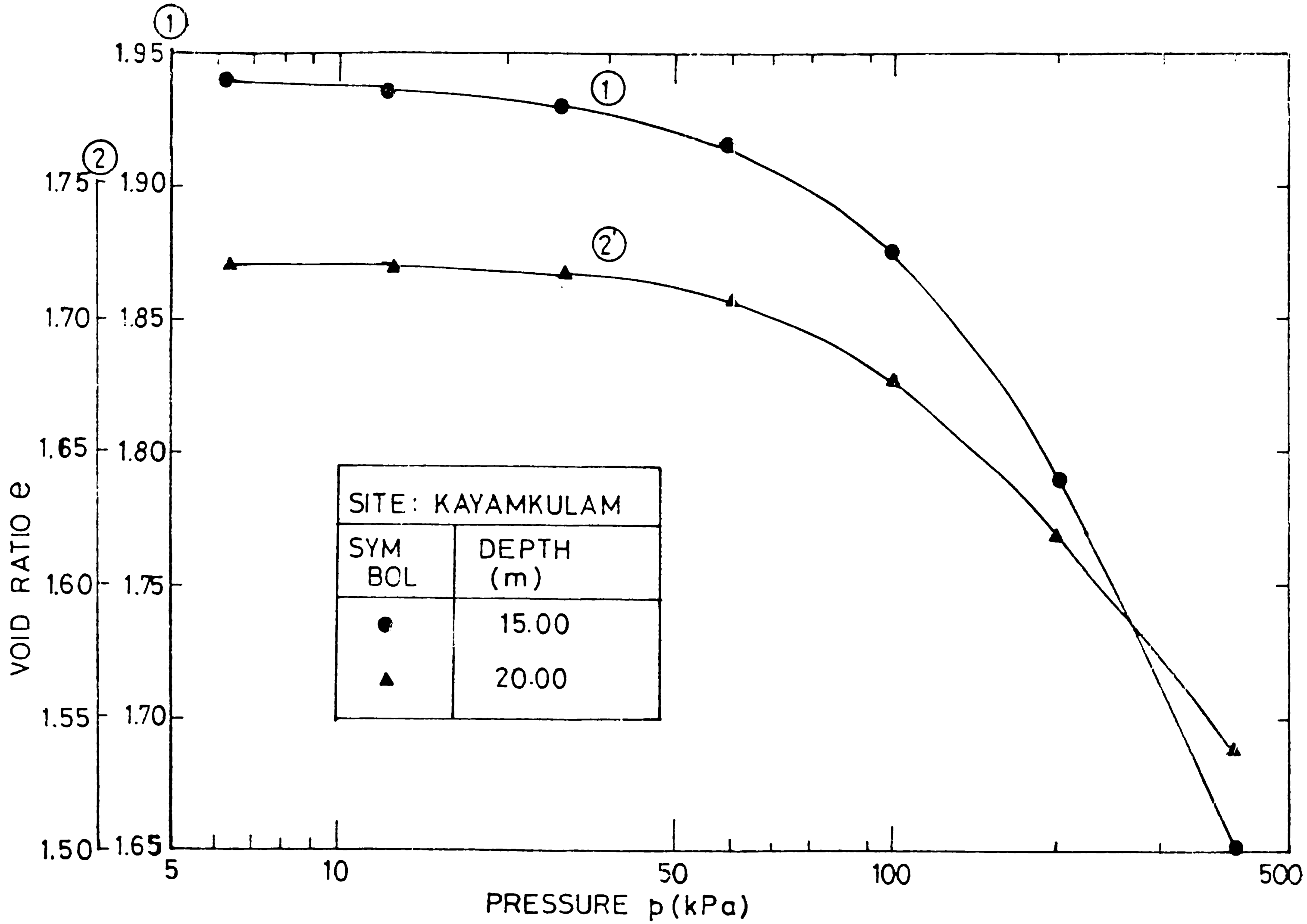


Fig.4.2.34 e-log p CURVES - MARINE CLAY FROM KAYAMKULAM

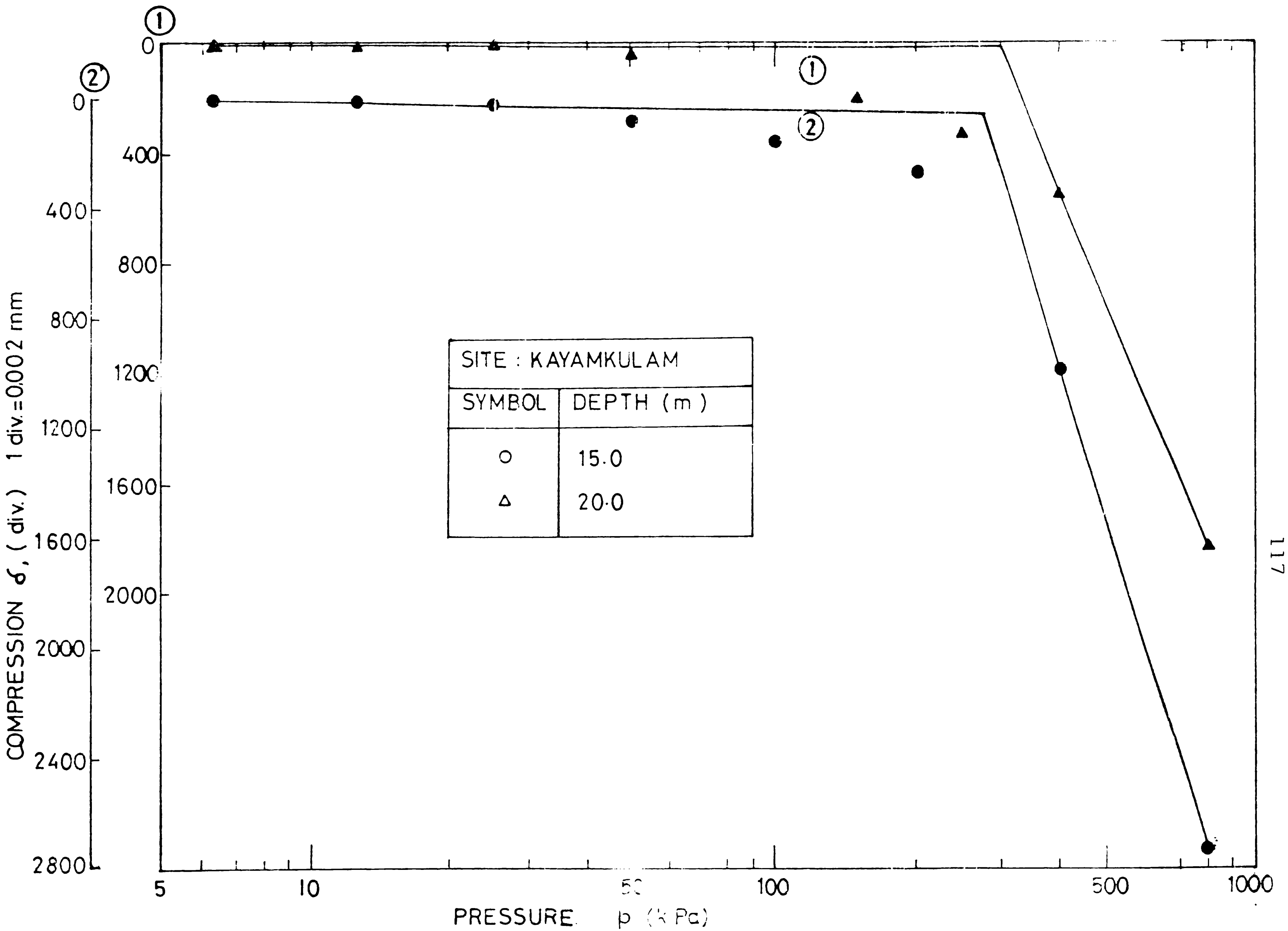


Fig.4.2.35 δ -log p PLOTS - UNDISTURBED MARINE CLAY FROM KAYAMKULAM

The e - log p curves for two samples collected from Kayamkulam NTPC site are shown in Fig.4.2.34. All the drawbacks of the Casagrande procedure obviously exist in these cases also. But the same data plotted as δ - log p (Fig.4.2.35) yield p_c values with least effort, but with precision and clarity.

Thus all the three new methods have a special advantage in the case of most of the e - log p curves obtained from routine consolidation tests conducted on undisturbed field samples. Out of these, the δ - log p plots may be singled out for further investigations due to several advantages over the other procedures taken up in this study.

It is felt that some minor modifications in the routine consolidation procedures will help to perfect these techniques. Any e - log p curve consists of a recompression portion which is almost linear, a virgin curve immediately after the p_c value and a region with reverse curvature at pressures far higher than the p_c values. It may be noted that consolidation settlements are computed from compression index, which is the slope of the virgin portion immediately after the p_c value. In the methods suggested, it is this portion, if defined with greater clarity, which can make the values more precise as the recompression portion is linear in almost all cases. In order to fit a straight line to the virgin compression curve, it is felt that more number of points in this zone will make the straight line more unambiguous. Hence it is desirable to have points at closer intervals in the virgin compression portion than those obtained now by keeping the load increment ratio equal to one. Thus the methods suggested can yield still better values with the suggested modifications in the test procedures.

4.3 Precompression of clays through preloading

Large tracts of poor subsoil including many that are partially or totally submerged are found in most of the port cities of the world and Cochin is no exception. The rapidly increasing demand for more and more habitable area have made it imperative to build on these soft soils too. While piling, quite often to depths ranging over 40 to 50 m, provides an answer to the highrise buildings and heavy structures in Cochin, ground improvement techniques perhaps provide the only economical solution for light and medium constructions. Precompression, wherein the probable consolidation settlement is achieved at an accelerated pace through preloading well before the construction is taken up, has been widely used in areas comprising of soft clays. This technique not only helps to eliminate or curtail the total and differential settlements, but simultaneously imparts considerable improvement in the shear strength characteristics of the soil.

4.3.1 The principle of precompression

The design or plan for the precompression stems from the concept of applying a temporary load in excess of permanent load thereby causing more consolidation to occur than caused by permanent load. Since the temporary load in excess accelerates the consolidation process, the total consolidation settlement can be achieved ahead of the time required for the consolidation under permanent load; eventhough only partial consolidation is permitted under the excess surcharge. When the surcharge is removed, due to the fact that only partial consolidation has been permitted, the centre of the clay layer is still only partially consolidated even under the permanent load, while for the layers closer to the drainage faces, the consolidation is complete. Consequently, when the surcharge is removed, the central layers which are underconsolidated even for the permanent load, would consolidate further and the outer layers, which are over consolidated would tend to rebound or swell. Since the consolidation settlements, still to take place, are higher than the swells experienced, the net effect will normally be additional consolidation settlement. The principles involved in the design of precompression of a clay layer will consist of counter balancing these two so that additional settlements are avoided once the surcharge loads are removed and construction is completed. Thus the parameters involved in the design of precompression will consist of

1. amount of primary consolidation
2. amount of secondary consolidation to be eliminated
3. thickness of the compressible layer
4. degree of consolidation at the centre of the clay layer
5. the equilibrium void ratio at the centre of the clay layer
6. swell when surcharge is removed
7. the overloading ratio
8. the time available for precompression
9. the economy

The principles involved in the preloading technique is shown in Fig.4.3.1. If Δf is the total consolidation settlement for permanent load and $\Delta f+s$ is the total consolidation for the permanent and surcharge loads, and U_{f+s} is the degree of consolidation required for zero settlement (Johnson, 1970),

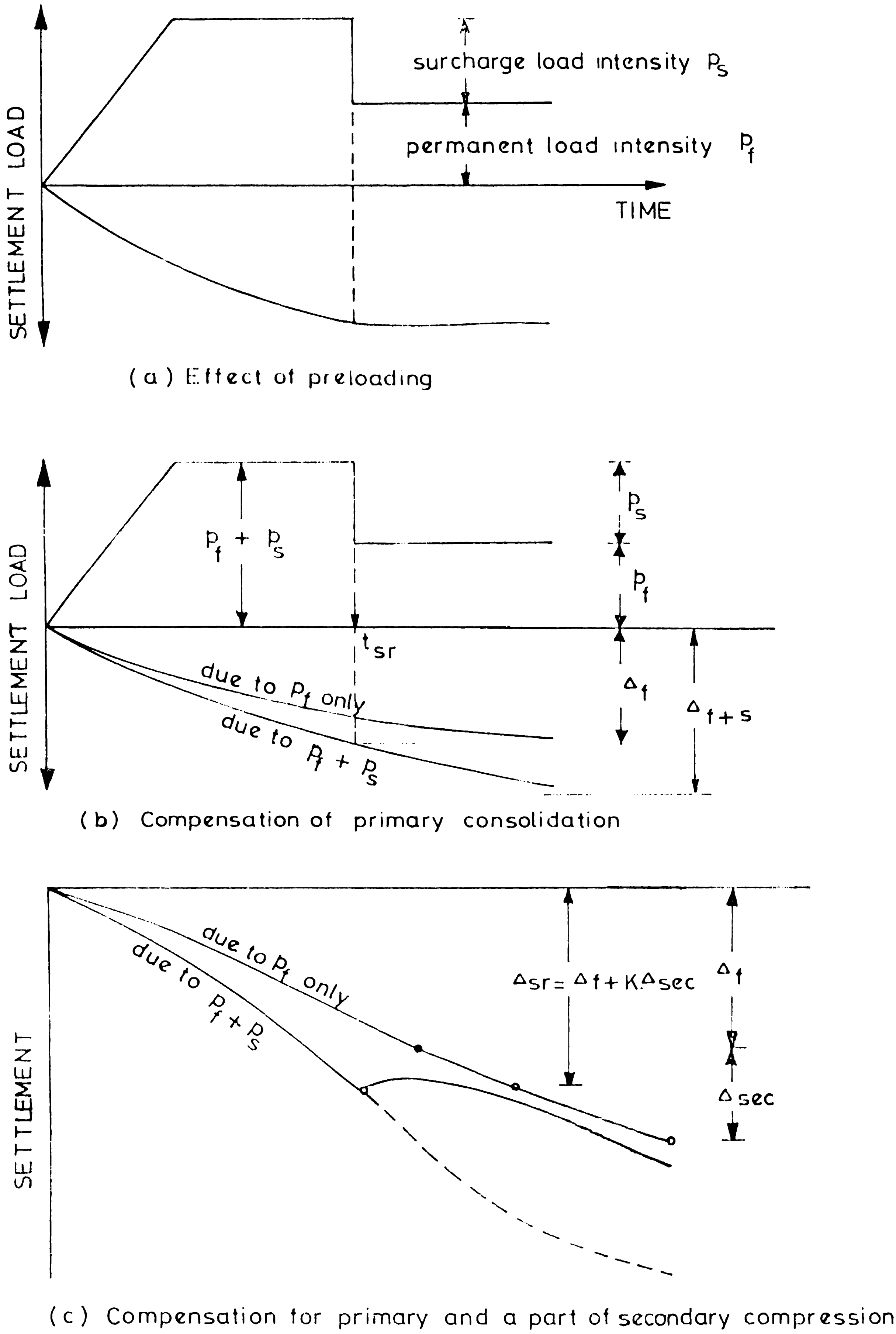


Fig.4.3.1 PRINCIPLE OF PRECOMPRESSION DESIGN

$$U_{f+s} \cdot \Delta_{f+s} = \Delta_f \quad (1)$$

or $$U_{f+s} = \frac{\Delta_f}{\Delta_{f+s}} \quad (2)$$

For permanent load,

$$\Delta_f = \frac{C_c \cdot H}{1+eo} \log \frac{P_o + P_f}{P_o} \quad (3)$$

Where, H is the thickness of clay layer

P_o - effective stress

P_f - increase in stress due to permanent load

For permanent load + surcharge load,

$$\Delta_{f+s} = \frac{C_c \cdot H}{1+e_o} \log \frac{P_o + P_f + P_s}{P_o} \quad (4)$$

Therefore for any particular degree of consolidation for the combined load

$$U_{f+s} = \frac{\Delta_f}{\Delta_{f+s}} = \frac{\log \frac{P_o + P_f}{P_o}}{\log \frac{P_o + P_f + P_s}{P_o}} \quad (5)$$

or

$$U_{f+s} = \frac{\log (1 + \frac{P_f}{P_o})}{\log[1 + \frac{P_f}{P_o} (1 + \frac{P_s}{P_f})} \quad (6)$$

Since the surcharge load applied over and above the permanent load controls the accelerated consolidation, the over loading ratio P_s/P_f which is the ratio of surcharge load to permanent load, controls the time required for precompression.

For a clay layer of considerable depth as in the case of marine clay deposits in Cochin area, the equation (6) cannot be used directly as the value of P_o, P_f and P_{f+s} will vary with depth due to surface load at finite areas. The degree of consolidation due to the combined permanent and surcharge loads will vary from layer to layer. Further, the time for the application of the surcharge is to be estimated by,

$$t_{sr} = T_v \, d^2/c_v \qquad (7)$$

where d is the length of drainage path.

Equations (6) and (7) show that the time for application of surcharge load for a particular degree of consolidation at the centre of a particular clay layer is controlled by the overconsolidation ratio P_s/P_f for that layer. Thus the two parameters which are important in precompression design are (1) the overloading ratio - ratio of surcharge load to the permanent load and (2) the time that can be permitted. The more the surcharge we put, the less will be the time required. An optimisation between these two parameters is the main criterion in the precompression design.

The precomposition should also account for the secondary compression due to the permanent load, which is given by,

$$\Delta_{sec} = C_{\alpha\epsilon} H \log t/t_p$$

where t - time at which Δ_{sec} is to be computed

t_p - time at which primary consolidation has been completed

Thus the settlement that should be achieved before surcharge load is removed can be expressed as,

$$\Delta_{sr} = \Delta_f + K.\Delta_{sec}$$

where K - the percentage of secondary compression which is to be achieved as primary consolidation by the surcharge load.

4.3.2 Laboratory studies on precompression

The principles of precompression and the parameters that influence the design, planning and execution of precompression have been discussed above. An indepth study into all aspects does not fall within the scope of the present study. But certain aspects of precompression were taken up for investigations vis-a-vis stabilisation of marine clays.

The effect of surcharge load applied and removed after a certain degree of consolidation is achieved, on consolidation of Cochin Marine clays is presented in Fig.4.3.2. It shows the full consolidation curve for the load increments 100-150 and 100-200 k Pa. As discussed in the above section, 100 k Pa can be taken as the effective stress (overburden pressure) at the centre of the clay layer and 50 k Pa is the permanent additional load coming on the clay layer. The primary and secondary consolidation settlements upto 7 days for a total load (overburden + permanent load) of 150 k Pa is 260 divisions approximately. faster rate of consolidation can be achieved by adding a surcharge load of 50 k Pa to the above permanent load of 50 k Pa which makes the total load to be applied equal to 100 k Pa. When the load increment is from 100-200 k Pa, the compression curve obtained is presented in the figure. The total compression is 590 divisions for the same period of 7 days where we can assume that the full consolidation has been achieved. In order to study the pattern of δ -log t curves,

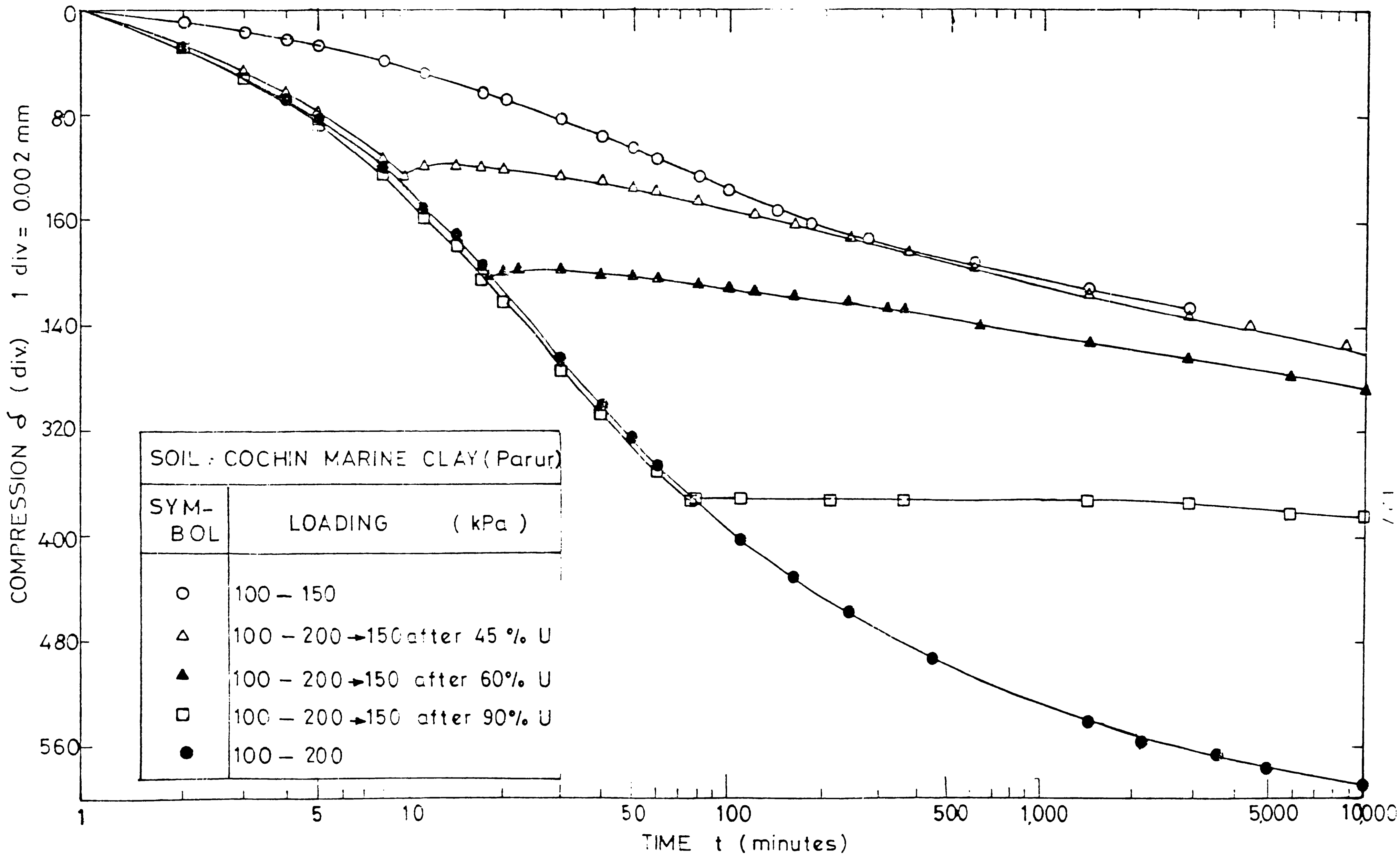


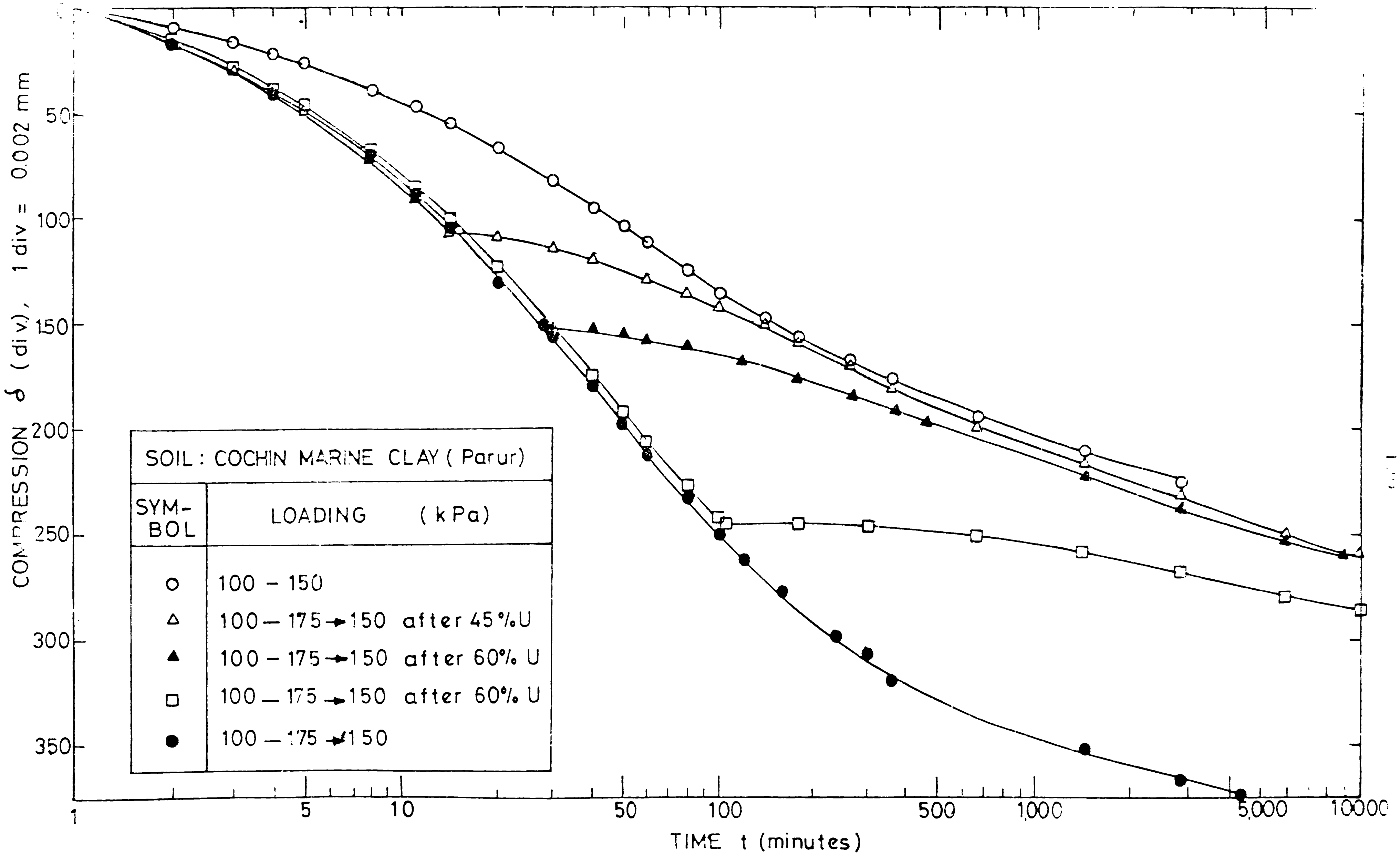
Fig.4.3.2 δ -log t CURVES FOR PRECOMPRESSED COCHIN MARINE CLAY

when the surcharge load is released before the consolidation is complete, three consolidation tests were carried out wherein during the load increment of 100-200 k Pa, the surcharge load of 50 k Pa was removed when the degrees of consolidation achieved were 45, 60 and 90%. It can be seen from the figure that the primary compression for 150 k Pa has been almost fully achieved even for the curve where U = 45%. The slope of the secondary compression portion becomes flatter as the degree of consolidation increases. In this case, the overloading ratio (P_s/P_f) is one.

It can be seen from Fig.4.3.2 that the full preconsolidation of about 260 divisions, which took 300 minutes for normal loading could be achieved in full within 9 minutes for an overloading ratio of 1. The tremendous advantage associated with preloading is quite obvious from the above.

But it can be seen from the pair of curves that there is no reduction in the secondary consolidation as the slopes in the latter portion are the same. The situation does not improve even when U = 60%. But for U = 90%, the curve is almost horizontal indicating that the settlement due to secondary consolidation is very negligible.

Fig.4.3.3 shows the δ -log t curves for an overburden pressure of 100 k Pa and P_f = 50 k Pa. The



SOIL : COCHIN MARINE CLAY (Parur)
SYM-BOL
LOADING (kPa)
100 – 150
100 — 175 → 150 after 45% U
100 – 175 → 150 after 60% U
100 — 175 → 150 after 60% U
100 — 175 → 150
COMPRESSION δ (div), 1 div = 0.002 mm
TIME t (minutes)

surcharge load is 25 k Pa which gives an overloading ratio of 0.5. It can be seen from the figure that primary consolidation has been fully achieved even by U = 60% curve. But, the secondary compression is almost the same for the first three curves. It becomes flatter for the U = 90% curve. The curves show that when overloading ratio and degree of consolidation are small, the reduction in secondary compression is almost insignificant.

The results of precompression studies carried out on Mangalore marine clays are presented in Figs. 4.3.4 and 4.3.5. In the former, the overloading ratio is 1.0 and for the latter, it is 3.0. It can be seen from the two figures that overloading ratio influences the coefficient of secondary consolidation considerably. For a 100 → 200 → 150 k Pa cycle, the values of $C'_{\alpha\epsilon}$ for U = 50, 70 and 90% are 6.9, 3.66 and 0.64 respectively. For the cycle of 100 → 200 → 125 k Pa with an overloading ratio of 3.0, the values of $C'_{\alpha\epsilon}$ for U = 50, 70 and 90% are 4.45, 0.85 and 0.14 respectively. The effect of overloading ratio is more pronounced for higher degrees of consolidation. It can be seen that the above points are valid for Cochin marine clays also.

The δ -log t curves of Cochin marine clay for a 100 → 300 → 150 k Pa cycle (overloading ratio = 3.0) for

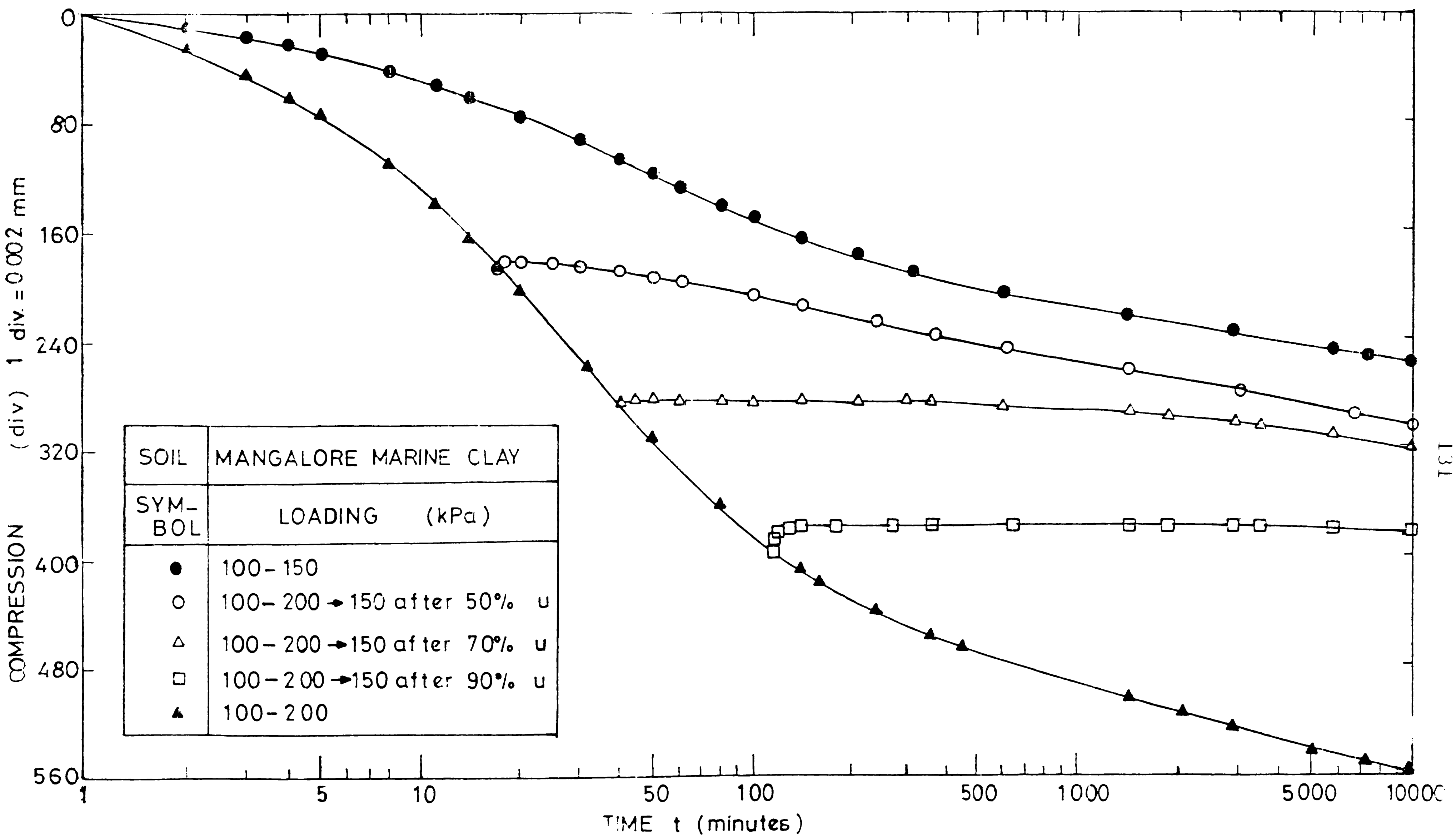


Fig.4.3.4 δ-logt CURVES FOR PRECOMPRESSED MANGALORE MARINE CLAY

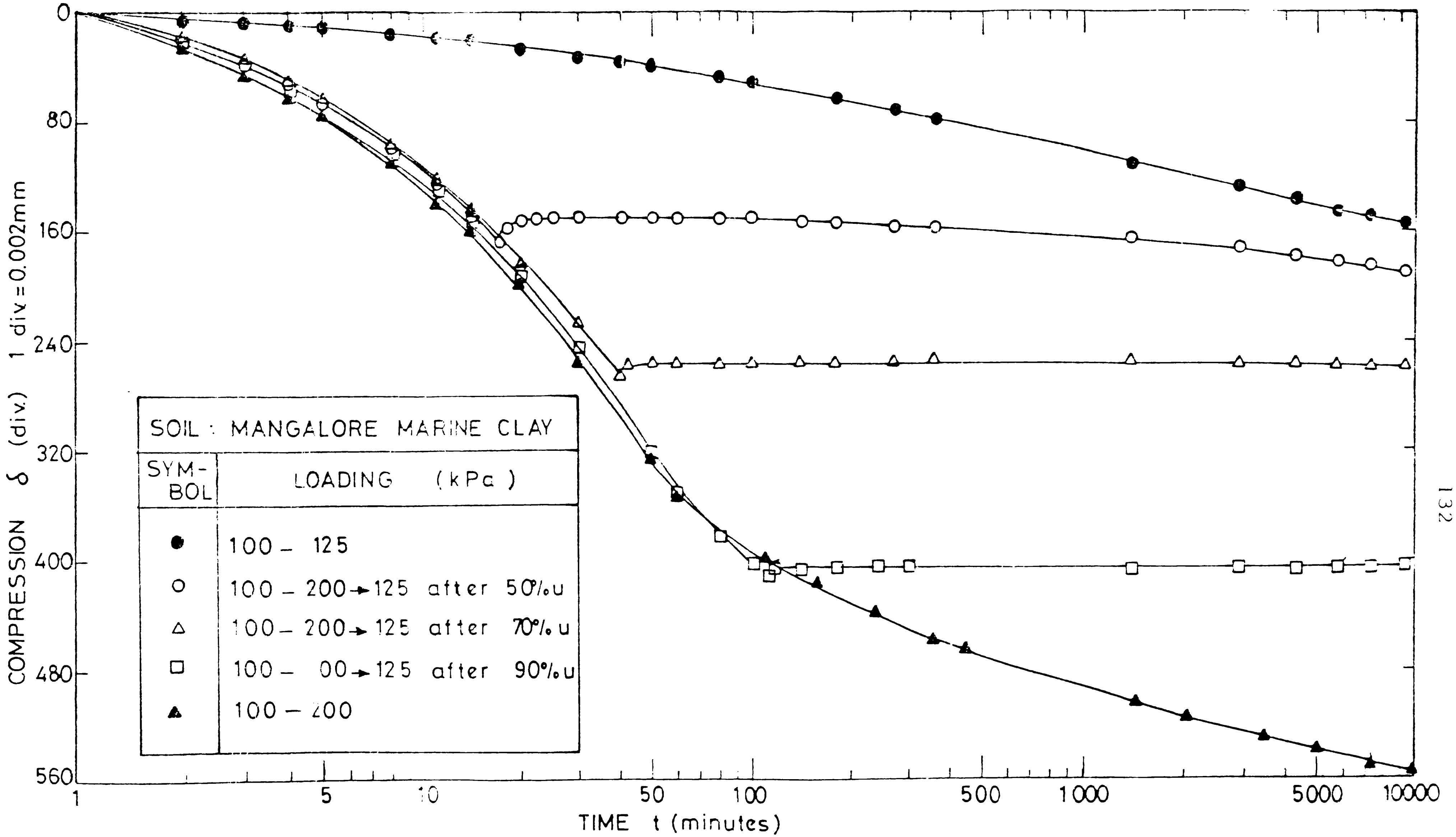


Fig.4.3.5 δ -log t CURVES FOR HIGHER OVERLOADING RATIO

U = 90% is shown in Fig.4.3.6. How a higher overloading ratio combined with a higher degree of consolidation almost totally eliminates both the consolidation settlements - primary as well as secondary, can be seen from the figure. The $C'_{\alpha\epsilon}$ is almost zero and there is a pronounced rebound when the surcharge load was removed after U = 90%.

Fig.4.3.7 shows the δ -log t curves for extremely high overconsolidation ratios. The marine clay samples from Mangalore consolidated at 100 k Pa was given a surcharge of another 100 k Pa. In three consolidation tests, the load was reduced to 110 k Pa (overloading ratio = 10) at three different degrees of consolidation equal to 50, 70 and 90%. It can be seen from the figure that all the three plots are parallel to each other and the secondary consolidation is almost zero.

Out of the nine parameters which could possibly influence the precompression behaviour of marine clays, it has been already shown that the parameters which will help to arrive at a design or plan for precompression are the overloading ratio and the degree of consolidation. A particular result could be arrived at by combining higher overloading ratios with lower degrees of consolidation, and vice-versa. In order to study the behaviour of clay layer

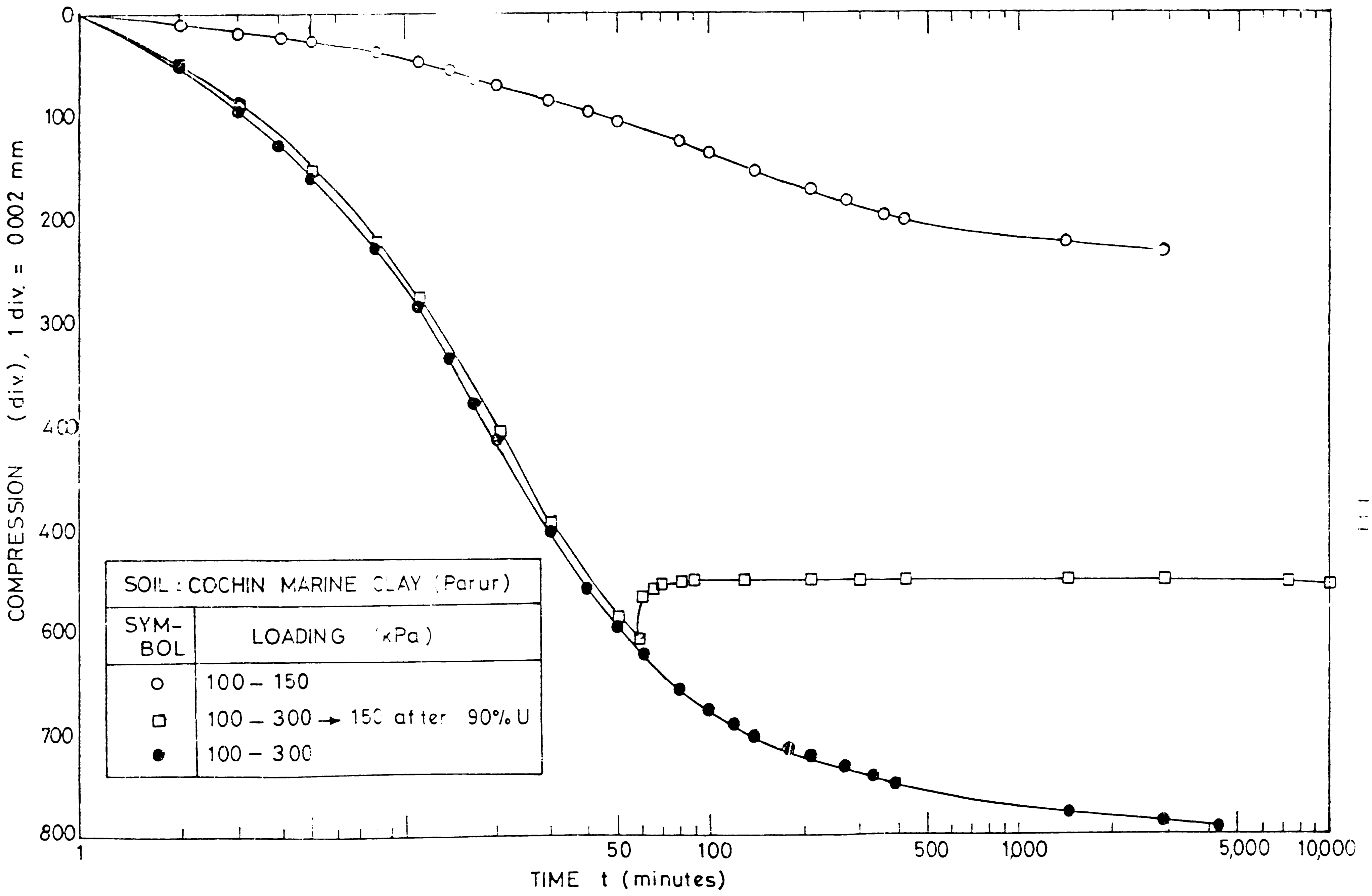


Fig.4.3.6 δ -log t CURVES FOR HIGHER OVERLOADING RATIO

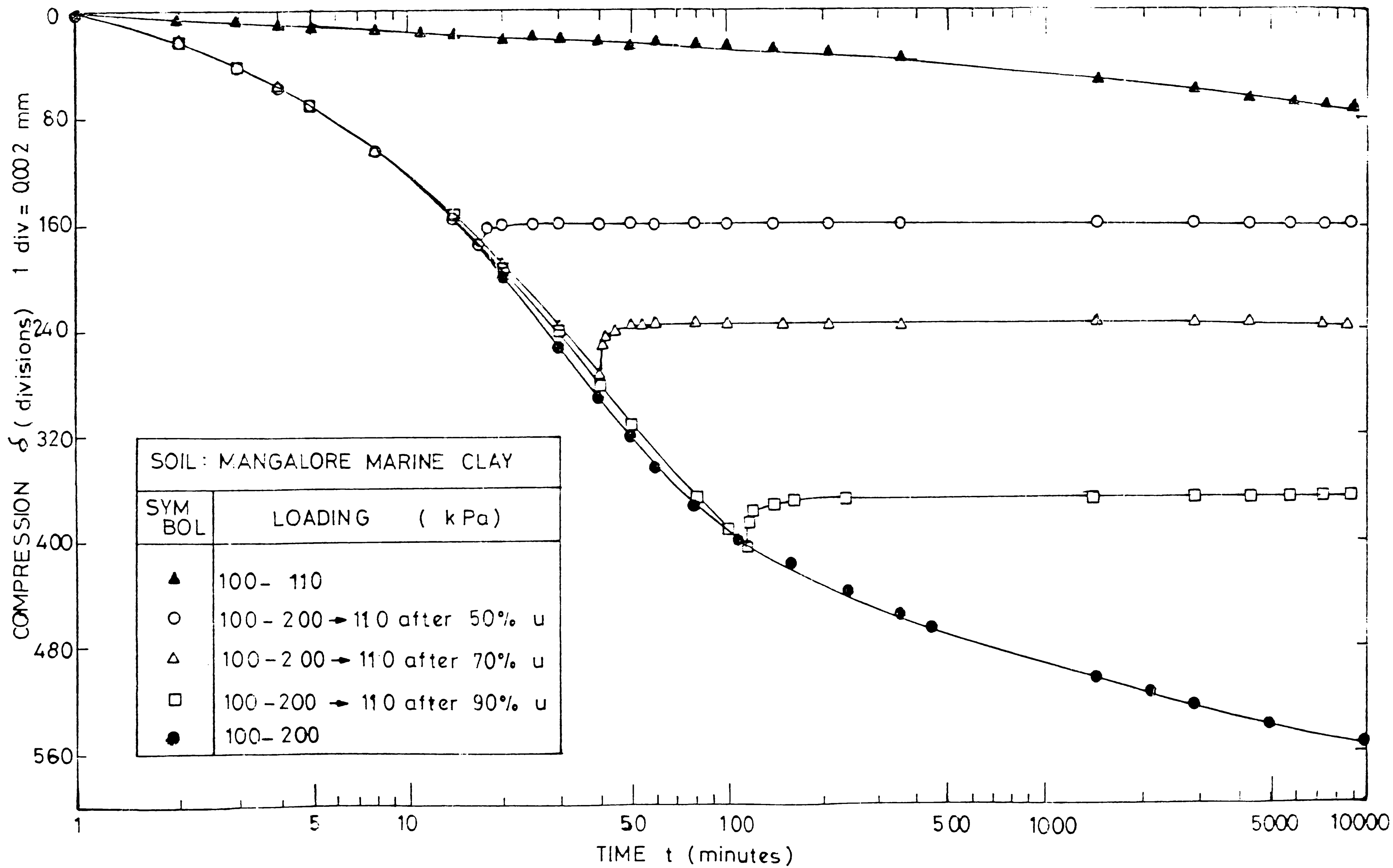
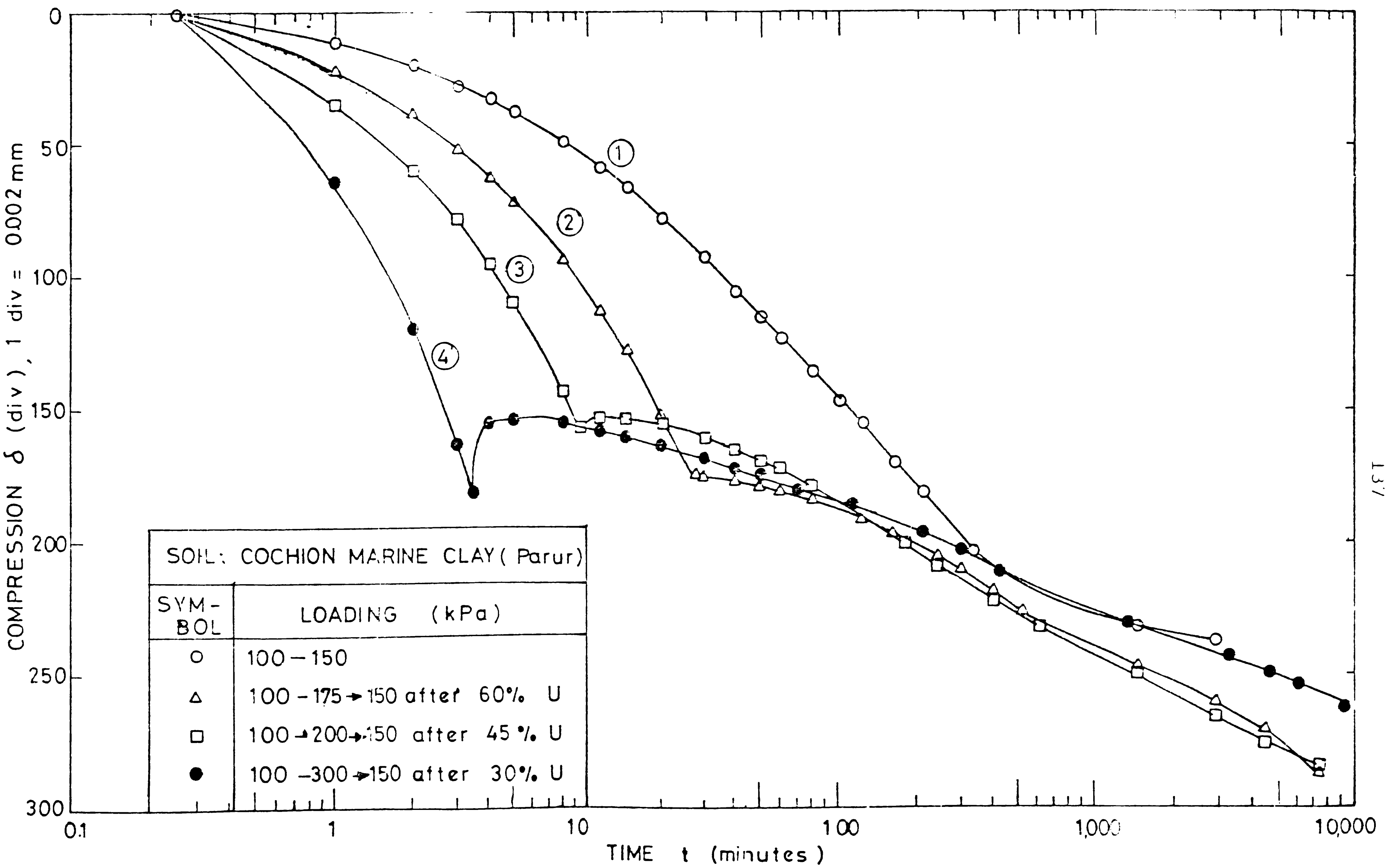


Fig.4.3.7 δ -log t CURVES FOR VERY HIGH OVERLOADING RATIO

subjected to precompression, a set of three consolidation tests were conducted whose results are presented in Fig.4.3.8 curve (1) shows the results of a routine consolidation test for a pressure increase from 100 to 150 k Pa. Curve (2) shows the results of a test wherein the pressure was increased from 100 to 175 k Pa and then reduced to 150 k Pa after realising 60% of the primary consolidation for the pressure increase 100-175 k Pa, ie., for a degree of consolidation of 60%. Here the overloading ratio is 0.5. Curve (3) shows the δ -log t plot for a pressure increase of 100 → 200 → 150 k Pa for U = 45% with overloading ratio equal to 1.0. Curve (4) is the δ -log t relationship for the pressure increment of 100 → 300 → 150 k Pa with U = 30% and overloading ratio of 3.0. It can be seen that the later portions of δ -log t curves merge with or fall close to curve No.(1).

The above figure shows that the variation in δ for the portion of the curves representing consolidation after precompression is negligible and even this variation can be easily eliminated by either a small variation in degree of consolidation or overconsolidation ratio. While in the test for curve (2) the surcharge was removed after 30 minutes, in case of curve (3) it was 9.5 minutes. For curve (4), it is just 3.5 minutes.



... CURVES FOR PRECOMPRESSION DESIGN

The above set of curves give a clue to the data required for the design of precompression. If δ -log t curves are made available for a particular clay at different pressure levels for different overloading ratios and for different degrees of consolidation, one can easily arrive at a precompression design wherein he can choose a combination of overloading ratio and degree of consolidation depending upon the time available at his disposal.

4.3.3 Consolidation of precompressed marine clays

A typical δ -log t curve for a precompressed specimen of Cochin marine clay for a degree of consolidation of consolidation of 60% is shown in Fig.4.3.9. The sample was consolidated initially at 100 k Pa, precompressed partially at 200 k Pa and further consolidated at 150 k Pa. Such a curve can have four distinct portions. The initial portion (1) shows the routine compression curve for a pressure increment of 100-200 k Pa. Till U = 60%, this should obviously follow the δ - log t curve for 100-200 k Pa.

When the surcharge load is released and consolidation pressure is brought down to 150 k Pa from 200 k Pa, an elastic rebound may be shown by the clay especially at higher degrees of consolidation. This will take place within a

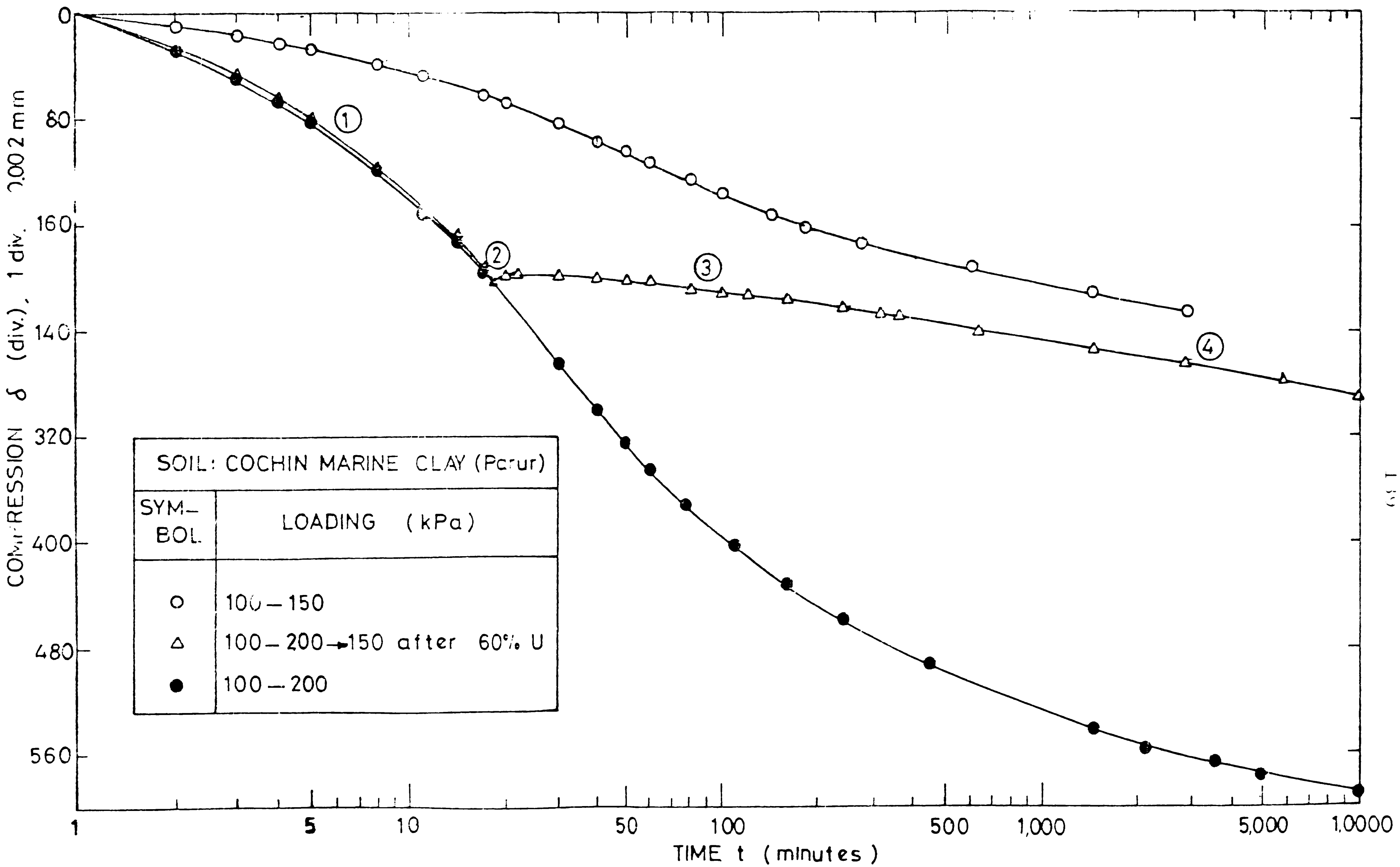


Fig.4.3.9 EFFECT OF OVERLOADING AND SURCHARGE REMOVAL

short while and could be formed as portion (2). Thereafter, for a certain period, the consolidation curve may represent a combination of primary and secondary consolidation. This is due to the fact that for a major portion of the clay, $U < 100\%$, where primary consolidation continues to take place. For the portions near the drainage surfaces, secondary consolidation will be taking place as the degree of consolidation is equal or close to 100%. This portion is designated as (3) in the figure. The last portion which may represent the secondary consolidation exclusively is represented as (4).

In the following discussions, the slope of the δ - log t curve in the portion (4) is designated as $C'_{\alpha\epsilon}$ and the ratio of change in void ratio from the point of surcharge removal to the final time reckoned, to the consolidation pressure applied for the portions (3) and (4) is termed as, $C'_c[de/d(\log p)]$

How the value of $C'_{\alpha\epsilon}$ varies with degree of consolidation and overconsolidation ratios is shown in Fig.4.3.10. As expected, $C'_{\alpha\epsilon}$ decreases with increase in either degree of consolidation or overloading ratio. The figure also shows that deformations can even be almost totally eliminated by choosing higher overloading ratios and

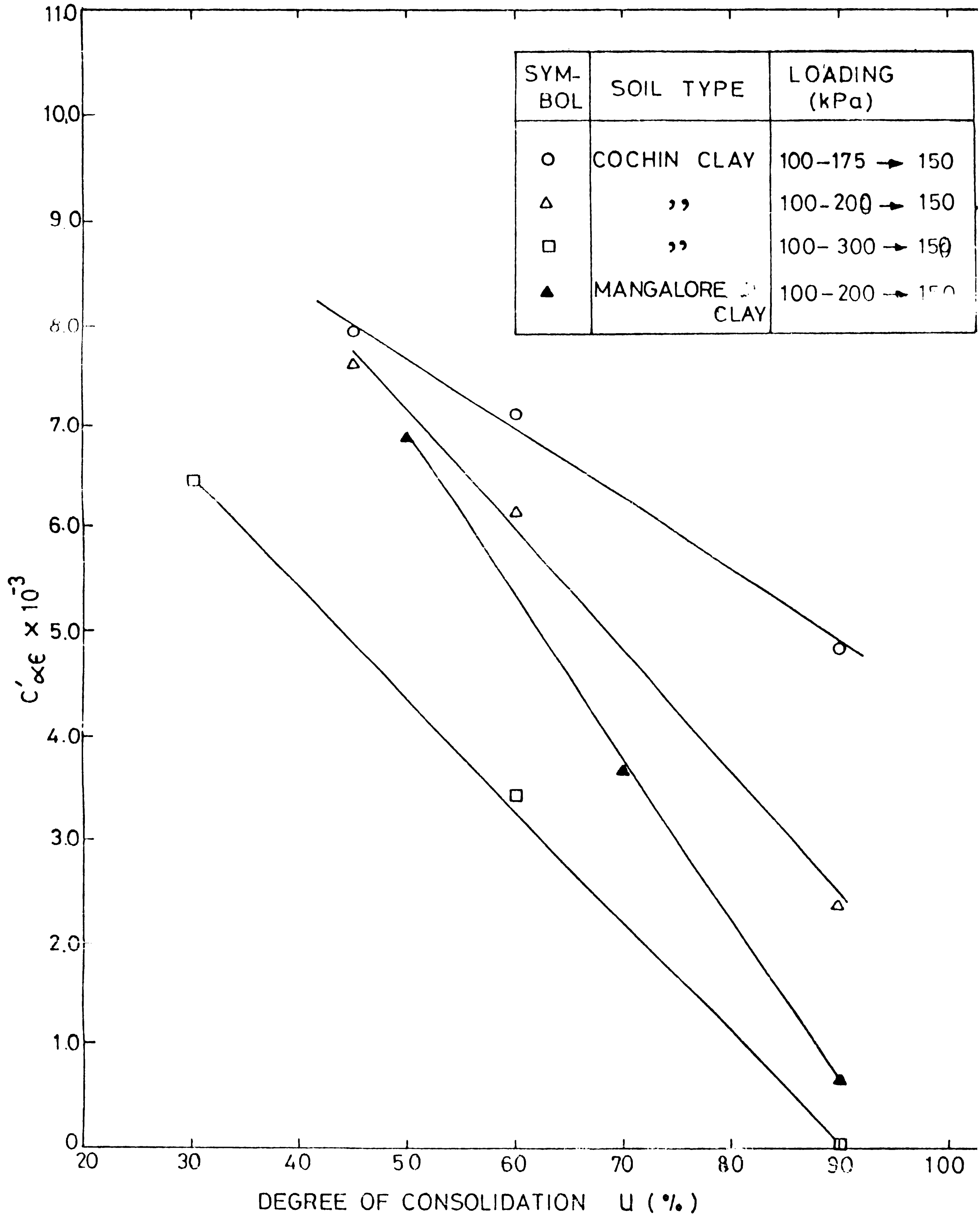


Fig.4.3.10 EFFECT OF PRECOMPRESSION ON SECONDARY CONSOLIDATION

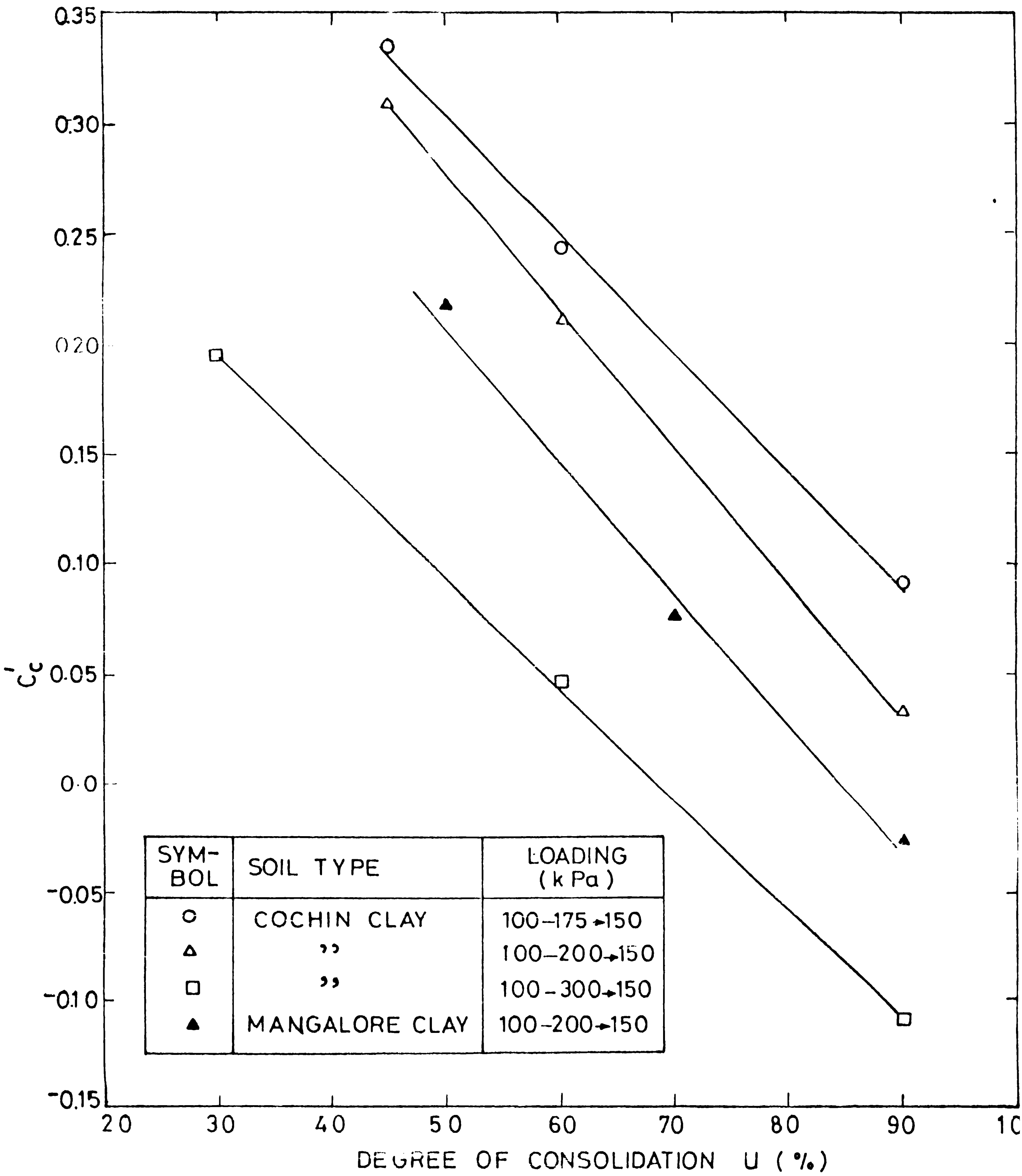


Fig.4.3.11 IMPROVEMENT IN COMPRESSIBILITY DUE TO PRELOADING

Table 4.3.1

Variation of $C'_{\alpha\epsilon}$ and C'_c with overloading ratio and degree of consolidation

Sl. No.	Soil type	Loading Details (k Pa)	Overloading ratio	$C_{\alpha\epsilon}$ or $C'_{\alpha\epsilon}$ $x(10^{-3})$	$\frac{de}{d(\log p)}$ or C'_c
1.	Cochin marine clay	100 - 150	--	6.32	0.687
2.	"	100 - 175 - 150 after 45% u	0.5	7.93	0.335
3.	"	" 60% u	0.5	7.11	0.243
4.	"	" " 90% u	0.5	4.82	0.91
5.	"	100 - 200 - 150 after 45% u	1.0	7.59	0.31
6.	"	' ' 60% u	1.0	6.12	0.213
7.	"	" " 90% u	1.0	2.34	0.031
8.	"	100 - 300 - 150 after 30% u	3.0	6.46	0.196
9.	"	" " 60% u	3.0	3.45	0.047
10.	"	" " 90% u	3.0	0.0078	-0.109
11.	Mangalore marine clay	100 - 150	--	6.33	0.625
12.	"	100 - 200 - 150 after 50% u	1.0	6.90	0.217
13.	"	" " 70% u	1.0	3.66	0.075
14.	"	" " 90% u	1.0	0.64	-0.028

(contd...)

Sl. No.	Soil type	Loading Details	Overloading ratio	$C_{\alpha\epsilon}$ or $C'_{\alpha\epsilon}$ x(10^{-3})	$\frac{de}{d(\log p)}$ or C'_c
15.	Mangalore marine clay	100-125	--	7.84	0.690
16.	"	100 - 200 - 125 after 50% u	3.0	4.45	0.061
17.	"	" 70% u	3.0	0.853	0.0058
18.	"	" 90% u	3.0	0.14	-0.0057
19.	"	100 - 190	--	--	0.829
20.	"	100 - 200 - 190 after 50% u	0.11	7.42	0.491
21.	"	" 70% u	0.11	5.85	0.245
22.	"	" 90% u	0.11	5.44	0.133

degrees of consolidation. For example in the case of the precompression cycle 100 → 300 → 150 k Pa, $C'_{\alpha\epsilon}$ is very close to zero (0.0078×10^{-3}), for U = 90%. Table 4.3.1 presents the values of $C'_{\alpha\epsilon}$ for a number of precompression tests for different combinations of overloading ratios and degrees of consolidation for marine clays from Cochin and Mangalore.

The variation of C'_c with respect to degree of consolidation for different overloading ratios are presented in Fig.4.3.11. They are akin to $C'_{\alpha\epsilon}$. The values are presented in Table 4.3.1 also. The table shows that the value of C'_c which was 0.687 for the load increment 100-150 k Pa in the routine consolidation test reduces to 0.031 for an overloading ratio of 1.0 and a degree of consolidation of 90%.

The non-technical parameters which control the design of precompression project are the time available for precompression and the expenditure that can be incurred. Since the time required to achieve the required precompression depends exclusively on the value of coefficient of consolidation, an accurate estimation of the value of C_v should receive prime consideration. Unfortunately the value of C_v is more or less treated as a constant in settlement computations. But the coefficient of consolidation depends on a number of parameters viz., preconsolidation pressure, stress level, pore fluid characteristics, sample disturbance etc. For a good undisturbed sample, C_v is relatively high for stresses less than P_c, decreases as P_c is reached and increases thereafter. For moderately disturbed samples, C_v is relatively low for stresses less than P_c and increases as P_c is exceeded (Johnson, 1970).

In the routine consolidation tests, load increment ratio is kept as one. But in case of precompression, the consolidation process is accelerated depending on the time available by overloading the clay layers. This might be substantially high and can influence the value of C_v considerably. Therefore in an investigation on precompression, a study of the parameters influencing C_v value is inevitable. Two factors viz., stress level and load increment ratio were taken up in the present study.

Fig.4.3.12 gives the values of C_v for different consolidation pressures and three load increment ratios of 1.0, 0.7 and 0.4 for an undisturbed sample of Cochin marine clay. It can be seen from the figure that C_v values are not significantly influenced by the consolidation pressure. But

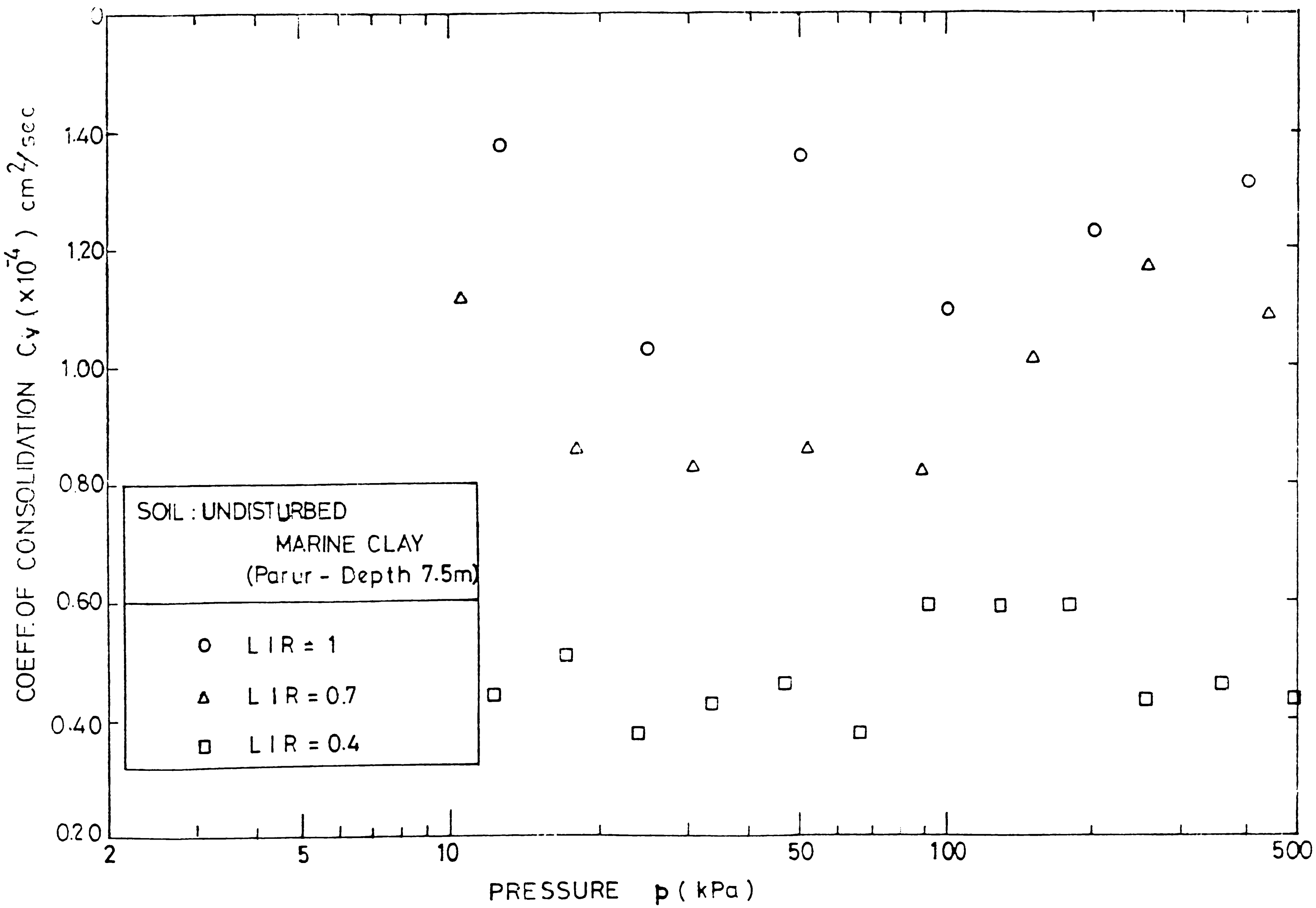


Fig.4.3.12 EFFECT OF LOAD INCREMENT RATIO ON COEFFICIENT OF CONSOLIDATION

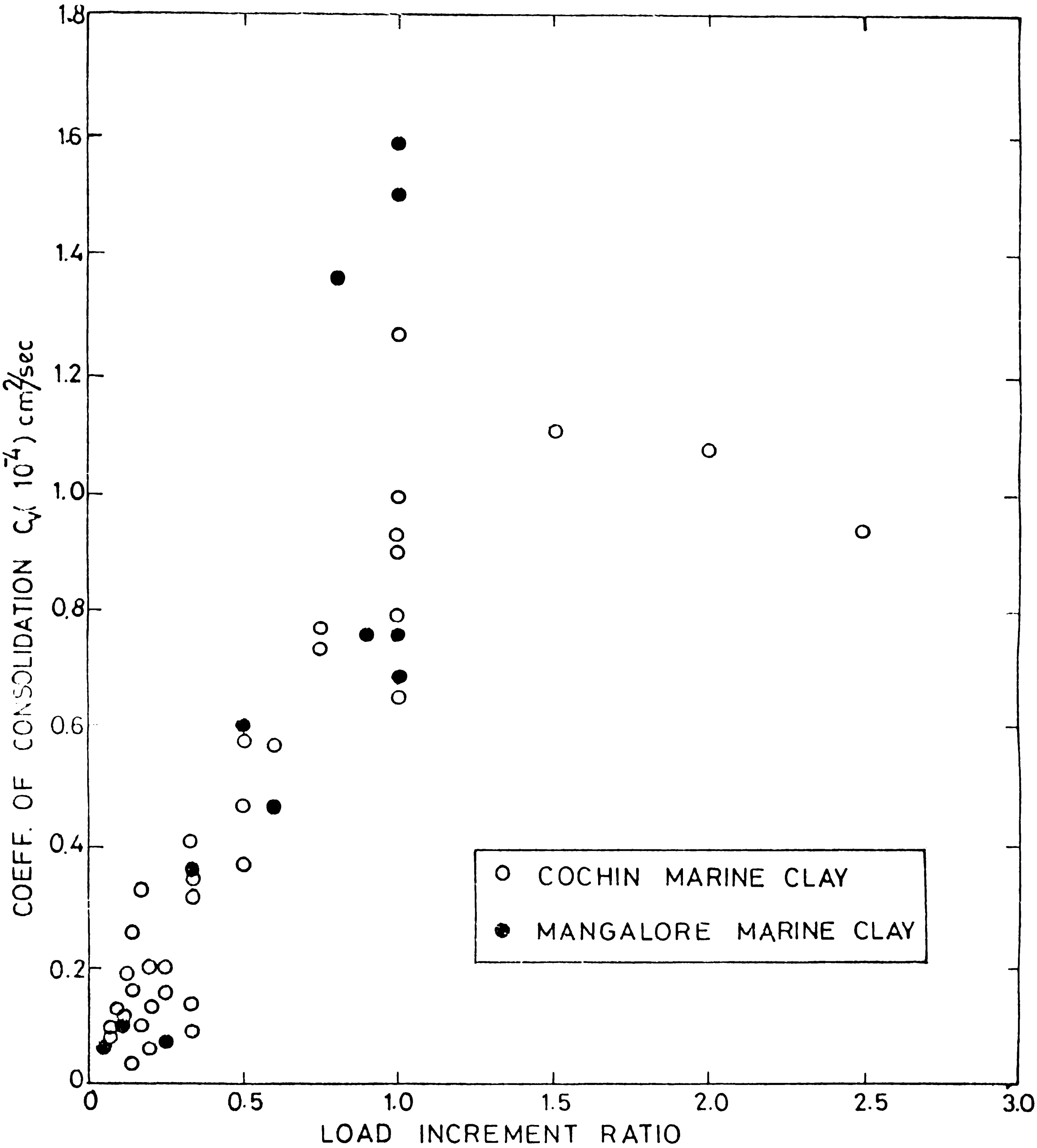


Fig.4.3.13 EFFECT OF LOAD INCREMENT RATIO ON C_v

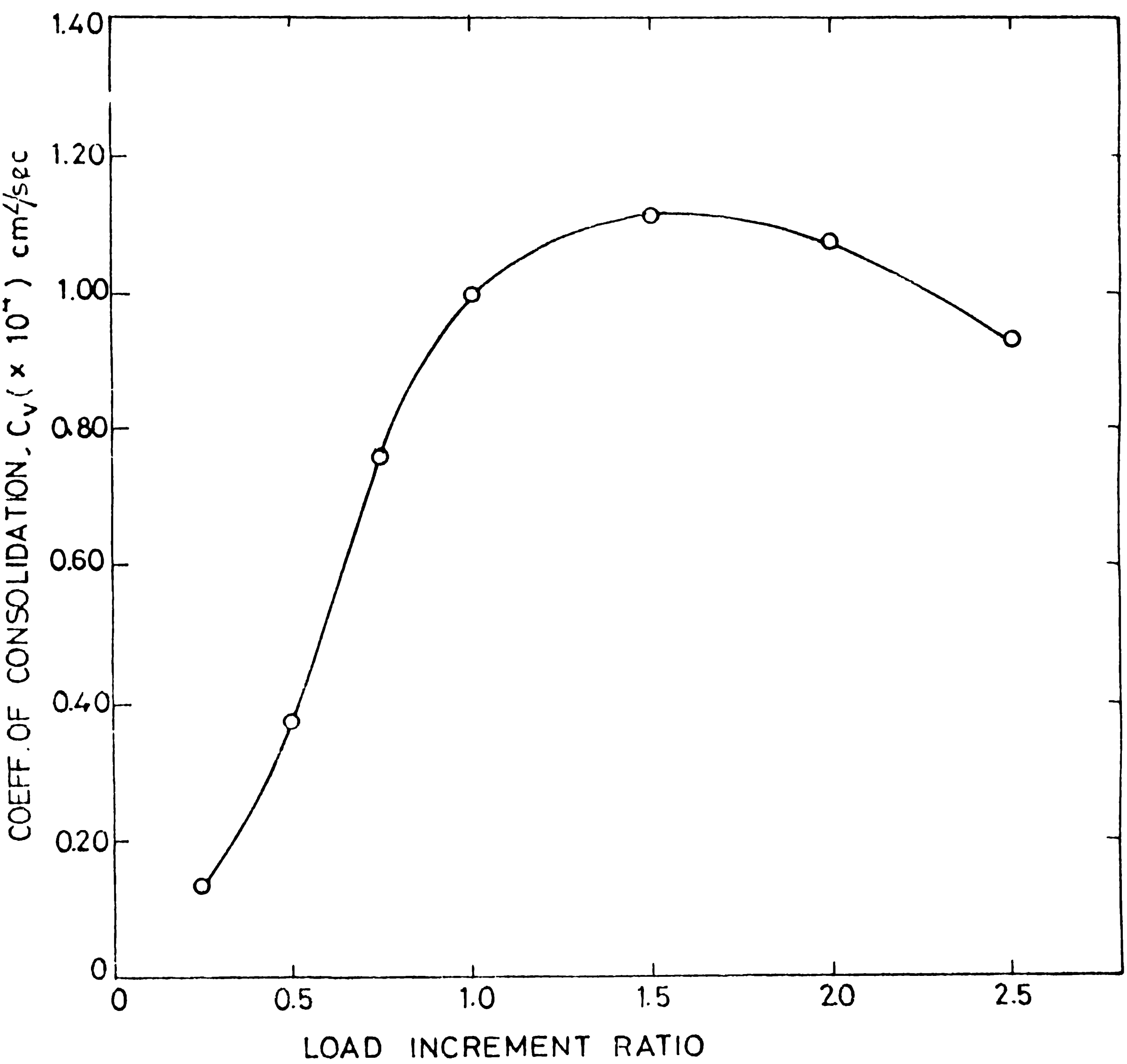


Fig.4.3.14 EFFECT OF LIR ON C_v FOR COCHIN MARINE CLAY

the load increment ratio influences the C_v values to a considerable extent. For LIR = 0.4, the values lie on a narrow band of 0.3×10^{-4} to 0.6×10^{-4} cm^2/sec. When the LIR is increased to 0.7, C_v ranges over 0.8×10^{-4} to 1.2×10^{-4} cm^2/sec. For LIR = 1, C_v values fall within 1.0×10^{-4} to 1.40×10^{-4} cm^2 sec. This shows that when LIR is increased from 0.4 to 1.0, the value of C_v increases by more than 100%.

The relation between C_v and load increment ratio is presented in Fig.4.3.13. It can be seen from the figure that C_v values steadily increase upto LIR = 1.0, but tend to decrease thereafter. In order to find out whether there is a range over which C_v values are highest, C_v values were determined for marine clay samples consolidated at 100 k Pa for load increment ratios in the range of 0.25 to 2.5. The results are presented in Fig.4.3.14. It indicates that the maximum values for C_v are obtained for LIR values around 1.5.

During the series of consolidation tests connected with precompression, it was observed that the time required for 50% and 100% consolidation (t_{50} and t_{100}) are influenced by LIR values. Fig.4.3.15 and 4.3.16 shows the relation between t_{50} and t_{100} with LIR. It can be seen that both the plots show very high values for t_{50} and t_{100} till LIR = 1.0. Thereafter the variations are negligible and steady values

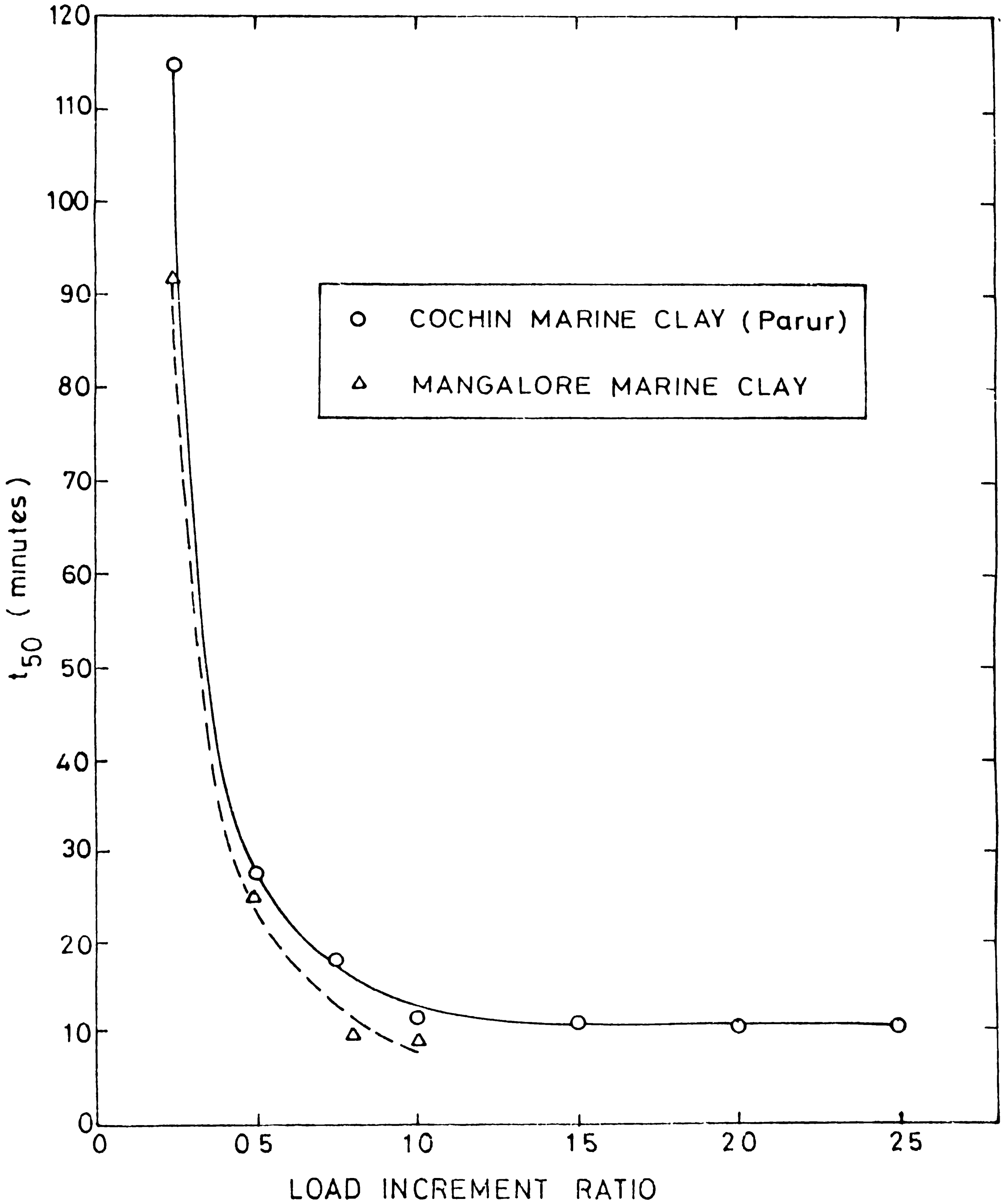


Fig.4.3.15 EFFECT OF LIR ON VALUES OF t_{50}

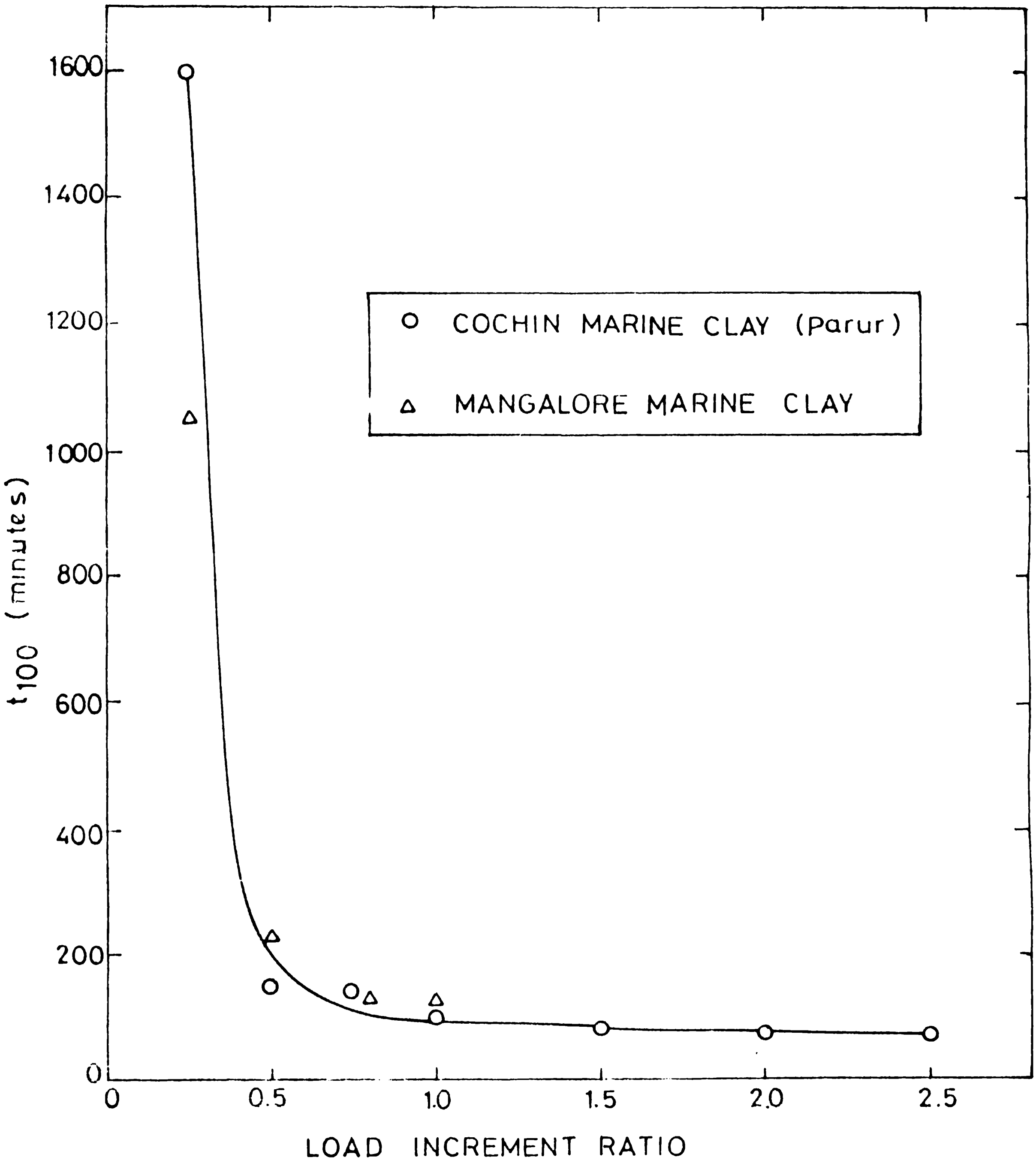


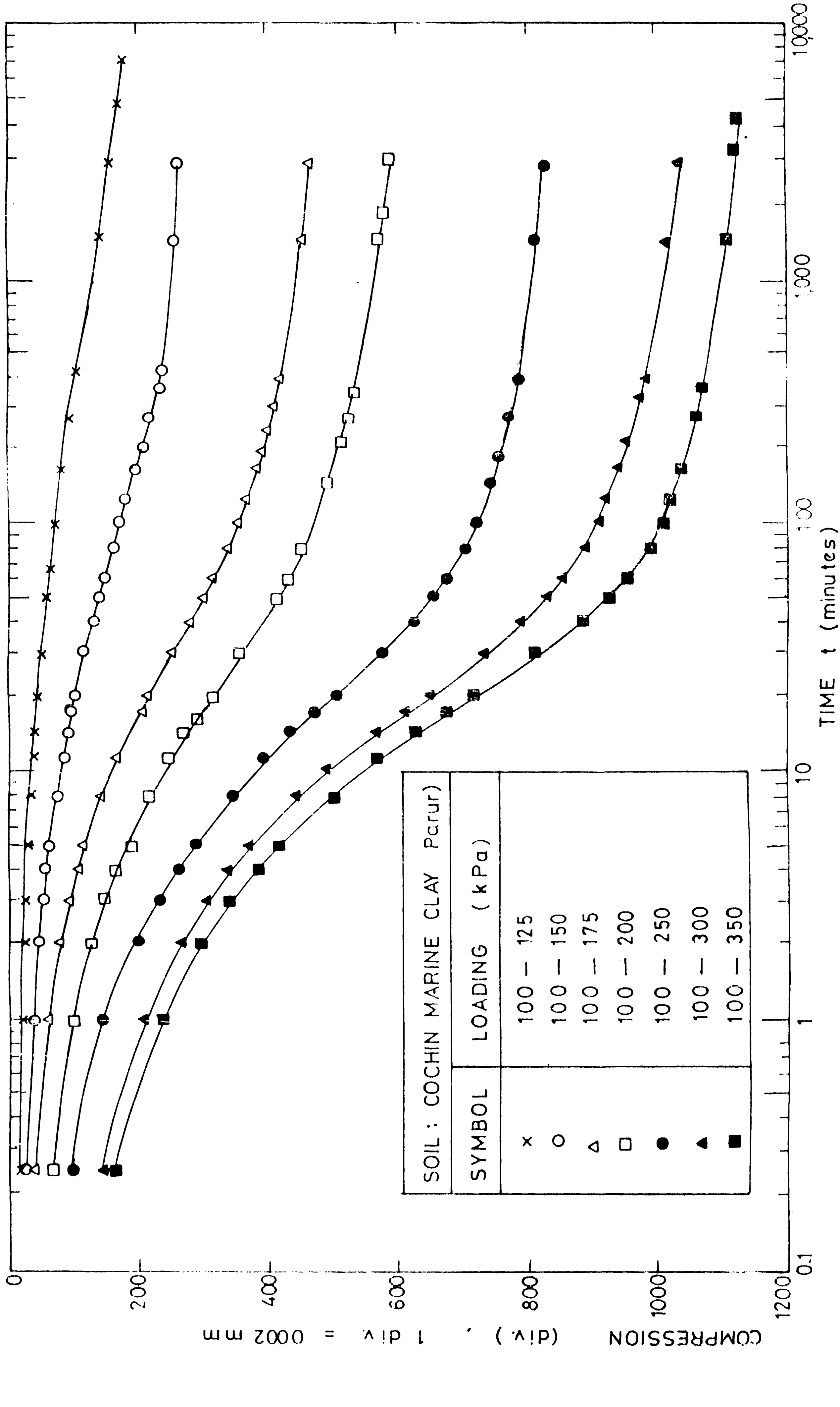
Fig.4.3.16 EFFECT OF LIR ON VALUES OF t_{100}

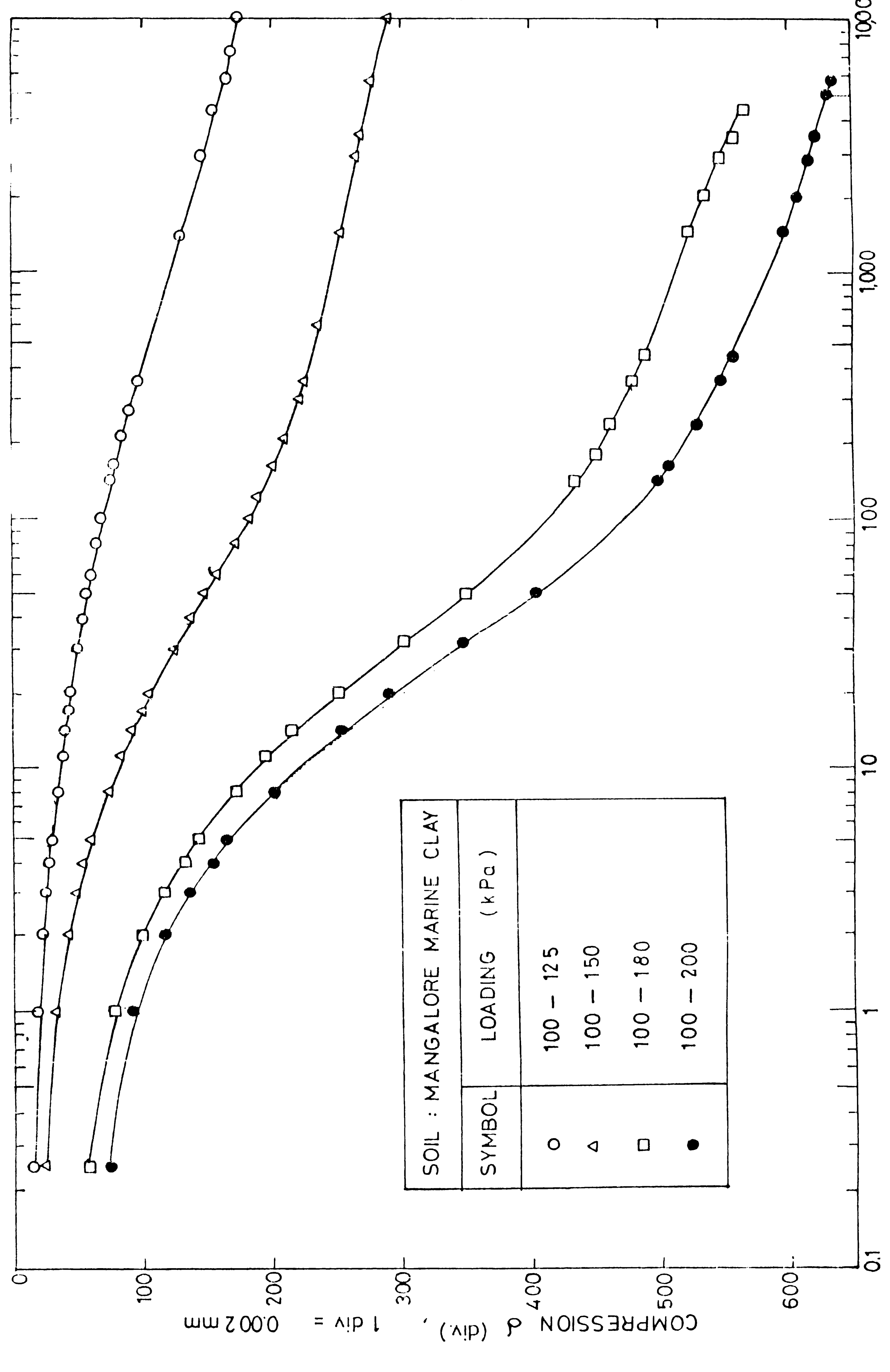
are obtained for t_{50} and t_{100}. This shows that the load increment ratio of one used for routine consolidation tests is an optimum value.

4.3.4 Instantaneous compression during overloading

In the methods for determination of C_v, it has been accepted that there is an initial compression which do not fall within the normal trends of the $\delta - \sqrt{t}$ plot or δ -log t plot. The difference between the initial dial reading R_o just before the load is increased and the corrected initial reading R_c is known as the initial consolidation or initial compression. In the case of routine consolidation tests, a specimen is loaded with a load increment ratio equal to one. Hence the increment in consolidation pressure is applied to a specimen which has been consolidated at a pressure equal to the pressure increment. Therefore there are no wide variations in the values of initial compression.

In precompression studies, the overloading ratios which can fall in a wide range play a key role. Since pressure increments could be several times the overburden pressure, the initial compressions can be significantly high. Fig.4.3.17 show the δ -log t curves for overloading ratios 0.25 to 2.5. The curves presented in 4.3.17 for Cochin marine clays and 4.3.18 for Mangalore marine clays show that initial compressions make a significant contribution to the


SOIL : COCHIN MARINE CLAY (Parur)
SYMBOL
LOADING (kPa)
100 — 125
100 — 150
100 — 175
100 — 200
100 — 250
100 — 300
100 — 350
TIME t (minutes)
COMPRESSION (div.) , 1 div. = 0002 mm
0.1
1
10
100
1,000
10,000
0
200
400
600
800
1000
1200


SOIL : MANGALORE MARINE CLAY
SYMBOL
LOADING (kPa)
100 – 125
100 – 150
100 – 180
100 – 200
COMPRESSION δ (div.) , 1 div = 0.002 mm
0
100
200
300
400
500
600
0.1
1
10
100
1,000
10,000

total compression values. For example, for a pressure increment 100-125 k Pa, the initial compression during the first 15 seconds is 15 divisions. When the load increment is 100-350 k Pa, the initial compression increases to 165 divisions, for the same time period. In order to study the information immediately after loading, the compression which takes place during the first 15 seconds is designated here as instantaneous compression.

Fig.4.3.19 shows the relation between instantaneous compression for different consolidation pressures applied on specimens of Cochin marine clay consolidated at 100 k Pa. It indicates that instantaneous compression steadily increases with load.

A plot between the instantaneous compression expressed as a percentage of total compression for corresponding consolidation pressuresfor Cochin marine clay is shown in fig.4.3.20. Interestingly, it presents a linear variation.

The values of instantaneous compression are more dominant in kaolinite clays. Fig.4.3.21 shows the δ -log t plots for kaolinite samples for load increments of 100-150 and 100-200 k Pa. It can be seen that instantaneous

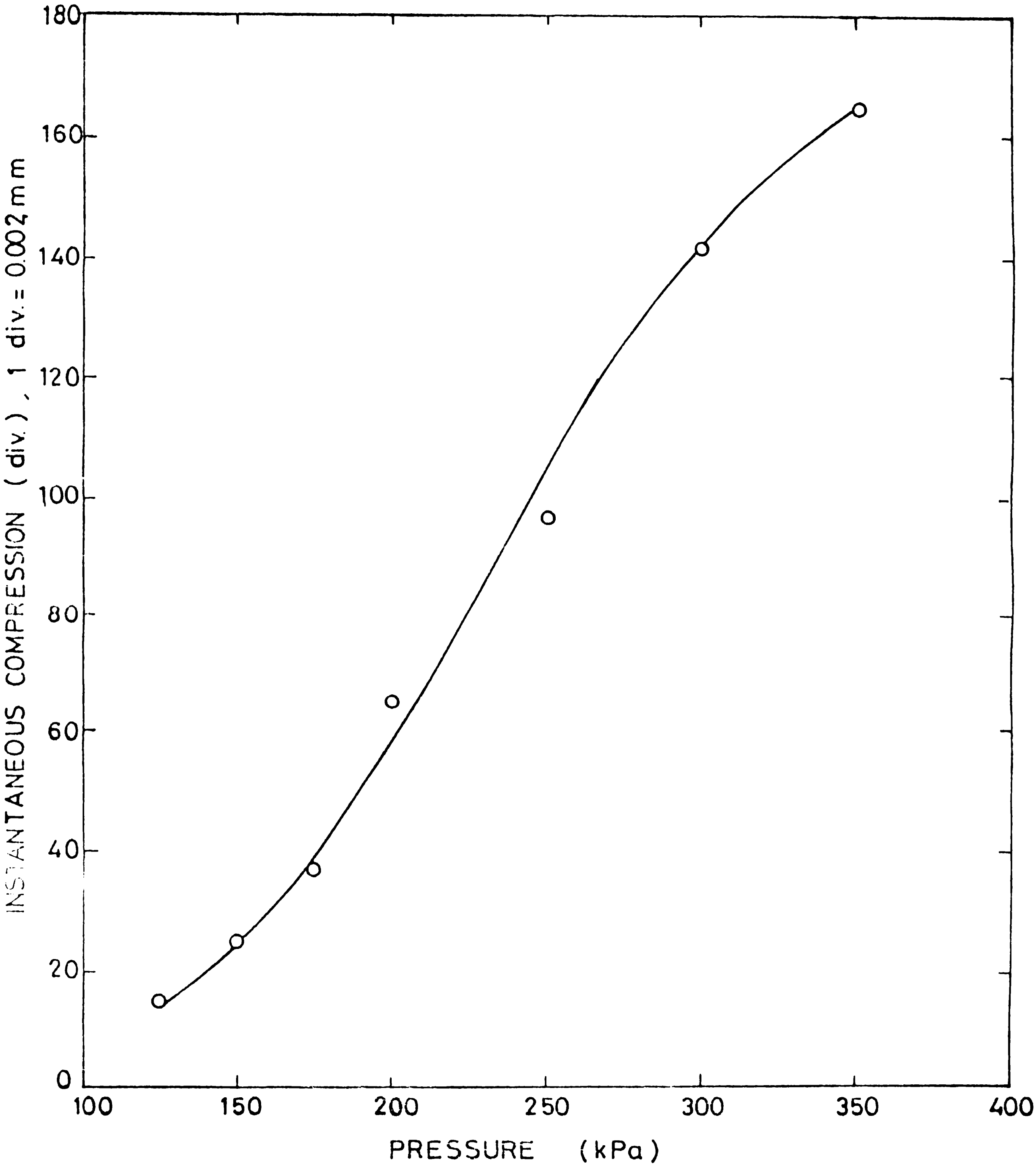


Fig.4.3.19 VARIATION OF INSTANTANEOUS COMPRESSION WITH PRESSURE FOR COCHIN MARINE CLAY

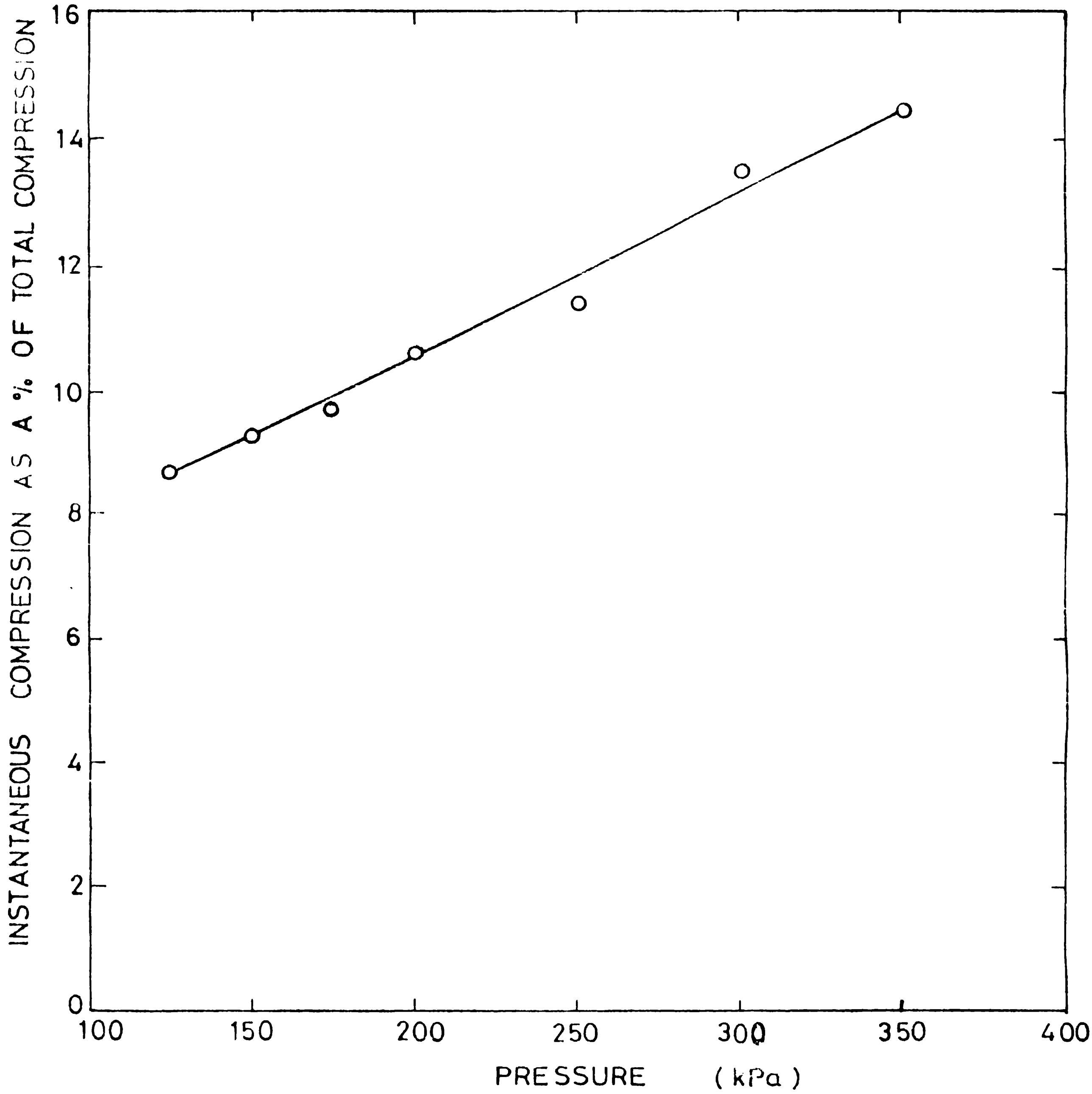


Fig.4.3.20 RELATION BETWEEN COMPRESSION RATIO AND PRESSURE

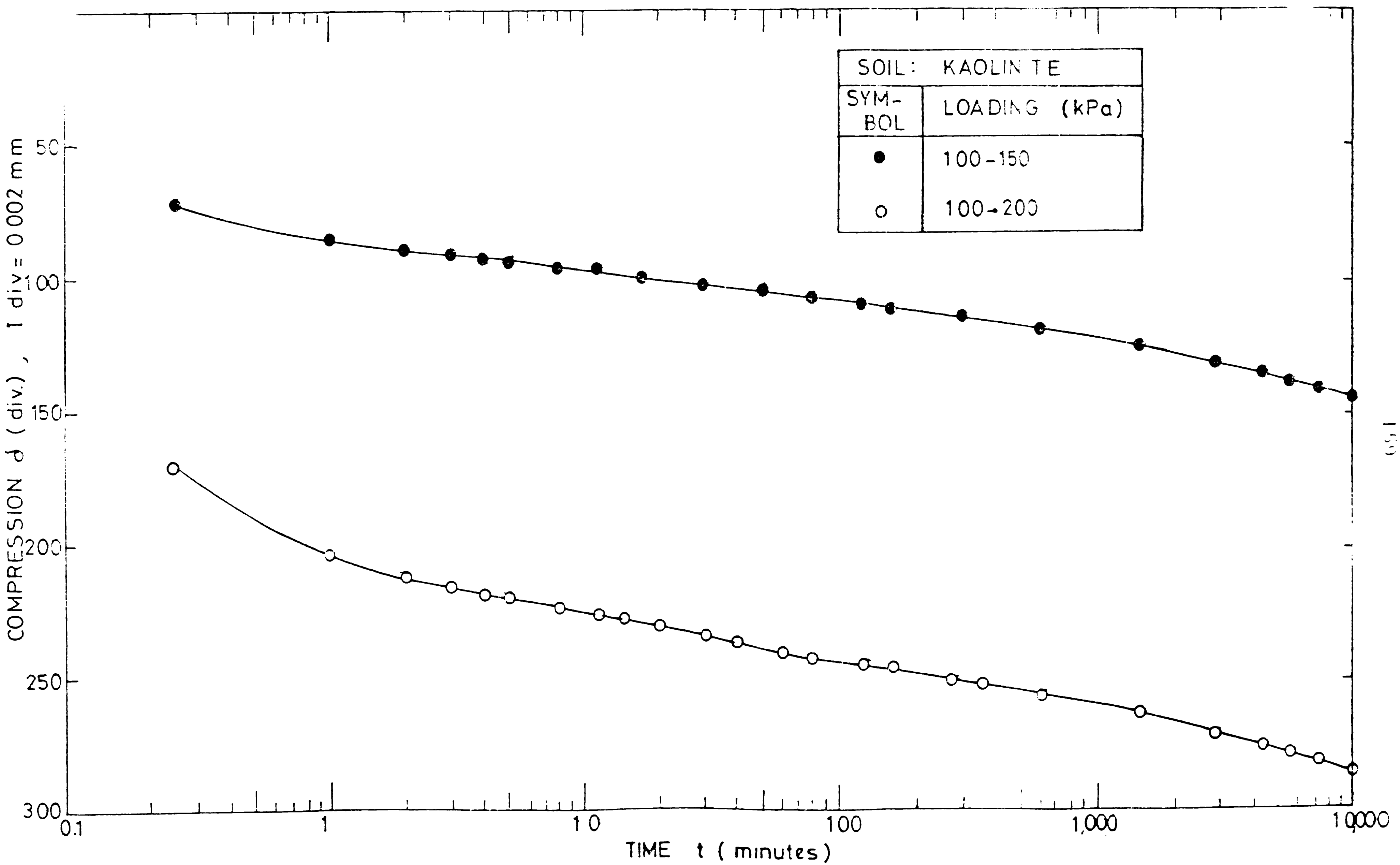


Fig.4.3.21 δ -log t CURVES FOR KAOLINITE

compressions are 72 and 170 divisions respectively. These form 51.4% and 59.2% of the total compression. This brings out the significance of instantaneous compression in precompression studies.

4.3.5 Recompression under permanent load

The design of a precompression project involve estimation of three distinct consolidation pressures - the overburden pressure and the pressure increments due to the permanent load of the structure and due to the surcharge applied for precompression. Once the precompression is completed, the loads accounting for the permanent and surcharge loads are removed and the clay can show some swell. As and when the structure is constructed, the permanent load causes a recompression of the clay.

In order to study the recompression behaviour, specimens of Cochin marine clay was subjected to precompression and recompression. In Fig.4.3.22, the first curve is the e - log curve for virgin compression. This is almost a straight line. When the pressure is released upto seating load and a routine consolidation test is carried out, the curve representing the 2nd cycle is obtained. The total deformation is considerably low and slopes of this recompression curve will help to estimate the settlements due

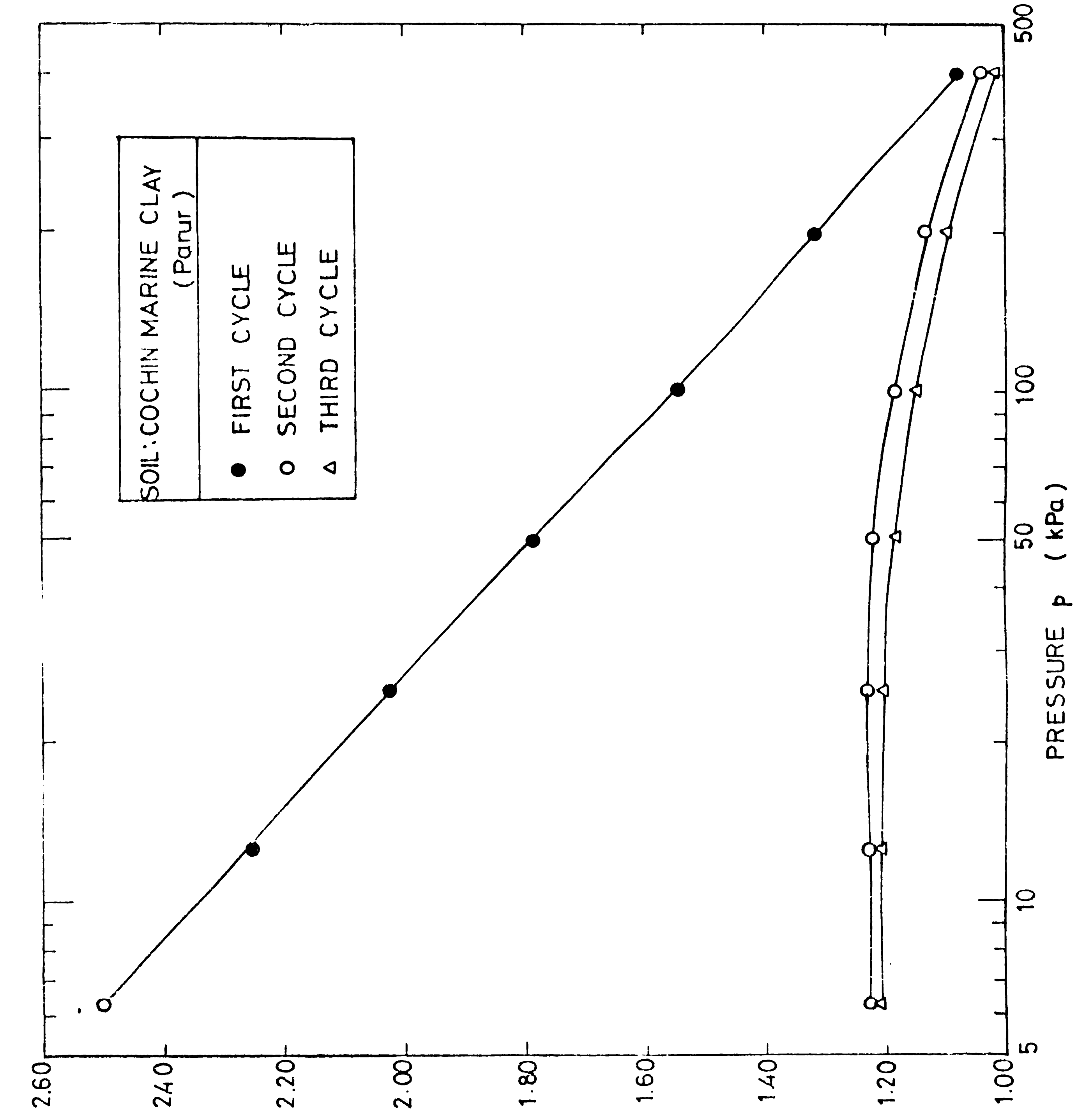


Fig.4.3.22 RECOMPRESSION CURVES FOR

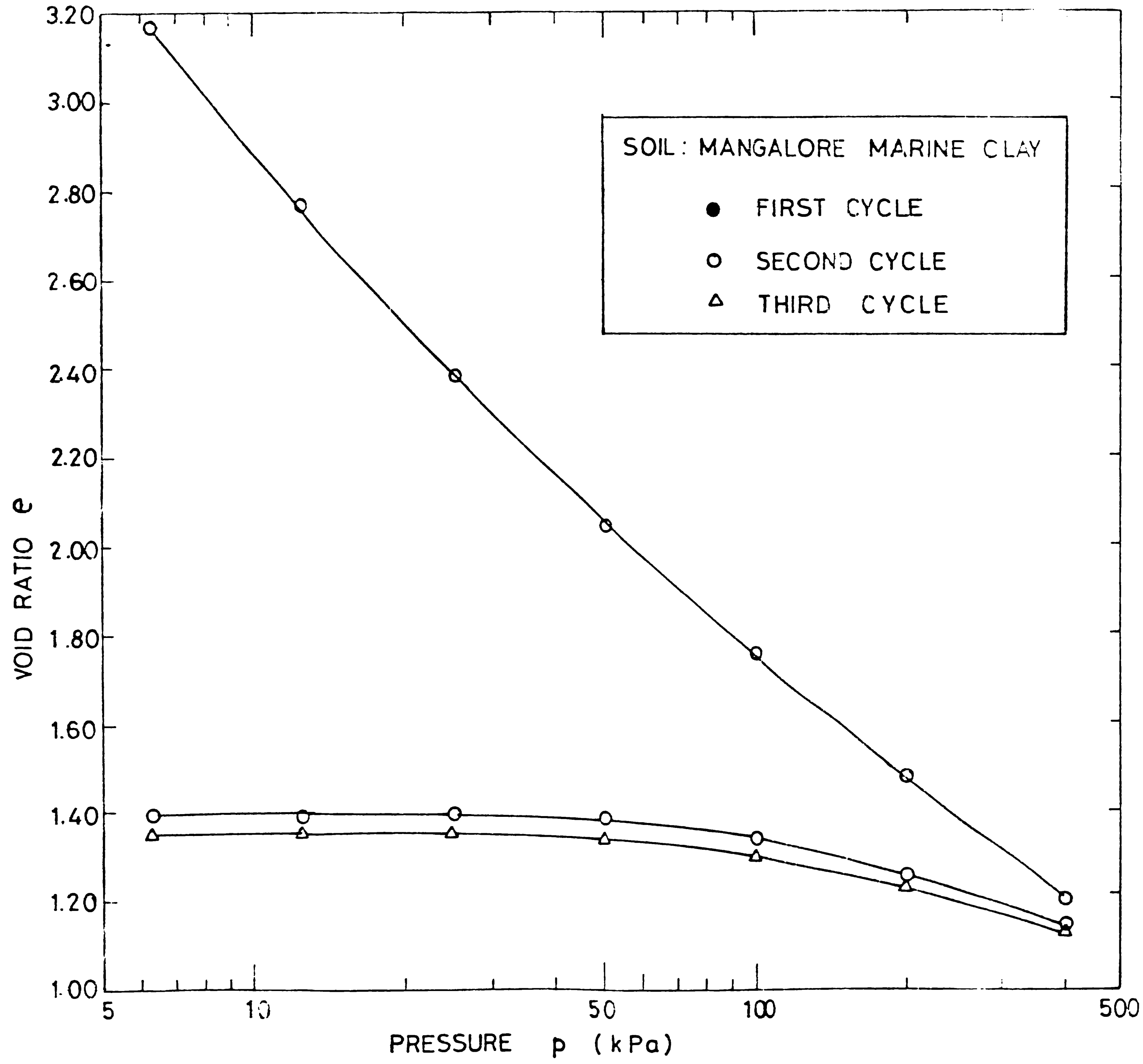
to the permanent load. The curve representing the third cycle run parallel to and close by the second cycle curve.

Fig.4.3.23 and 4.3.24 show the results of tests carried out on Mangalore marine clay and kaolinite. The behavioural pattern is exactly the same.

The values of de/(d log p) ~~of~~ (C_c) for the three curves in Fig.4.3.22 are presented in Fig.4.3.25. The significant reduction in C_c values especially at the lower pressures where it is close to zero indicates the tremendous potential of recompression in controlling settlements due to permanent loads.

The results of recompression studies for particular pressure increment of 100-200 k Pa and 200-400 k Pa for Cochin marine clay are presented in Figs.4.3.26 and 4.3.27 in the form of δ -log t curves. The figure shows that for lower pressures, the recompression curves get stabilised faster and merge with each other. But for higher pressures the curves do not get stabilised so easily.

The present investigations had only a limited scope wherein the factors influencing precompression were identified. In order to get the required data for the design of a precompression project, more detailed studies have to be carried out.


SOIL : MANGALORE MARINE CLAY
FIRST CYCLE
SECOND CYCLE
THIRD CYCLE
VOID RATIO e
3.20
3.00
2.80
2.60
2.40
2.20
2.00
1.80
1.60
1.40
1.20
1.00
5
10
50
100
500
PRESSURE p (kPa)

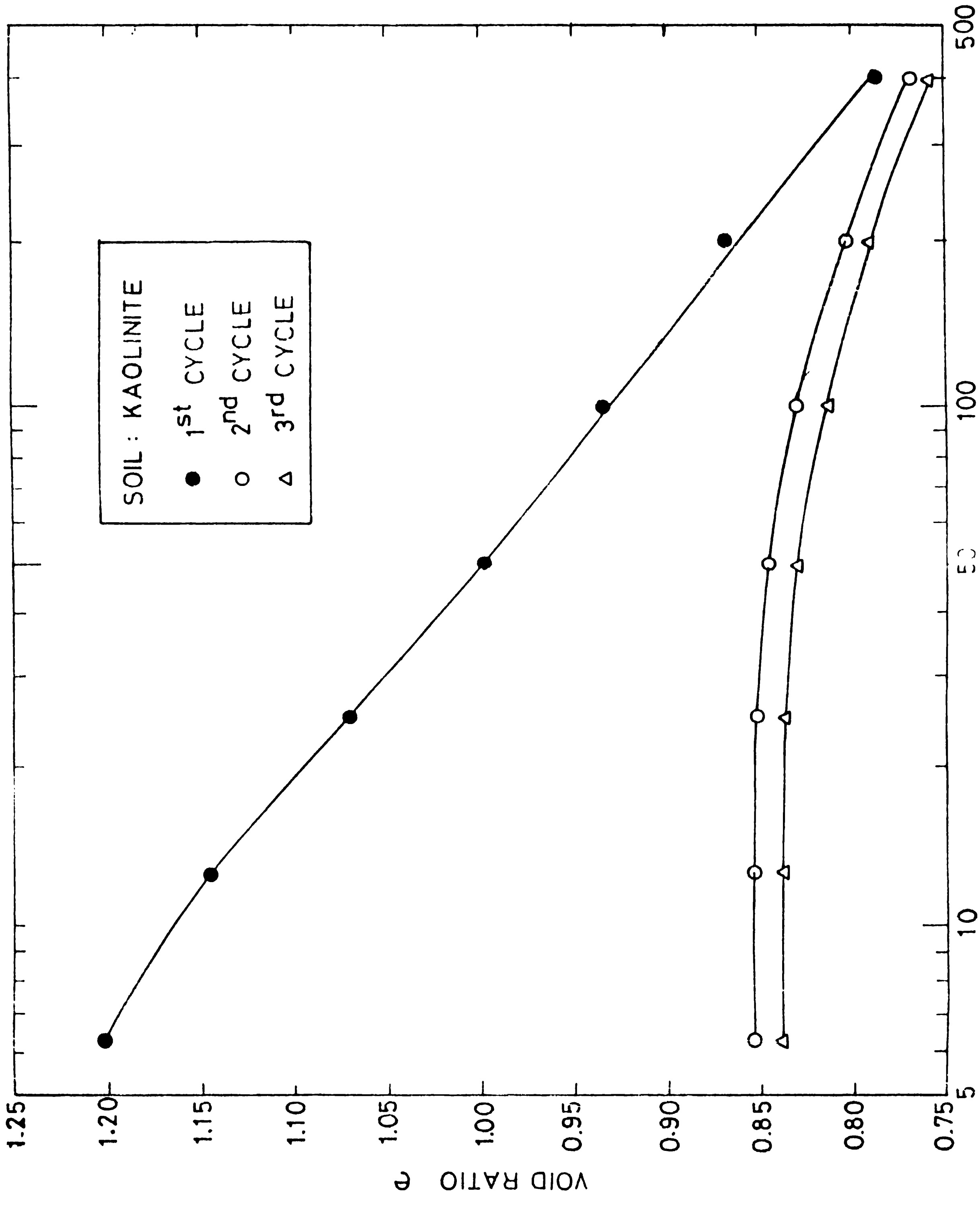


Fig.4.3.24 RECOMPRESSION CURVES FOR KAOLINITE

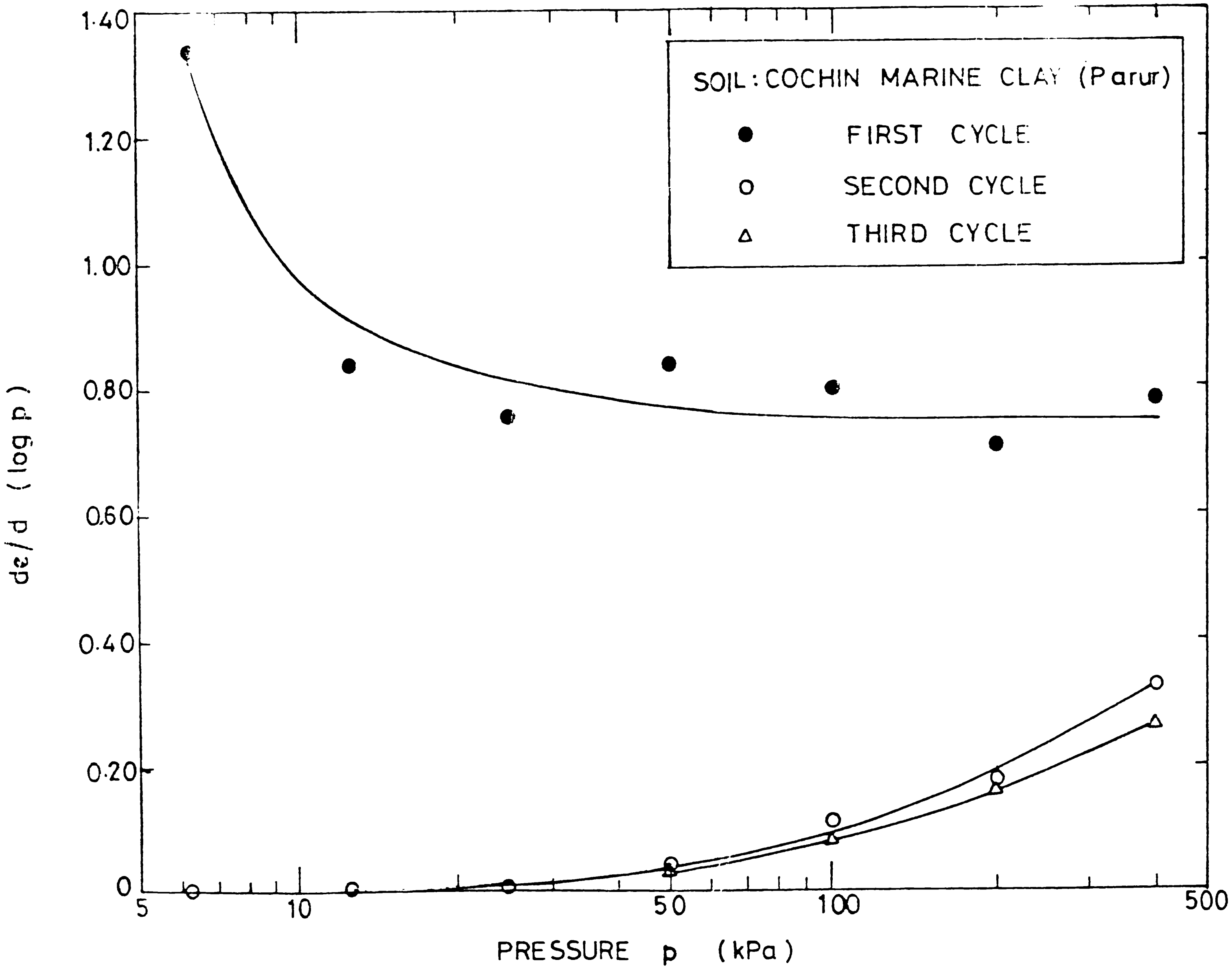
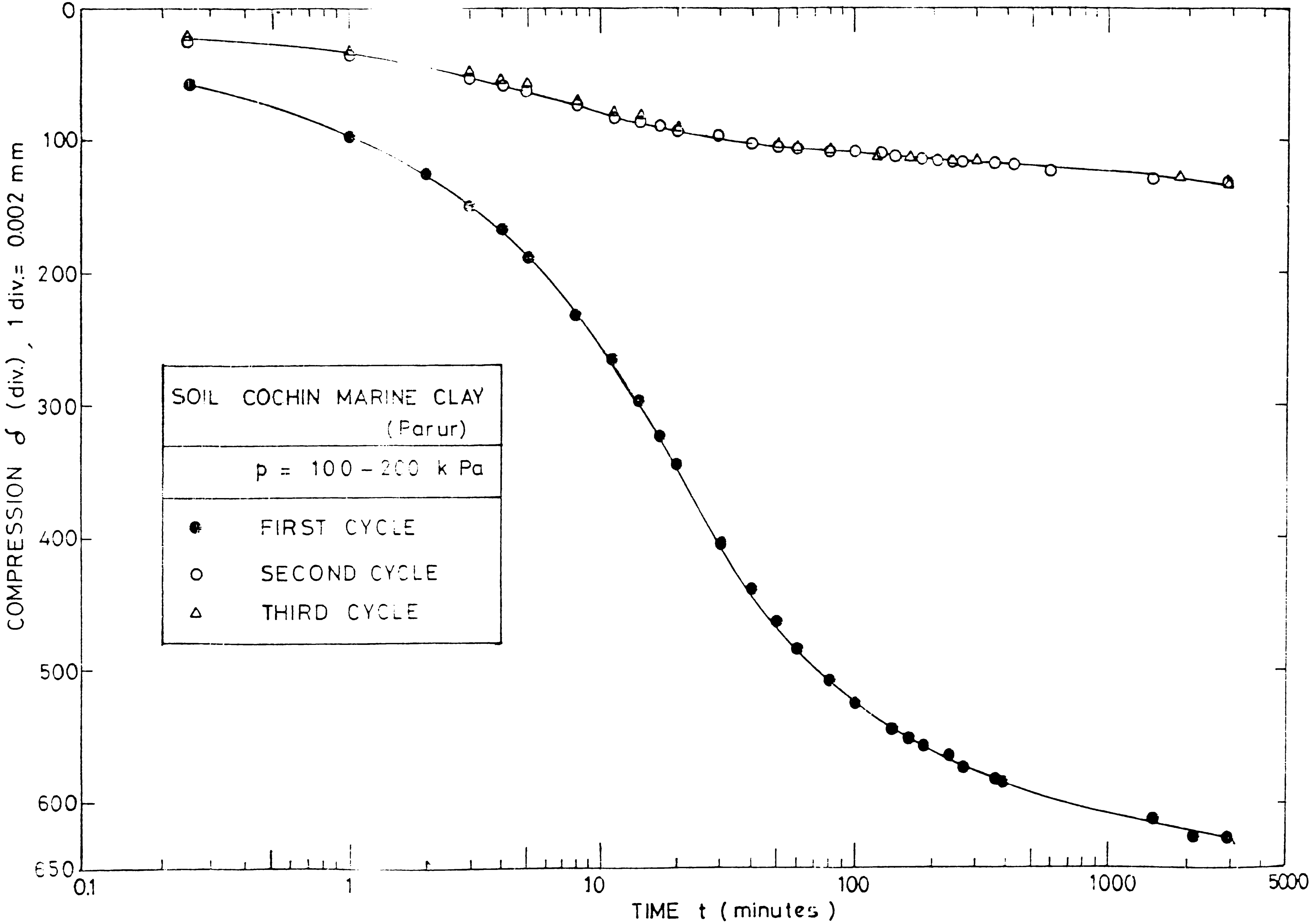
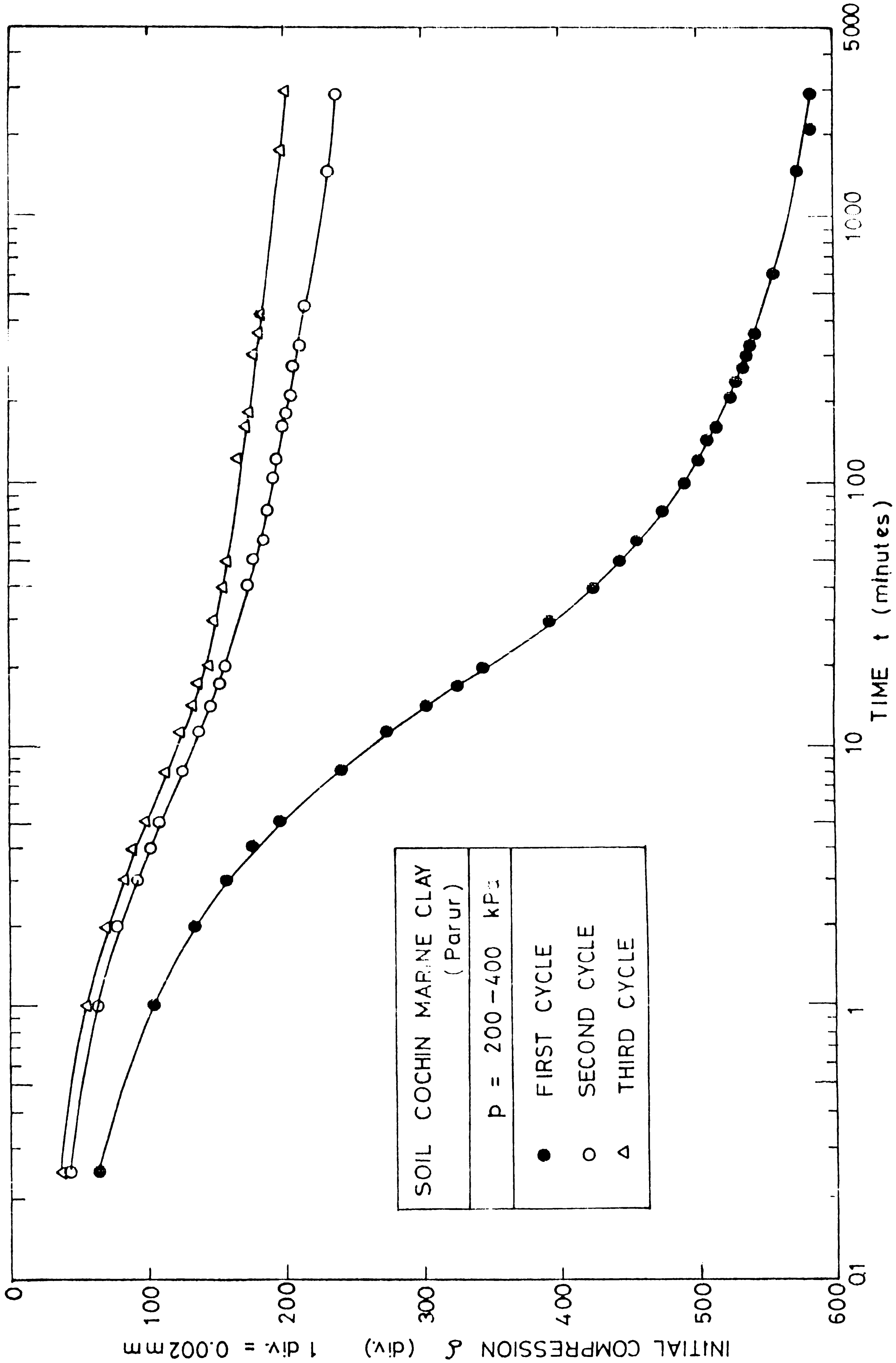


Fig.4.3.25 EFFECT OF PRELOADING ON de/d(log p) VALUES



SOIL COCHIN MARINE CLAY (Parur)
p = 100 - 200 k Pa
FIRST CYCLE
SECOND CYCLE
THIRD CYCLE
COMPRESSION δ (div.), 1 div.= 0.002 mm
TIME t (minutes)
0
100
200
300
400
500
600
650
0.1
1
10
100
1000
5000


SOIL COCHIN MARINE CLAY
(Parur)
p = 200 - 400 kPa
FIRST CYCLE
SECOND CYCLE
THIRD CYCLE
TIME t (minutes)
0.1
1
10
100
1000
5000
INITIAL COMPRESSION δ (div.) 1 div. = 0.002 mm
0
100
200
300
400
500
600

4.4. Compressibility behaviour of stabilised clays

4.4.1 Introduction

It has been man's incessant endeavour, since the dawn of civilisation to improve soil to meet his requirements. More than 3000 years ago, soil stabilisation was already in practice as indicated by the remnants of the Babylonian temple. Contemporary to this, the Chinese used wood, bamboo or straw to reinforce the soil. The Chinese ideogram for 'Civil Engineering' means nothing other than soil and wood.

Attempts on soil stabilisation through centuries have left development of numerous techniques to improve the strength characteristics of soils, whose growing importance in the solution of more and more complicated problems can hardly be overemphasised. The methods to improve soil can be drawn into three distinct groups - temporary soil improvement techniques limited to the period of construction will include dewatering, ground freezing etc. Permanent soil improvement techniques will comprise of practices to improve the natural soil without addition of materials, such as compaction, thermal treatment etc. Permanent soil improvement with addition of materials has made the maximum contribution to soil stabilisation through improvised stabilising techniques, development of stabilising equipments and identification of most appropriate stabilising agents. Field practices over the [illegible] few centuries and the exhorbitant research data made available by recent workers conclude in unequivocal terms that lime and cement can yield the best results out of the numerous stabilising agents tried.

The use of lime as stabilising agent dates back to the early Romans who used it in the construction of their roads, especially the famous 'Via Appian' - a traffic artery which has given outstanding service since its construction (Leonards, 1962). Clayey soils can be stabilised by the addition of a small percentage of lime and montmorillonite clays which forms the basic mineral for Cochin marine clays respond more readily to lime treatment than illites and kaolinites. Eventhough the chemistry of lime stabilisation is yet to be explained unambiguously (Thompson, 1966) it has been shown that mainly cation exchange, flocculation, carbonation and pozzolanic action contribute to the improvement in engineering characteristics.

Cement stabilisation has been very popular since the turn of this century especially in the case of road and air field pavements. Since then millions of square metres of pavements and air strips have been stabilised by soil-cement mixtures (Leonards, 1962).

When hydration of cement takes place in the cement stabilisation process, it results in the formation of calcium hydroxide and the pH value of the aqueous phase is raised to approximately 12.2. The products formed after short periods of ageing are largely gelatinous and amorphous, but with further curing, poorly ordered varieties of hydrated calcium silicate and hydrated calcium aluminate develop. During the early stages of hydration, gelatinous products form around anhydrous cement cores and are precipitated from solution in the interstices between particles. Hardening is due to the gradual desiccation of these gelatinous products and the crystallisation of new materials (Bell, 1978).

As the cement hydrates, the strength of the stabilised soil improves and it becomes water resistant. The hydrated cement coatings form a skeletal structure of considerable strength, the actual strength depending on the lump size and the amount of cement used.

An indication of whether or not a clay is amenable to stabilisation by cement is provided by its Atterberg limits (Croft, 1968). The suitability of a clay soil for cement stabilisation is controlled by its texture and chemical and mineralogical composition. The results of several mineralogical examinations of clay-cement mixtures (Croft, 1967) suggested that certain clay minerals interfere with the stabilising action of cement. For instance, the expansive clay minerals have a profound influence on the hardening of cement.

4.4.2 Physical properties of cement treated marine clays

Treatment of cohesive soils with cement imparts a number of desirable changes in their physical properties. First and foremost is the reduction in the plasticity index of the soil which makes them more friable and workable (Dandson and Handy, 1960). This is accompanied by appreciable changes in compressibility and shear strength characteristics.

The results of a detailed study on the physical properties of cement treated Cochin marine clays are presented in Table 4.4.1. The liquid limit shows an instantaneous reduction immediately after cement treatment. But the value steadily increases with curing period. The liquid limit of the marine clay increases from 129% to 145% on treatment with 9% cement after curing for 3 months. The variation in plastic limit on cement stabilisation is much more pronounced. From a plastic limit value of 56.5% for an untreated clay the value goes up as high as 78%. Eventhough several mineralogical examinations of clay-cement mixtures (Croft, 1967) suggested that certain clay minerals interfere with the stabilising action of cement. For instance, the expansive clay minerals have a profound influence on the hardening of cement.

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Table 4.4.1

Physical [illegible]rties of cement treated soil (Cochin marine clay from Parur)

Additive (%)	Curing period	CEMENT Liquid limit (%)	CEMENT Plastic limit (%)	CEMENT Plasticity index (%)	CEMENT Shrinkage limit (%)	LIME Liquid limit (%)	LIME Plastic limit (%)	LIME Plasticity index (%)	LIME Shrinkage limit (%)
0	Untreated	129	56.5	72.5	20.3	129	56.5	72.5	20.3
3	Nil	118	45	73	22.4	--	--	--	--
	1 week	125	48	77	24.7	144	50.8	93.2	22.3
	1 month	123	47	76	25.0	134	54.2	79.8	20.0
	2 months	122	45	77	26.0	--	--	--	--
	3 months	120	45	75	25.5	--	--	--	--
6	Nil	131	60	71	24.5	138.5	71.5	67.0	38.8
	1 week	135	69	66	27.0	145	83.0	62.0	31.5
	1 month	137	72	65	34.5	147	73.1	73.9	32.7
	2 months	137	78	59	34.5	146	76.0	70.0	32.1
	3 months	142	80	62	35.0	144	76.0	68.0	32.5
9	Nil	116	49	67	24.0	--	--	--	--
	1 week	139	69	70	38.5	127.5	55.8	71.7	33.7
	1 month	136	70	66	40.0	136	76.7	59.3	33.3
	2 months	145	80	65	40.0	149	80.7	69.3	36.9
	3 months	143	79	64	43.5	144	71.4	72.6	35.2

both liquid limit and plastic limit increases on cement stabilisation, the higher rate of increase in plastic limit brings down the plasticity index as shown in the table. While the plasticity index for the untreated soil is 72.5%, the been brought down to around 65% upon treatment with cement.

The fact that stabilising agents causes aggregation of grains is brought out by the shrinkage limit values listed in the table. While the shrinkage limit is only 20.3% for the untreated soil, the value goes up as high as 43.5% on treatment with 9% cement cured for three months.

It is well known that clays with montmorillonite as the basic clay mineral respond more readily to lime treatment. To have a comparative study of the variations in physical properties of samples treated with cement and lime, the values for lime treated Cochin marine clays (Jose, 1989) are also presented in Table 4.4.1. It can be seen that the variation in Atterberg limits and shrinkage limits are more or less identical, as far as Cochin marine clays are concerned. Eventhough the immediate response is much more pronounced in case of lime treated samples, for longer curing periods, the values are more or less same. For example, the liquid limit for a sample treated with 6% cement is 131%

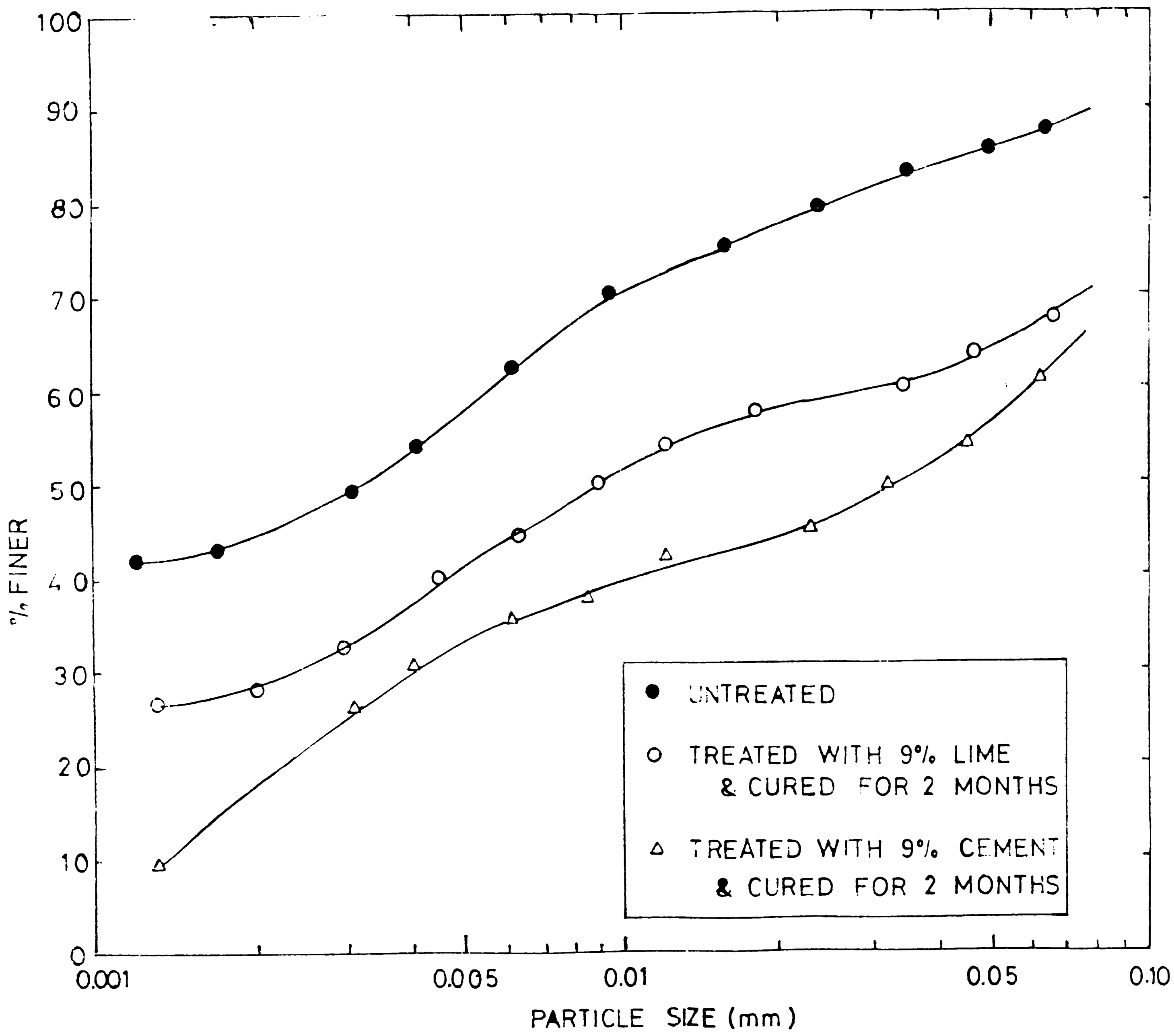


Fig.4.4.1 TYPICAL GRAIN SIZE DISTRIBUTION CURVES FOR TREATED COCHIN MARINE CLAY

compared to 138.5% in case of lime. These values respectively increase to 142 and 144 upon curing for three months.

Typical grain size distribution curves for untreated Cochin marine clay and for samples treated with 9% lime and 9% cement and cured for two months are presented in Fig.4.4.1. Similar results have been obtained for other percentages and other curing periods also. In spite of the addition of the dispersing agent (section 3.5), the aggregation of fines due to stabilisation is clearly brought out by the relative positions of the grain size distribution curves and decrease in the percentages of clay size fraction. While the natural Cochin marine clay sample had a clay fraction of 45%, the clay size fraction for lime treated soil is 28% and the corresponding value for cement treated soil is only 22%.

4.4.3 Compressibility characteristics of cement stabilised clays

4.4.3.1 Improvement in compressibility

Stabilisation of fine grained soils with lime has been in practice for quite a few centuries. Considerable research work has gone into the study of improvement of strength characteristics of clays by lime stabilisation. The compressibility characteristics of lime stabilised Cochin marine clays have been studied in great detail by Jose (1989). But studies on stabilisation of clays using cement are mostly confined to shear strength characteristics of soil-cement used for highway and airfield pavements. In these [illegible], the main concern is improvement of bearing capacity of the top layers. The compressibility characteristics especially under sustained load have not been investigated properly as evident from an almost total lack of research papers in this specific area. Hence a detailed study of the various aspects of consolidation behaviour of cement stabilised Cochin marine clay has been taken up in the present work.

The improvement in compressibility characteristics on treatment of Cochin marine clay with 6% cement is shown in Fig.4.4.2. The figure shows the relation between the percentage reduction in thickness and the consolidation pressure for different treated samples - one for which no curing period was allowed and the other three cured for periods of one week, one month and three months. For purposes of comparison, the plot between $\Delta H/H_o$ (%) and consolidation pressure for an untreated remoulded sample of Cochin marine clay is also given. It can be seen from the set of curves that the cement stabilised clay becomes stronger and better depending on the curing period allowed. For shorter periods

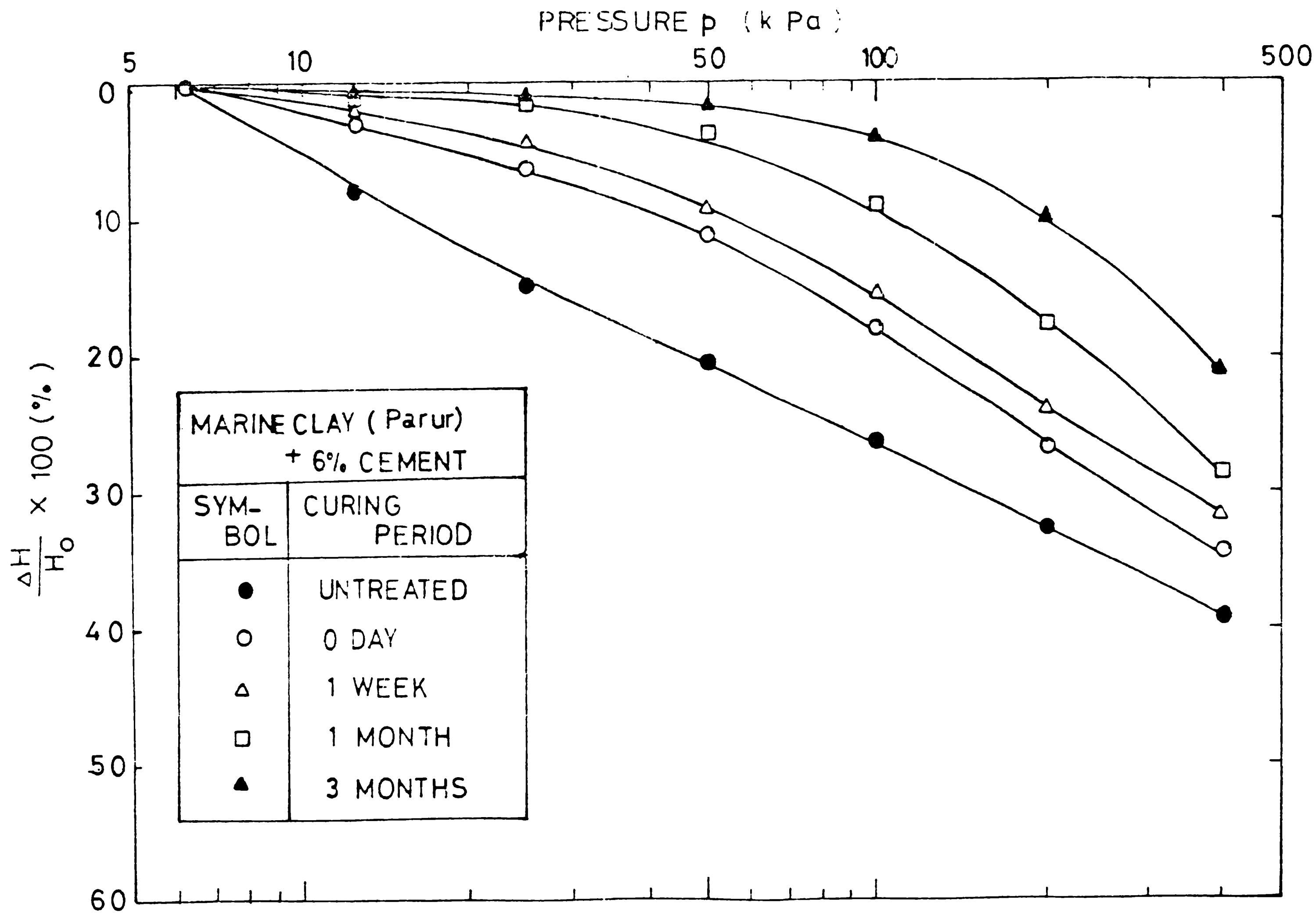


Fig.4.4.2 IMPROVEMENT IN COMPRESSIBILITY ON CEMENT TREATMENT

of curing, the curves are almost parallel to that of untreated sample. As bonds develop with respect to time allowed, the specimens become less compressible especially for the lower pressure ranges. As the consolidation pressures increase beyond around 50 k Pa, the compressions observed are substantial. For example, for a sample cured for three months, the deformation is significantly low - about 2.9% of the deformation of the untreated sample for a consolidation pressure of 12.5 k Pa. But when the consolidation pressure is [illegible] Pa, the deformation for the same specimen is 47% of the [illegible] of the untreated sample. Obviously the compression curve has two portions - the first one in which the deformation is significantly low and the second portion in which the rate of increase in deformation is substantial.

In case of a standard consolidation test, the compression curve has two distinct portions - the recompression portion and the virgin compression curve. These two fall on either side of the preconsolidation pressure which is a distinct and unique value for any normally consolidated clay. But in the case of the cement stabilised Cochin marine clay, the change over from the almost linear initial portion wherein the consolidation pressures are below the strength of the bonds developed, to segment representing the consolidation at higher pressures is gradual, with the rate of deformation steadily increasing with pressure. This indicates that unlike the routine compression curve where there is a sudden failure of the soil fabric, the type of failure exhibited by cement stabilised clay is of a progressive nature.

Figure 4.4.3 shows the relation between consolidation pressure and the ratio between the cumulative deformation of the cement stabilised clay and the corresponding values for untreated remoulded clay expressed as a percentage. While the sample for which no curing period was allowed gives a compression of about 37% of the compression of the untreated clay for a consolidation pressure of 12.5 k Pa, the corresponding value for a sample [illegible] for one month is just 6.5%. On curing for three months, the deformation ratio further reduces to 2.9%. But for consolidation pressures higher than the bond strength, the deformation ratios are much higher. For example, for a consolidation pressure of 400 k Pa, the deformation ratios are 80, 63.8 and 47% respectively for curing periods of 0, 1 and 3 months.

Figure 4.4.4 shows the relation between $\Delta H/H_o$ and consolidation pressure for marine clay specimens treated with 9% cement and cured for periods of 0 to 3 months. The

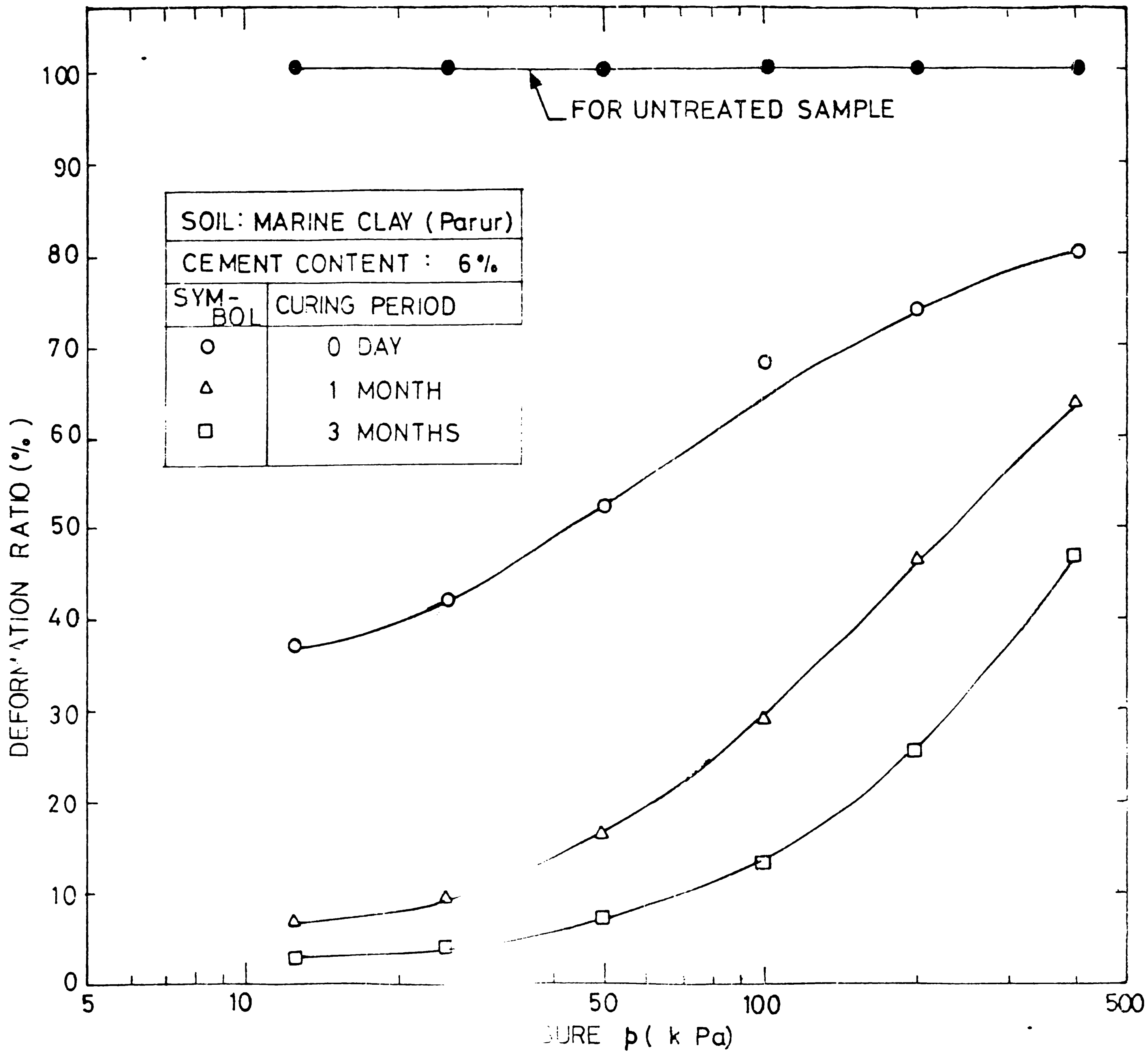


Fig.4.4.3 EFFECT OF C ATION PRESSURES ON DEFORMATION OF CEMENT STABIL COCHIN MARINE CLAY

behaviour is similar to those of samples treated with 6% cement.

It has been reported by earlier workers (Ingles and Metcalf, 1972; Bell, 1978) from studies on soil-cements that a certain minimum percentage of cement is essential to ensure [illegible]n in strength. A minimum percentage of 5 to 6 has been rec[illegible]ed in literature. In order to investigate the consolidation behaviour of specimens treated with lower percentages of cement than the value recommended, few consolidation tests were carried out on clays treated with 3% cement and cured for periods of 0 to 3 months. The results are presented in Fig.4.4.5. It can be seen from the set of curves that the effect of stabilisation is very negligible especially at higher consolidation pressures, where the curves come closer and closer.

The significance of the finding of the earlier workers that there is a minimum percentage required to impart the expected strength to the stabilised soils is clearly broughtout by Fig.4.4.6. Compression curves for three different specimens of Cochin marine clay treated with 3, 6 and 9% cement and cured for 3 months are presented in the figure. While the pair of curves representing soil treated with 6 and 9% cement shows a bond strength between 50 and

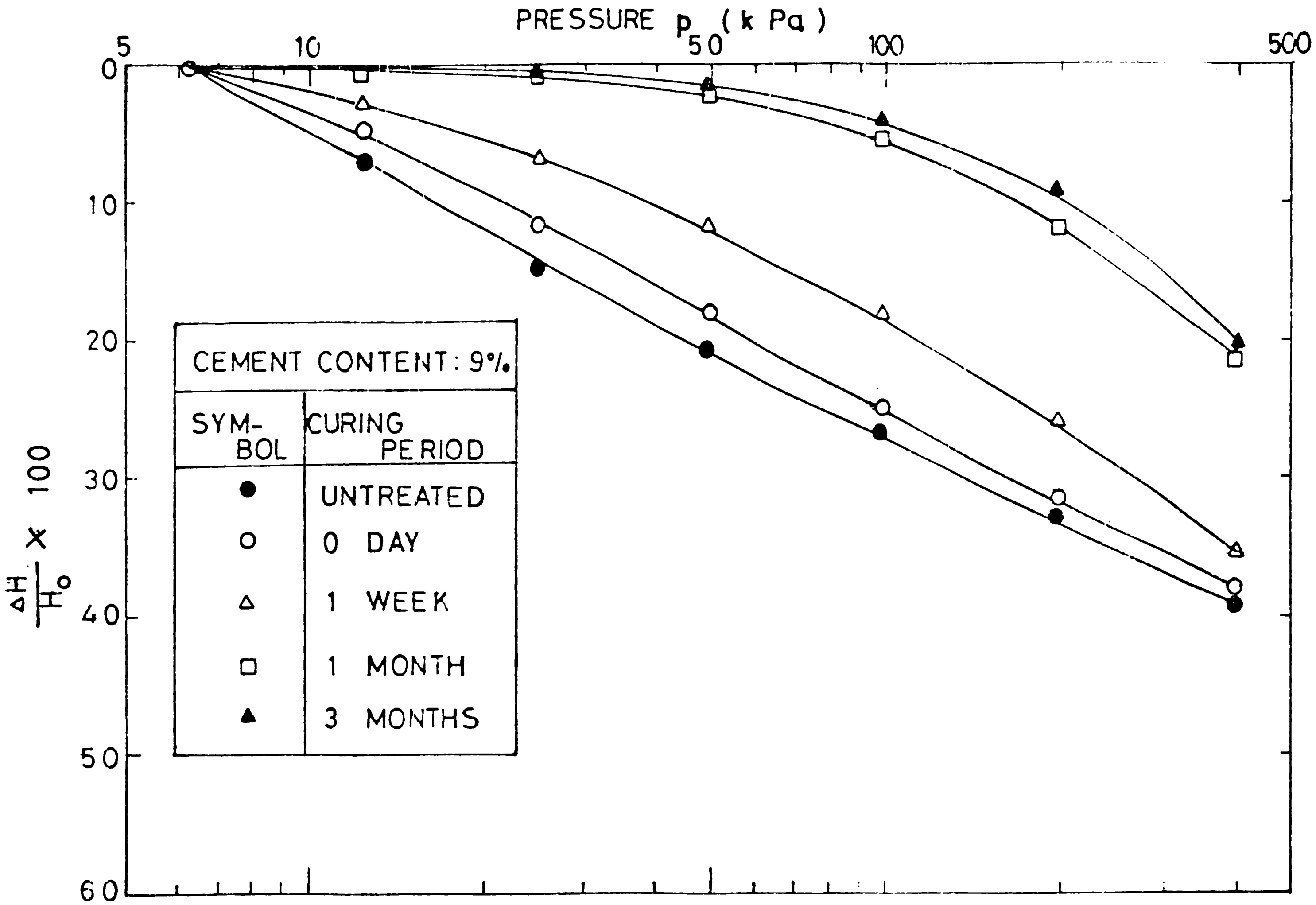


Fig.4.4.4 IMPROVEMENT IN COMPRESSIBILITY ON CEMENT TREATMENT

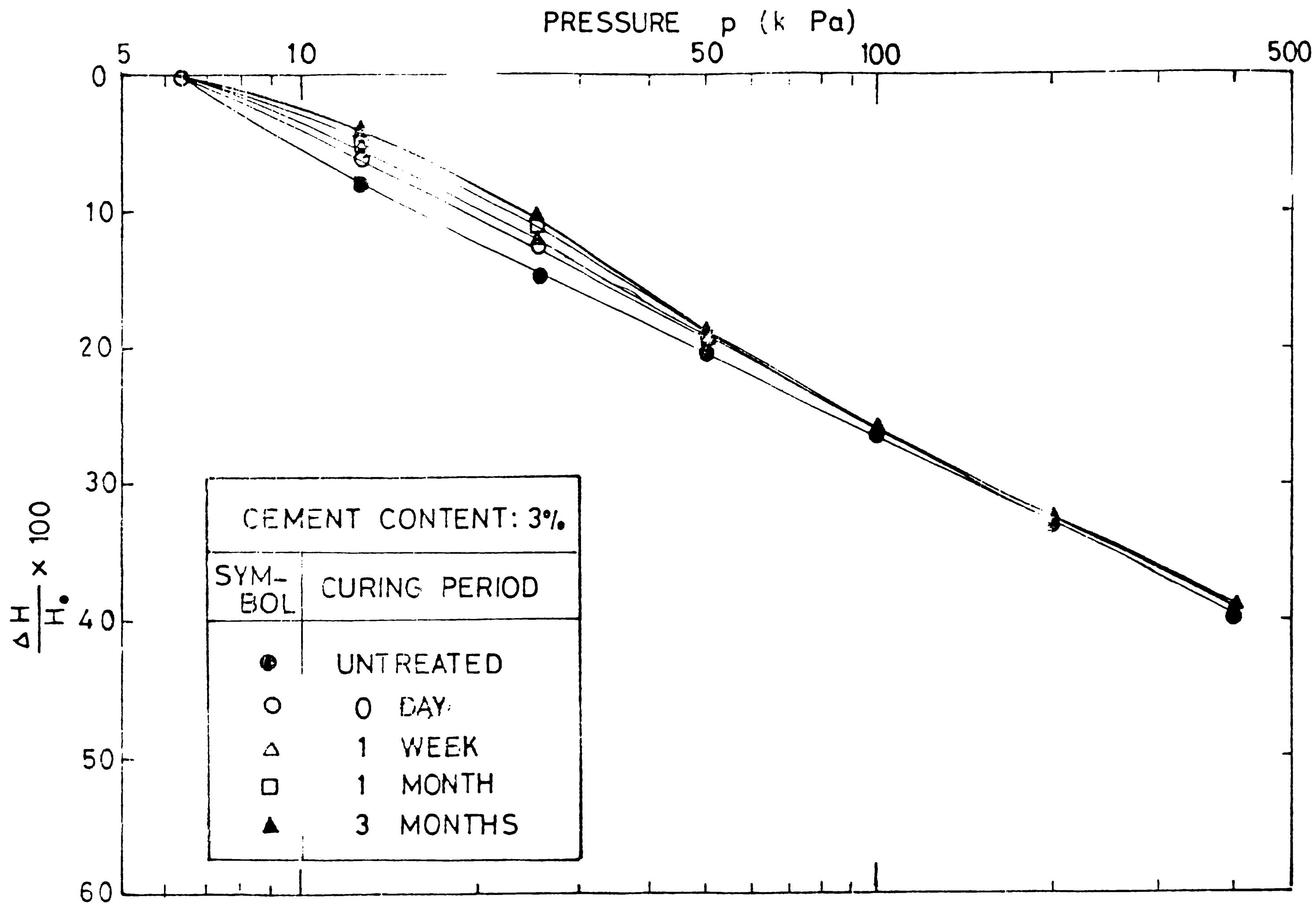


Fig.4.4.5 EFFECT OF CEMENT TREATMENT (LOWER %) ON COCHIN MARINE CLAY

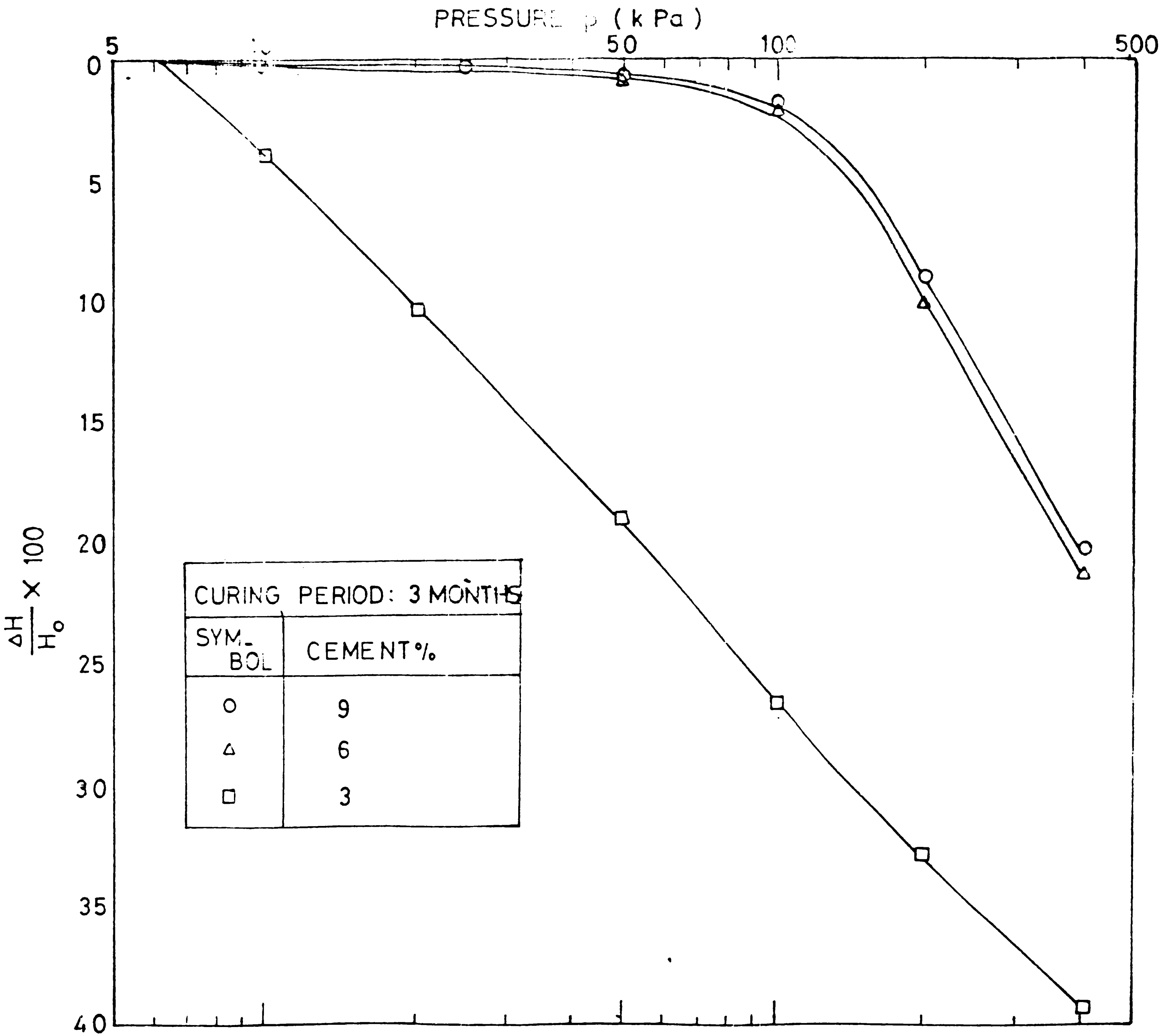
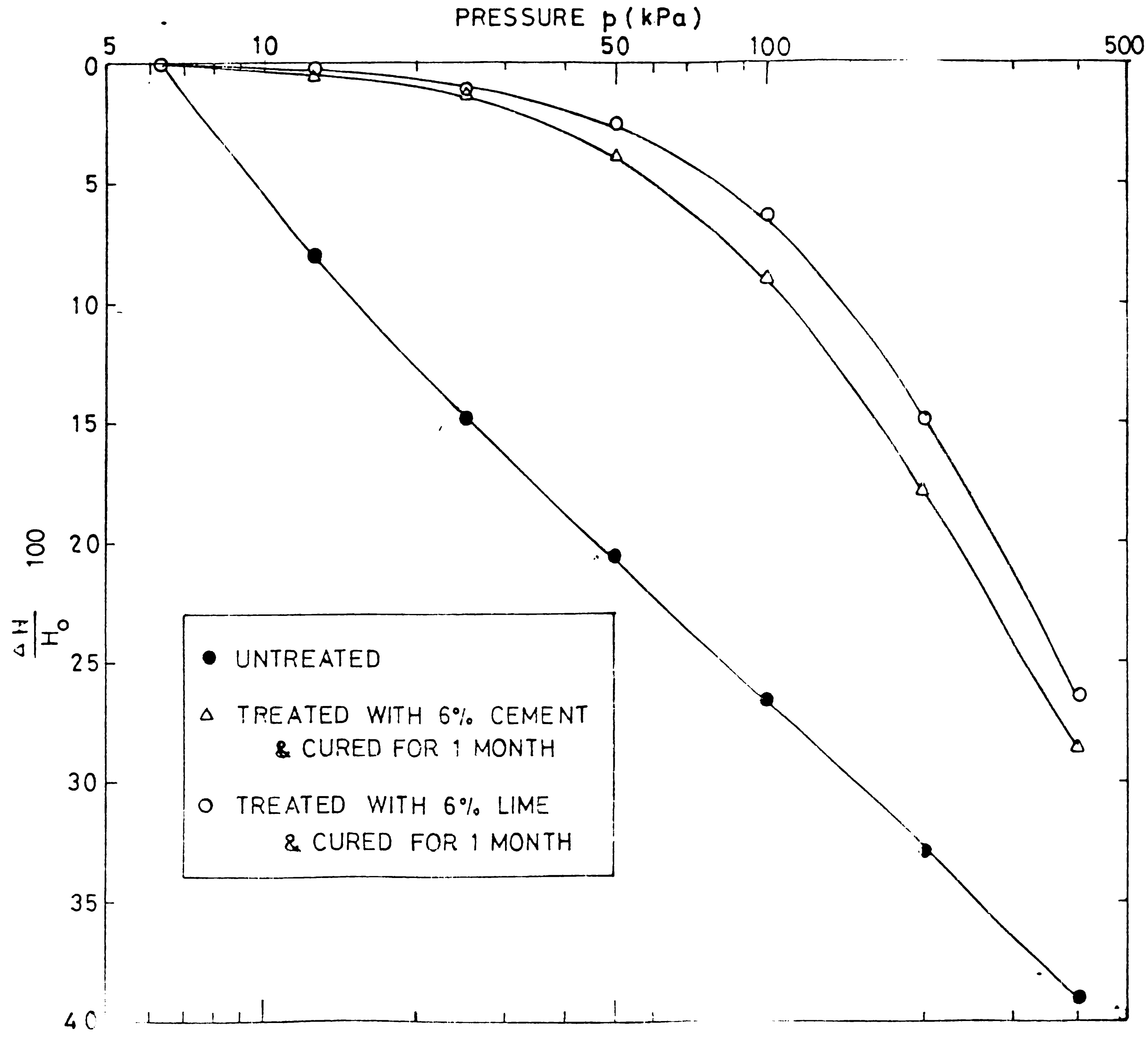


Fig.4.4.6 EFFECT OF CEMENT CONTENT ON COMPRESSIBILITY

100 k Pa, accompanied by considerable reduction in deformation, the curve representing clay treated with 3% cement does not show development of any sort of bond strength in spite of curing for 3 months. The compression curve is [illegible] that of untreated soil.

4.4.3.2 Comparison with lime treated soil

There have been extensive investigations on various aspects of the compressibility characteristics of fine grained soils stabilised with lime. Compared to this there have been very few studies on soils stabilised with cement. It has been confirmed by earlier workers (Jose 1989; Balasubramaniam et al., 1989) that the optimum range for stabilisation with lime and cement is 6 to 10%. In order to find out which gives better results within this range, a comparative study has been made in Figs. 4.4.7 and 4.4.8. The first figure shows the compression curves for clays treated with 6% lime and 6% cement cured for one month along with the curve for an untreated specimen. This figure shows that lime stabilisation helps to develop stronger bonds. But the set of curves in Fig.4.4.8 for identical specimens treated with 9% lime and 9% cement indicate that cement stabilisation gives marginally better results. The pair of curves in Figs. 4.4.7 and 4.4.8 are very close to each other indicating that both lime and cement give more or less identical results in case



F 1.4.7 COMPARISON BETWEEN CEMENT AND TREATMENT ON COCHIN MARINE CLAY

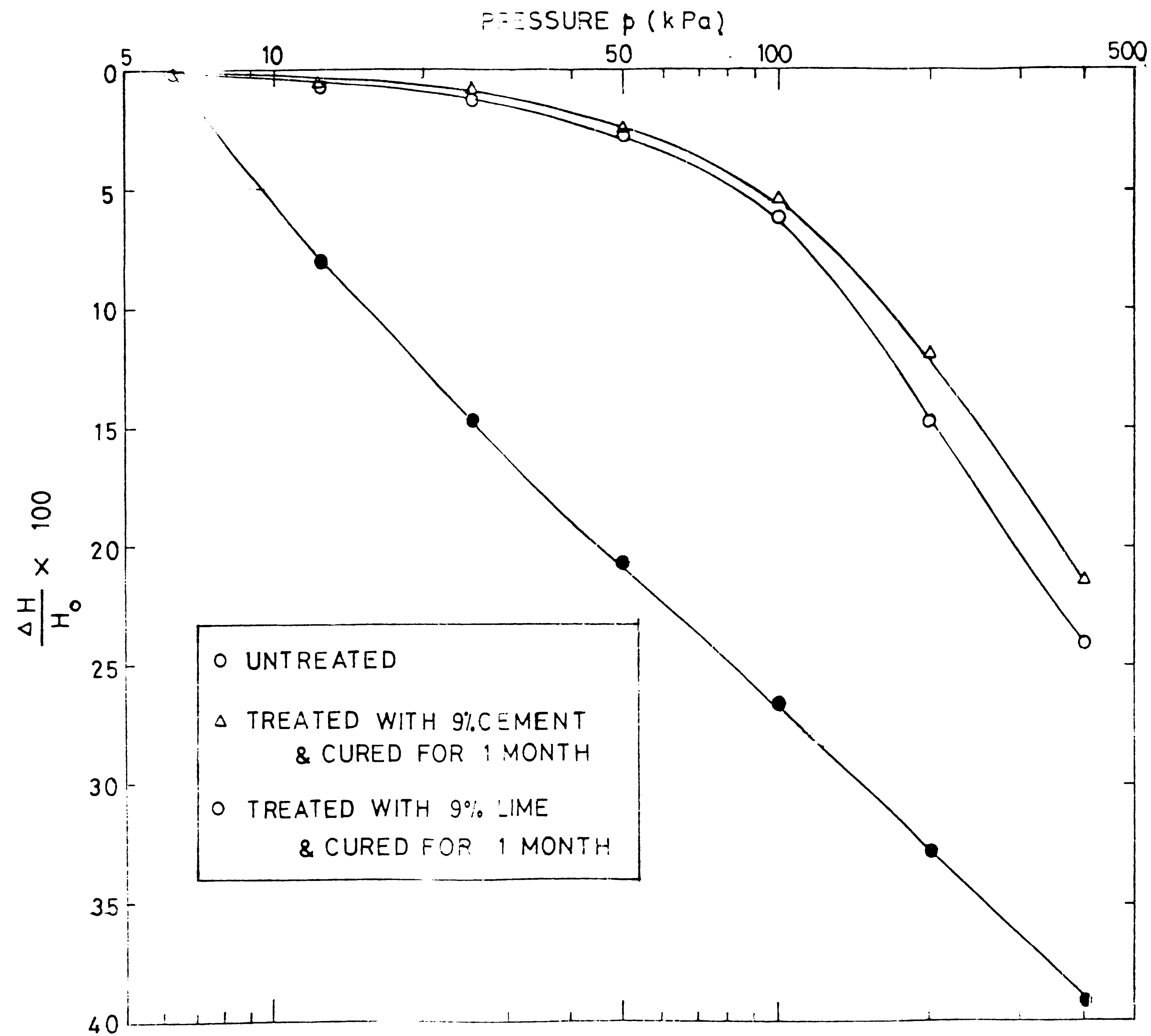


Fig.4.4.8 EF OF CEMENT AND LIME TREATMENT ON COCHIN MA LAY

of stabilisation of Cochin marine clays.

4.4.3.3 Effect of cement stabilisation on [de/d(log p)] values

The consolidation curves for cement stabilised soils presented in Figs.4.4.6 to 4.4.8 shows that the rate of deformation is significantly lower than that observed in the case of untreated remoulded marine clay. But once the bonds are broken, there is considerable reduction in the void ratio. These aspects of the consolidation behaviour are reflected in the curves presented in Fig.4.4.9. The figure shows the relation between ratio of change in void ratio to the corresponding pressure increment with consolidation pressure. The values of [de/d(log p)] for untreated clay varies from 1.13 to 0.94. But in the case of soils stabilised with 6% cement the values are considerably lower. They are 0.216, 0.06 and 0.03 for curing periods of 0, 1 and 3 months respectively, for the pressure increment 6.25-12.5 k Pa. In case of the cement stabilised clay for which no curing period was allowed, the value increases at a faster rate due to the low bond strength developed. It reaches a peak value of 1.3 and drops down thereafter. In case of the samples cured for 1 month, the rate increases drastically after a consolidation pressure of 25-50 k Pa, and drops down after a consolidation pressure of 200 k Pa. The reduction in the value of

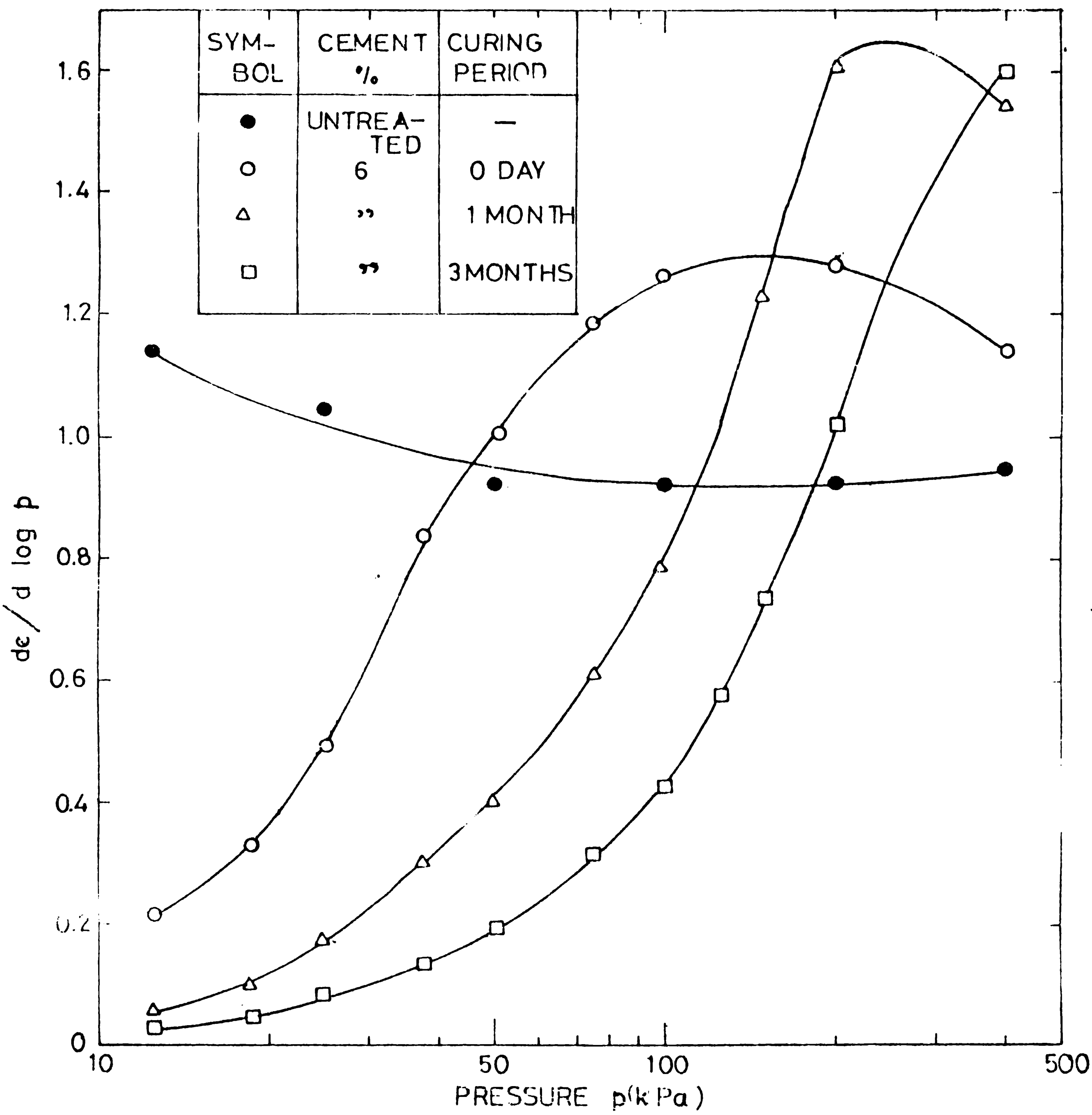


Fig.4.4.9 VARIATION OF de/d log p WITH PRESSURE FOR SAMPLES TREATED WITH 6% CEMENT

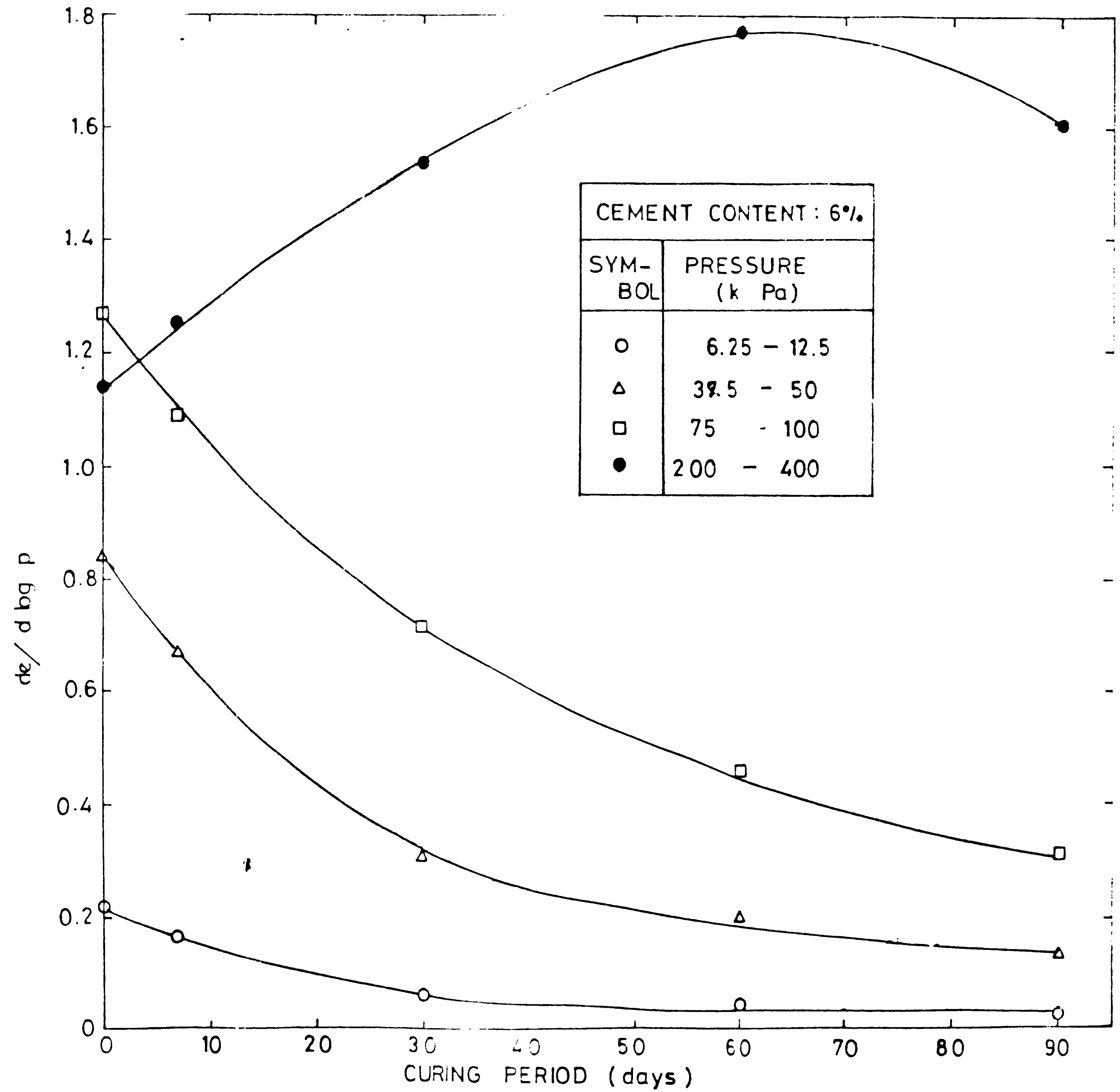


Fig.4.4.10 VARIATION OF de/d log p WITH CURING PERIOD FOR VARIOUS PRESSURES

[de/d(log p)] indicates a higher resistance to the consolidation pressure, which can be attributed to a rearrangement of the soil grains after the bonds due to stabilisation are broken. In case of the clay cured for 3 months, the increase in [de/d(log p)] is observed after about 100 k Pa. A study of the three curves shows that the rate of change of void ratio with respect to consolidation pressure can be controlled by allowing more curing period and keeping the consolidation pressure well below the bond strength.

As mentioned earlier, attempts to reduce deformation through stabilisation will meet with greater success at consolidation pressures well below the bond strength developed. This is clearly broughtout in Fig.4.4.10, wherein [de/d(log p) is seen decreasing with lower consolidation pressures and longer curing periods.

4.4.3.4 C_v and $C_{\alpha\epsilon}$ values for cement stabilised clays

During the discussion on typical grain size distribution curves for cement treated Cochin marine clays (Fig.4.4.1), it was pointed out that there is an aggregation of fines into coarser particles due to stabilisation. It was also broughtout that cement causes greater aggregation than lime. This effect is manifested from the results of the studies on coefficient of consolidation, C_v of cement stabilised clays. Figure 4.4.11 shows the relation of C_v with consolidation pressure for three samples. It can be seen that for the same specimen treated with a certain percentage of cement and cured for a finite period, the variation with respect to consolidation pressure is negligible. But the values increase considerably depending on the percentage of cement and curing period. The value of C_v for a consolidation pressure of 50 k Pa increases from $0.63 \times 10^{-4} cm^2/sec$ for the untreated clay to $3.08 \times 10^{-4} cm^2/sec$ and $10.8 \times 10^{-4} cm^2/sec$ for specimens treated with 3 and 9% cement and cured for one month.

Figure 4.4.12 shows the relation between coefficient of secondary consolidation $C_{\alpha\epsilon}$ with consolidation pressure for samples treated with 9% cement and cured for periods of 0 to 3 months. For purposes of comparison, $C_{\alpha\epsilon}$ for an untreated specimen is also presented. It can be seen from the figures that there is considerable reduction in the value of $C_{\alpha\epsilon}$ with respect to the curing period permitted. While $C_{\alpha\epsilon}$ for the untreated specimen is 9.75×10^{-3} for a pressure increment of 25-50 k Pa, the corresponding values for periods of 0 day, 1 week, 1 month and 3 months are 4.03×10^{-3}, 1.73×10^{-3}, 1.11×10^{-3}, 0.91×10^{-3} and 0.71×10^{-3} respectively. This shows that $C_{\alpha\epsilon}$ for the sample cured for 3 months is just 7.3% of the corresponding value for untreated clay. The value

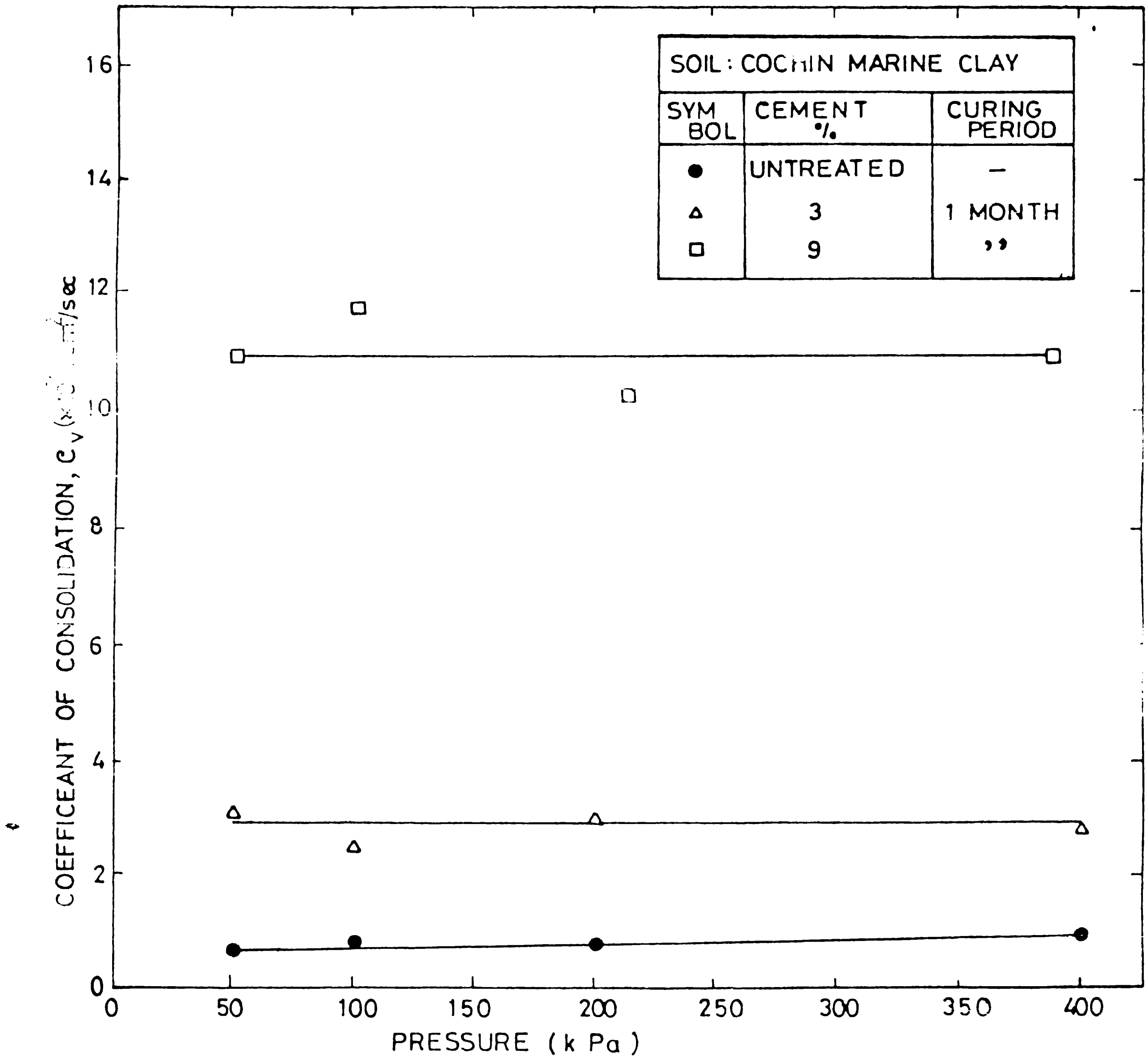


Fig.4.4.11 EFFECT OF CEMENT TREATMENT ON COEFFICIENT OF CONSOLIDATION

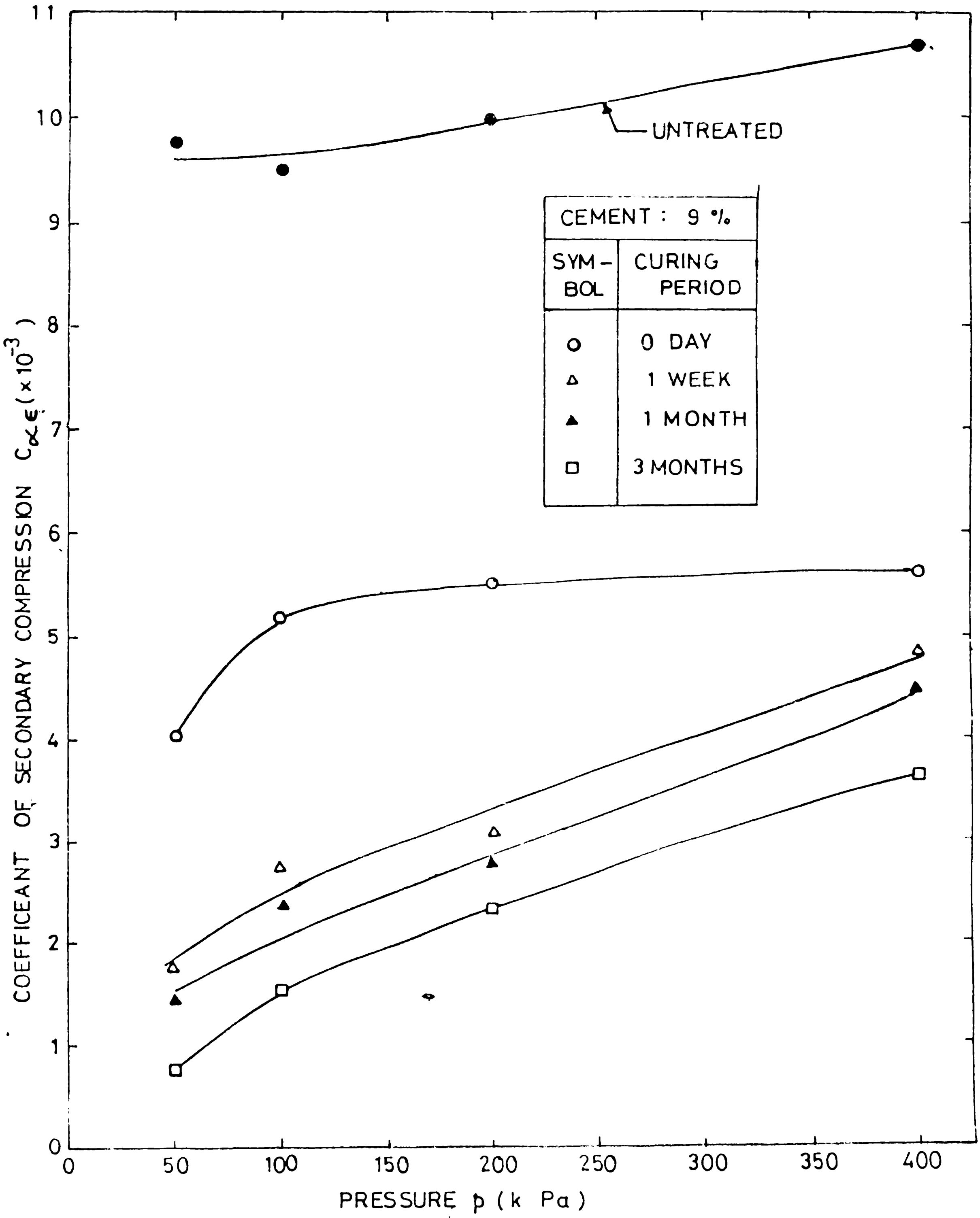


Fig.4.4.12 EFFECT OF CEMENT TREATMENT ON SECONDARY COMPRESSION FOR COCHIN MARINE CLAY

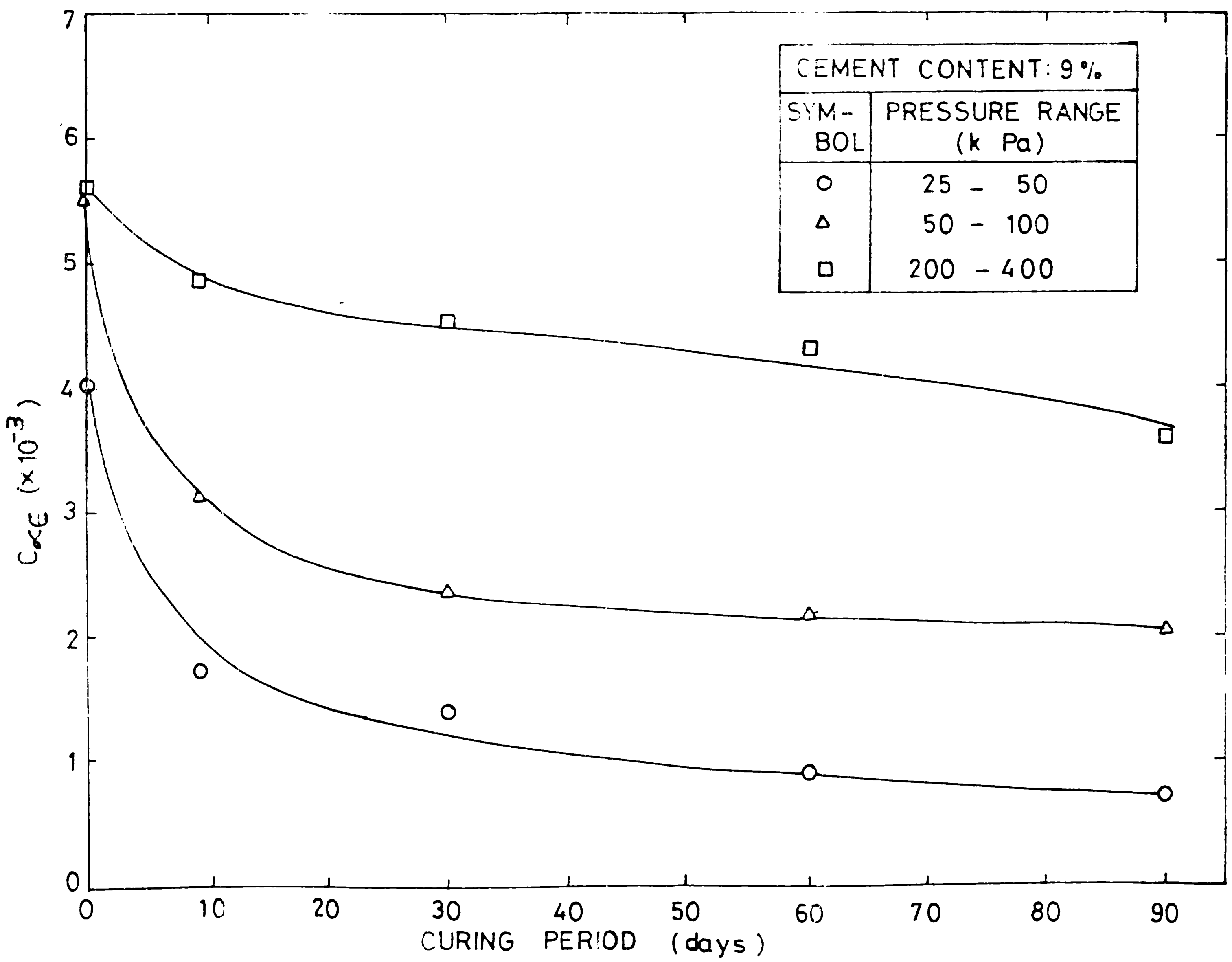


Fig.4.4.13 EFFECT OF CURING PERIOD ON COEFFICIENT OF SECONDARY CONSOLIDATION

of $C_{\alpha\epsilon}$ for the specimen for which no curing period was allowed does not vary considerably as the consolidation pressure increases from 50-400 k Pa. But for the cured samples, the value of $C_{\alpha\epsilon}$ increases considerably with pressure. $C_{\alpha\epsilon}$ for specimen cured for 3 months is 33.7% of that of the untreated specimen for the pressure increment 200-400 k Pa. Thus as indicated earlier, with regard to deformation criteria the cement stabilised clay will have more beneficial effects at lower pressures.

It can be seen from Fig.4.4.13 that the value of $C_{\alpha\epsilon}$ decreases with increase in curing period. The reduction is considerable for the first 30 days and marginal thereafter. For example, for a pressure increment of 25-50 k Pa, the $C_{\alpha\epsilon}$ is 4.03×10^{-3} when no curing period was allowed. When cured for 30 days, the value reduced to 1.41×10^{-3} (34.9%). For 90 days, the value further reduces to 0.71×10^{-3} (17.7%).

4.4.3.5 Development of bond strength

The development of bond strength on curing specimens with different cement content for different curing periods is presented in Fig.4.4.14. It can be seen from the figure that the development of bond strength (preconsolidation pressure) with time for specimens treated

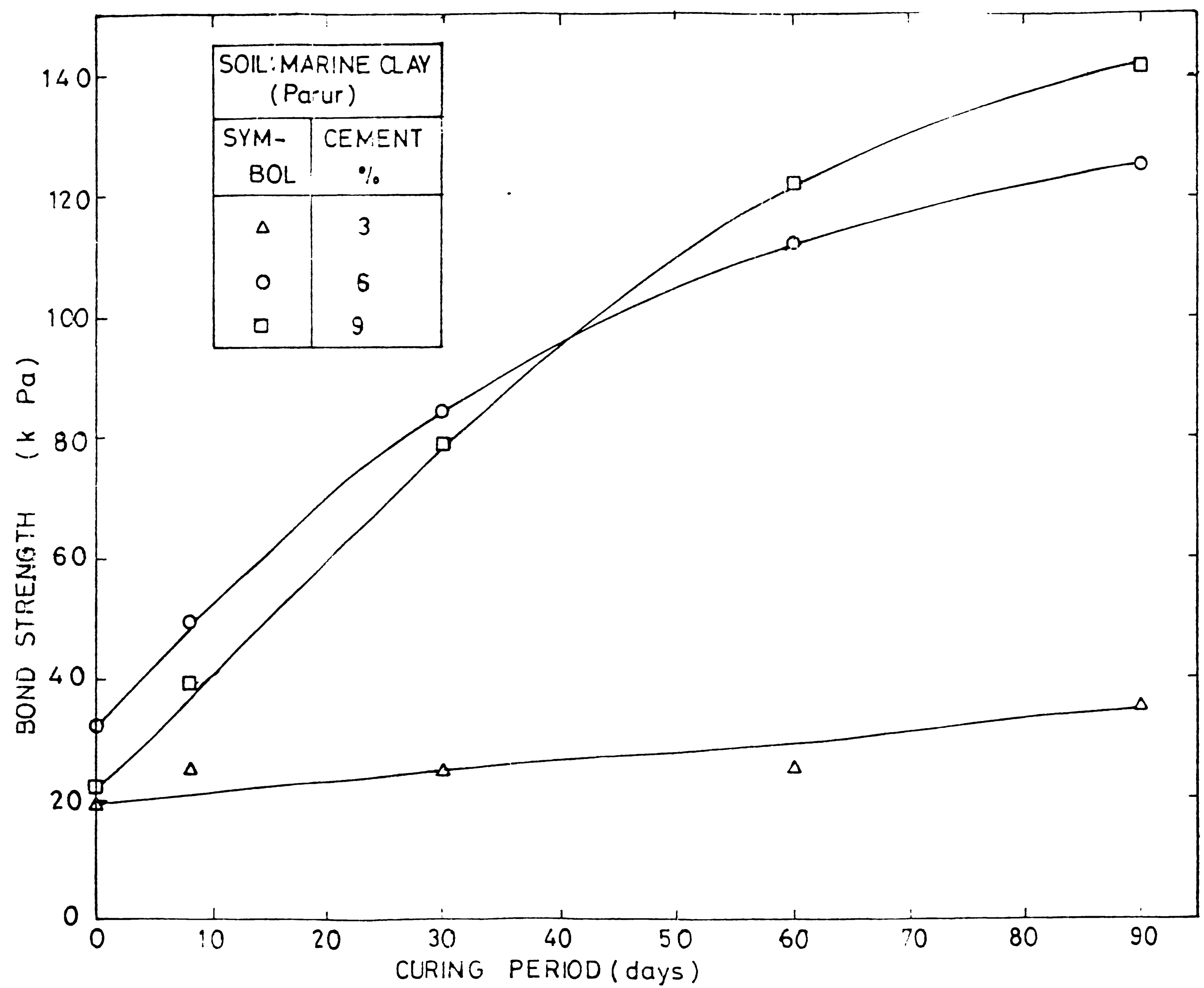


Fig.4.4.14 DEVELOPMENT OF BOND STRENGTH ON CURING FOR CEMENT TREATED SAMPLES

with 3% cement is marginal. Seventyone percent of the maximum bond strength (90 days) is developed within one week itself. As the cement content increases, rate of development of bond strength decreases. For instance, for the curing period of 1 week, the bond strengths developed are 40% and 26.7% of the maximum bond strength developed for 6% and 9% respectively. The bond strength (preconsolidation pressure) was determined by the log-log method.

The fact that the rate of gain in strength reduces with increase in percentage of stabilising agent has been established in case of lime also. For purposes of comparison, results obtained from lime stabilisation of Cochin marine clays are presented in Fig.4.4.15. It can be seen that the trend for development of bond strength is almost the same for both cement and lime.

The rate of increase in bond strength in case of marine clays treated with lime and cement are different. Fig.4.4.16 shows the development of bond strength with curing period for two specimens of marine clay treated with 6% cement and lime. It can be seen from the figure that there is an almost instantaneous gain in strength on lime stabilisation and most of the bond strength is achieved within 1 month itself. But in case of cement, the initial

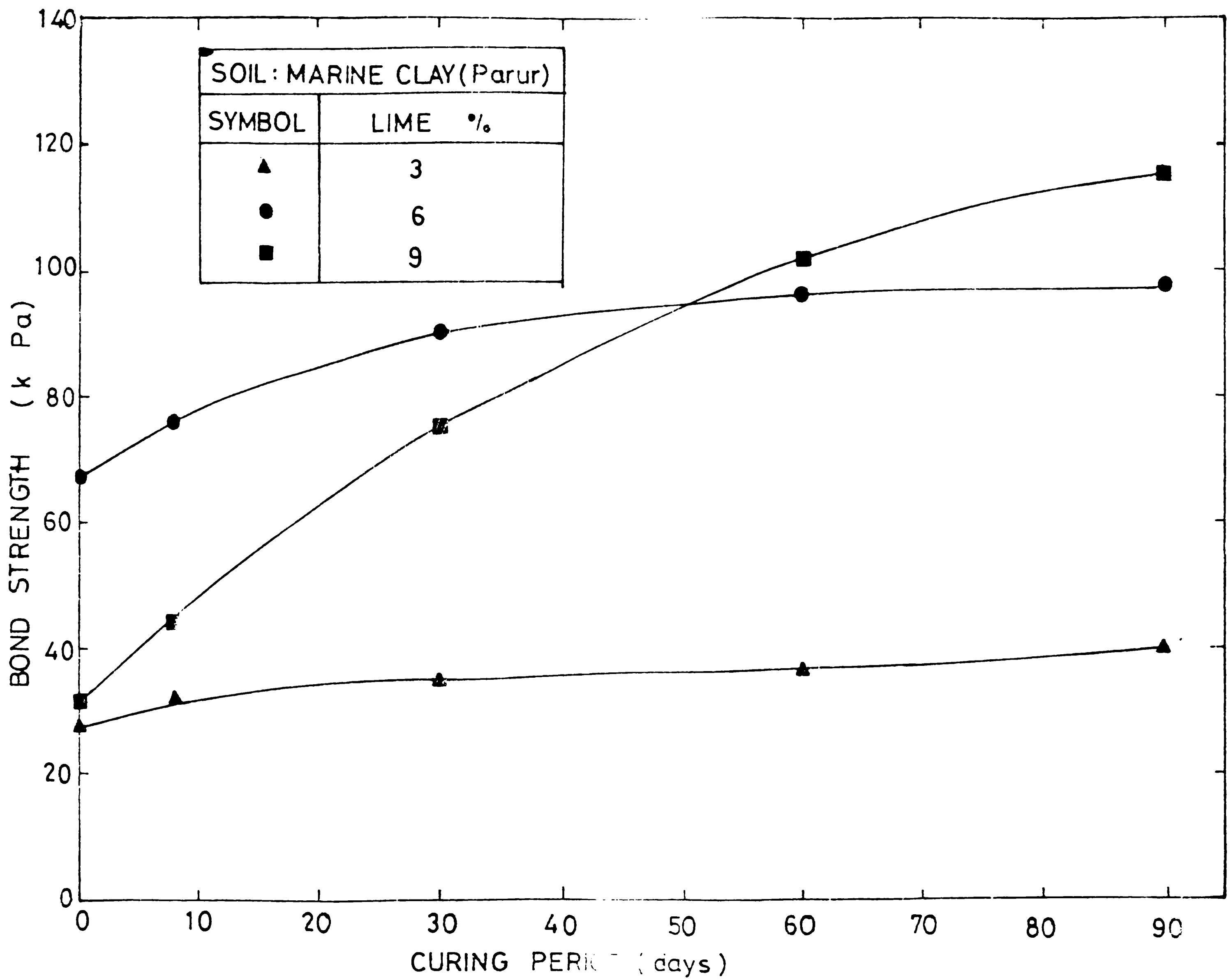


Fig.4.4.15 DEVELOPMENT OF BOND STRENGTH ON CURING FOR LIME TREATED COCHIN MARINE CLAY

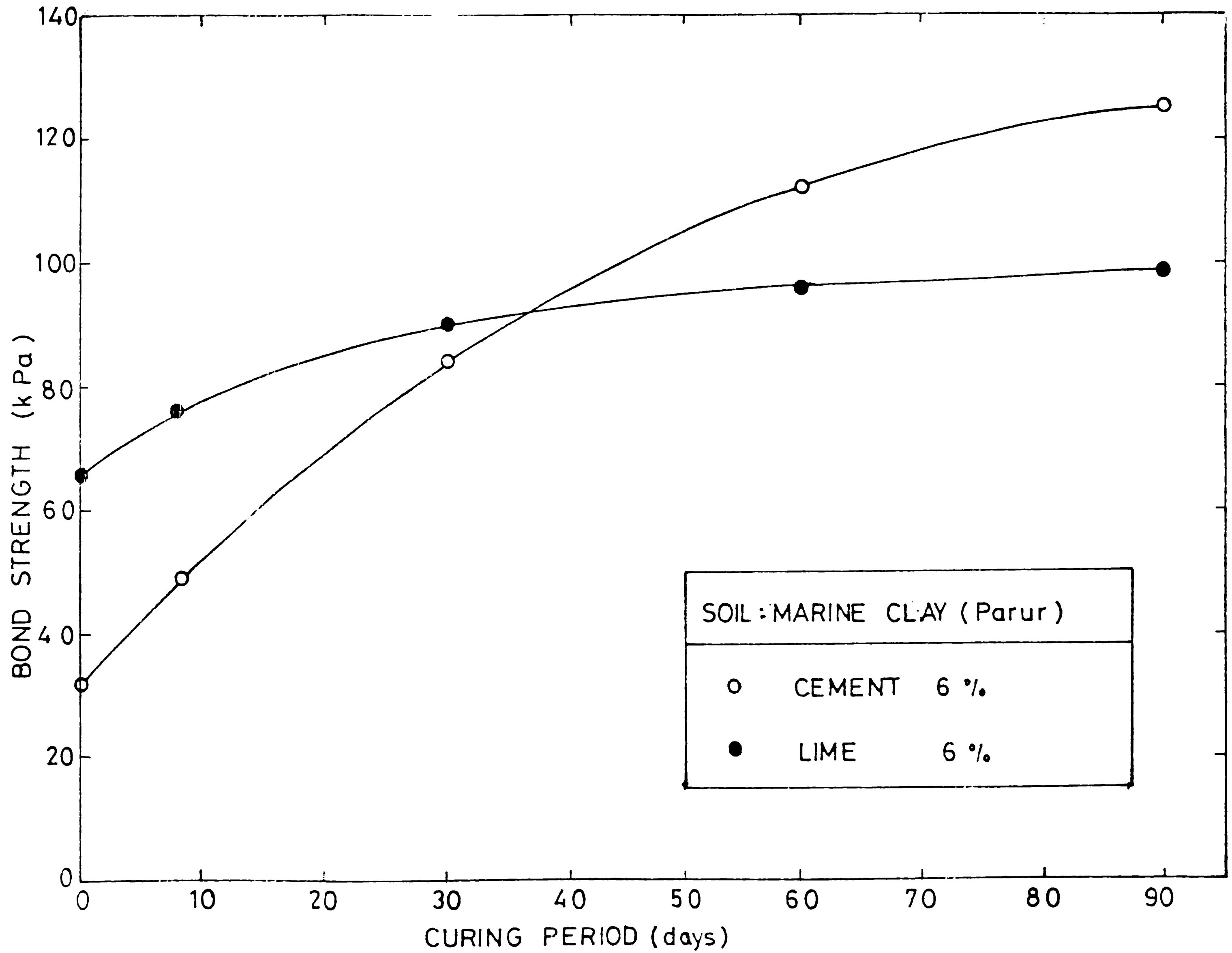


Fig.4.4.16 DEVELOPMENT OF BOND STRENGTH ON CURING FOR CEMENT AND LIME TREATED SAMPLES

gain in strength is less than 50% of that of lime stabilised specimen. But the cement treated samples keep up a steady pace of growth and exhibit greater bond strength than lime stabilised soil after the first 35-40 days.

4.4.4 Rebound characteristics

According to Terzaghi's theory of consolidation, any force exerted on a fully saturated soil mass is taken up instantaneously by an increase in the pore water pressure. During consolidation, the excess pore pressure slowly dissipates as the pressure is transferred to the soil fabric by the development of effective stress. Thus the stress is resisted by the soil skeleton. Upon removal of the external load, there will be an inevitable elastic rebound. The soil structure will manifest the strength it has gained due to development of bonds between clay particles during consolidation. Thus the rebound characteristics can throw some light on the elastic characteristics of the soil fabric in case of both treated and untreated clays.

Figure 4.4.17 shows a set of rebound curves for untreated and cement treated Cochin marine clays. Specimens consolidated upto 400 k Pa were allowed to rebound/swell at the same loading stages. The unloading curve of the untreated Cochin marine clay shows the maximum rebound. When the load was released to the original seating load of 6.25 k Pa, a swell of 533 divisions was observed. In the case of specimen treated with 6% cement and consolidated without allowing any curing period the rebound was as low as 157 divisions, ie., just below 30% of that of the untreated sample. While the significant swell in the case of untreated soil can be attributed to the elasticity of the original soil fabric, the poor rebound in case of treated soil may indicate a thorough rearrangement of soil grains due to the physico-chemical changes consequent to the interaction with the stabilising agent. Eventhough the treated specimens show higher strength, the fact that there have been significant changes in the nature of interparticle forces is broughtout by the contrast in the behaviour of the two specimens. As we allow more curing period, the chemical bonds developed becomes stronger as shown by the marginal increases in swell for longer curing periods.

In order to compare the behaviour with that for lime treated Cochin marine clay, the studies were repeated on specimens stabilised with 6% lime. It can be seen from Fig.4.4.18 that the pattern of rebound curves are in excellent agreement with the observations in the case of cement treated clays, eventhough the magnitudes of swell are slightly lower.

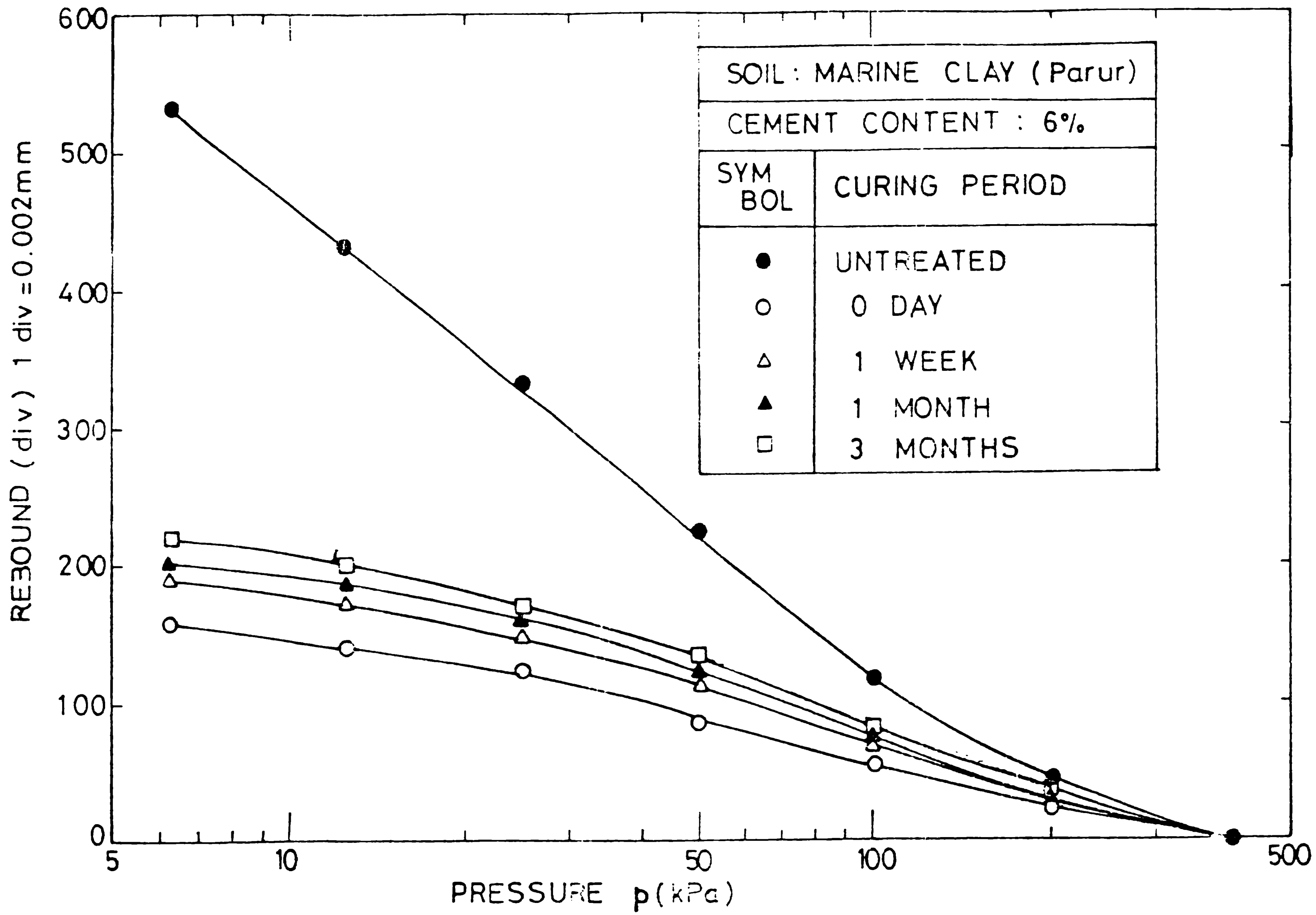


Fig.4.4.17 REBOUND CURVES FOR CEMENT TREATED COCHIN MARINE CLAY

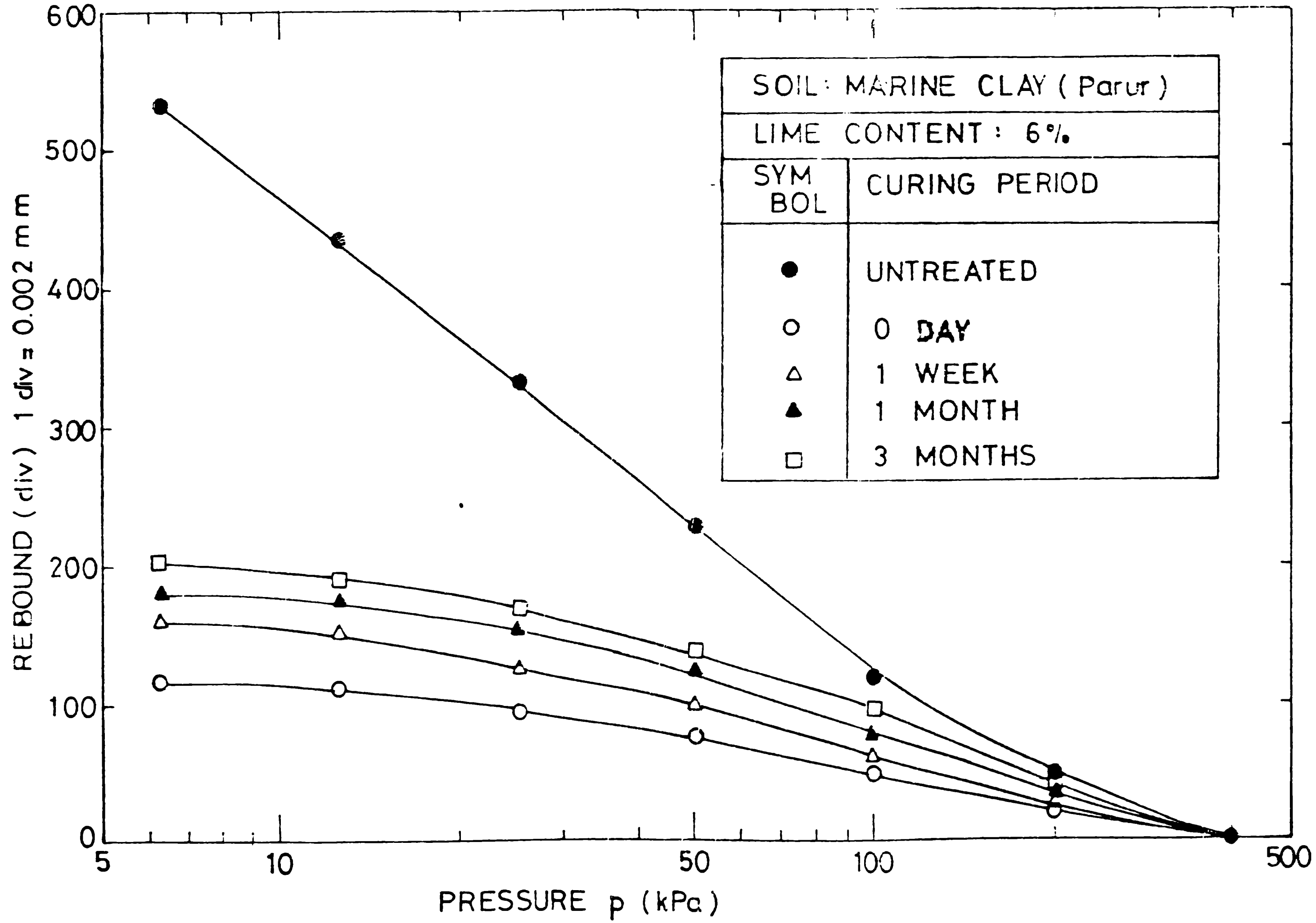


Fig.4.4.18 REBOUND CURVES FOR LIME TREATED COCHIN MARINE CLAY

Chapter V

SHEAR STRENGTH CHARACTERISTICS OF COCHIN MARINE CLAYS

5.1 Introduction

For an engineer in pursuit of solutions for geotechnical problems, the need for an accurate assessment of shear strength of soil can hardly be overemphasized. In a variety of problems such as bearing capacity of footings and piles, stability of slopes, dams and embankments and earth pressures, the design virtually stems from the shear strength parameters C and ϕ.

Compared to other civil engineering materials, soil, with its multicomponent polyphase system at times even makes even a definition of strength or failure, most intriguing. The problem is further compounded by the numerous factors which influence the shear strength such as type of clay mineral, water content, nature of pore water, stress history, drainage during shear, stress path, rate of loading, temperature, soil structure, anisotropy, sensitivity etc. Thus a unique value for shear strength can hardly be attained in case of soils especially in case of clays.

It is comparatively easier to visualise the shear resistance of coarse grained materials. But in case of cohesive soils, the mechanism is yet to be explained conclusively, as the physico-chemical forces within the system play an equally important role as the external loads. Here, an attempt has been made to evaluate the shear strength of Cochin marine clays in its natural state and under certain specific conditions in addition to investigations to improve the strength characteristics through stabilisation.

5.2 Shear strength characteristics of marine soils

Consolidated Undrained Tests

Series of triaxial shear tests were conducted on undisturbed samples of marine clays collected from Cochin as well as Mangalore, in order to study the stress-strain characteristics of marine sediments. Fig.5.2.1 shows the deviator stress vs. axial strain for ambient pressures ranging over 50 to 400 k Pa for Cochin marine clay collected from a depth of 11 m below the bed of backwaters. Pore pressure measurements were also made in these consolidated undrained tests. It can be seen from the figure that pore pressure initially increases with consolidation pressure (50-200 kPa). $\overline{CU}$ tests with σ_3 = 300 and 400 k Pa showed lower pore pressures. This can be attributed to the dilatency of marine clay samples behaving as overconsolidated clays. The stress-strain curves for the lower chamber pressures show a peak value, which remains unchanged as strain increases further.

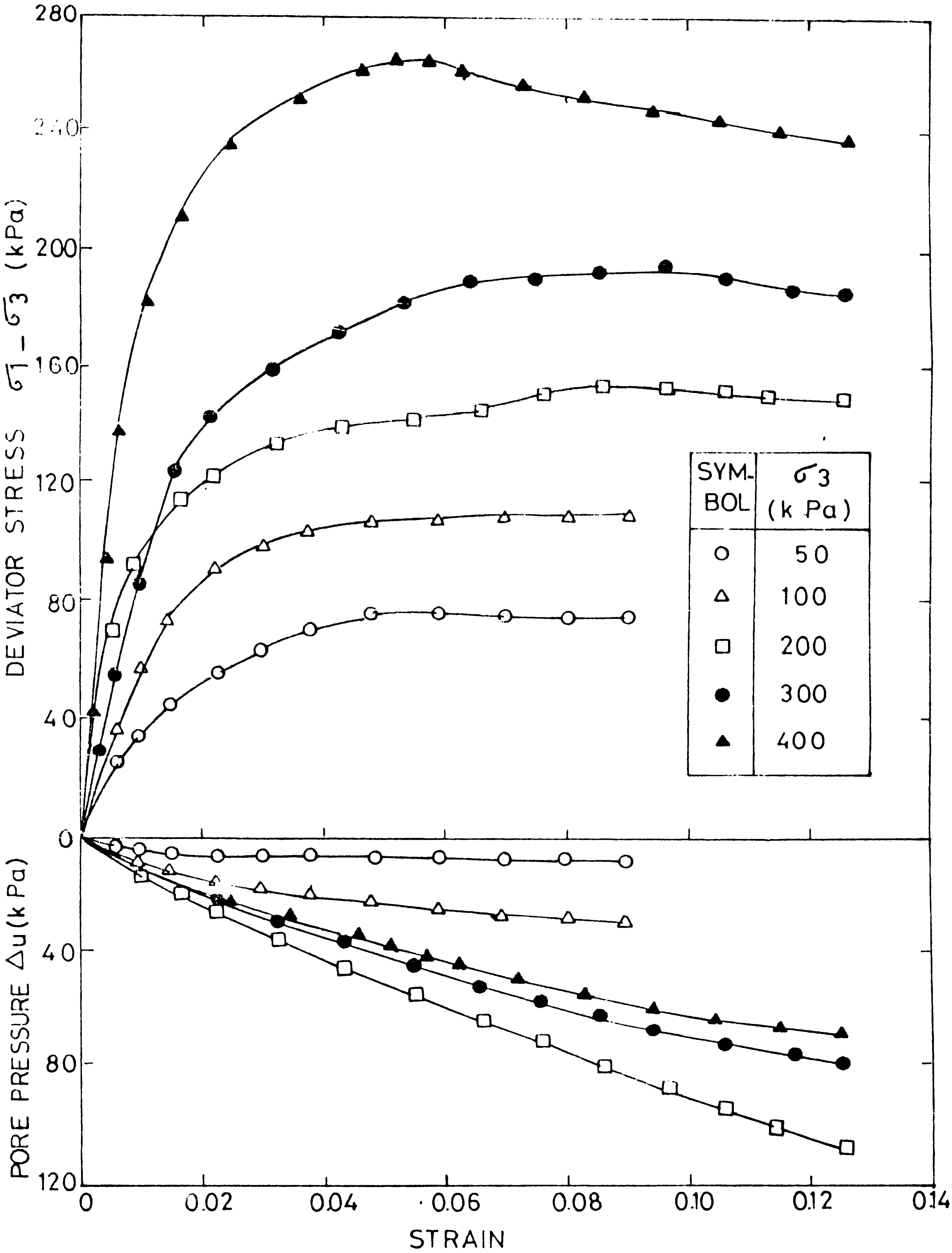


Fig.5.2.1 VARIATION OF DEVIATOR STRESS AND PORE PRESSURE FOR UNDISTURBED COCHIN MARINE CLAY (CU TEST)

But in the case of tests carried out with ambient pressures of 300 and 400 k Pa, the deviator stress reaches a peak value and tend to decrease as strain increases.

Figure 5.2.2 shows similar stress-strain curves for an undisturbed marine clay sample collected from 4.5 m below sea bed about 2 km off Mangalore coast. It can be seen that the stress-strain behaviour and the pore pressure plots are quite consistent with the results of the tests on Cochin marine clay. The effects of dilation for higher ambient pressures and the similar nature of the stress-strain curve can be noticed here also.

The variation of initial tangent modulus with confining pressure for marine clays collected from Cochin and Mangalore are shown in Fig.5.2.3. It can be seen that they are directly proportional to confining pressures with Cochin marine clay giving higher values.

Figure 5.2.4 shows the two series of effective stress paths obtained from Cochin clays and Mangalore clays. It can be seen from the set of curves that the stress paths are inclined almost at 45° initially. This slow rate of development of pore pressure is indicative of the availability of a higher fabric strength in marine sediments compared to

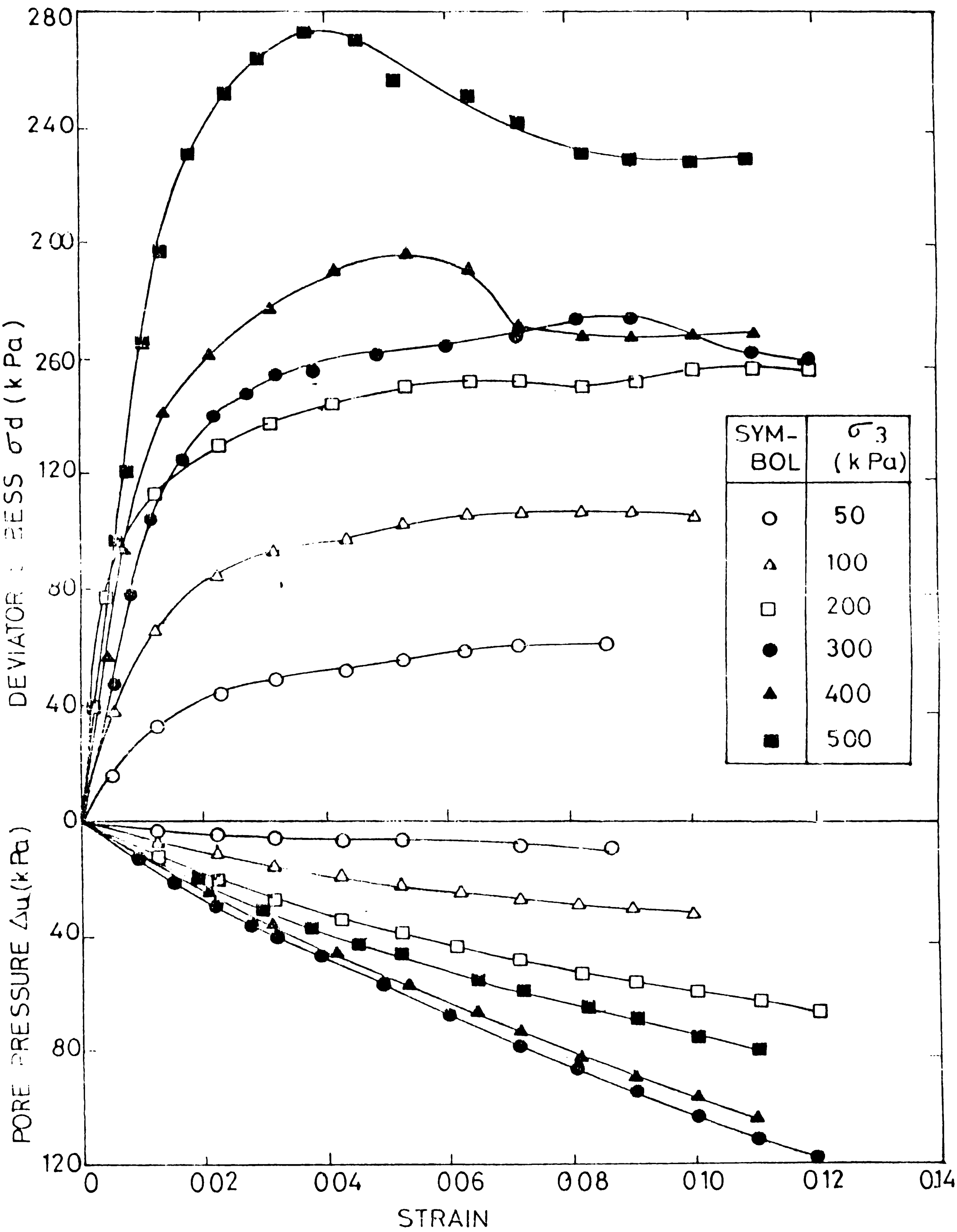


Fig.5.2.2 VARIATION OF DEVIATOR STRESS AND PORE PRESSURE FOR UNDISTURBED MANGALORE MARINE CLAY (CU TEST)

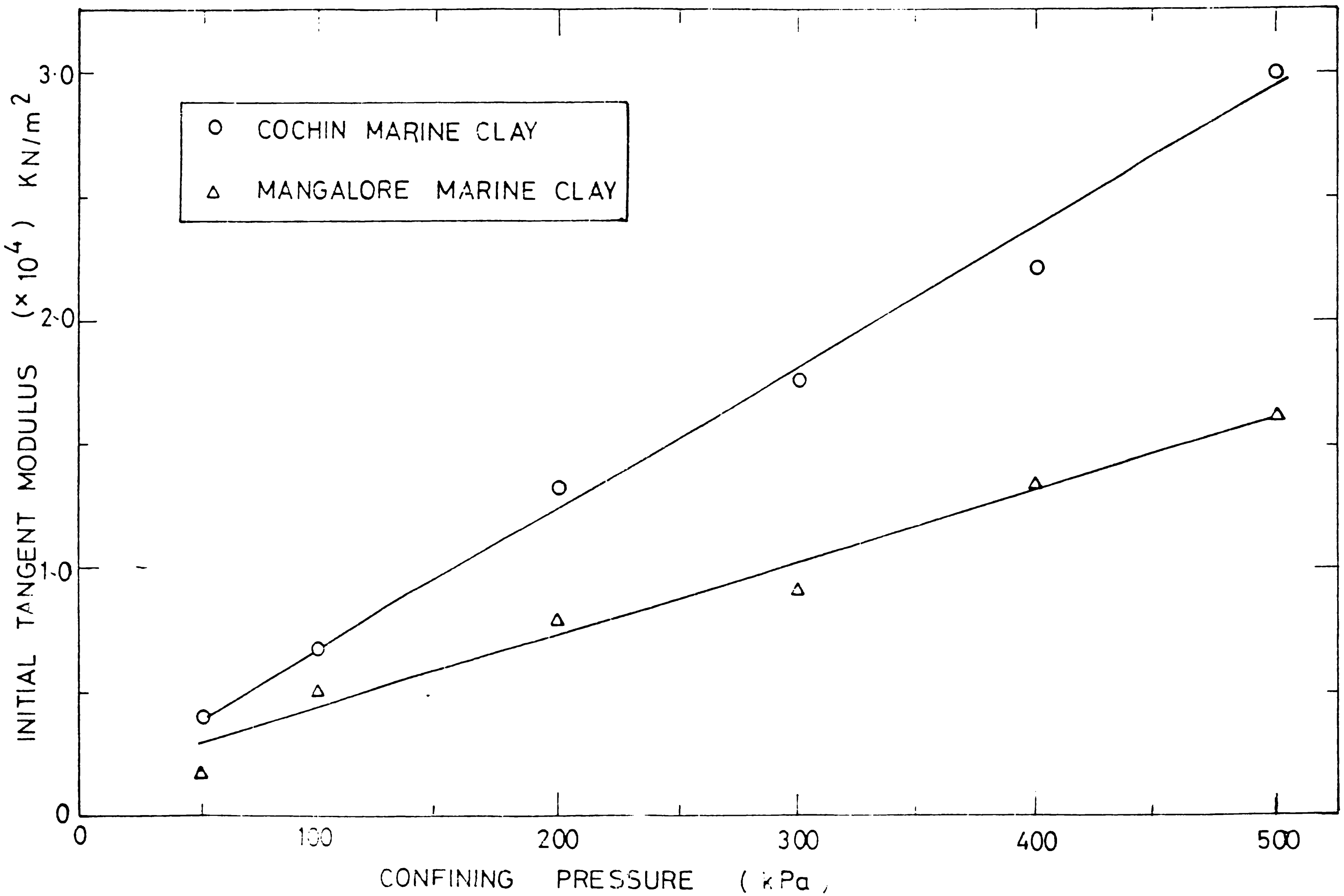
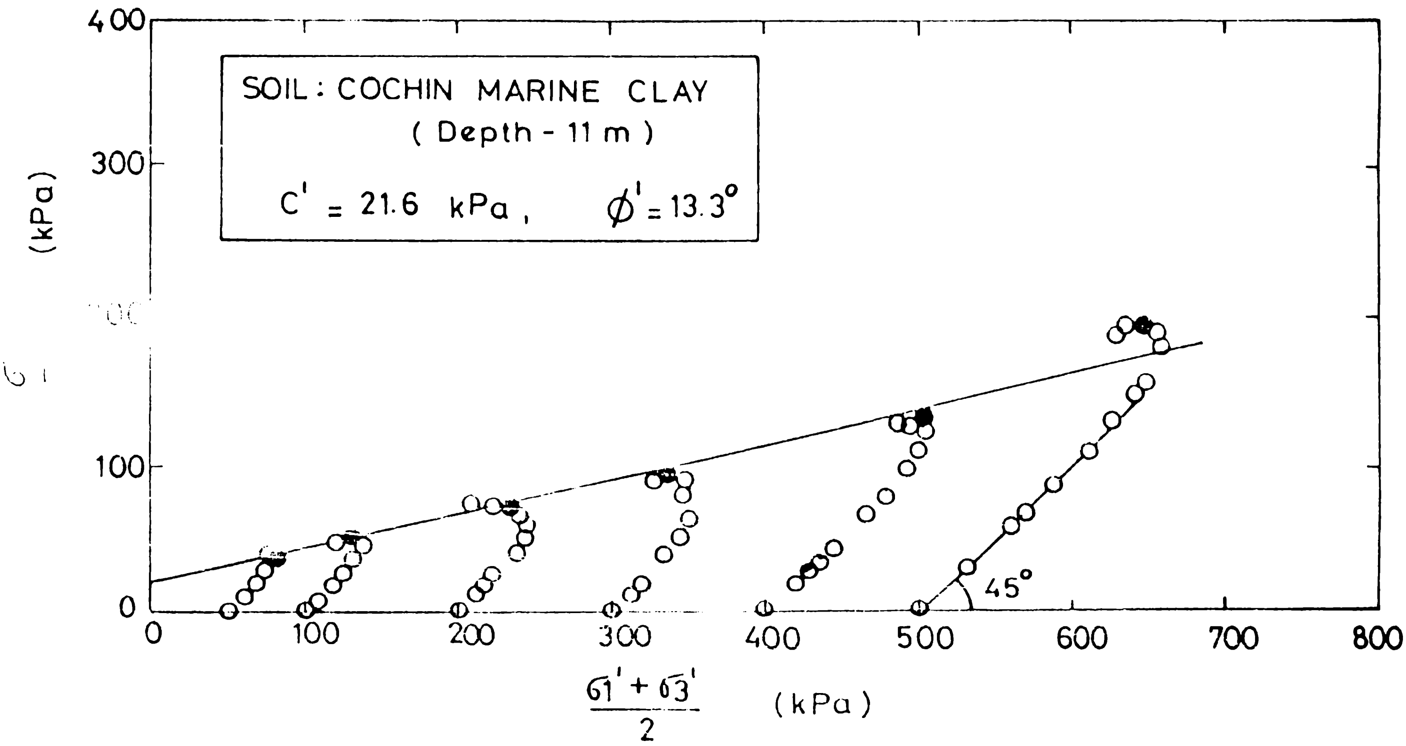


Fig.5.2.3 VARIATION OF INITIAL TANGENT MODULUS WITH AMBIENT PRESSURE (FROM CU TESTS)



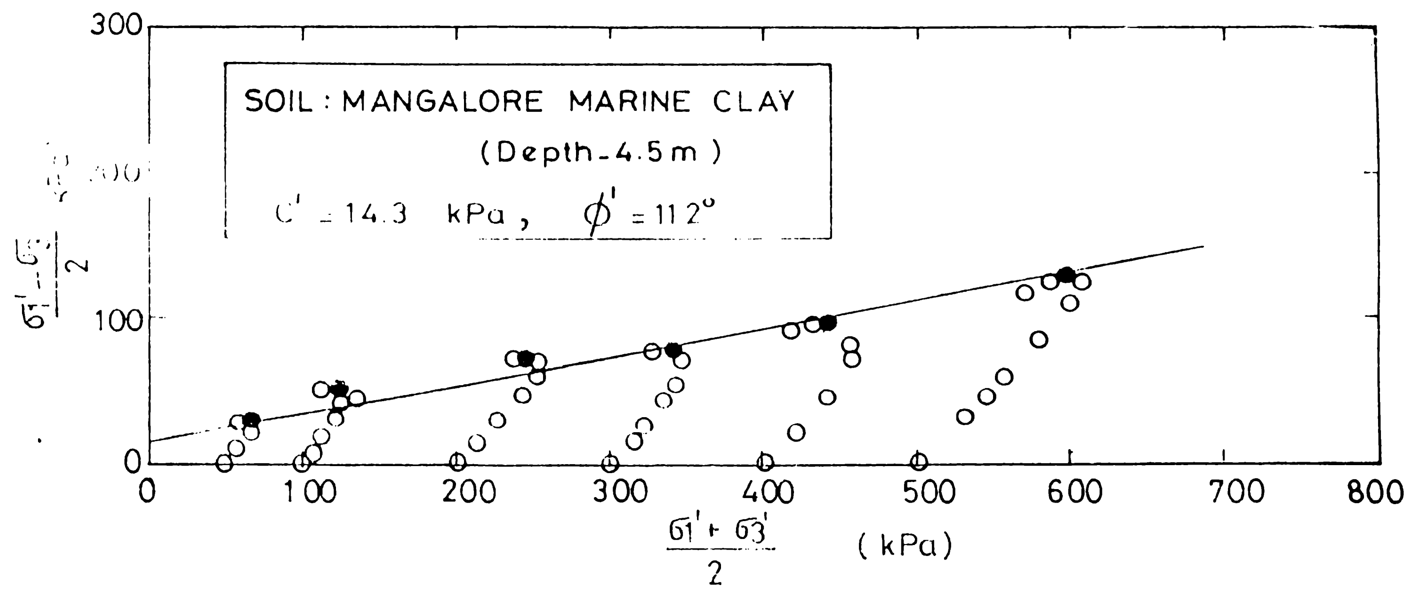


Fig.5.2.4 EFFECTIVE STRESS PATHS FOR COCHIN AND MANGALORE CLAYS

normal clays. As deviator stress increases, the fabric gets compressed accompanied by a faster increase in pore pressure. The stress paths gets shifted off the 45° line and failure is reached without any significant increase in deviator stress, indicative of the flocculant clay fabric.

Strength envelope drawn through the coordinates [illegible]cting the failure in each triaxial shear test gives a value of $C' = 21.6$ k Pa and $\phi' = 13.3°$ for Cochin clay and $C' = 14.3$ k Pa and $\phi' = 11.2°$ for Mangalore clay, as can be seen in Table 5.2.1.

The variation of A_f for various consolidation pressures are shown in Fig.5.2.5 for undisturbed marine clay samples collected from Cochin and Mangalore. It varies from 0.09 to 0.33 for Cochin marine clays and 0.16 to 0.65 for Mangalore clays. The maximum values are obtained for consolidation pressures around 200-250 k Pa. The A_f values decrease thereafter which may be due to microdilation at higher stress levels.

According to Leonards (1962), for normally consolidated clays, A_f values range over 0.7 to 1.3. As preconsolidation increases, A_f values tend to decrease. The range of the present values indicate that the samples behave

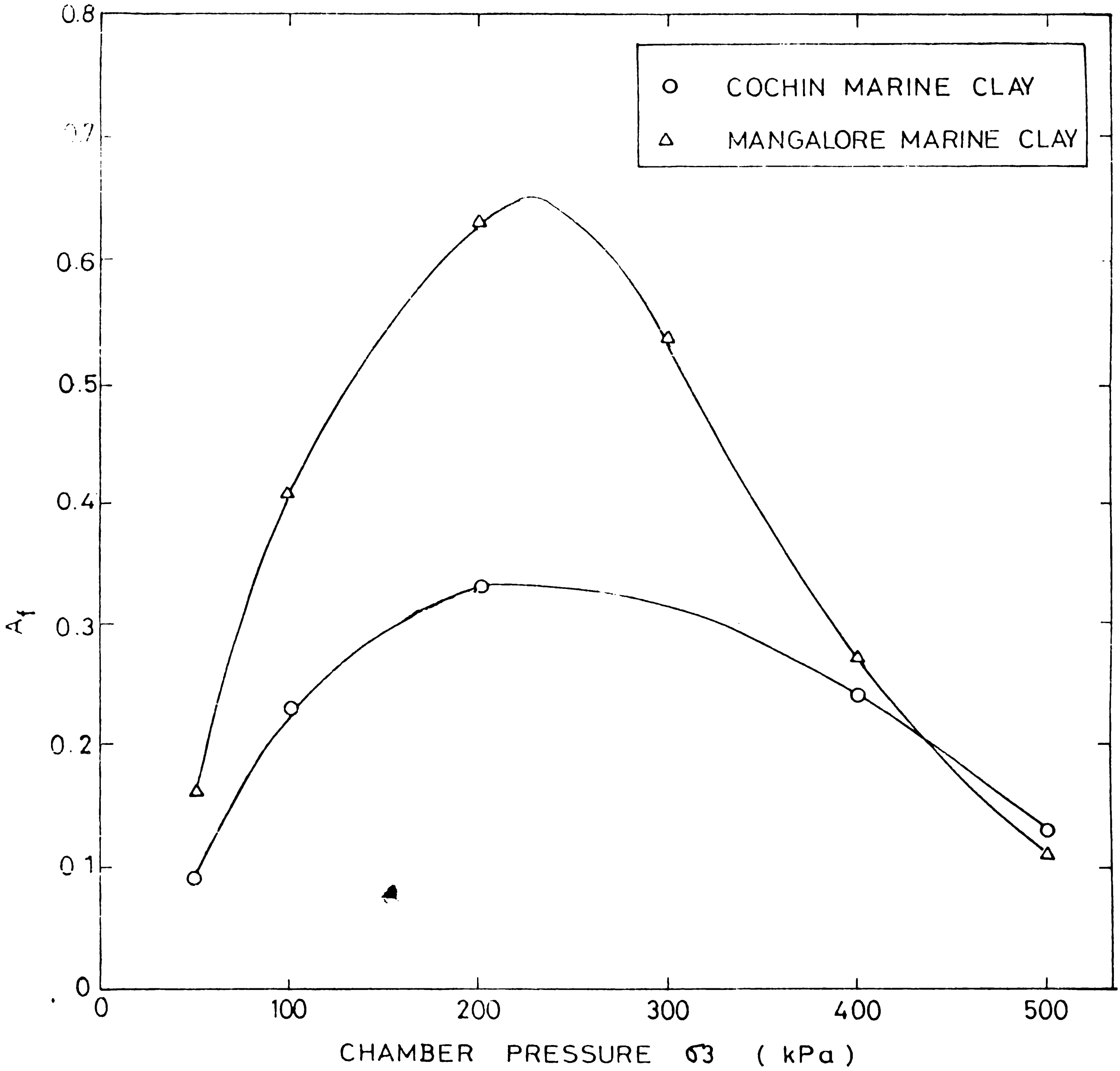


Fig.5.2.5 VARIATION OF 'A' PARAMETER WITH AMBIENT PRESSURE

as though they are lightly preconsolidated. This could be attributed to the additional bond strength, typical of marine clays.

Drained Tests

The results of consolidated drained tests conducted on Cochin marine clay samples collected from 8 m below bed of backwaters are presented in Fig.5.2.6. As in the case of pore pressure measurements in CU tests, the volume changes are low to start with, pick up as consolidation pressures increase to record a maximum σ_3 = 200 k Pa and decrease thereafter. Unlike $\overline{CU}$ tests, wherein the stress-strain curves recorded steady values after the peak, the results of CD tests show ups and downs after the peak values, a behaviour similar to direct shear test results. This can be attributed to dilation effects.

Fig.5.2.7 shows the results of similar tests performed on samples of Mangalore marine clay. The trends are akin to those of Cochin clays. The values of the strength parameters are given in Table 5.2.1.

Unconsolidated Undrained Test

The stress-strain curves obtained from Unconsolidated undrained tests conducted on samples from a

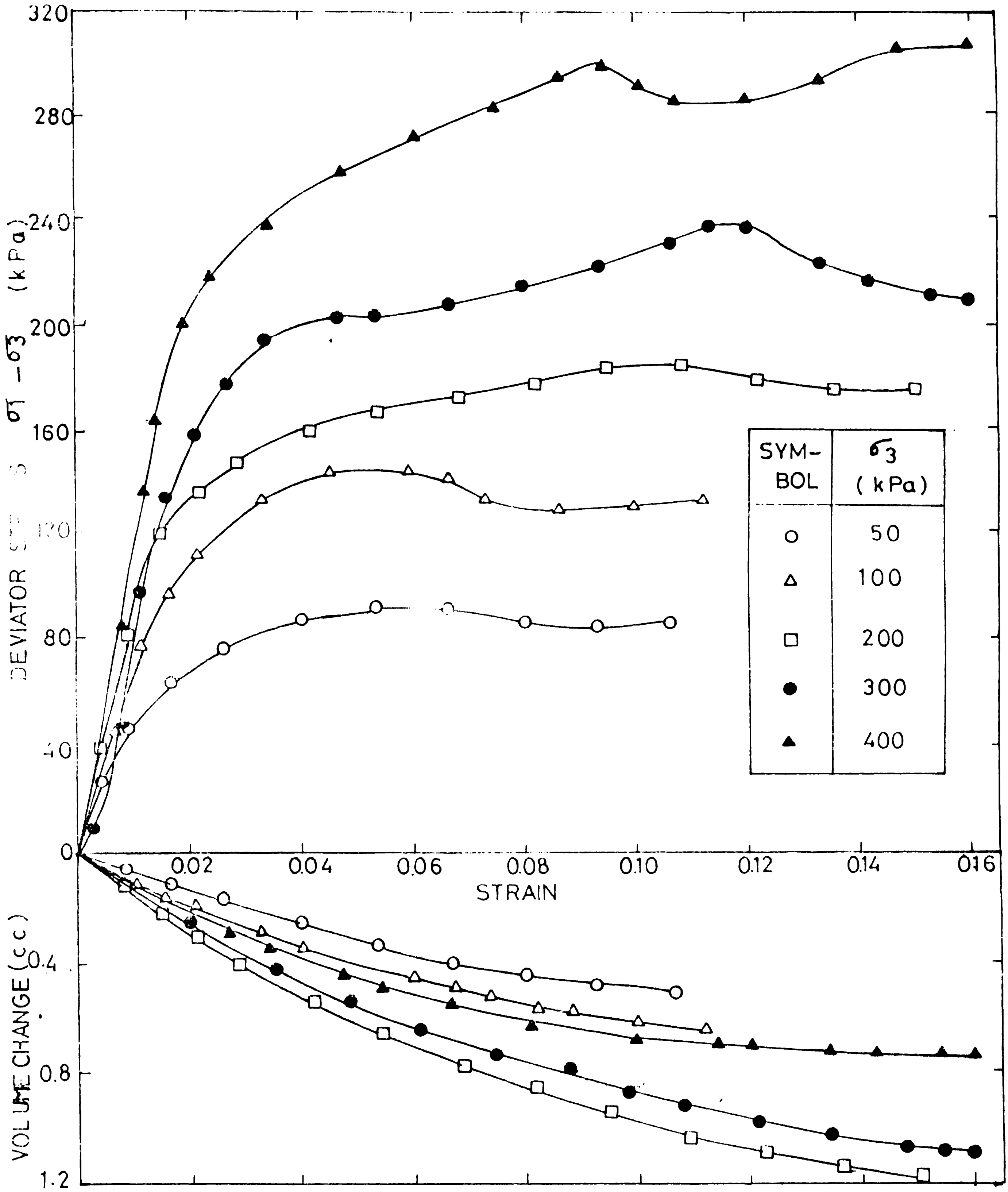


Fig. 5.2.6 STRESS-STRAIN AND VOLUME CHANGE BEHAVIOUR OF UNDISTURBED COCHIN MARINE CLAY (CD TESTS)

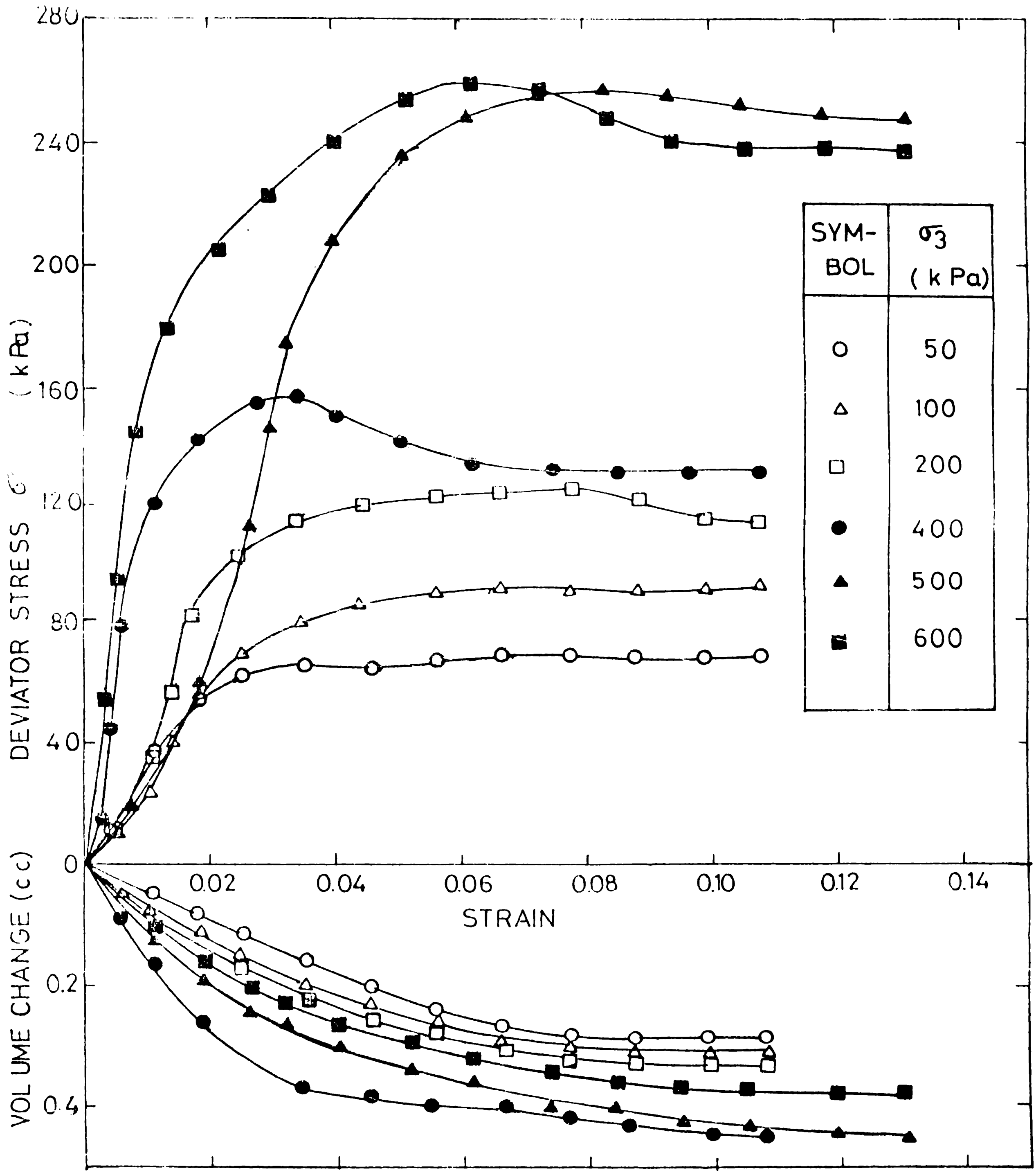


Fig.5.2.7 STRESS-STRAIN AND VOLUME CHANGE BEHAVIOUR OF UNDISTURBED MANGALORE MARINE CLAY (CD TESTS)

Table .2.1

Results of triaxial shear tests on Cochin and Mangalore marine clays

Sl. No.	Location	Depth	Type of test	Ambient pressure σ_3(k Pa)	Deviator stress σ_d(k Pa)	Total stress parameters C (k Pa)	Total stress parameters ϕ (deg.)	Effective stress parameters C' (k Pa)	Effective stress parameters ϕ' (deg.)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1.	Cochin	11m below back water bed	Consolidated undrained with pore pressure measurement	50	78				
				100	110				
				200	153	20.4	11.7	21.6	13.3
				300	195				
				400	262				
2.	Mangalore	4.5m below sea bed	-do-	50	60				
				100	106				
				200	152	16.3	10.7	14.3	11.2
				300	176				
				400	194				
				500	274				

(contd...)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
3.	Cochin	11m below back water bed	Unconsolidated Undrained	50	52				
				100	52				
				200	49	25.2	--	--	--
				300	48				
				400	51				
4.	Cochin	8m below back water bed	Consolidated drained test	50	90				
				100	148				
				200	200	--	--	25.8	14.4
				300	254				
				400	330				
5.	Mangalore	5.5m below sea bed	-do-	50	64				
				100	90				
				200	124	--	--	12.2	11.2
				300	157				
				500	257				
				600	260				

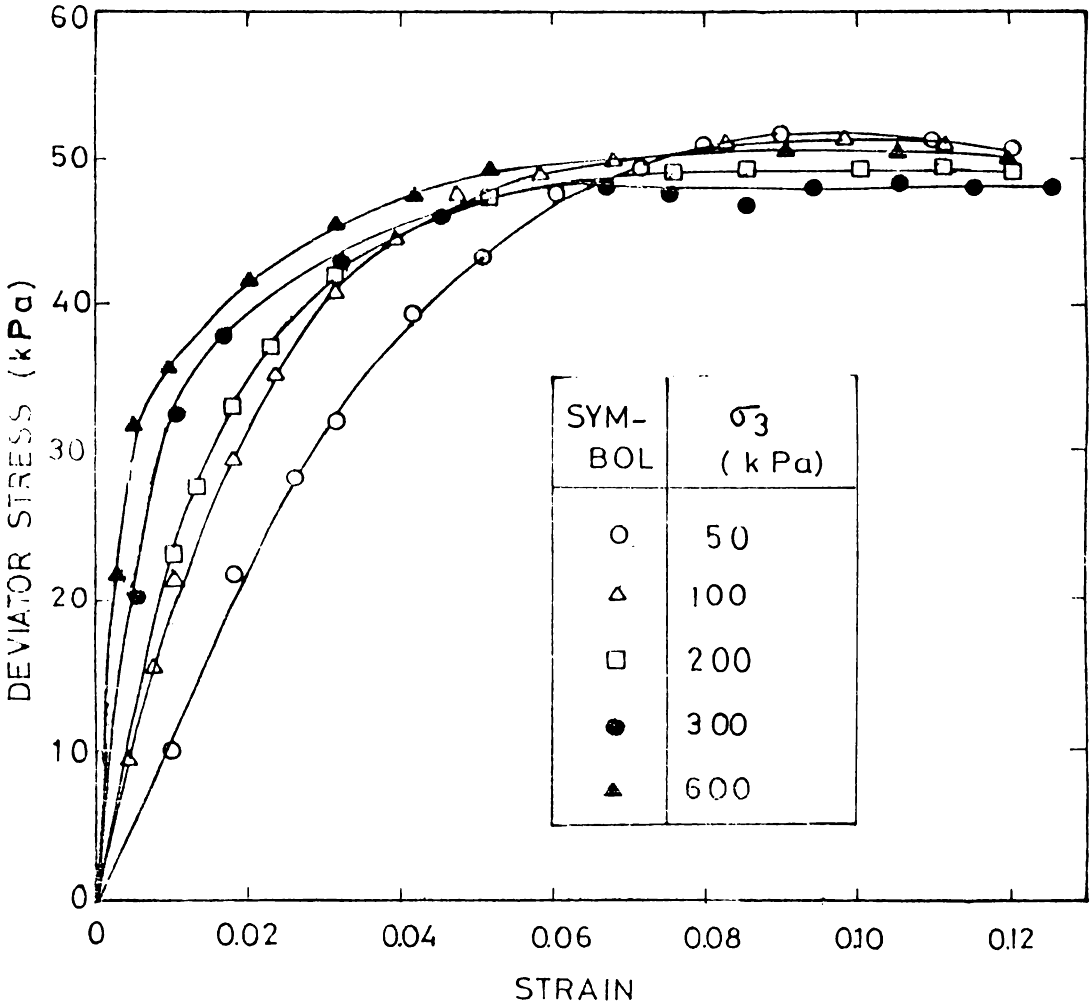


Fig.5.2.8 STRESS-STRAIN CURVES FOR UNDISTURBED COCHIN MARINE CLAY (UU TEST)

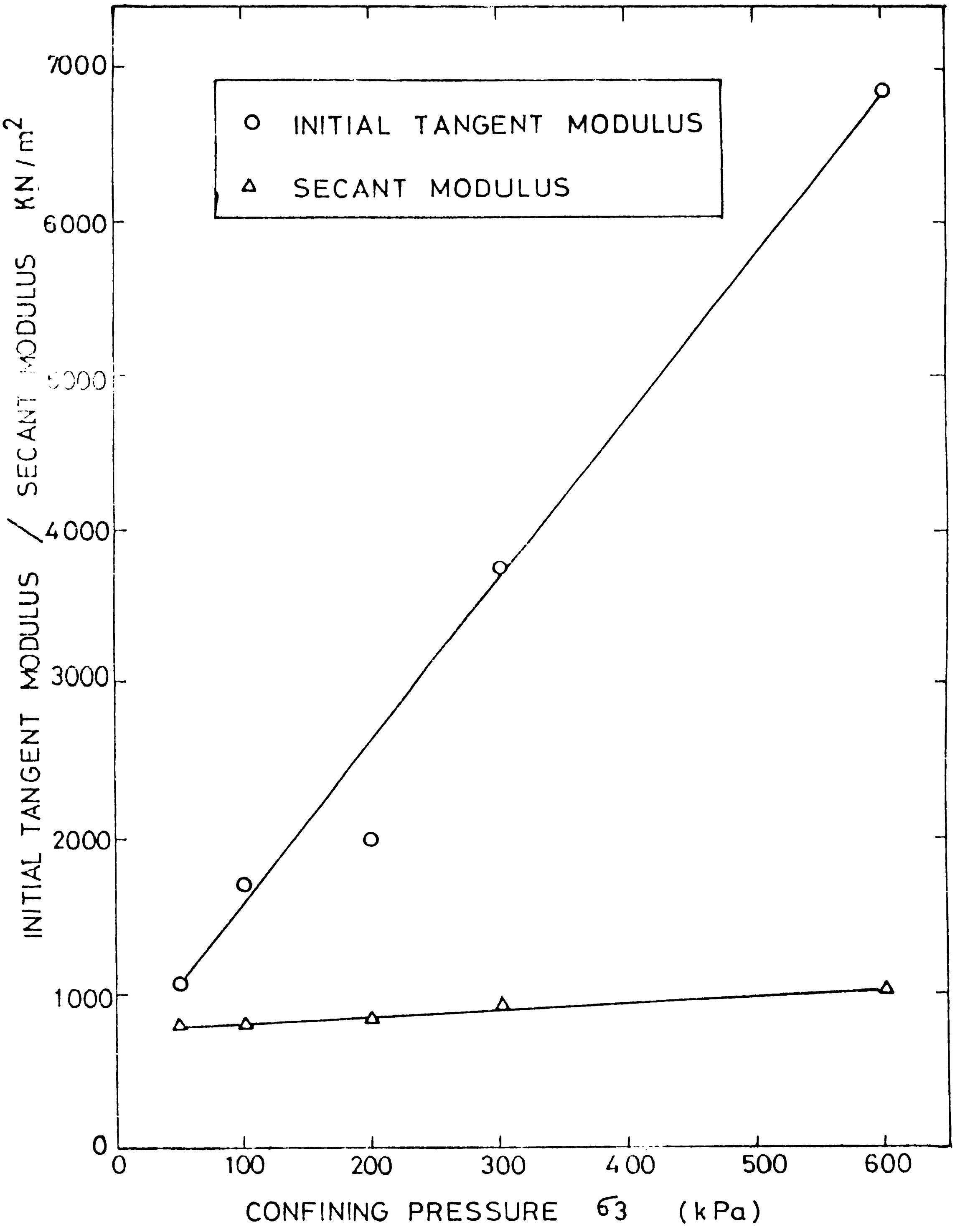


Fig.5.2.9 VARIATION OF INITIAL TANGENT MODULUS AND SECANT MODULUS WITH CELL PRESSURE FOR COCHIN MARINE CLAY (UU TEST)

depth of 11 m below the backwater bed in Cochin are presented in Fig.5.2.8. As expected, the failure values do not vary with consolidation pressure. They fall within a very narrow range, well within permissible limits of sample variations used for the different tests. But as the confining pressure increases, the samples become stiffer and stiffer and the initial tangent modulus increases steadily, as shown in Fig.5.2.9. The figure also shows that the variation of secant modulus is not significant.

5.3 Shear strength characteristics of compacted marine clays

The phenomenal changes that are brought in by air drying and oven drying of Cochin marine clays in their index properties and consolidation characteristics have already been studied in detail (Jose et al, 1988b; Rao et al, 1989; Sridharan et al, 1991; Pandian et al, 1991). The aggregation of finer particles into coarser grains brought about during drying improves the compressibility characteristics considerably. For example, the compression index of moist marine clay is around 1.5. Upon air drying, this gets reduced to about 0.8. When there is such a drastic reduction in the compression index due to air drying, this has to be necessarily accompanied by improvement in shear strength characteristics also. Hence a study on the shear strength characteristics of air dried Cochin marine clay was taken up in the present study. Since air drying of the top layers of Cochin marine clay regions by cheap methods such as ploughing during summer, the potential of such air dried marine clays as an embankment or construction material, was taken up. The details and results of compaction studies on air dried Cochin marine clay are presented below.

Standard Proctor Compaction and Modified Compaction tests were conducted on air dried Cochin marine clay samples. The results are presented in Fig.5.3.1 and Table 5.3.1.

Samples for triaxial shear tests were prepared by both wetting process and drying process. As mentioned earlier, air drying improves the strength characteristics of the moist marine clay. When the soil is used as a construction material as in the case of an embankment, the optimum moisture content obtained from Proctor tests can be reached by the wetting process or by drying process. In the drying process, the moist clay samples are dried to the extent when the natural moisture content gets reduced to OMC. This process is a little involved since drying the moist sample exactly to OMC is a tedious task. But certain advantages of this process will be discussed later.

In case of wetting process, the moist samples are

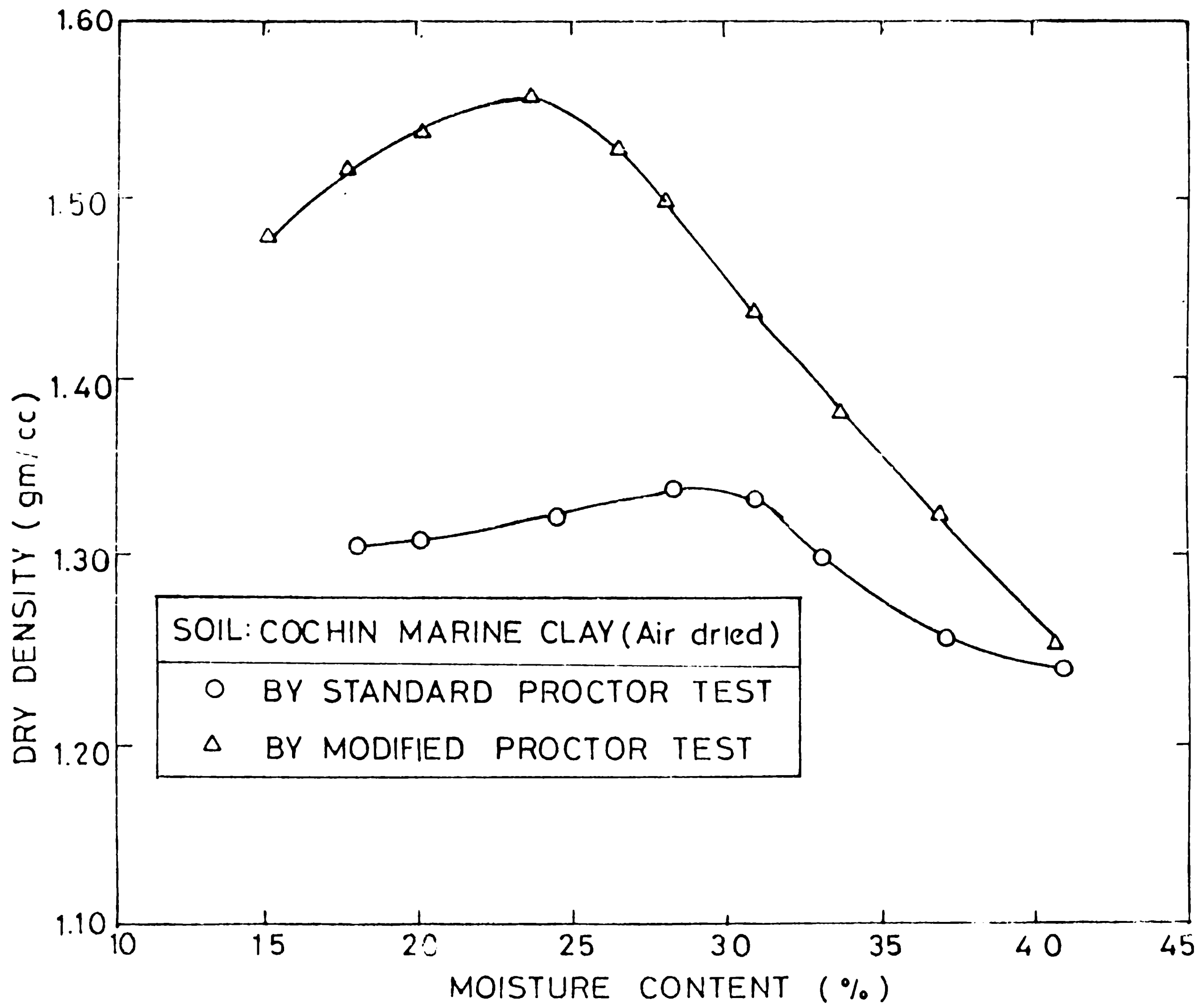


Fig.5.3.1 COMPACTION CURVES FOR AIR DRIED COCHIN MARINE CLAY

Table 5.3.1

Index properties and compaction test results

Sl. No.	Property/test	Moist soil	Soil dried to a m.c. of 25%	Fully air dried soil
1.	Liquid limit (%)	129.0	105.0	93.0
2.	Plastic limit (%)	53.5	38.9	36.2
3.	Plasticity index (%)	75.5	66.1	56.8
4.	Shrinkage limit (%)	20.0	20.5	20.5
5.	Grain size distribution			
	(a) Clay size (%)	47	36	33
	(b) Silt size (%)	38	45	48
	(c) Sand size (%)	15	19	19
6.	Standard Proctor Test			
	(a) Optimum moisture content (%)	--	--	30.4
	(b) Max. dry density (g/cc)	--	--	1.33
7.	Modified Proctor Test			
	(a) Optimum moisture content (%)	--	--	23.2
	(b) Max. dry density (g/cc)	--	--	1.56

full air dried and powdered. The exact quantity of water required to bring the water content to OMC is then added and compacted. Obviously this method has the advantage of better field contro.

The results of triaxial shear tests ($\overline{C}U$ test) carried out on samples prepared by the wetting process and subjected to standard compaction are presented in Fig.5.3.2. The effective stress parameters C' and Ø' obtained from these tests are presented in Table 5.3.2 and they are 34.3 k Pa and 23.5° respectively.

The results of consolidated undrained tests carried out on identical specimens, but prepared by the drying process are presented in Fig.5.3.3. The shear strength parameter C' and Ø' obtained from these tests are 42.8 k Pa and 23° respectively.

For a comparative study of the stress-strain characteristics of specimens prepared by both processes, the test results are combinedly presented in Fig.5.3.4. It can be seen that for all the three ambient pressures, the drying process shows higher shear strength. The C' and Ø' values given in Table 5.3.2 shows that the unit cohesion for drying process is 42.8 k Pa compared to 34.3 k Pa obtained by the

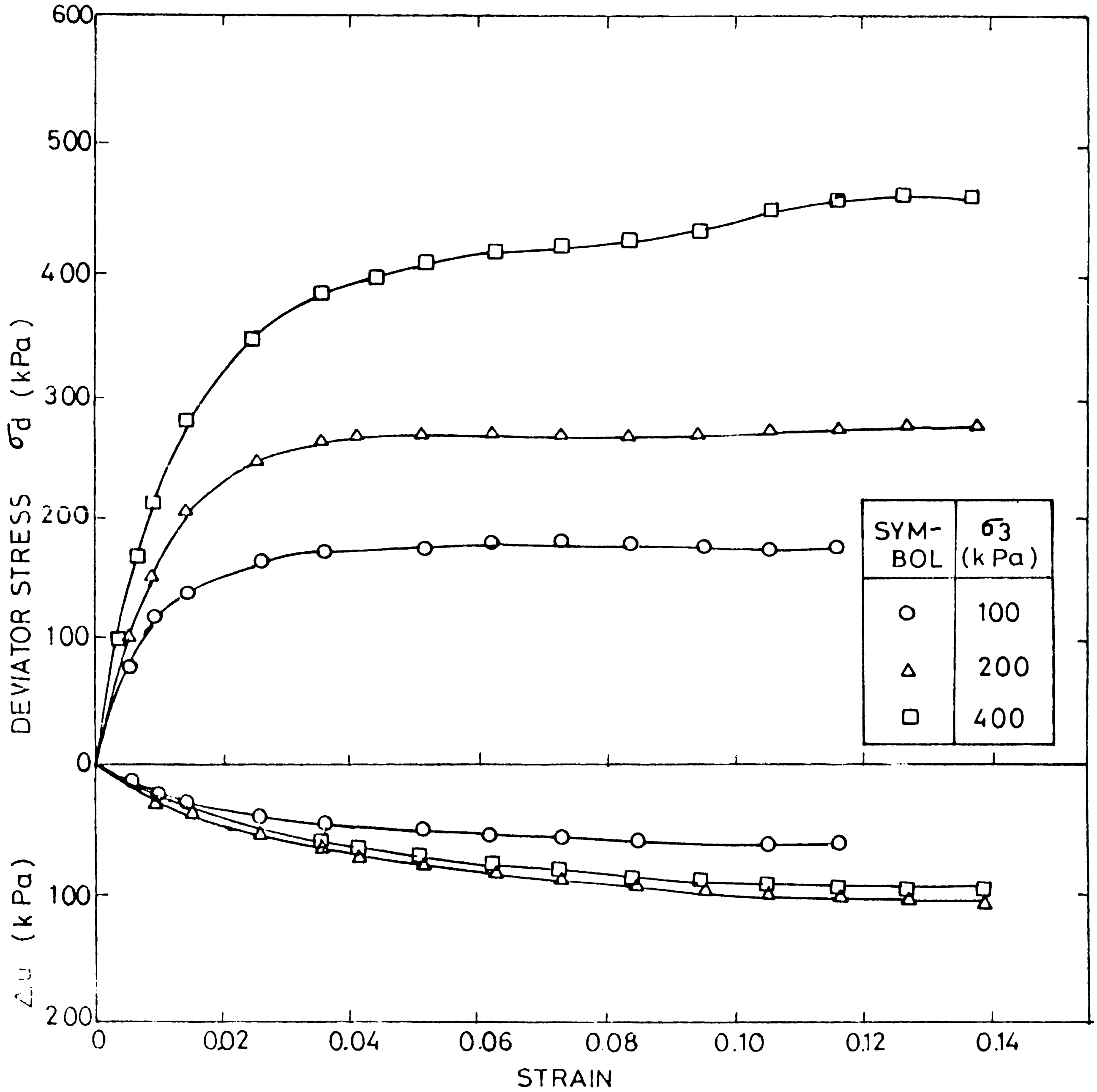


Fig.5.3.2 STRESS-STRAIN CURVES FOR COMPACTED MARINE CLAY (BY WETTING PROCESS) - SAMPLES PREPARED AT STANDARD PROCTOR DENSITY

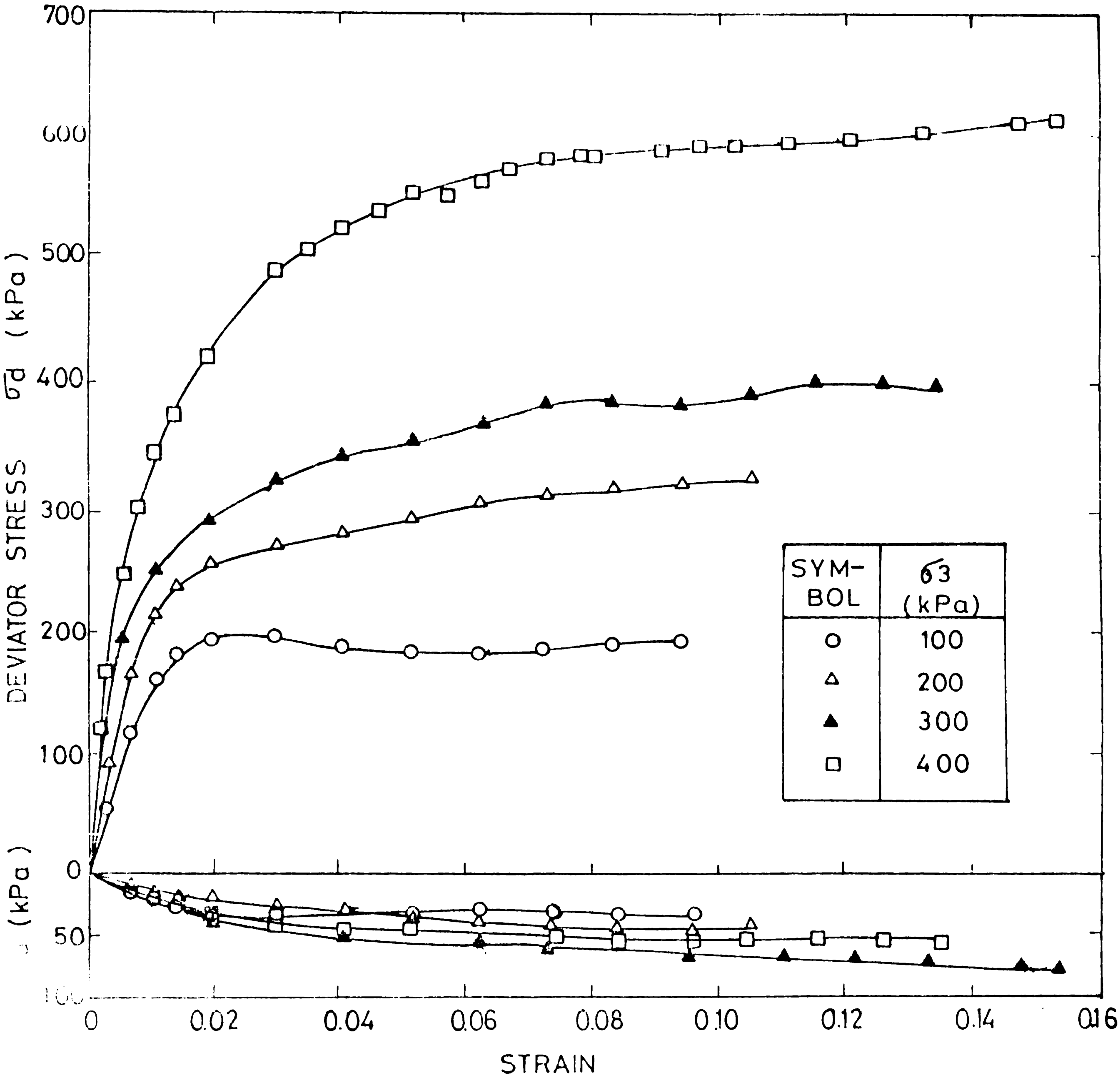


Fig.5.3.3 STRESS-STRAIN CURVES FOR COMPACTED MARINE CLAY (BY DRYING PROCESS) - SAMPLES PREPARED AT STANDARD PROCTOR DENSITY

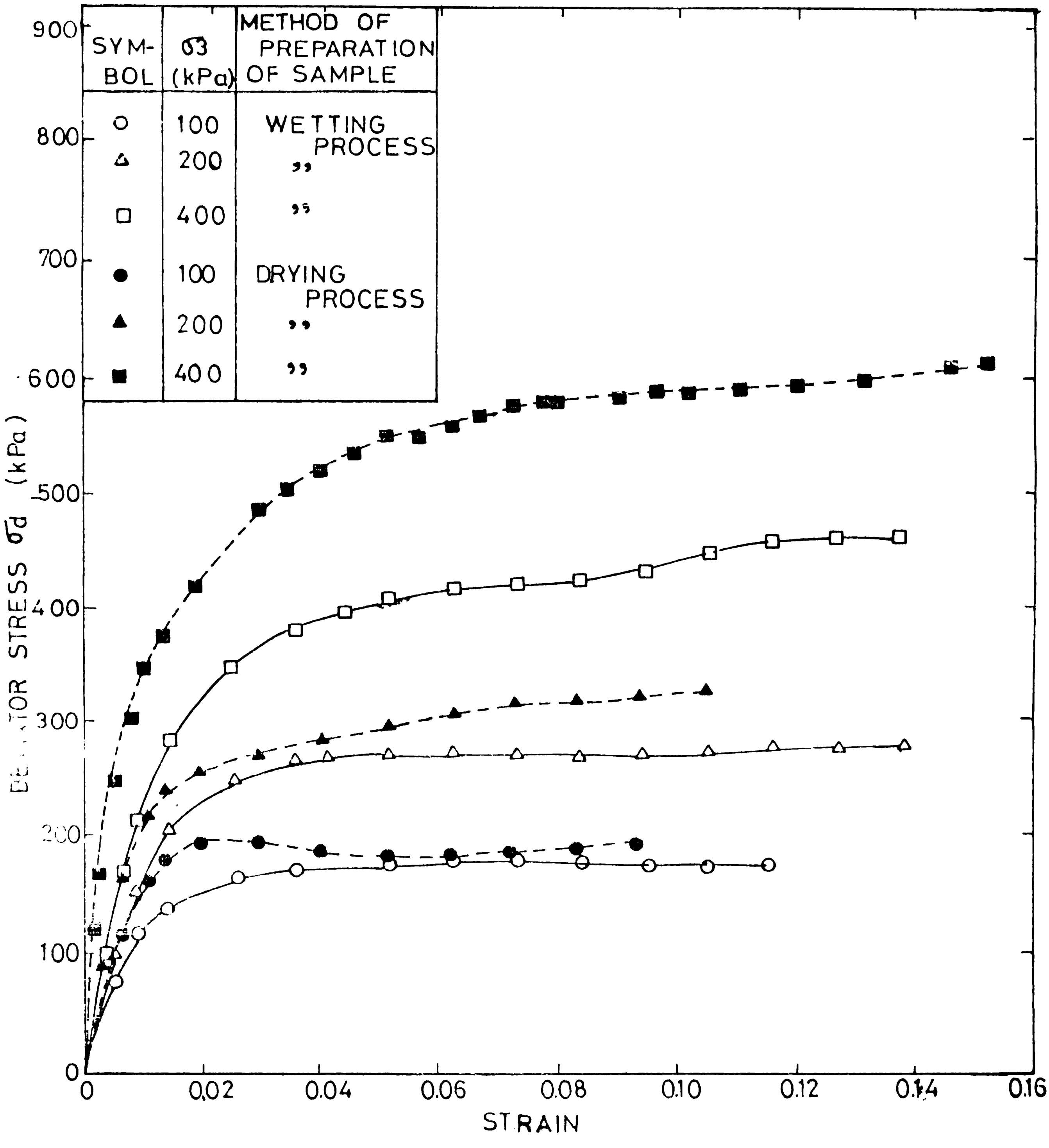


Fig.5.3.4 COMPARISON BETWEEN WETTING AND DRYING PROCESS - SAMPLES PREPARED AT STANDARD PROCTOR DENSITY

Table 5.3.2

Results of triaxial shear ($\overline{CU}$) tests on compacted Cochin marine clay

Sl.	Description of sample	Chamber pressure σ_3 (k Pa)	Deviator stress at failure σ_d (k Pa)	Pore pressure at failure u (k Pa)	Total stress parameters		Effective stress parameters	
					C (k Pa)	ϕ (deg.)	C' (k Pa)	ϕ' (deg.)
1.	Wetting process - Standard compaction	100 200 400	180 280 470	56 105 95	18.5	21.0	34.5	23.5
2.	Drying process - Standard compaction	100 200 300 400	200 330 405 620	33 46 73 53	29.0	20.5	42.8	23.0
3.	Wetting process - Modified compaction	100 200 400	450 585 900	40 46 40	95.8	23.5	110.4	24.5
4.	Drying process - Modified compaction	100 200 300 400	620 710 905 945	5 14 94 34	144.8	23.5	160.4	23.0

wetting process. The Ø' values being almost equal, the gain in strength can be exclusively attributed to the gain in the value of C'.

The above behaviour gives a clue to the changes in the soil fabric obtained from both the processes. On full air drying, a large portion of the clay content undergoes aggregation leaving limited quantity of fines. In case of samples for drying process, the percentage of fines are higher which helps to bind the aggregated particles thereby improving the value of unit cohesion.

The liquid limit values of samples obtained by wetting process and drying process are 93 and 105 respectively as given in Table 5.3.1. Jose et al (1988a, 1988b) and Jose (1989) have brought out that liquid limit is directly proportional to clay content in case of Cochin marine clays. Thus it could be inferred from the above liquid limit values that the samples from drying process have greater clay size fraction and this accounts for the increase in unit cohesion.

Figures 5.3.5 and 5.3.6 shows the stress-strain curves from $\overline{CU}$ tests for specimens prepared at modified Proctor density through the wetting process and drying process respectively. As in the earlier case, the drying process gives

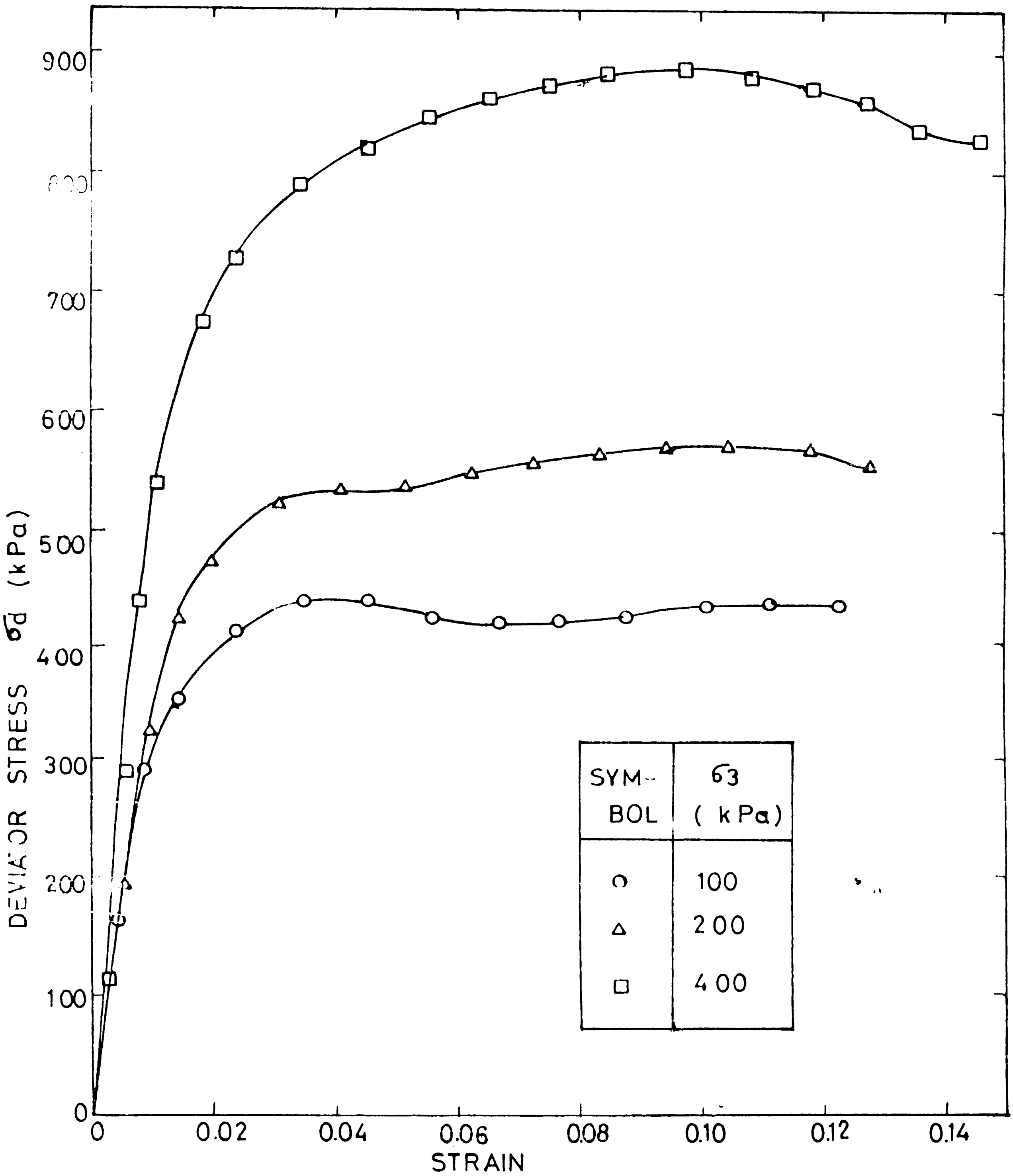


Fig.5.3.5 STRESS-STRAIN CURVES FOR COMPACTED MARINE CLAY (BY WETTING PROCESS) - SAMPLES PREPARED AT MODIFIED PROCTOR DENSITY

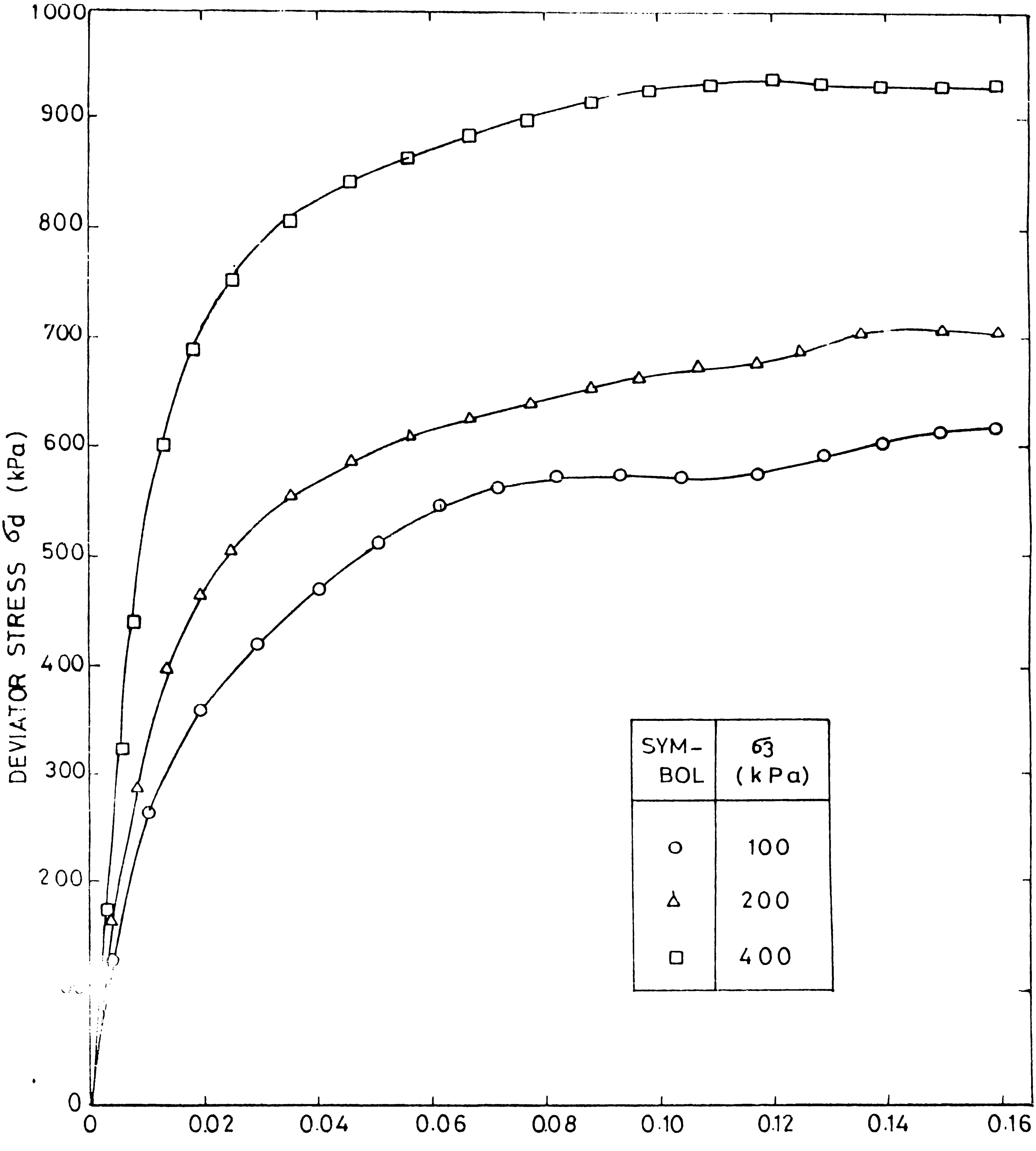


Fig.5.3.6 STRESS-STRAIN CURVES FOR COMPACTED MARINE CLAY (BY DRYING PROCESS) - SAMPLES PREPARED AT MODIFIED PROCTOR DENSITY

higher strength. The values of C' and Ø' as given in Table 5.3.2 are 110.4 k Pa and 24.5° for the wetting process. The corresponding values in case of samples prepared by drying process are 160.4 k Pa and 23° respectively. Here again it is obvious that the increase in unit cohesion from 110.4 k Pa to 160.4 k Pa is responsible for the gain in strength, as the variation in the value of Ø' is marginal. These results are in good agreement with the findings of Seed, Mitchell and Chan (1960).

For purposes of comparison between specimens prepared under standard and modified Proctor density, the stress-strain curves are presented in Fig.5.3.7. It can be seen from the figure that there is significant increase in shear strength, as compaction increases. The increase in density between the two compactive efforts is 17.2%; the corresponding increase in shear strength (unit cohesion) is as high as 320%.

Figure 5.3.8 shows the relationship between initial tangent modulus and confining pressures. It can be seen that the variation between the two are linear in all the cases. The values of initial tangent modulus are higher in the case of drying process. This is consistent with the earlier inferences. Further, it can be seen from the figure that the

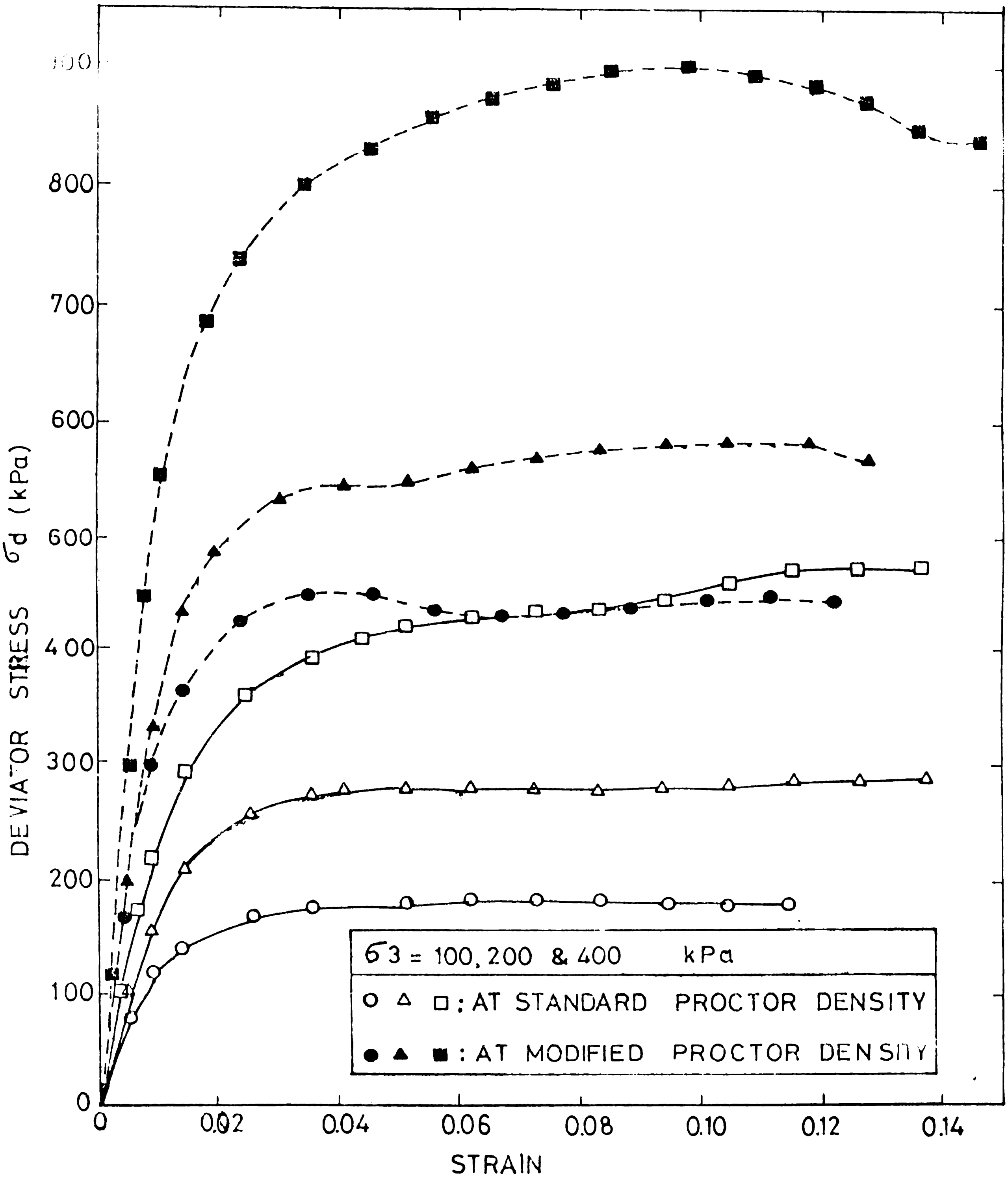


Fig.5.3.7 EFFECT OF INCREASE IN COMPACTIVE EFFORT

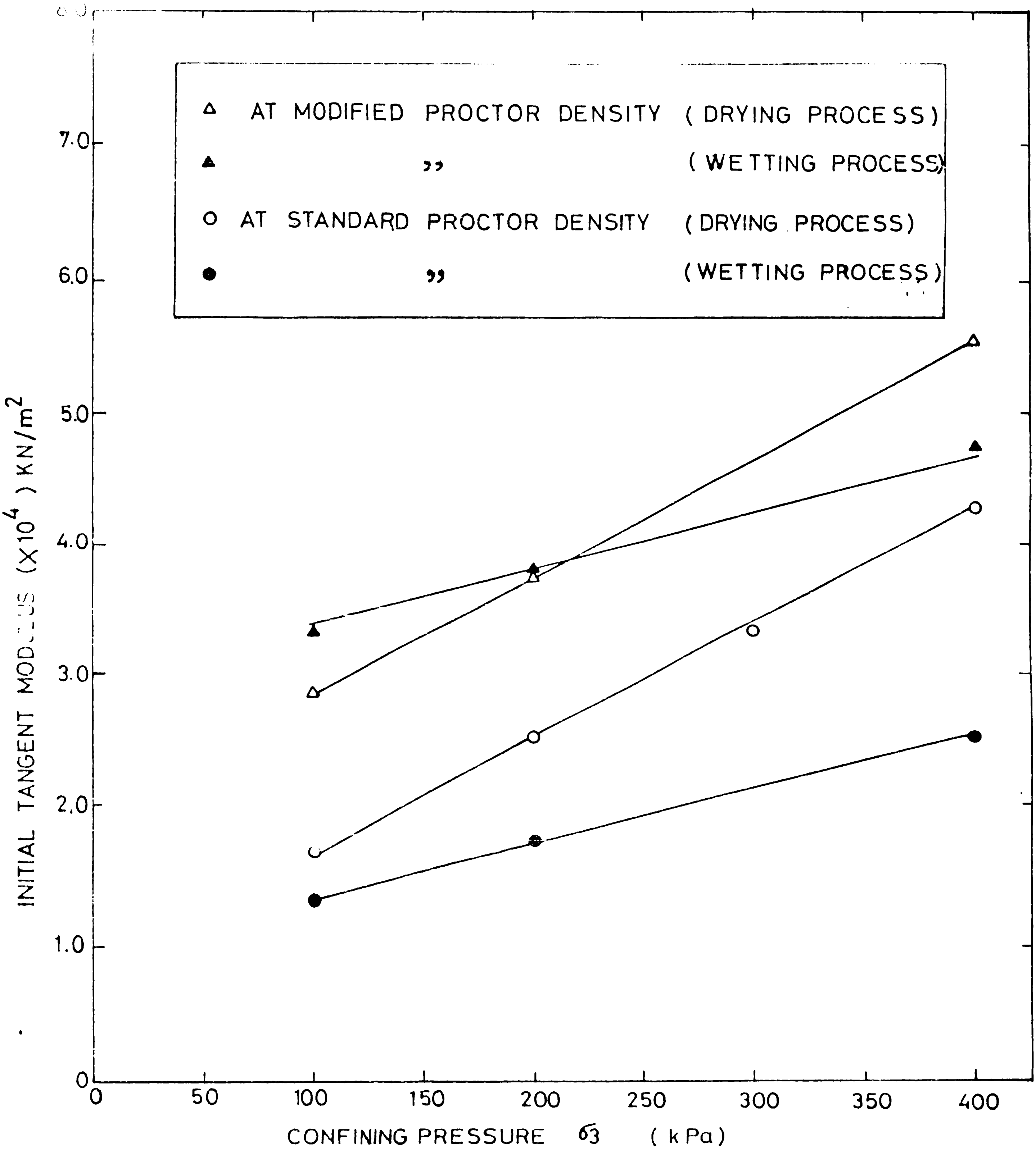


Fig.5.3.8 VARIATION OF INITIAL TANGENT MODULUS WITH CELL PRESSURE

slopes of each pair of curves for the two compactive efforts are parallel. This indicates that Ø' remains the same - only C' gets affected due to the increase in compactive effort.

5.4 Shear strength characteristics of lime treated clays

The results of the consolidation tests clearly brought out the improvement in compressibility characteristics of Cochin marine clays on treatment with lime. The shear strength characteristics of lime treated specimens were investigated in detail through a series of vane shear tests, unconfined compressive strength tests and triaxial shear tests.

5.4.1 Laboratory vane shear tests

As discussed earlier in chapter III, the natural moisture content of the samples collected from field were close to the liquid limit values. Hence the shear strength values are very low and could be measured only by laboratory vane shear tests. On lime treatment, the shear strength improved considerably and unconfined compressive strength tests could be conducted on samples cured for about 15 days.

In order to select an optimum percentage of lime, specimens were prepared using 2, 4, 6 and 8% of lime by dry weight of soil and cured upto 60 days. The results are presented in Fig.5.4.1. The specimens treated with 2% lime did not show any notable improvement in strength. On treatment with 4% lime, there was significant increase in shear strength (about 10 times), but the full strength gain is achieved within a week and there is no further increase in strength for the next 50 days. This indicates that the lime added is insufficient for the full potential of chemical interaction. Addition of 6% lime showed a steady improvement right upto 60 days of curing. Eventhough 8% registered still higher strength, 6% was selected as an optimum value which was consistent with the finding of other research workers (Bell, 1988; Jose, 1989; Balasubramaniam et al, 1989).

Figure 5.4.1 also shows an interesting characteristic in the rate of strength gain with strain on lime treatment. While lower percentage of lime shows a higher rate of strength gain initially, in case of specimens treated with higher percentages of lime, the rate of strength gain was low for the first few days of the curing period. As curing period increased, the peak strength obtained were consistent with percentage of lime. Similar observations have already been reported while discussing the results of consolidation tests (section 4.4).

The variation of the shear strength (unit cohesion)

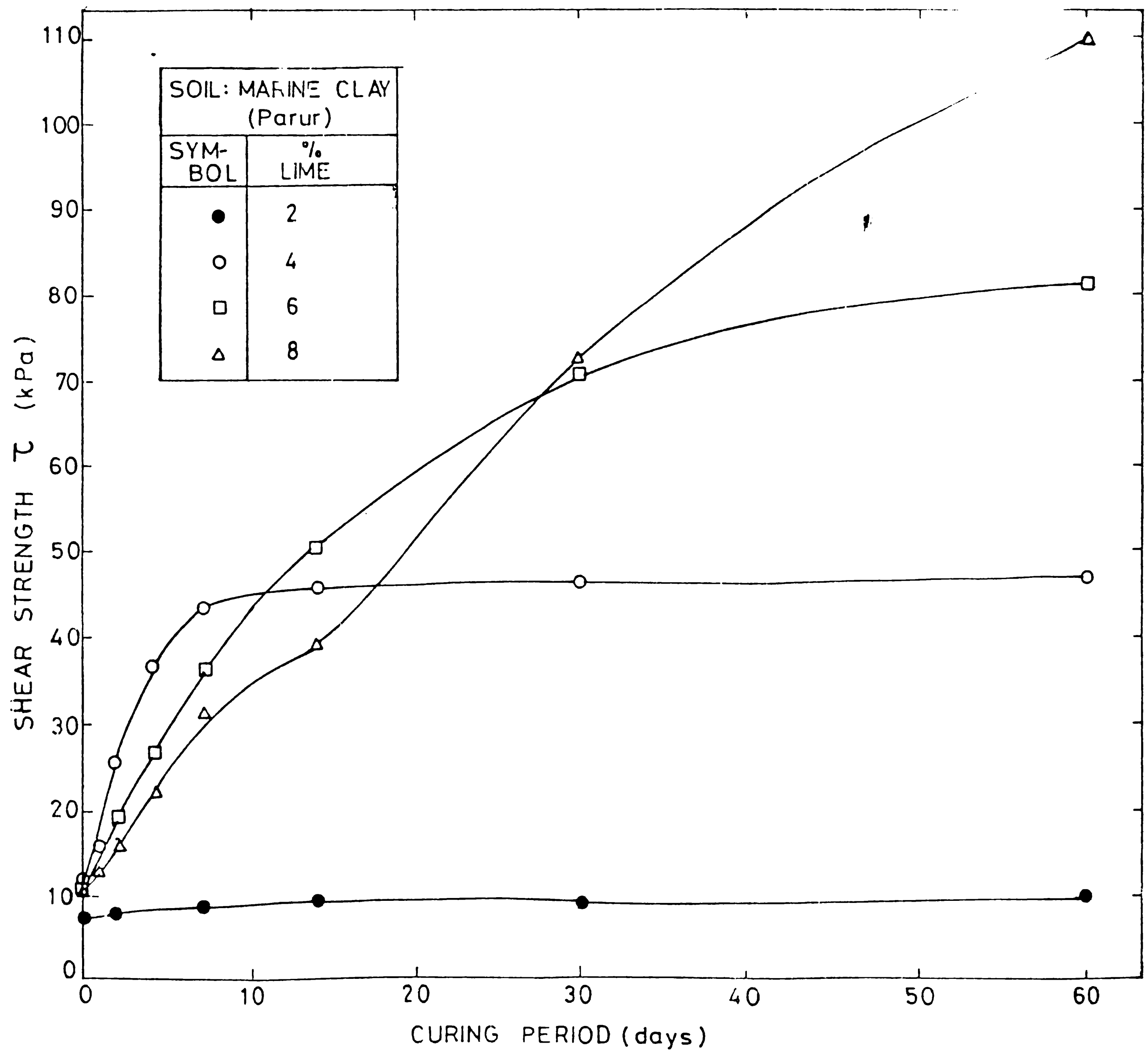


Fig.5.4.1 EFFECT OF LIME PERCENT AND CURING PERIOD ON SHEAR STRENGTH

of samples treated with 3, 6 and 9% of lime for curing periods upto 90 days are shown in Fig.5.4.2. The trends are in agreement with those in the previous figure. Since the gain in strength through lime stabilisation is mostly attributed to bonds developed, it was decided to study the strength characteristics of specimens which were remoulded after the strength tests as in the case of sensitivity studies. It can be seen from the figure that the remoulded strength very closely follow the strength of treated cured specimens. The ratio of strength of the cured specimens and remoulded samples after the initial strength tests varies from 3.5 to 4 for specimens treated with 3 and 6% lime.

The $\tau - \theta$ curves for specimens treated with 6% lime and cured for different periods are shown in Fig.5.4.3. The development of bonds with respect to curing period can be clearly seen from the curves. The specimens cured for 1 month show a peak strength of 50 k Pa at an angular rotation of 102°. At this point, the bonds developed through action of lime get broken and the stress suddenly drops to 18 k Pa. Thereafter the curve undergoes a strain hardening stage wherein the shear strength again picks up to a limited extent. The $\tau - \theta$ curve for the specimen cured for two months behaves exactly in a similar manner. But in the case of specimens cured for one week and that not cured at all, the behaviour is

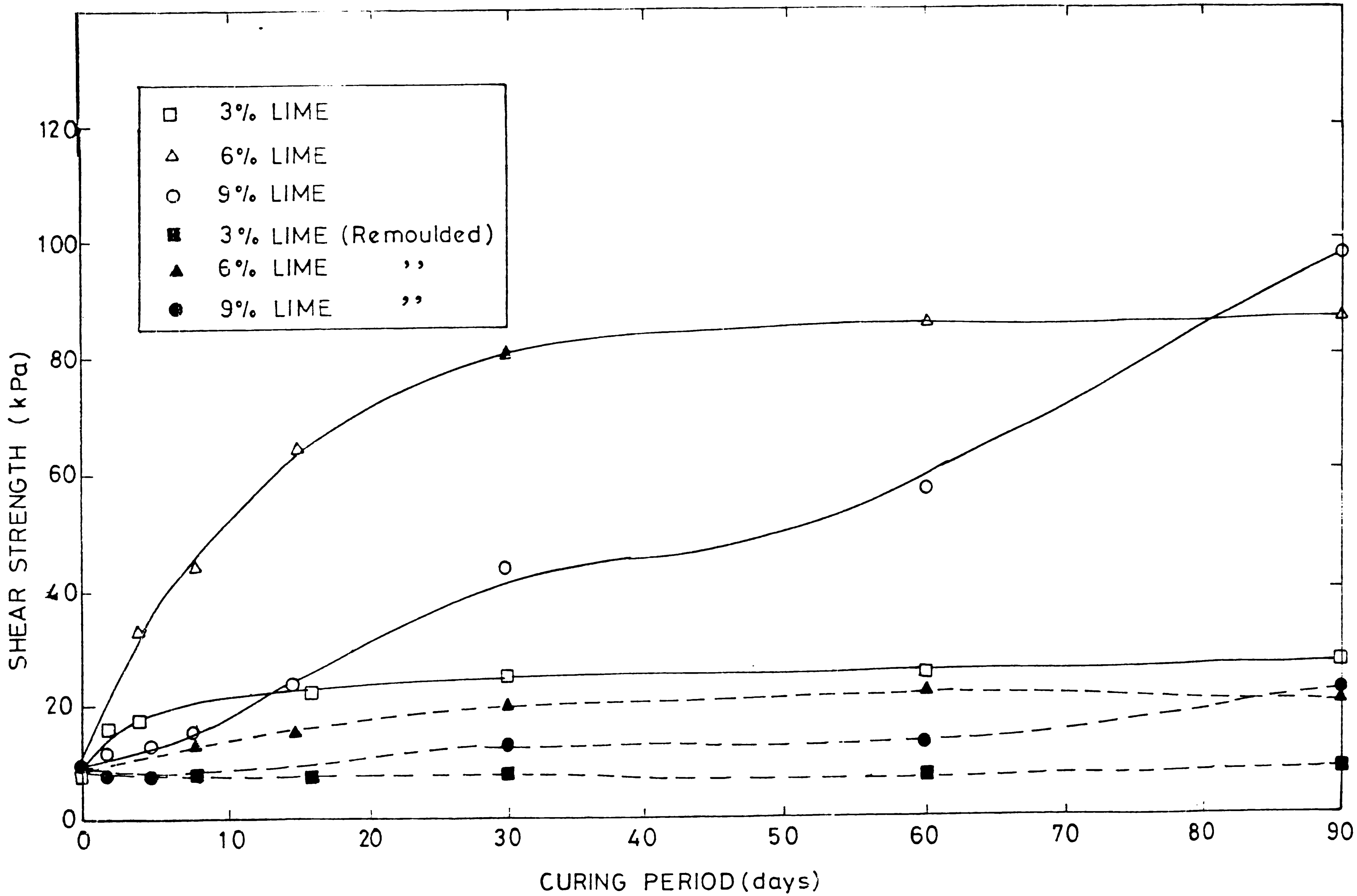


Fig.5.4.2 DEVELOPMENT OF BOND STRENGTH WITH LIME PERCENT AND CURING PERIOD

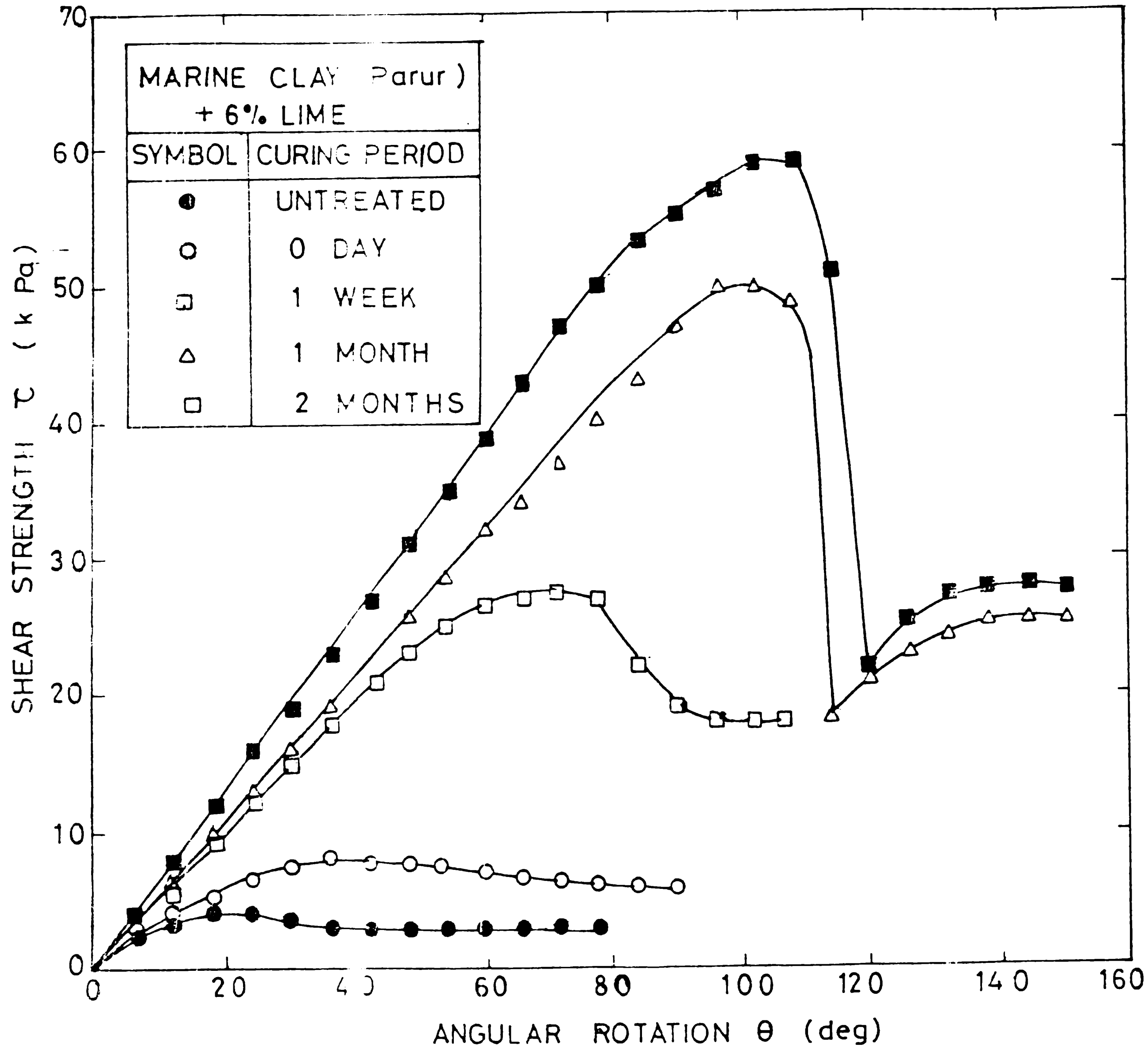


Fig.5.4.3 TYPICAL τ - θ CURVES

somewhere in between the untreated sample and the sample cured for one month.

In order to study whether the shear strength is influenced by the initial water content, in case of treated samples also, specimens were prepared with three different moulding water contents. The results of vane shear tests on these specimens, treated with 6% lime and cured upto 90 days are shown in Fig.5.4.4. It can be seen that the strength increases as moulding water content decreases. As the water content increases from 93 to 114%, the shear strength decreases from 85 to 53 k Pa.

Figure 5.4.5 shows similar results for specimens treated with 9% lime. Eventhough the gain in strength is incomplete, it can be seen that the shear strength varies inversely with water content. These results are in agreement with the observations made by Holm (1979).

The results of vane shear tests on marine clay samples collected from Cochin (Parur) and Kuttanad (Kidengara) treated with 6% lime and cured upto 60 days are presented in Fig.5.4.6. The results of vane shear tests on remoulded specimens are also presented. It can be seen from the pair of curves that the samples from Kidengara with a water content of

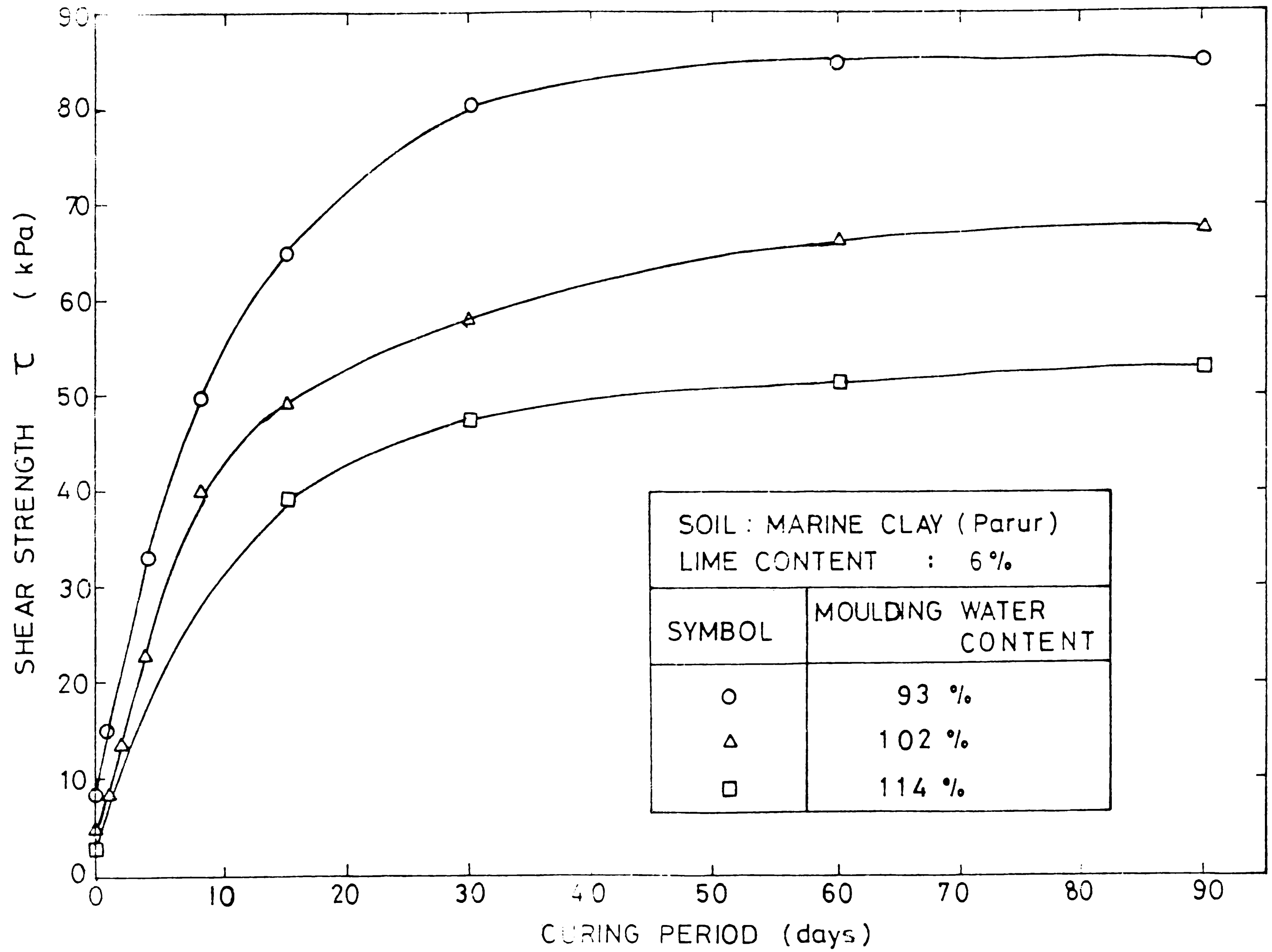


Fig.5.4.4 EFFECT OF INITIAL WATER CONTENT ON SHEAR STRENGTH OF LIME TREATED COCHIN MARINE CLAY

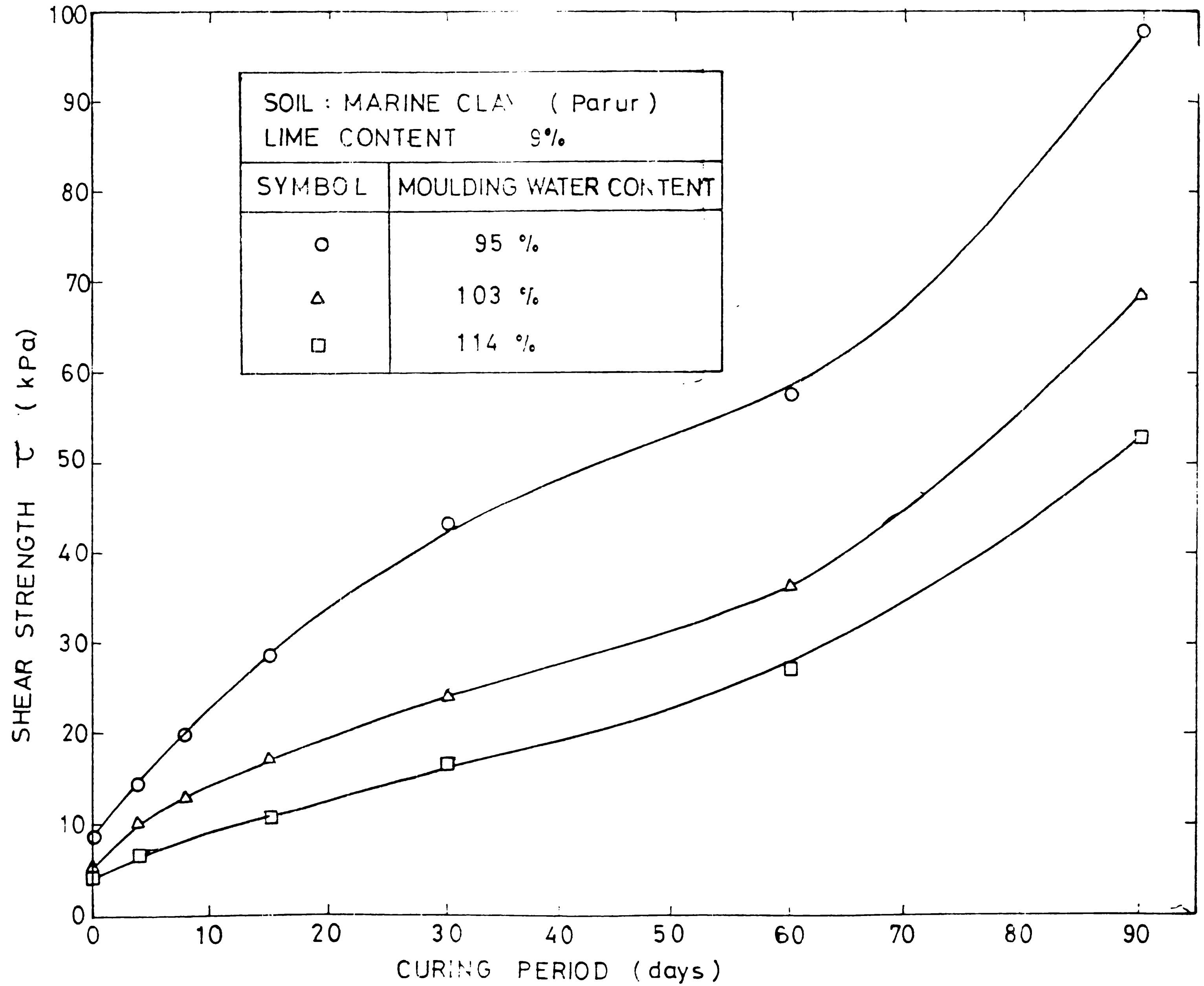
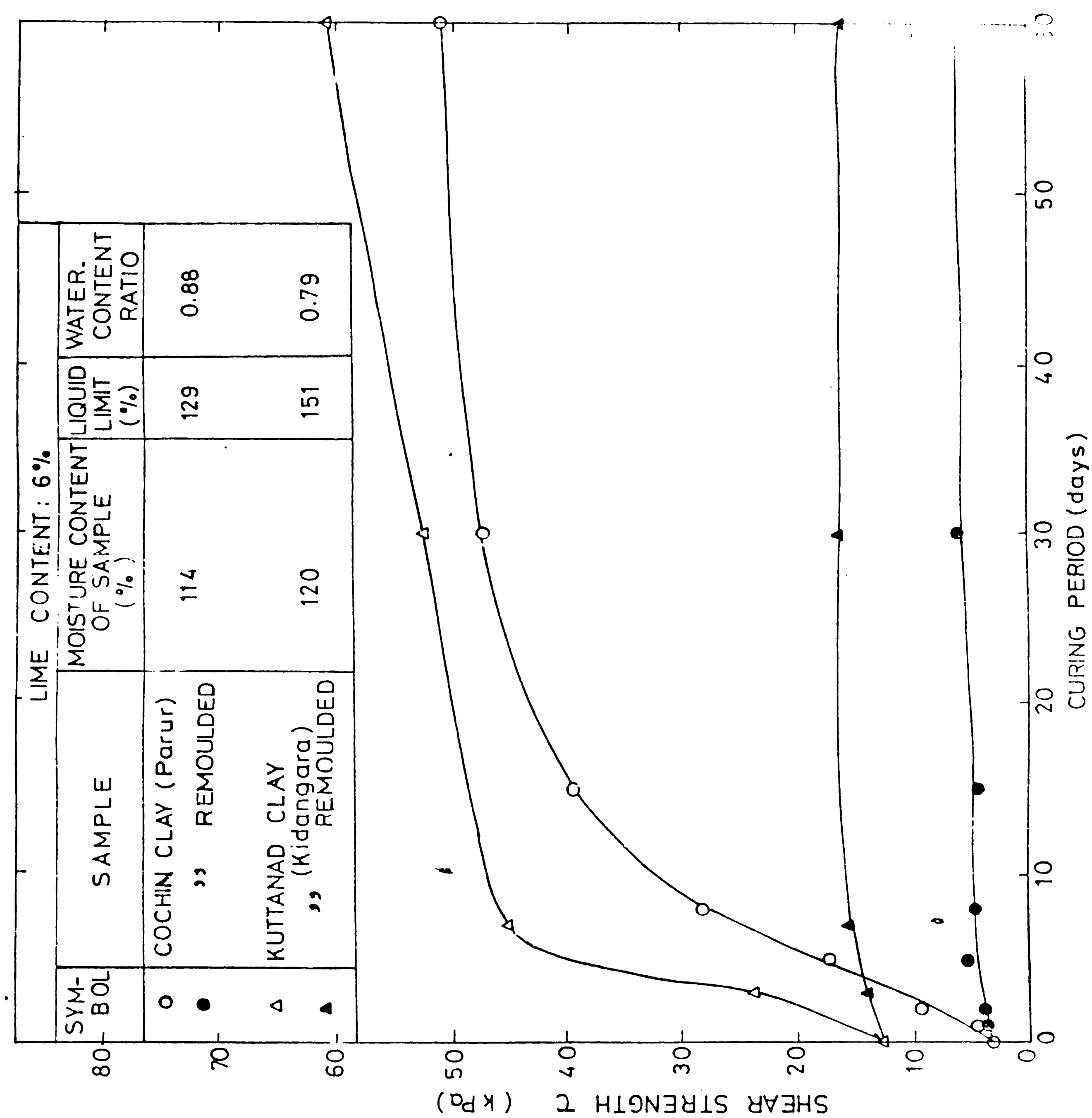


Fig.5.4.5 EFFECT OF INITIAL WATER CONTENT ON SHEAR STRENGTH OF LIME TREATED COCHIN MARINE CLAY


LIME CONTENT: 6%
SYM-BOL
SAMPLE
MOISTURE CONTENT OF SAMPLE (%)
LIQUID LIMIT (%)
WATER-CONTENT RATIO
COCHIN CLAY (Parur)
114
129
0.88
,, REMOULDED
KUTTANAD CLAY (Kidangara)
120
151
0.79
,, REMOULDED
SHEAR STRENGTH τ (kPa)
CURING PERIOD (days)
0
10
20
30
40
50
60
70
80

120% gives higher values than the samples from Parur whose water content is only 114%. But it can be seen from the legend that the water content ratio (ratio of water content to liquid limit) for Kidengara samples is 0.79, while that for Parur is 0.88. Thus, as in the case of untreated samples (Jose, 1989) the shear strength is inversely proportional to water content ratio, in case of lime treated specimens also.

5.4.2 Correlation between vane shear strength and UCC strength

Cochin marine clays with their natural water contents very close to liquid limits possess very poor shear strength. These low values can be measured only by laboratory vane share tests. But when studies on effect of lime treatment on Cochin marine clays were taken up, the strength characteristics of specimens which comprised of untreated samples, treated samples cured for short durations and treated samples cured for long periods had to be investigated. This wide range cannot be covered by either laboratory vane shear tests or by unconfined compressive strength tests. While, the unit cohesion for remoulded marine clay sample was as low as 2.5 k Pa, the corresponding value for a specimen treated with 12% lime and cured for 8 months was as high as 175 k Pa. While the low strengths could be measured by laboratory vane shear tests and high strengths could be measured with unconfined strength tests with reasonable accuracy, it was noticed that in the overlapping range there was considerable difference between the shear strength values obtained from the two different tests for identical specimens. A discontinuity was observed in every series of strength tests on treated marine clay samples.

The present problem is similar to that of results of crushing strength tests on concrete cubes and cylinders, which was overcome by establishing a correlation between cube strength and cylindrical strength. Attempts in these lines to ...in a correlation between the shear strength τ obtained from vane shear tests and unit cohesion C from UCC tests are presented in Fig.5.4.7.

Specimens for laboratory vane shear tests and UCC tests were prepared using remoulded natural Cochin marine clay and marine clay treated with lime. Eventhough vane shear tests can be performed on even extremely soft soils, specimens for UCC tests can retain the shape with a minimum shear strength of around 3 k Pa. Similarly, specimens with shear strength higher than around 60 k Pa cannot be tested in the laboratory vane shear test equipment. Thus the range over which a correlation between τ and C has been defined by these limits. While shear strength of natural marine clay was varied

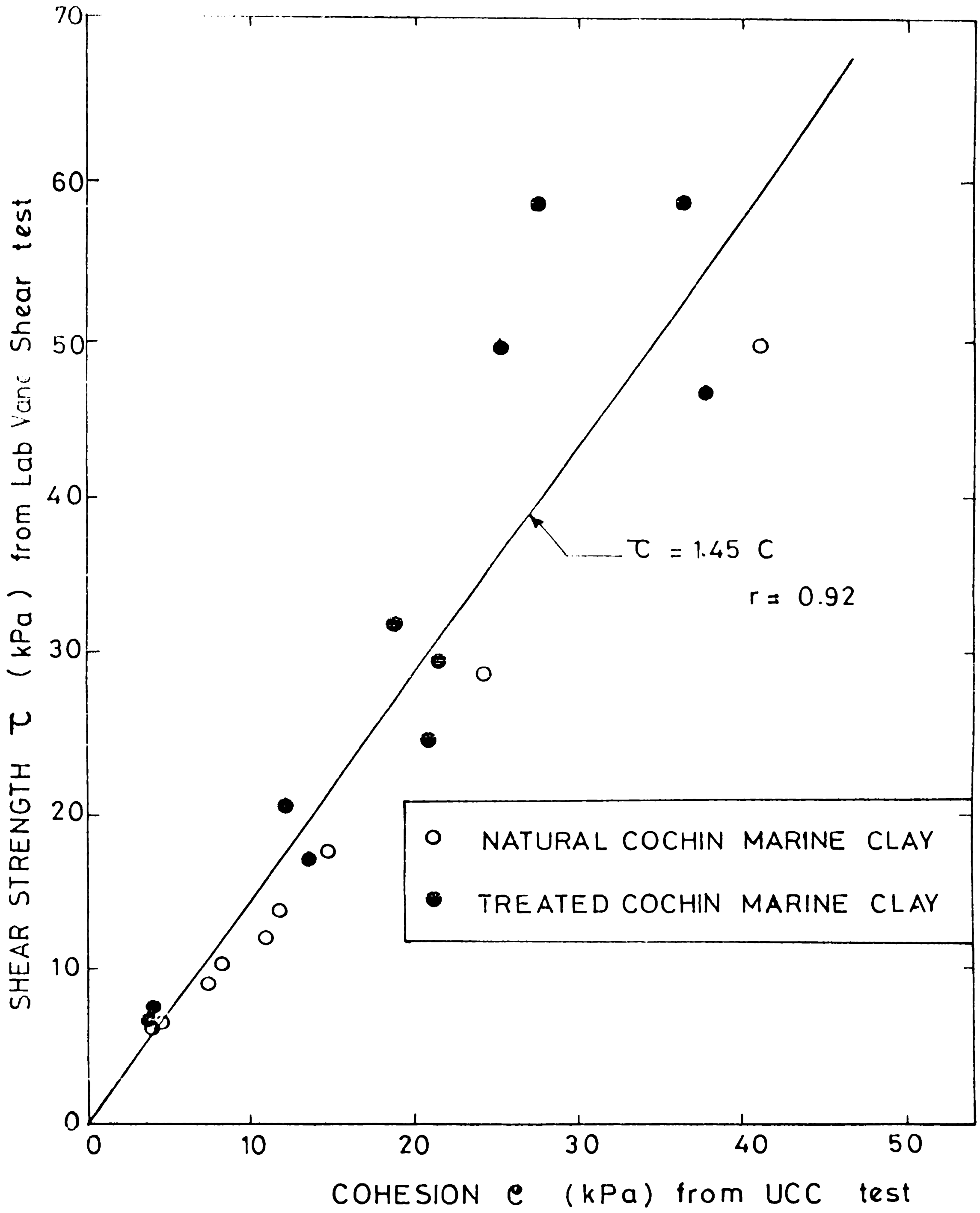


Fig.5.4.7 RELATION BETWEEN C AND

by changing the moulding water content, that of treated clay samples were varied by changing the lime content and curing period.

The statistical fit of the experimental results yielded the relation

$$\tau = 1.45 \, c$$

with a correlation coefficient, r = 0.92.

The relation for natural Cochin marine clay as obtained from the straight line fit is

$$\tau = 1.19 \, c$$

with a high correlation coefficient r = 0.998.

This relation can be made use of to estimate the strength of Cochin marine clays irrespective of the method adopted for the strength test.

5.4.3 Unconfined compressive strength tests

Through a series of unconfined compressive strength tests on specimens prepared with varying lime contents and curing periods, the effect of lime stabilisation on Cochin marine clays has been studied in detail. How the strength and elastic characteristics vary with lime content and curing period is presented in Figs.5.4.8 to 5.4.10.

The three figures show the relation between q_u and strain for samples treated with 6%, 9% and 12% lime and cured for 2 weeks to 8 months. The three sets of curves show that there is considerable change in the elastic properties of the soil associated with the gain in strength. As curing period increases, there is considerable gain in strength, but this is accompanied by a partial loss in elasticity. The specimens tend to become more and more brittle as curing period advances. The residual strength of the specimens treated with a particular lime content and cured for different periods fall within a narrow range. While the peak strengths show a remarkable variation, the variation of the residual strength is very low. This is as ought to be since most of the bonds gets broken at peak stress level.

The above aspect has been clearly brought out in Fig.5.4.11 which shows the stress-strain curves for samples treated with 6, 9 and 12% lime and cured for 2 months. While the peak strength vary from 54 to 79 k Pa, the residual strength for the three specimens fall within a narrow band of 18 to 22 k Pa. The phenomenon that higher lime contents take longer duration for development of strength is brought out in

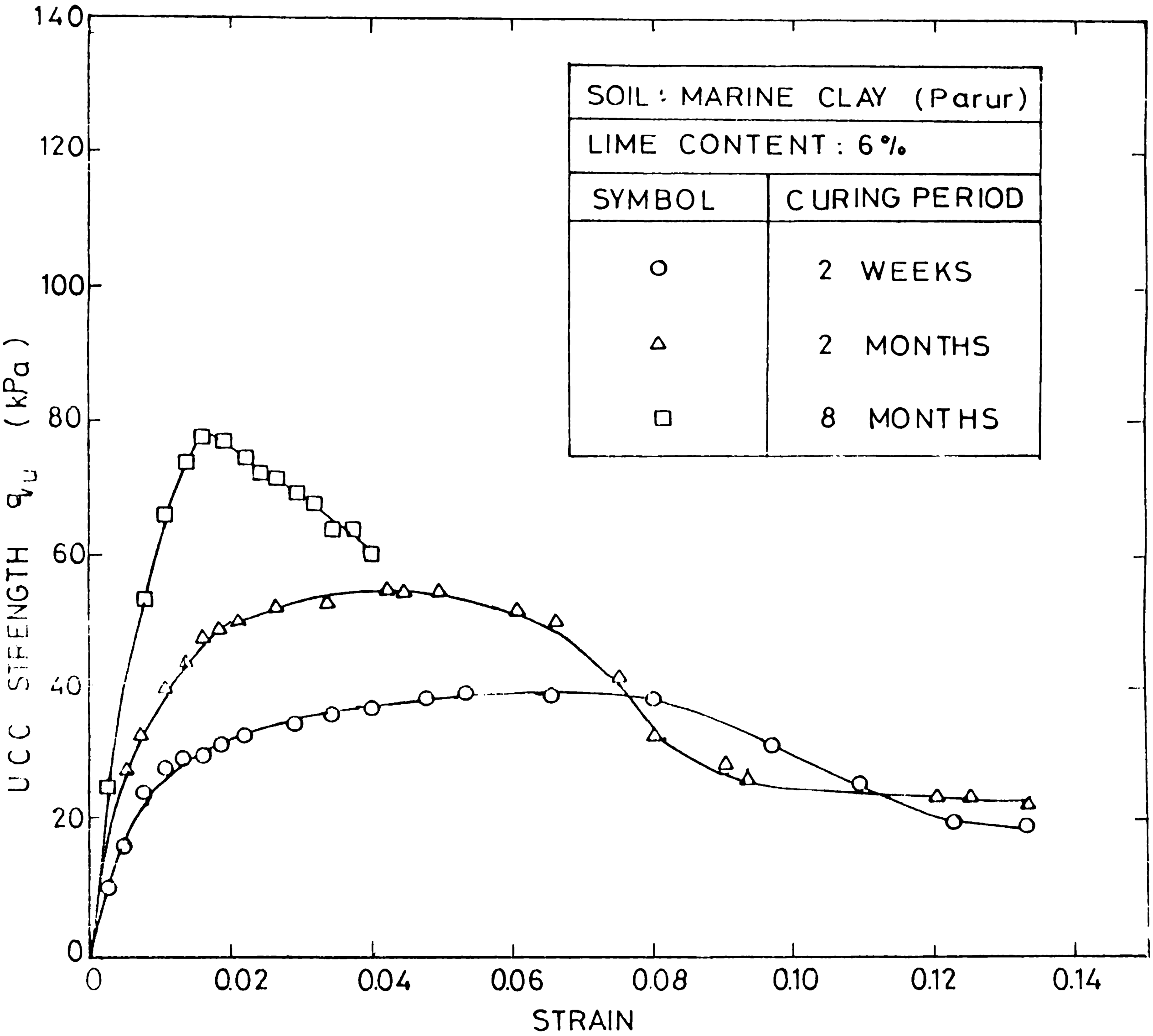


Fig.5.4.8 STRESS-STRAIN CURVES FOR LIME CONTENT OF 6%

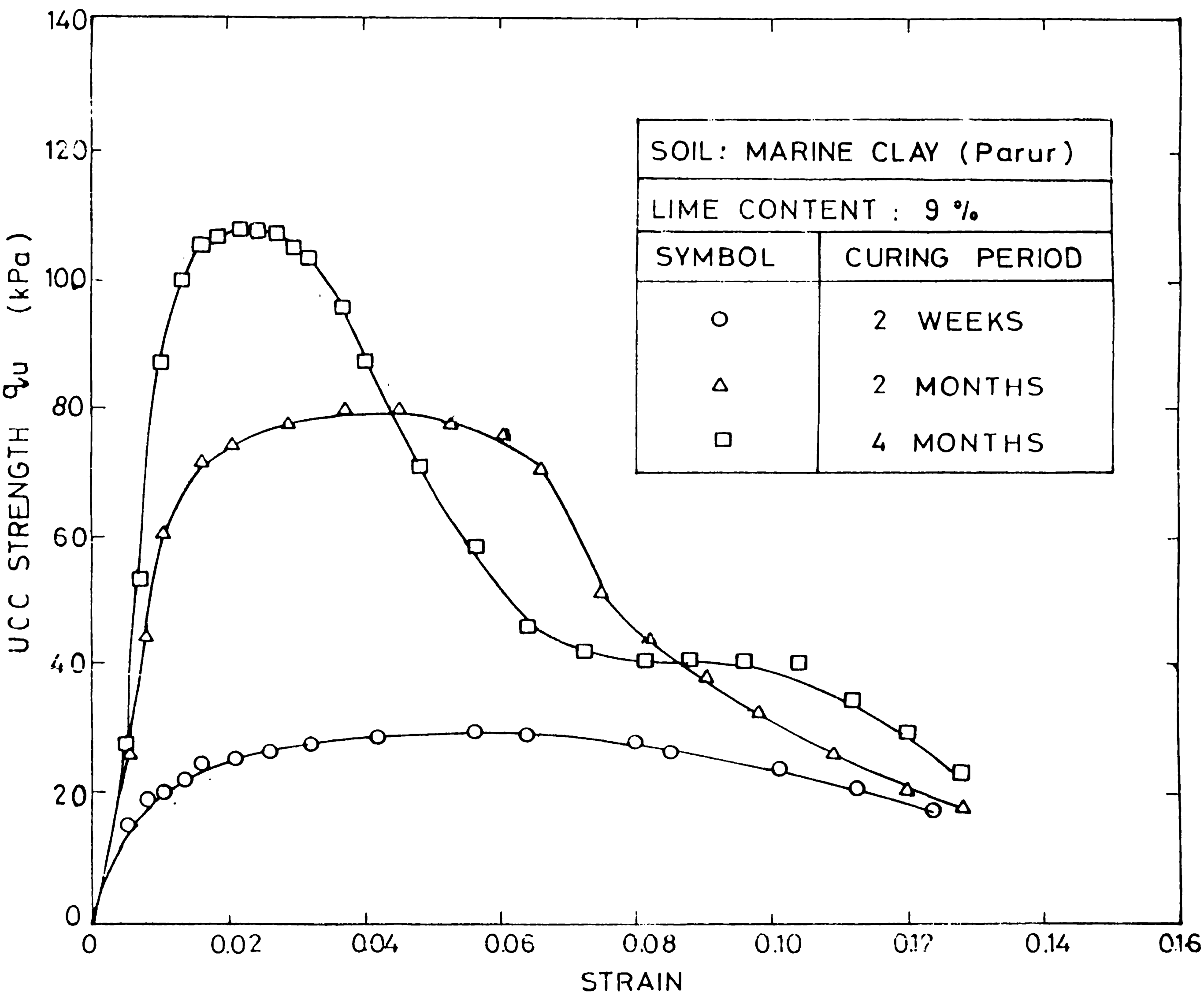


Fig.5.4.9 STRESS-STRAIN CURVES FOR LIME CONTENT OF 9%

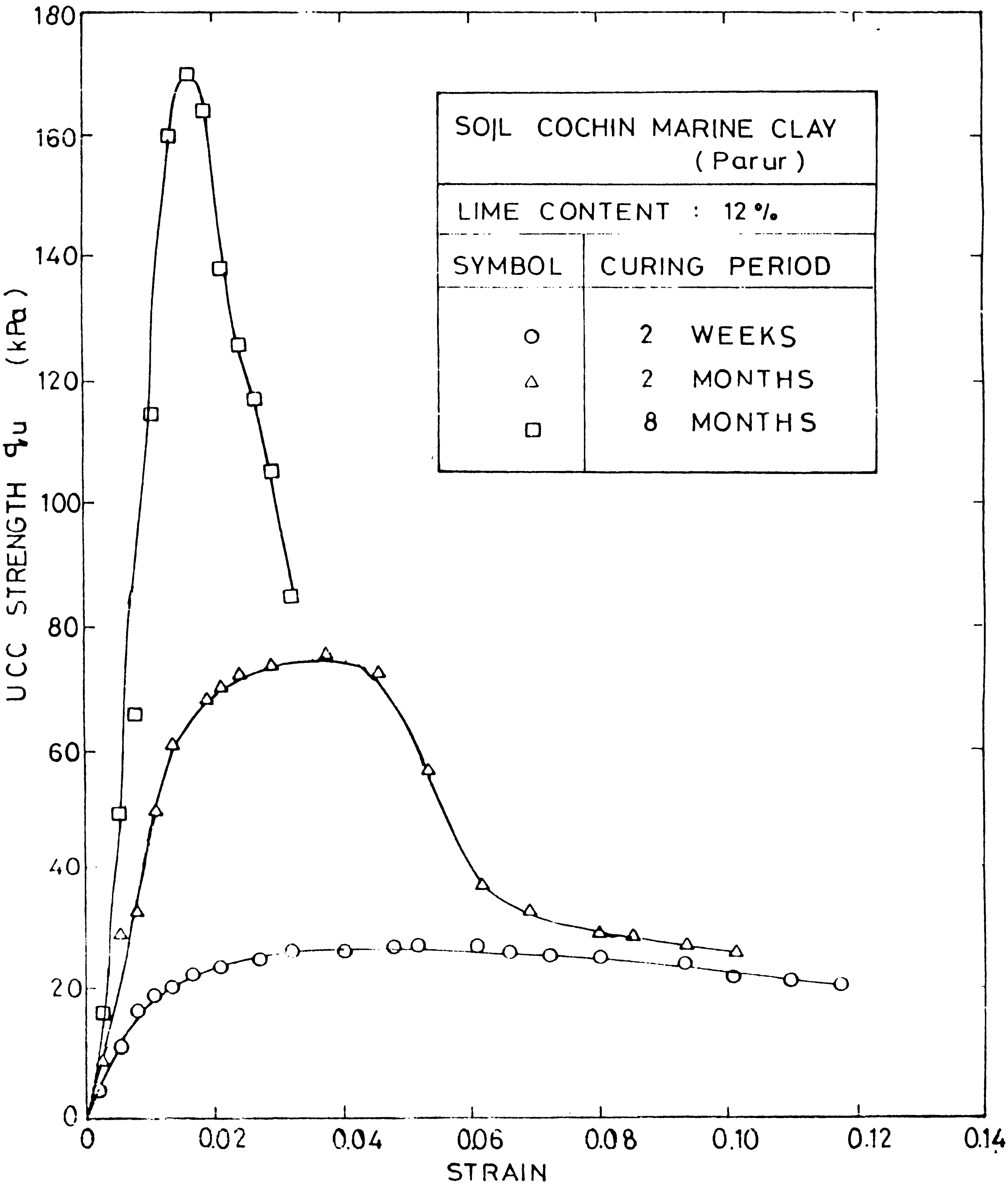


Fig.5.4.10 STRESS-STRAIN CURVES FOR LIME CONTENT OF 12%

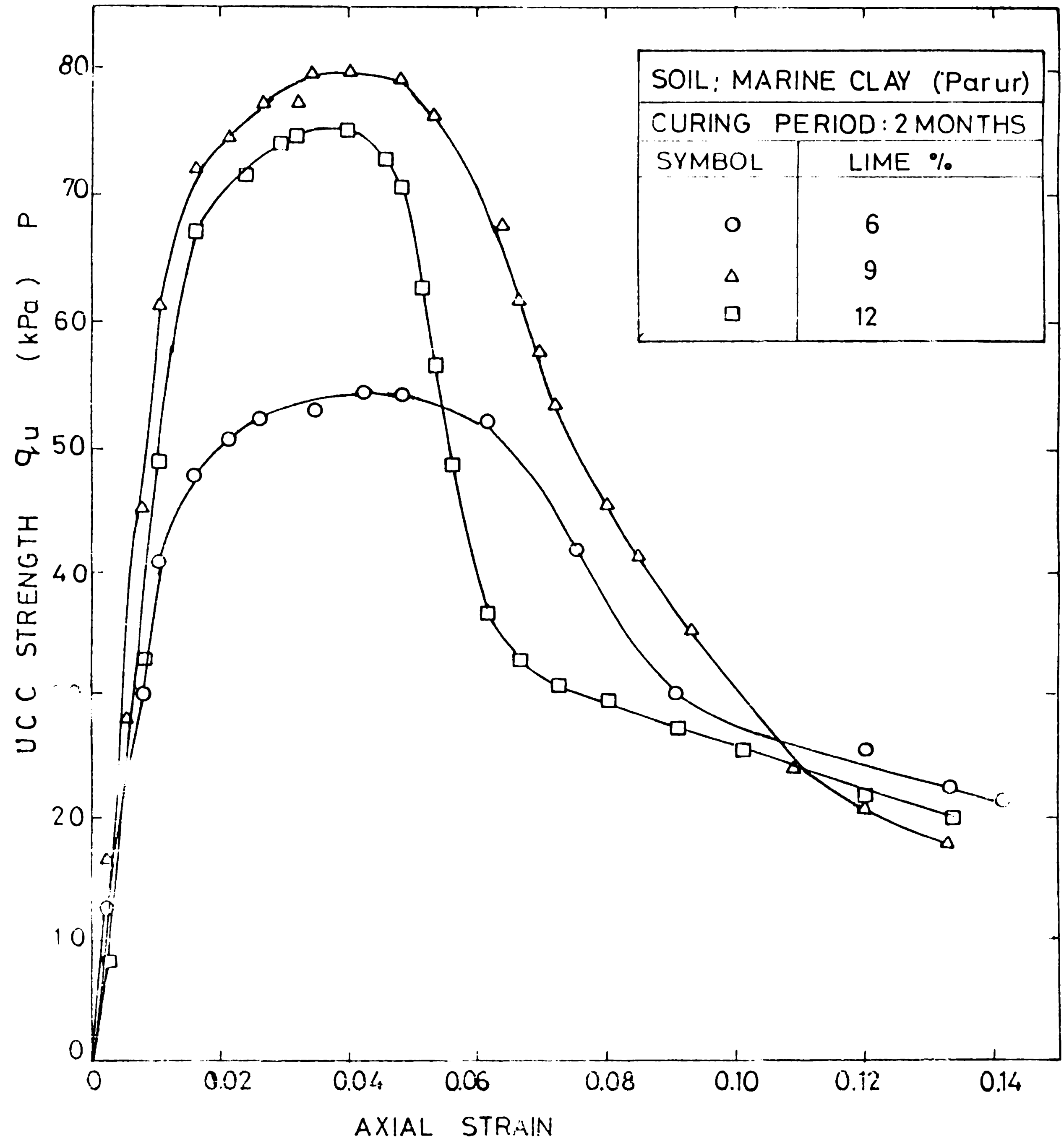


Fig.5.4.11 EFFECT OF LIME PERCENT ON STRESS-STRAIN BEHAVIOUR

this case also by the stress-strain curve for 12% lime.

The bonds developed through lime stabilisation grow stronger as well as more brittle with lime content and curing period, is clearly shown in Fig.5.4.10. While the peak strength region is spread over a strain of .02 - 0.118 for a curing period of 2 weeks, this gets reduced to .018 - .046 for 2 months curing. Compared to these two, there is a phenomenal reduction in the strain range for the sample cured for 8 months wherein the region reduces to almost a point.

A close examination of figures 5.4.8 to 5.4.10 reveals that the peak values for UCC strength occur at some regular strain values for specific curing periods. This prompted a study of strains at peak values and curing periods. Table 5.4.1 shows the strain at peak values of q_u for specimens treated with 6, 9 and 12% lime and cured between periods varying from 15 days to 8 months. It can be seen from the table that for a particular curing period, the strain at $q_{u_{max}}$ has an almost unique value. In case of samples cured for 8 months, the values are exactly the same.

Fig.5.4.12 shows a graphical representation of the data. As curing period increases, the strain at $q_{u_{max}}$ decreases. The rate of fall is higher at the initial

Table 5.4.1

Values of strain at peak stress (from UCC tests)

Sl. No.	Curing period	Values of strain for lime content of		
		6%	9%	12%
1.	2 weeks	0.053	0.056	0.058
2.	1 month	0.043	0.045	0.045
3.	2 months	0.043	0.035	0.037
4.	4 months	0.029	0.021	0.025
5.	8 months	0.016	0.016	0.016

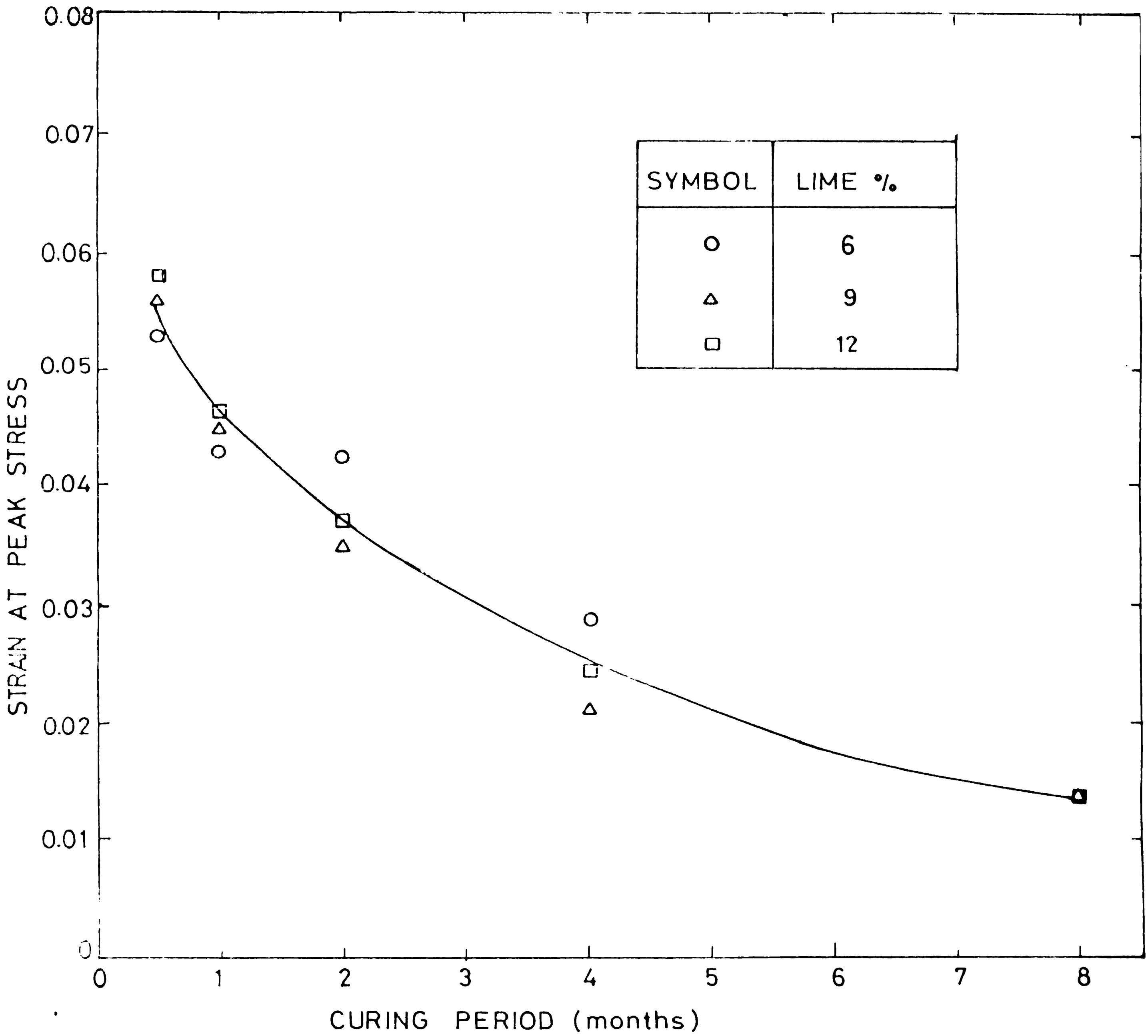


Fig.5.4.12 EFFECT OF CURING PERIOD ON STRAIN AT PEAK STRESS

stages of the curing period and tends to a constant value of around .016.

As has already been reported during discussions on the compressibility characteristics upon lime stabilisation and as reported in literature (Bell, 1988) the rate of gain in strength is inversely proportional to the lime content. Higher the lime content, longer will be the curing period required by the samples to develop the maximum shear strength. This has been brought out in Fig.5.4.13. While, only one week was required to get $q_{u_{max}}$ for a specimen treated with 3% lime, it took more than 8 months to reach maximum value in case of the sample stabilised with 12% lime.

The above aspect has been brought out clearly in Fig.5.4.14. It shows the relation between lime content and curing period for $q_{u_{max}}$. The latter increases steadily with lime content.

It has been proved by earlier workers (Nagaraj, 1992; Jose, 1989) that shear strength of fine grained soils is inversely proportional to the water content ratio - ie., ratio of natural water content and liquid limit. For a specific water content ratio, one can obtain a reliable relationship between lime content and $q_{u_{max}}$. Fig.5.4.15 shows the

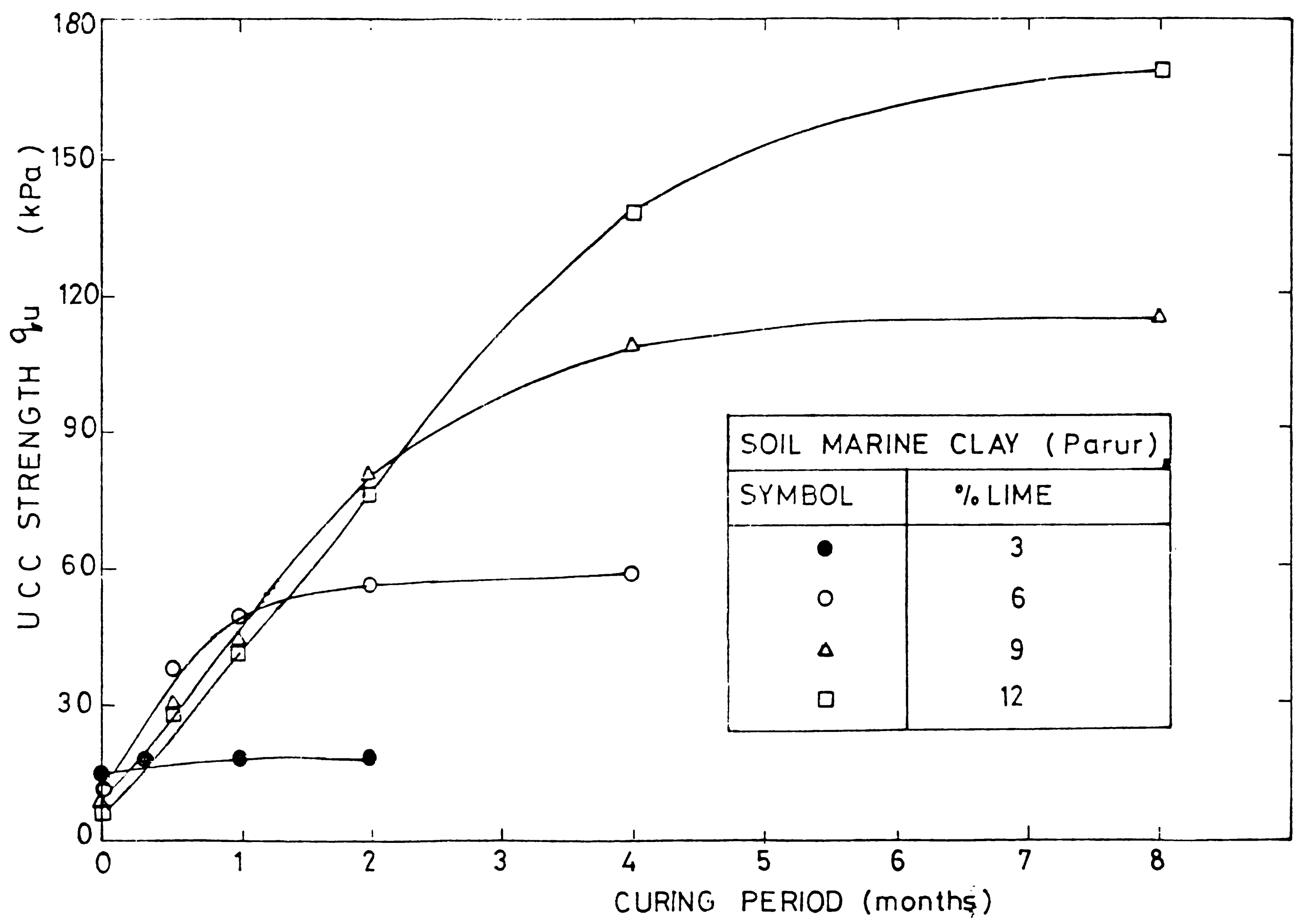


Fig.5.4.13 EFFECT OF LIME PERCENT AND CURING PERIOD ON UCC STRENGTH

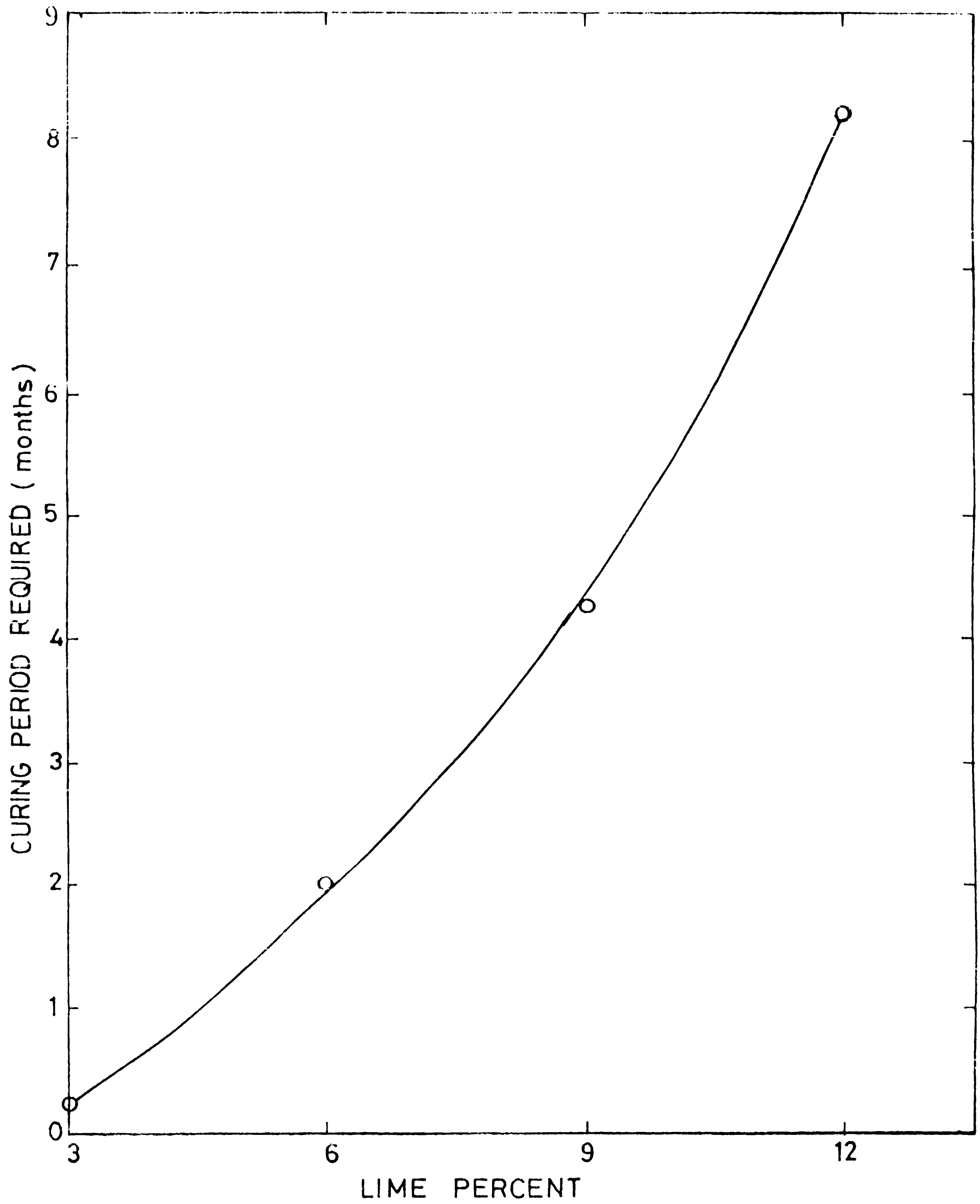


Fig.5.4.14 EFFECT OF LIME PERCENT ON CURING PERIOD REQUIRED

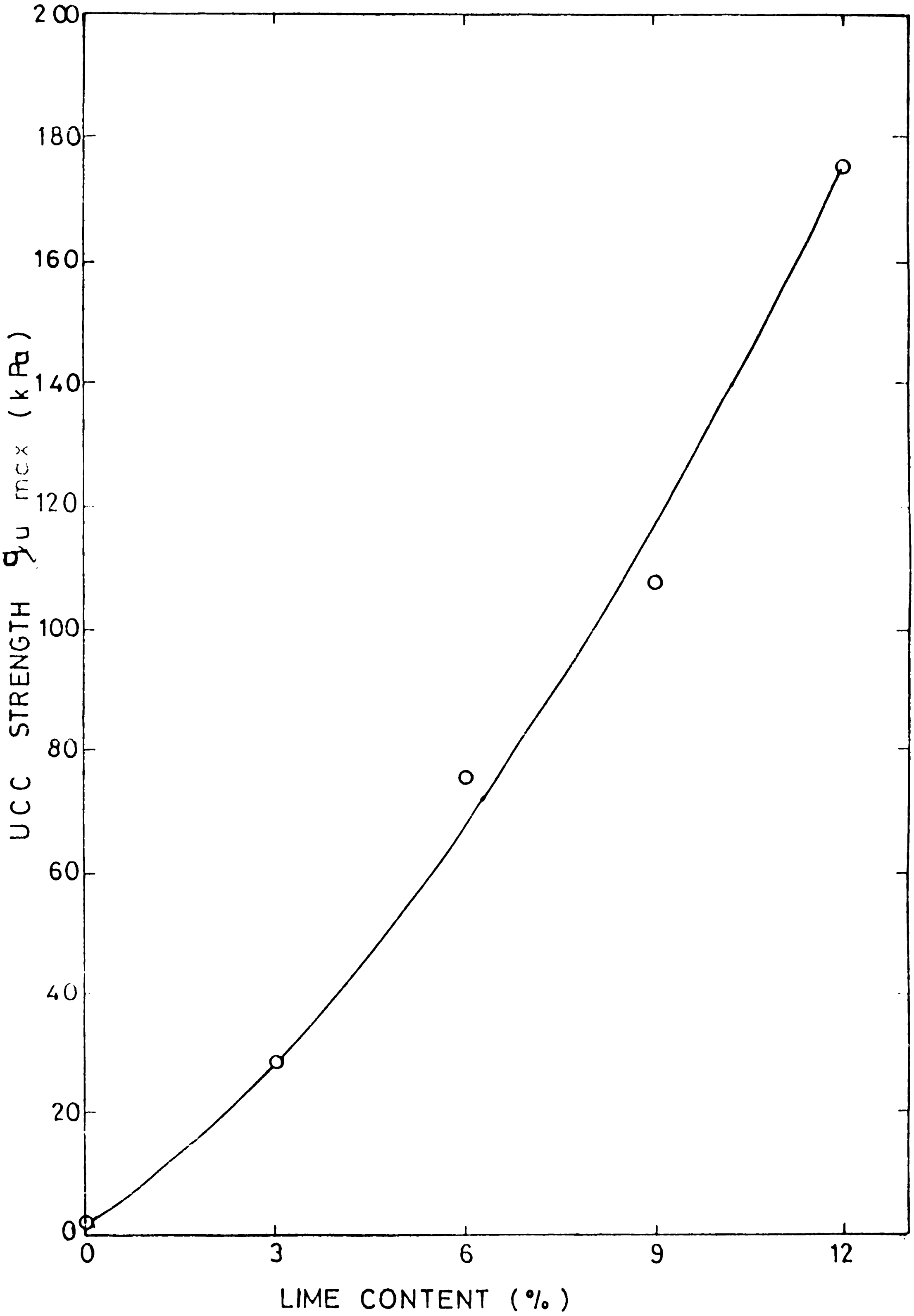


Fig.5.4.15 EFFECT OF LIME CONTENT ON $q_{u_{max}}$

relationship between $q_{u_{max}}$ and % of lime. The values of $q_{u_{max}}$ shown are the absolute maximum value of q_u allowing the required curing periods for development of maximum shear strength. The normal range of lime percentage recommended by research workers and adopted by field engineers fall within 12% (Bell, 1988; Jose, 1989).

Figures 5.4.14 and 5.4.15 form a very useful pair of charts for the design of Cochin marine clays stabilised with lime. For a particular clay, with specified physical and engineering properties, a relationship could be established between the maximum q_u that will be developed for various % of lime. Similarly a relation between lime percentage and curing period required for $q_{u_{max}}$ can also be established. Using these two relationships established for any particular soil, one can find out the lime content that is required to achieve a particular shear strength and the curing period that has to be allowed to ensure that full development of shear strength takes place.

5.4.3 Triaxial shear tests on lime treated clays

It has been already established that the highly compressible soft Cochin marine clays can be stabilised effectively with addition of lime. In order to study the effect of lime treatment on the individual shear strength parameters C and ϕ, the nature of pore pressure development and the pattern of volume change behaviour during shear, a series of triaxial shear tests were conducted.

Figure 5.4.16 shows the deviator stress-strain relationship for marine clay specimens treated with 6% lime and cured for one month. A comparison with Fig.5.2.1 for the untreated undisturbed Cochin marine clay will help to compare the stress-strain behaviour and pore pressure development for treated clays. Due to the additional bond strength developed the deviator stress at failure is higher than that of untreated marine clay. While for treated marine clay, the peak value remains steady at failure, it tends to decrease in case of untreated specimens at higher chamber pressures. The pore pressure development shows a steady increase in case of undisturbed sample. But in case of treated clay, the value picks up faster and gets stabilised earlier. The dilatency behaviour in both cases is identical. It occurs after a σ_3 of around 200 k Pa.

A comparison between the shear strength parameters obtained for $\overline{CU}$ tests on remoulded samples and those for samples treated with 6% lime and cured for one month, can give an idea about the improvement in strength characteristics and the shear strength parameter which contributes more in the

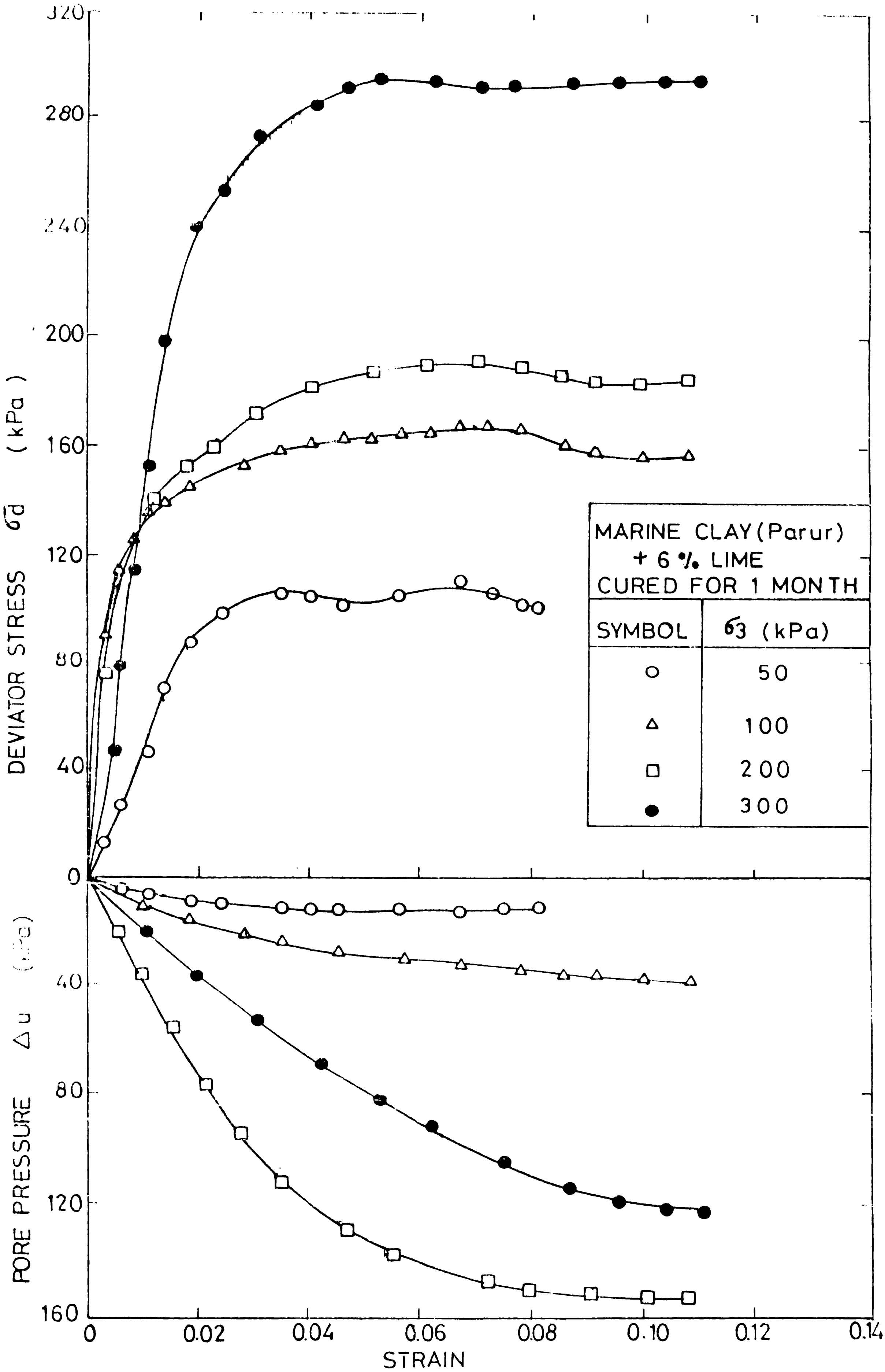


Fig.5.4.16 VARIATION OF DEVIATOR STRESS AND PORE PRESSURE FOR LIME TREATED COCHIN MARINE CLAY (CU TESTS)

strength development. While the value of c is 2 k Pa for remoulded soil, the shear strength parameters obtained from triaxial shear tests are 31.1 k Pa and 14° respectively. The effective strength parameters C' and Ø' are 31.8 k Pa and 14.4° respectively.

Figure 5.4.17 shows the stress-strain relationship along with pore pressure for samples treated with 6% lime and cured for 3 months. The characteristics observed in the earlier figure holds good in this case also. From the value of strength parameters given in Table 5.4.2 it can be seen that the value of C increases with curing period whereas Ø remains almost constant. This is in agreement with the findings of Wissa et al (1965).

The variations in stress-strain relationship of marine clays treated with 6% lime with varying curing periods are shown in Fig.5.4.18. As curing period increases, the maximum deviator stress increase along with initial tangent modulus. The curves are quite consistent with one another with peak values remaining more or less constant.

The variations of the value of A_f with ambient pressure for marine clay samples treated with 6% lime and cured for two different periods viz., one month and three

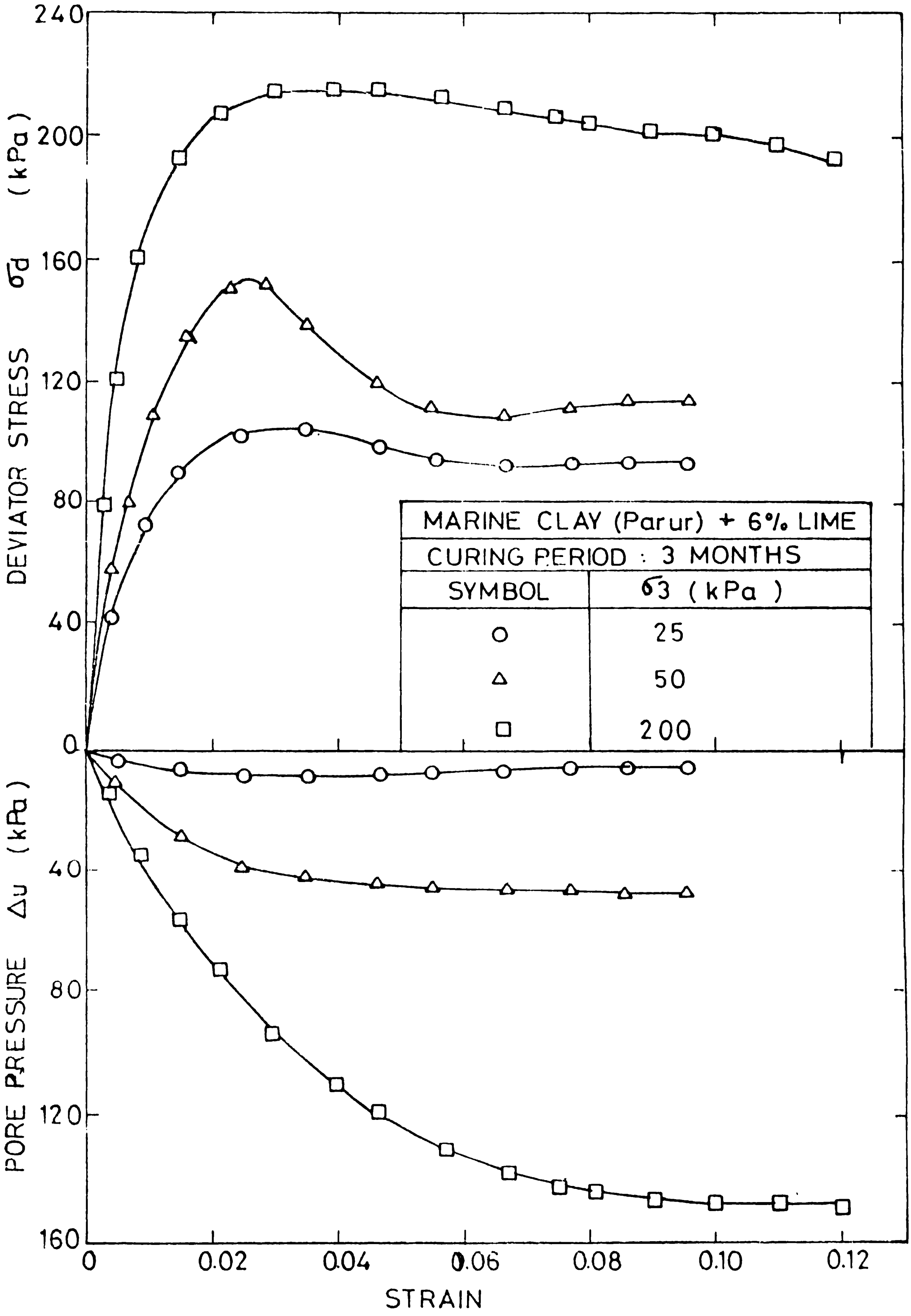


Fig.5.4.17 VARIATION OF DEVIATOR STRESS AND PORE PRESSURE FOR LIME TREATED COCHIN MARINE CLAY (CU TESTS)

Table 5.4.2

Results of triaxial shear tests on lime treated soils

(Sample: Cochin Marine clay (Parur) + 6 % lime)

Sl. No.	Curing period	Type of test	Chamber pressure σ_3 (k Pa)	Deviator stress σ_d (k Pa)	Total stress parameters C (k Pa)	Total stress parameters ϕ (deg.)	Effective stress parameters C (k Pa)	Effective stress parameters ϕ' (deg.)
1.	1 month	Consolidated Undrained	50 100 200 300	110 166 192 296	31.1	14.0	31.8	14.4
2.	3 months	Consolidated Undrained	25 50 75 150 200 250	105 154 166 172 216 244	36.9	13.5	38.8	15.8
3.	1 month	Consolidated Drained	50 100 200 300	144 188 242 330	--	---	41.3	14.5

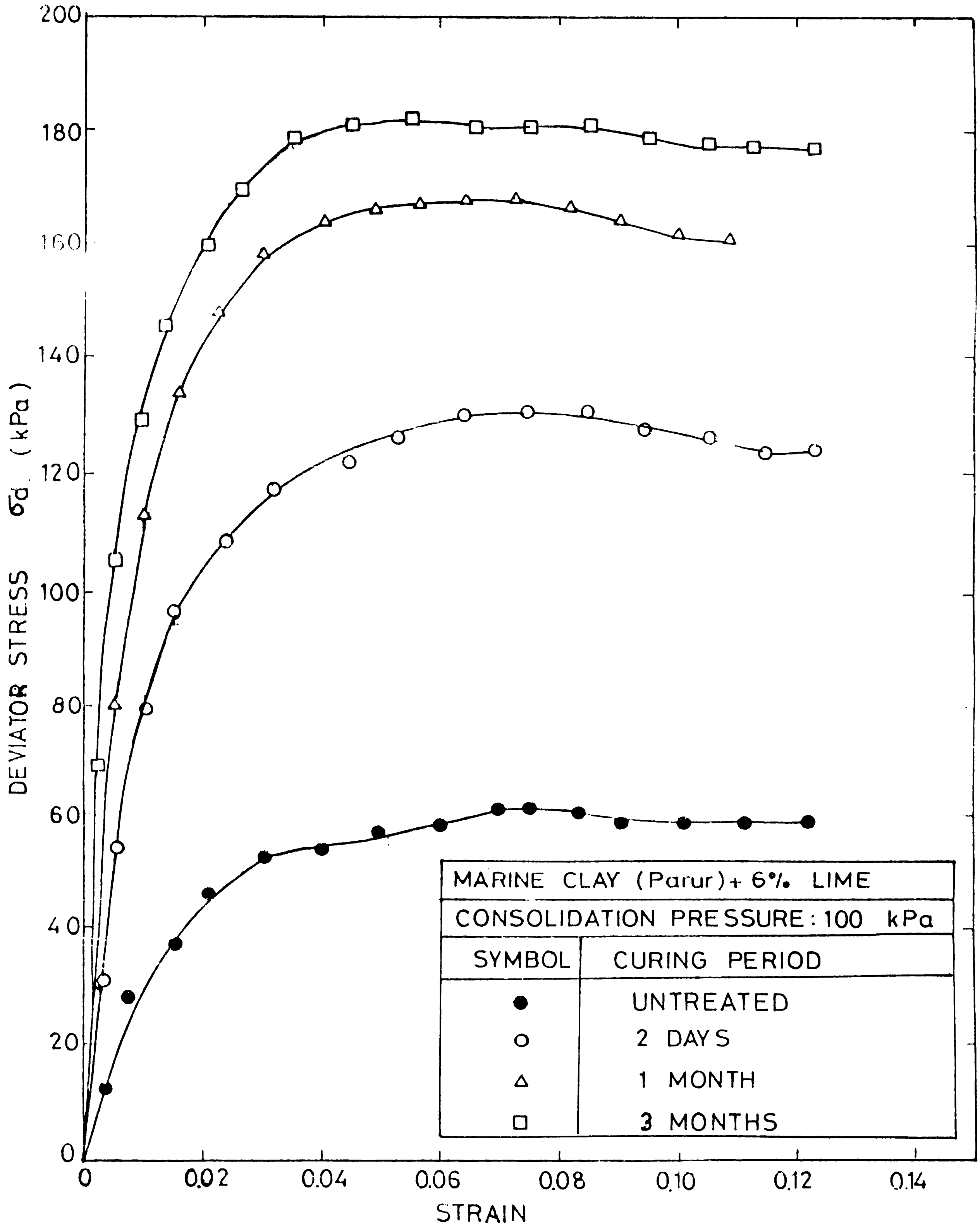


Fig.5.4.18 EFFECT OF LIME TREATMENT ON COCHIN MARINE CLAY (FROM CU TESTS)

months are presented in Fig.5.4.19. Normally, soils with higher shear strength give low values for Skempton's pore pressure parameter A at failure. But in case of lime treated samples, the specimen with longer curing period which is obviously associated with higher strength show larger values for A_f. As discussed earlier, this phenomenon is exhibited by untreated Cochin marine clays. The reason for this unusual behaviour need to be examined.

The effective stress path from the results of triaxial tests with different chamber pressures are presented in Fig.5.4.20. While in Fig.(a) the results of samples cured for three months are presented, Fig.(b) shows stress paths for specimens cured for one month. It can be seen from the two figures that the pattern of the effective stress paths are similar to that of untreated undisturbed clays.

The results of consolidated drained tests conducted on samples treated with 6% lime and cured for one month are presented in Fig.5.4.21. Unlike the $\overline{CU}$ test results presented in Fig.5.4.16, the stress-strain curves do not flatten out and do not show a specific peak value. They register a steady marginal increase and the failure can be defined only against a specific strain. The fact that there is some aggregation of fines during stabilisation can perhaps be explained by a

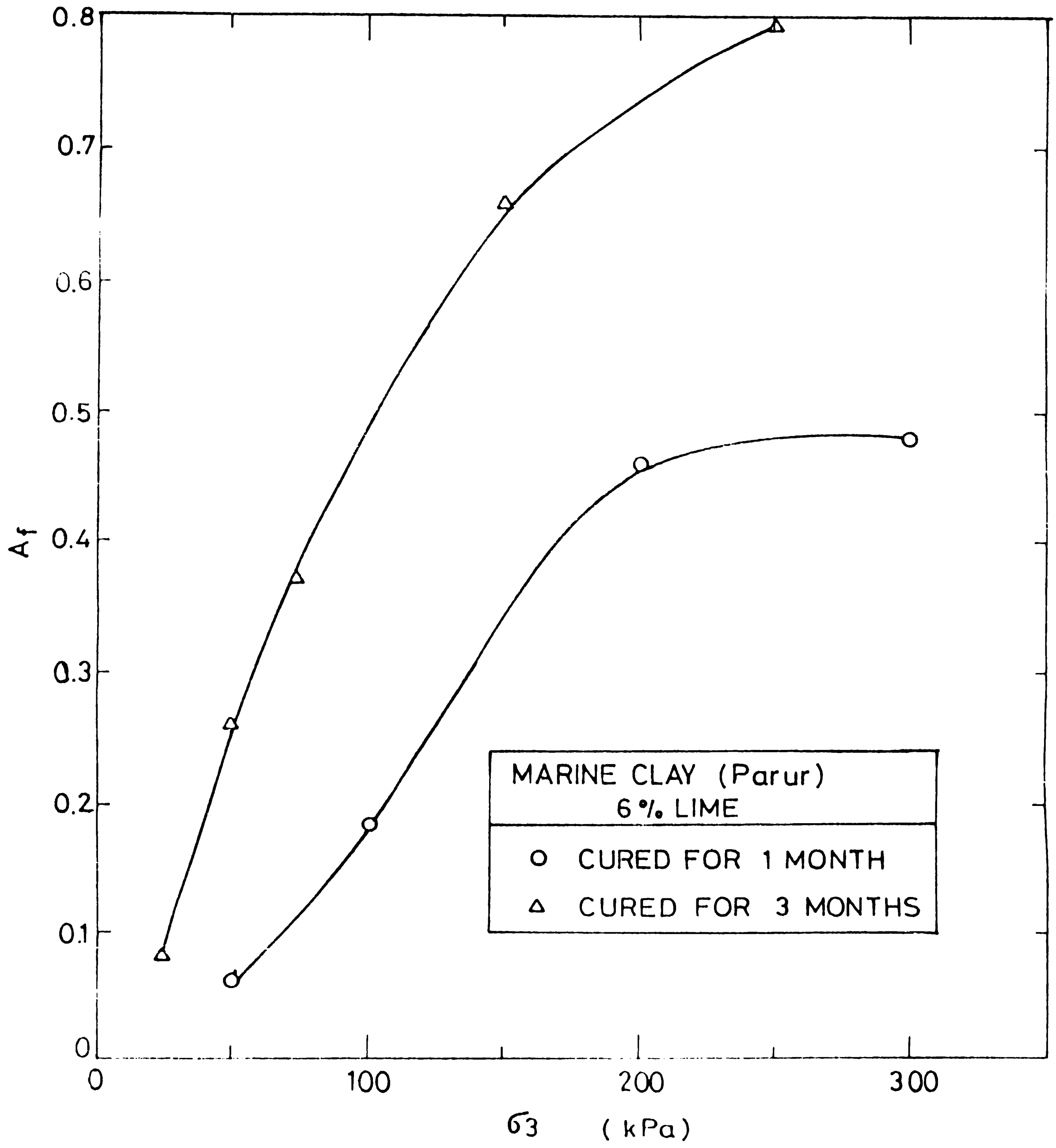


Fig.5.4.19 VARIATION OF A_f WITH AMBIENT PRESSURE FOR LIME TREATED COCHIN MARINE CLAY

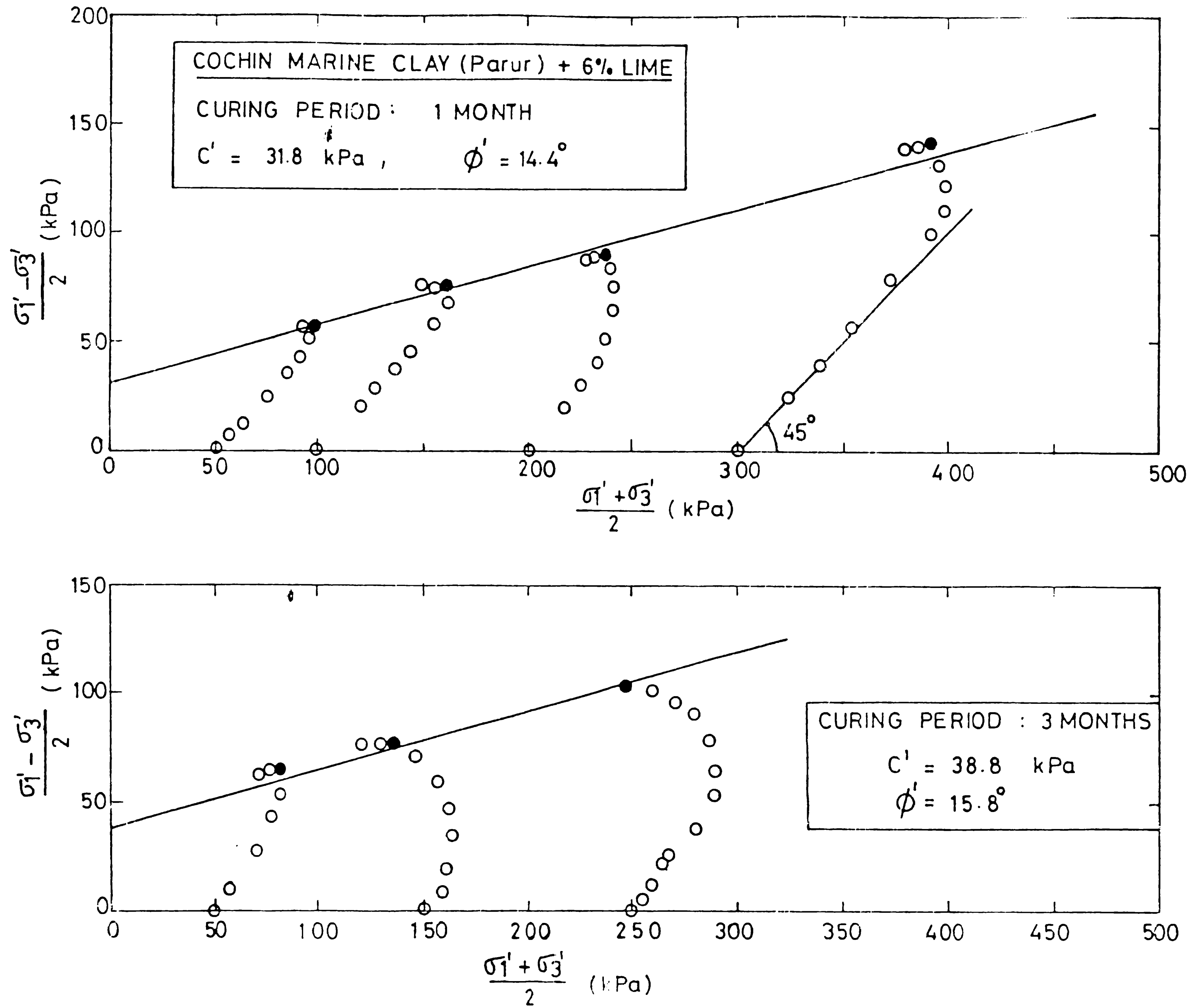


Fig.5.4.20 EFFECTIVE STRESS PATHS FOR LIME TREATED COCHIN MARINE CLAY

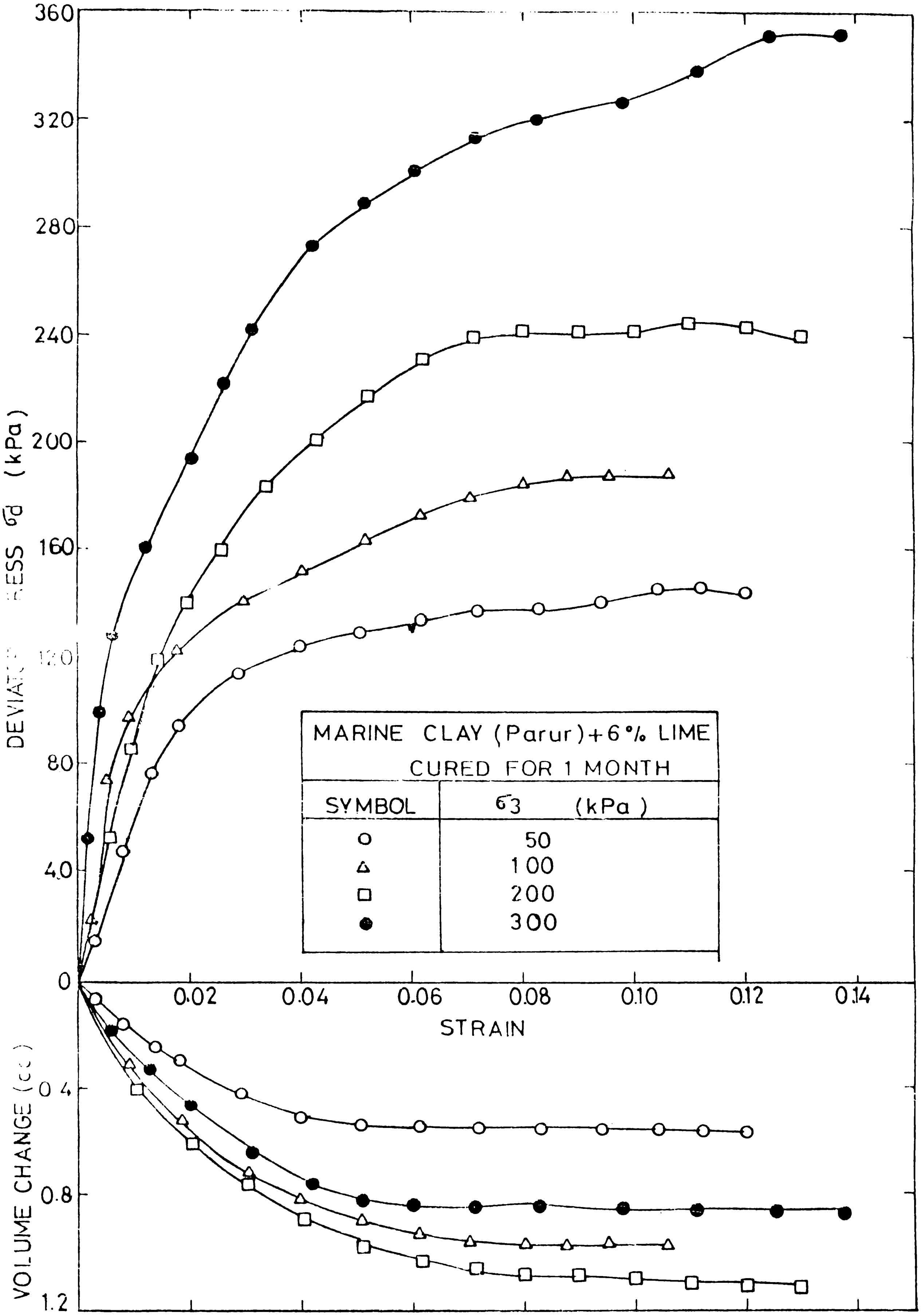


Fig.5.4.21 STRESS-STRAIN CURVES FOR LIME TREATED COCHIN MARINE CLAY (FROM CD TESTS)

comparison of the stress-strain curves obtained from $\overline{CU}$ and CD tests. In case of the former, since there is only limited development of intergranular pressure due to the fact that the pore pressure parameter B is not unity, the shear strength is almost solely depend on unit cohesion and angle of internal friction does not have any significant role. Therefore, when the bonds developed through lime stabilisation are broken, there is no further increase in shear strength. Further, the initial tangent modulus has considerably higher value.

In case of drained test, there is a steady increase of deviator stress with strain without showing any pronounced peak. This possibly indicates that the dilation component is negligible.

The results of the three types of triaxial shear tests viz., UU, CU and CD carried out on treated marine clay samples are presented in figures 5.4.22 and 5.4.23. The former shows the stress-strain relationship for a chamber pressure of 50 k Pa, while the latter shows the results for σ_3 = 200 k Pa. It can be seen that the behavioural patterns are quite in tune with the earlier discussions. In both cases (σ_3 = 50 and σ_3 = 200 k Pa), the UU tests showed the lowest resistance to deviator stress and CD test the highest with CU falling in between, as expected. While UU and CU tests show no further

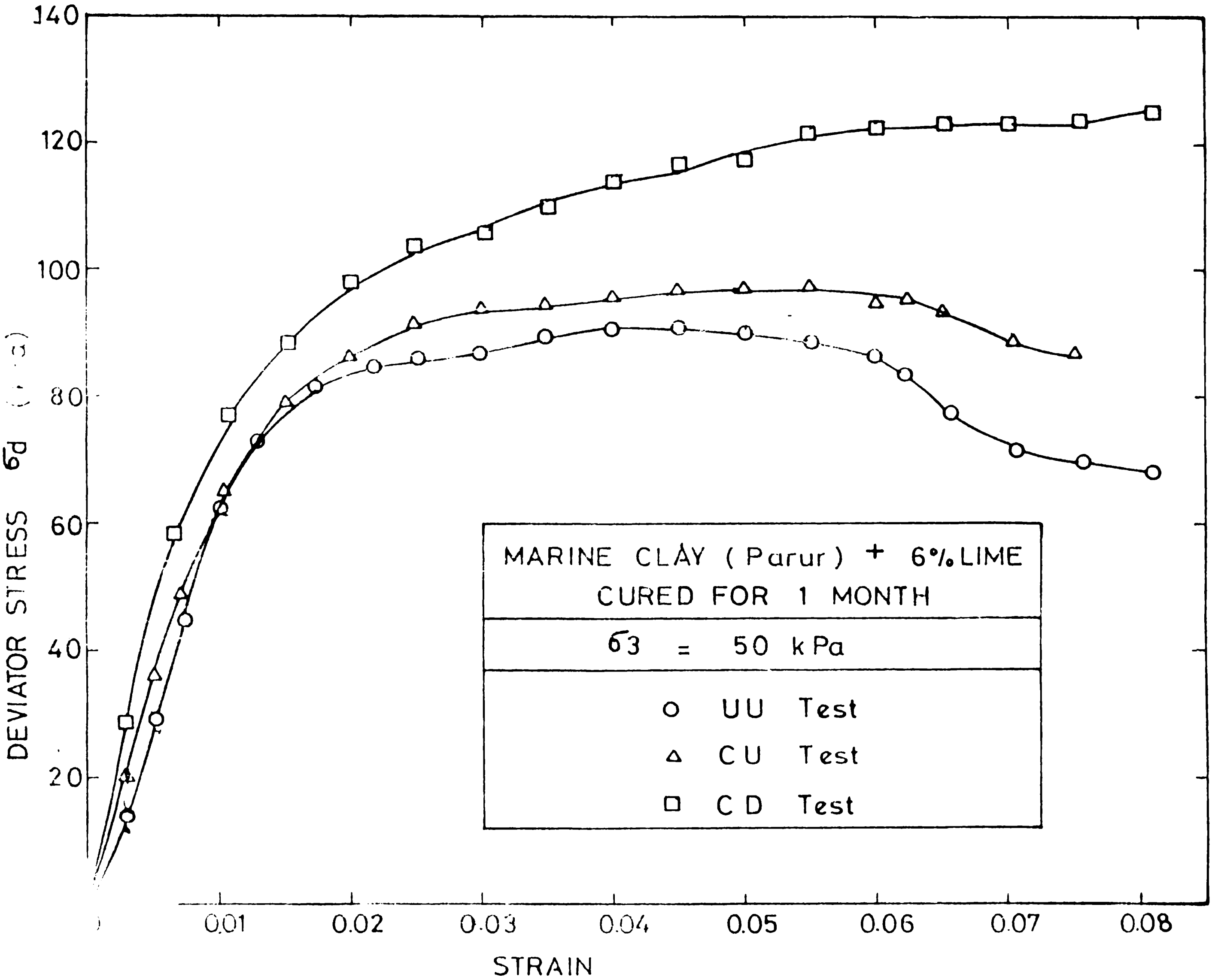


Fig.5.4.22 EFFECT OF THE TYPE OF TRIAXIAL SHEAR TEST ON STRESS-STRAIN BEHAVIOUR

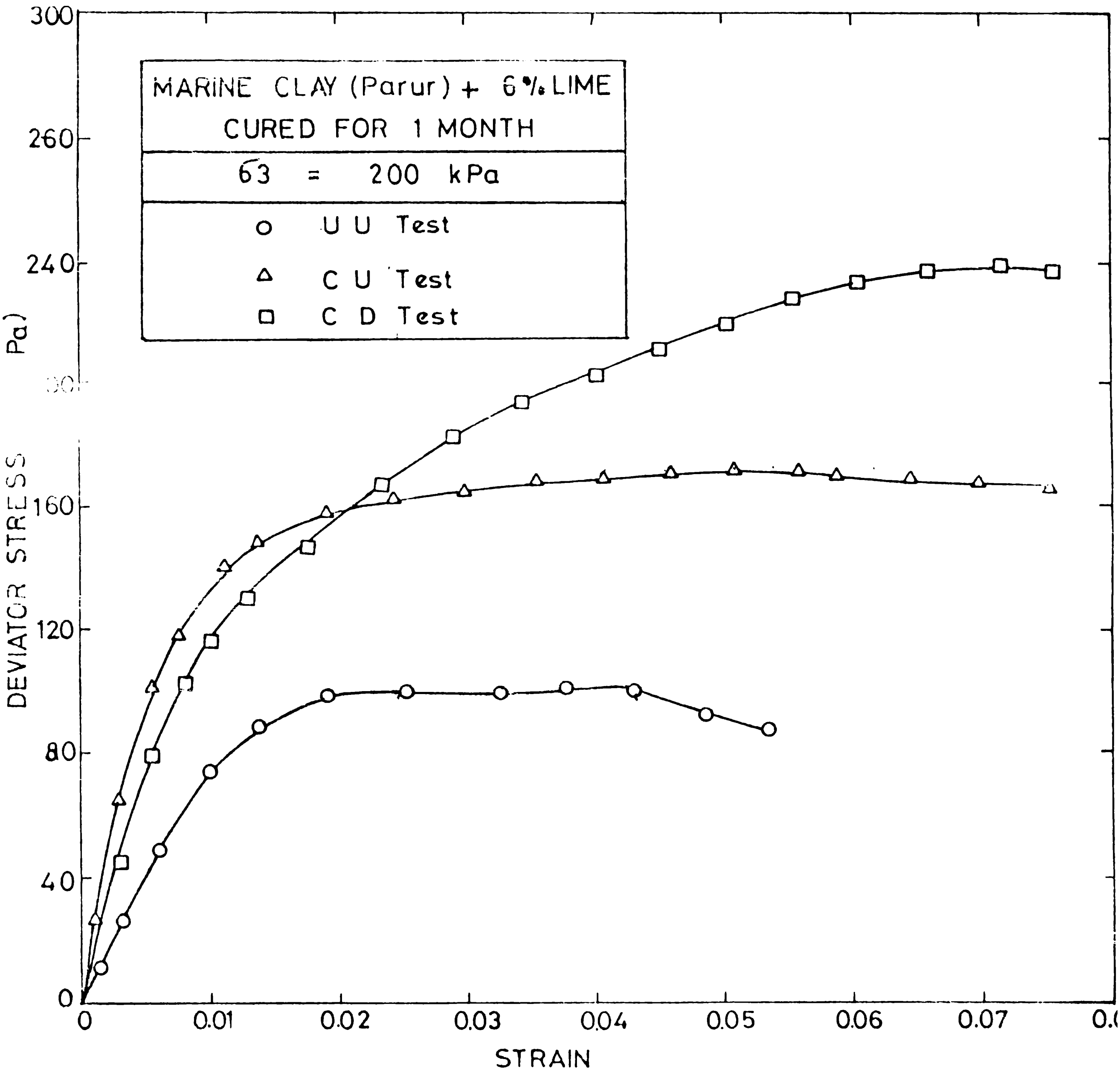


Fig.5.4.23 EFFECT OF THE TYPE OF TRIAXIAL SHEAR TEST ON STRESS-STRAIN BEHAVIOUR

increase in resistance after a peak value, the results of the CD tests show some increase in strength even at large strains, making it necessary to redefine the failure criterion with respect to a specified strain.

5.5 Model studies on treated marine clay beds

Having established the advantages of stabilisation of the soft marine clay deposits in Cochin, using lime, some model studies were conducted in the laboratory in similar lines of some field techniques currently in practice in developed countries.

The two techniques attempted in the laboratory are: (1) rodding method and (2) sand-lime columns method.

In the first method, quick lime approximately 6% by dry weight of the soil was spread over marine clay filled in a tank of size 25 cm x 25 cm x 25 cm, at the natural moisture content (very close to liquid limit). The lime was pushed down into the clay using a 45 cm long, 2 cm dia. rod. This was done continuously at the closest spacing possible so that a large quantity of lime penetrated into the clay by about 20 cm. The top surface was relevelled and kept in humid condition for curing.

Five identical samples were prepared as described above and kept for curing under moist conditions. Samples were tested after 1, 2, 3, 4 and 6 months. The arrangements for testing the stabilised bed as in the case of model studies for square footings is shown in Fig.5.5.1. The load-deformation curves obtained from the five tests are presented in Fig.5.5.2. It can be seen that there is significant increase in the bearing capacity of the treated clay bed. While for the untreated clay bed, the maximum load taken by the plate was only 9.23 kg, the corresponding load for a curing period of one month was 58 kg, which is about 6.5 times to original value. When the bed was cured for six months, the load taken by the plate increased to 102 kg, which is about 11 times the value for untreated clay.

In order to assess the efficacy of the mixing procedure or to check whether the procedure yielded a uniform mass treated with lime, the foundation bed cured for six months was divided into three layers of approximately 7 cm thickness each, after the load test. Each layer was then thoroughly mixed and index properties were determined.

The results of the above tests are presented in Table 5.5.1 and Fig.5.5.3. The grain size distribution curves in the above figure clearly show that only the top layer got

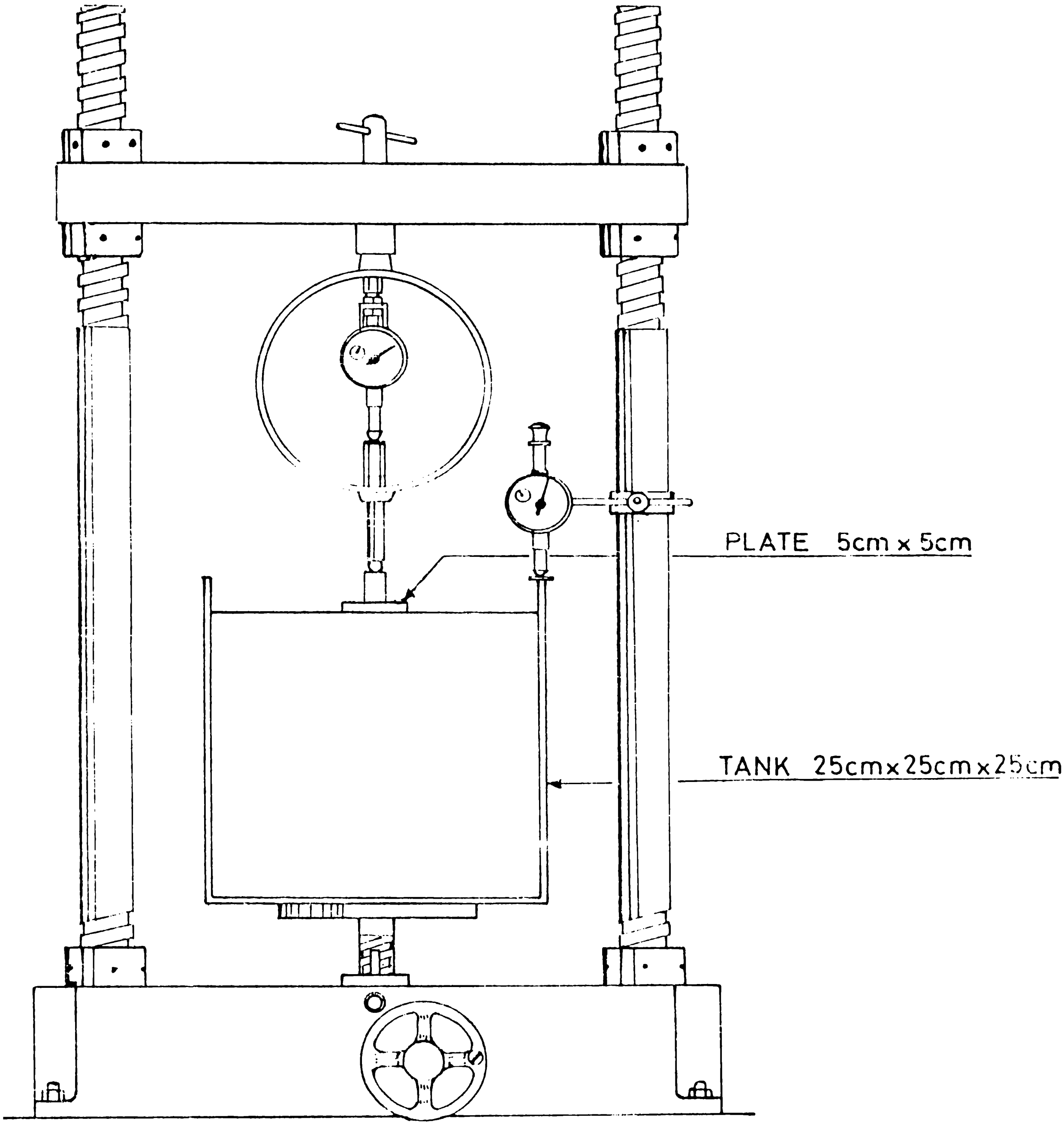


Fig.5.5.1 LOAD TEST SET UP IN THE LABORATORY

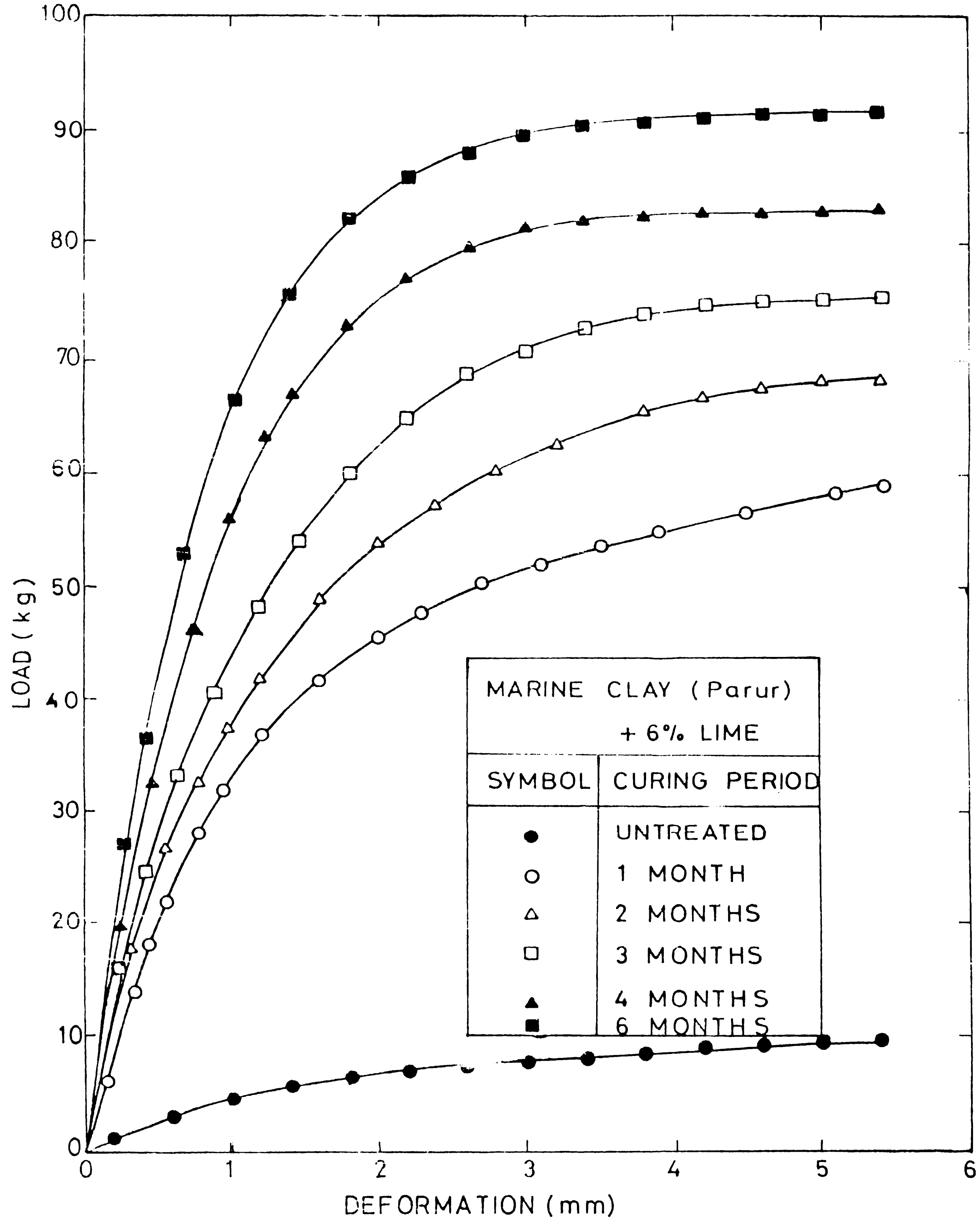


Fig.5.5.2 LOAD - DEFORMATION CURVES FOR TREATED FOUNDATION BED (RODDING PROCESS)

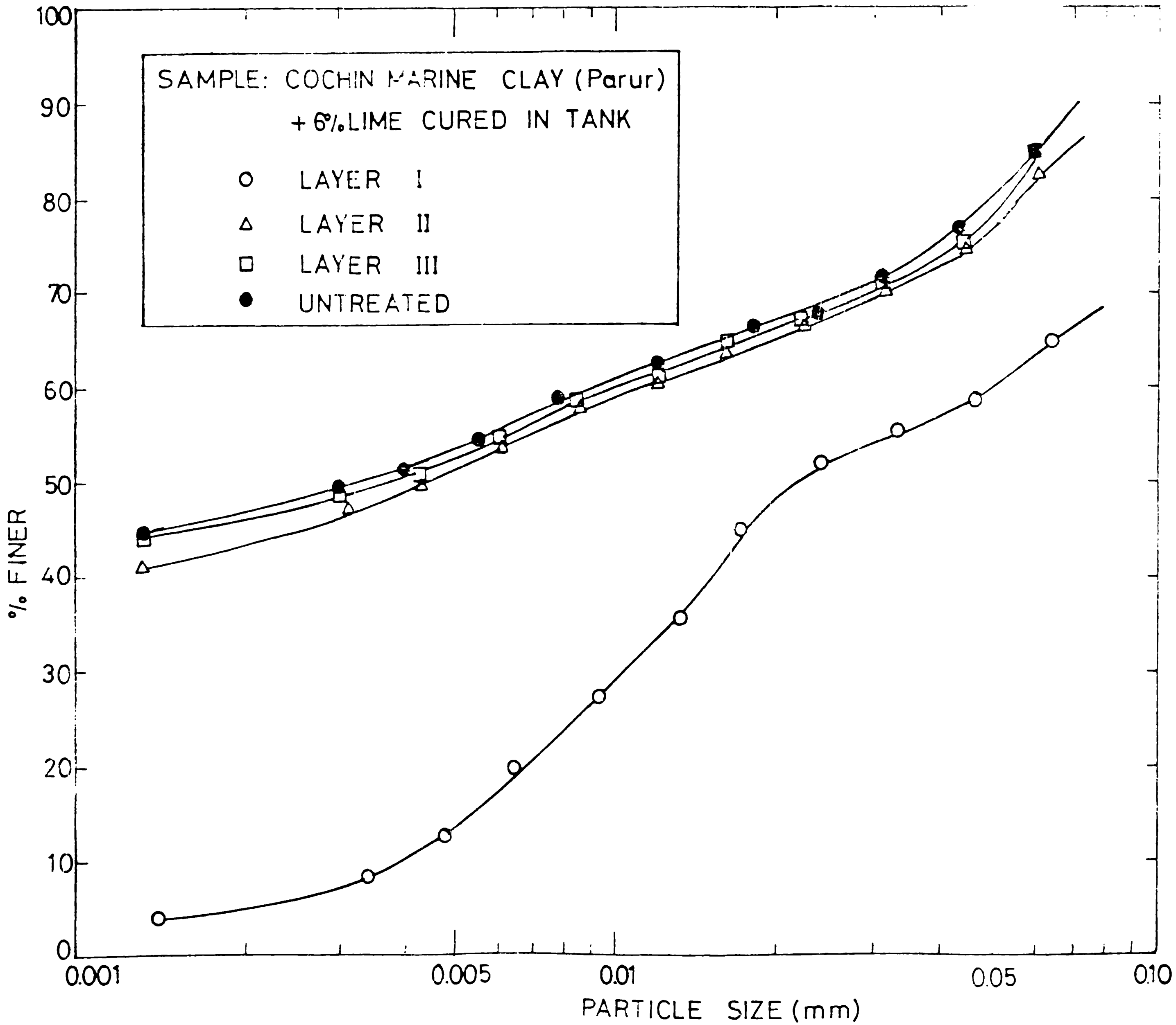


Fig.5.5.3 GRAIN SIZE DISTRIBUTION CURVES

Table 5.5.1

Physical properties of marine clay + 6% lime cured for six months in tank (Rodding process)

Sl. No.	Description of soil sample	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)	Free swell index (cc/g)	Grain size distribution		
						Clay size (%)	Silt size (%)	Sand size (%)
1.	From layer I	122	69.0	53.0	5.82	5	63	32
2.	From layer II	108	42.5	65.5	5.52	43	44	13
3.	From layer III	114	45.0	69.0	5.68	46	45	9
4.	Untreated	114	45.0	69.0	5.70	47	44	9

the benefits from lime stabilisation, as indicated by the low percentage of clay size fraction and high liquid limit.

It is well established that sand-lime mixes will help the stabilisation process and yield better results than treatment with lime alone. In order to have a qualitative assessment of such a treatment, 10% sand and 6% lime were thoroughly mixed and spread over the moist clay in the tank. The mixing was then done by the rodding procedure. Four specimens thus prepared were tested after curing periods of 1, 2, 3 and 6 months. The results are presented in Fig.5.5.4. It be seen from the figure that there is an additional gain in strength by around 20% by adding sand with lime.

As in the earlier case, the clay bed was divided into three layers and tests were conducted for the index properties. They are presented in Table 5.5.2. It can be seen from the liquid limit values and percentage of sand that the method is quite efficient for the top layer. The efficiency decreases with depth.

Strengthening of soft clay beds using sand-lime piles is an established practice in Cochin area. In order to make a quantitative assessment of the bearing capacity of a marine clay foundation bed stabilised with sand-lime piles,

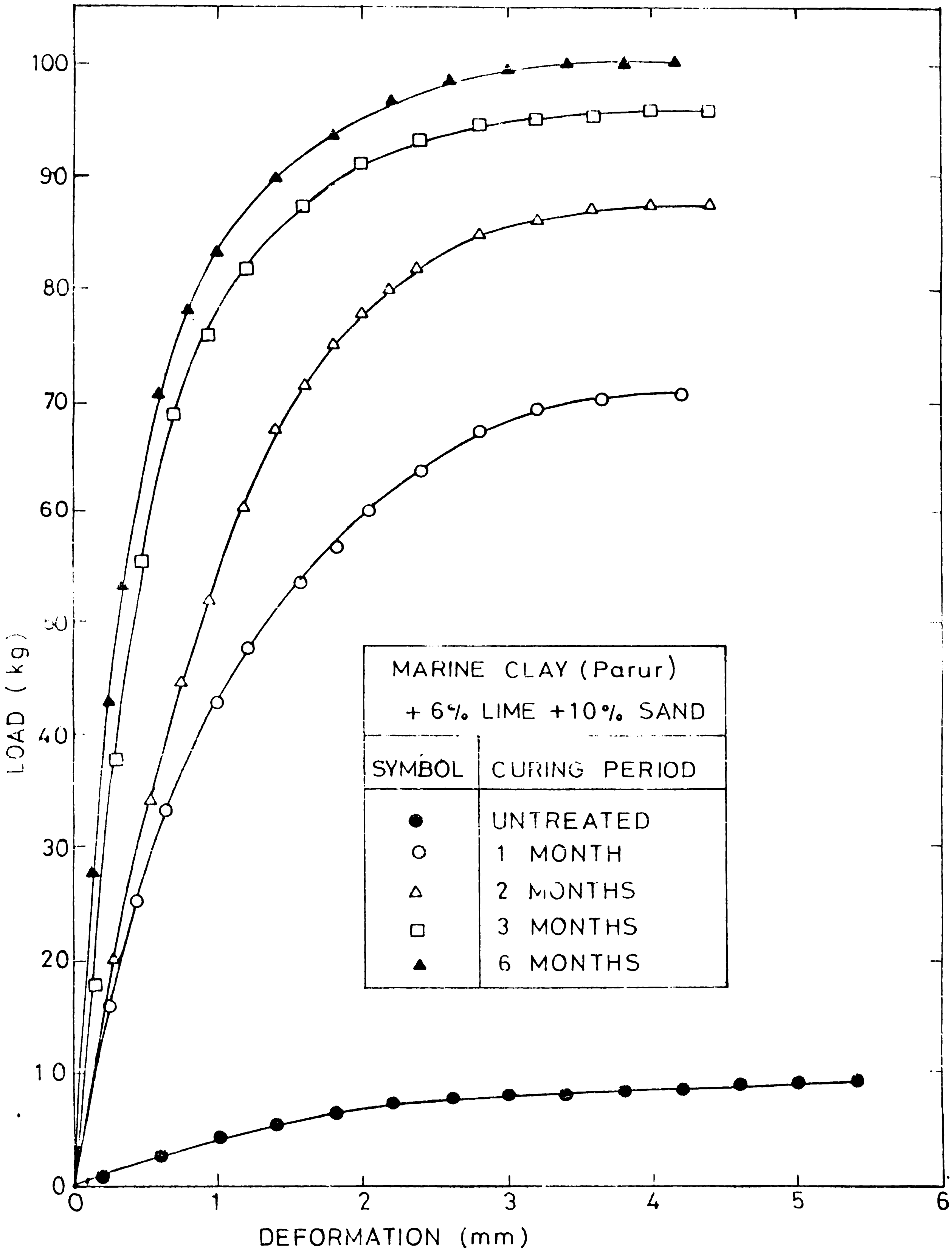


Fig.5.5.4 LOAD - FORMATION CURVES FOR TREATED FOUNDATION BED (RODDING PROCESS)

two specimens were prepared in small tanks (25 cm x 25 cm x 25 cm). Vertical holes 1.5 cm in diameter were made in a grid pattern upto the bottom using a small auger. This was then refilled with sand-lime mixture (6% lime and 10% sand by dry weight of clay). The spacing chosen for the grid was 4 cm.

The results of load tests conducted on these stabilised beds are shown in Fig.5.5.5. While the untreated clay gave a strength of about 18 kg, a marine clay bed stabilised with sand lime piles cured for 1 and 3 months gave strengths of 182 kg and 278 kg respectively. The gain in strength is about 10 times for the former and about 15.5 times for the latter.

Eventhough the gain in strength is substantial in all the cases, the percentage increases are not in tune with similar values in case of unconfined compressive strength test results reported earlier. This is due to the difference in the method of mixing the lime with clay. While a thorough and uniform mixing was possible for specimens in case of UCC strengths, the field practices inevitably suffer from some limitations depending on the method adopted for lime stabilisation. Large scale field tests are essential to establish reliable norms for estimation of gain in strength

Table 5.5.2

Physical properties of marine clay + 6% lime - 10% sand cured for six months in tank (Rodding process)

Sl. No.	Description of soil samples	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)	Free swell index (cc/g)	Grain size distribution		
						Clay size (%)	Silt size (%)	Sand size (%)
1.	From layer I	48	--	--	2.14	12	21	67
2.	From layer II	74	38	36	3.45	22	44	34
3.	From layer III	96	40	56	4.18	40	35	25
4.	Untreated	114	45	69	5.70	47	44	9

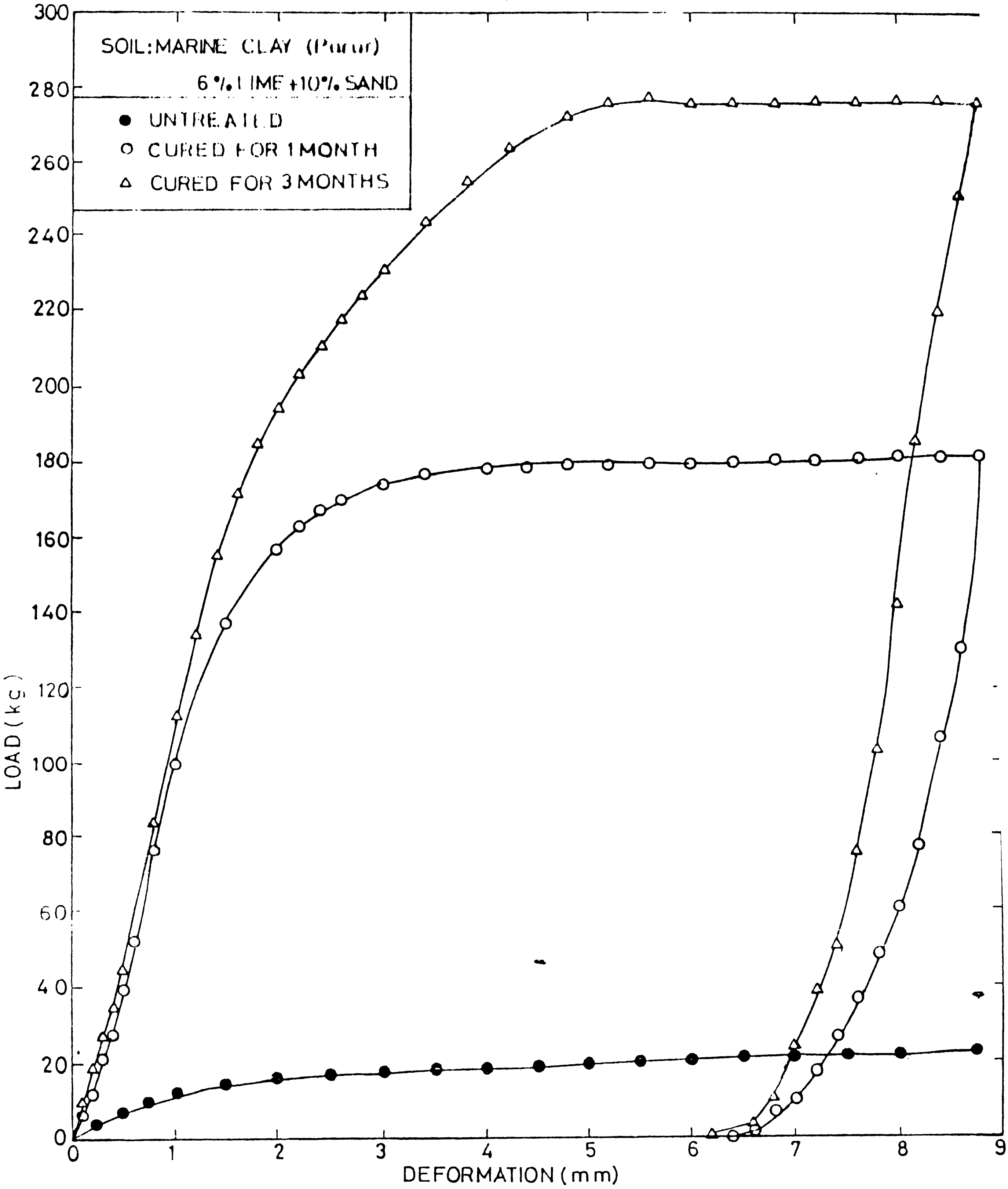


Fig.5.5.5 LOAD - DEFORMATION CURVES FOR TREATED FOUNDATION BED (SAND-LIME PILES)

obtained in field, from results of laboratory tests. The model studies reported herein are intended to serve as a pointer for further work.

Chapter VI

CONCLUSIONS

Based on the detailed investigations carried out on samples of Cochin marine clays, the following conclusions have been drawn on the compressibility and shear strength characteristics.

Through a detailed analysis, the accuracy and reliability of the existing methods for determination of preconsolidation pressure have been compared, bringing out the limitations of each. Eventhough Jose et al. (1989) had proposed a method for preparation of specimens consolidated at predetermined pressures, the efficacy of the method has been verified and confirmed in this work, through a series of consolidation tests with different types of soils and periods of consolidation.

Five available methods for estimation of p_c viz., Casagrande (1936), Burmister (1951), Schmertmann (1955), Becker (1987) and log-log method (1989) were considered in this work. Burmister's and Schmertmann's methods were not taken up for detailed study as they were trial and error procedures which are cumbersome and time consuming. The results from the log-log method were found to be the most advantageous.

A new method wherein the plot between log H and log p was tried and this had the advantage that it does not involve any arithmetical computations. The results obtained were remarkably close to those of log-log method.

Obviously, deformations will be minimal when the clay specimens are loaded within the recompression range. But once p_c is reached and exceeded, the deformations will be considerably higher. A new method for determination of p_c has been suggested in this work wherein the dial gauge readings are plotted against log p. The value of p_c is given by the point at which the lines drawn through the initial and final sets of points intersect each other. The method can claim superiority over other methods in that there are no computations involved and p_c is obtained as soon as the test is over or even before. In addition, compared to Casagrande method, errors that can creep in during the selection of point of maximum curvature and the geometrical construction are eliminated. Through a series of consolidation tests, the applicability of the method to undisturbed samples has been established. The usefulness or efficacy of the method can be further enhanced by reducing the load increment ratio such that points are obtained at closer intervals on the virgin curve.

Precompression is essentially a method of accelerating the consolidation process by preloading the clayey layers by a surcharge load over and above the permanent load to be applied. The principal parameters which can influence a precompression programme have been identified. The effect of overloading ratio and degree of consolidation on consolidation of clay specimens has been studied with marine clay samples collected from Cochin as well as Mangalore.

It has been proven through a series of consolidation tests that the coefficient of secondary consolidation can be made almost zero by selecting higher values of overloading ratio as well as degree of consolidation. The coefficient of consolidation plays a pivotal role in the design and execution of a precompression project. It has been shown that C_v increases with load increment ratio for load increment ratios upto 1.5. Thereafter it tends to decrease. Similarly the values of t_{50} and t_{100} are very high for lower values of load increment ratios. For load increment ratios higher than 1.0, both values tend to be constant.

Eventhough overloading ratio or load increment ratio helps precompression, this is necessarily accompanied by an instantaneous compression which increases with consolidation pressure. Ratio of instantaneous compression to total compression is directly proportional to the consolidation pressure. Compared to montmorillonitic soils, the ratio of instantaneous and total compression in kaolinite soils is very high, at times crossing 50%.

When samples are subjected to repeated loading and unloading, there is considerable improvement in the compressibility. There is little effect on deformation of specimens when the cycle of loading is three or more.

On stabilisation with lime and cement the Atterberg limits of Cochin marine clays are increased. There is significant increase in shrinkage limit. The grain size distribution curves show that there is aggregation of fines. Cement content as low as 3% does not improve the compressibility characteristics. There is a marked improvement in compressibility with 6 and 9% cement. The compression curves for clays treated with lime and cement are more or less identical.

C_v increases considerably with cement content, but remains almost constant with consolidation pressure variations. $C_{\alpha\epsilon}$ of stabilised marine clays reduces with curing period allowed. As cement content increases, the rate of gain in strength of stabilised soil decreases.

In case of untreated samples, the rates of rebound are higher than treated samples, but rebound characteristics are identical for both lime and cement treated clays.

Compared to consolidation, the studies on the shear strength aspects of marine clays have been very limited. Detailed investigations have been carried out in the present study on the stress-strain behaviour through a series of CU, CD and UU tests on Cochin marine clays which have a sensitivity of around 3.0. Most of the tests have been repeated on marine clay samples collected from Mangalore, to obtain a comparative study.

The CU tests showed that the rate of development of pore pressure decreases at higher values of $_3$. This can be attributed to dilation at higher stress levels. The effective stress paths are inclined almost at 45° in the initial stages which shows that the development of pore pressure takes place at a lower rate. This is indicative of a higher fabric strength in marine sediments.

The pore pressure parameter at failure, A_f has low values for Cochin marine clays compared to the values for normally consolidated clays as reported in literature. This is due to the additional bond strength, typical of marine clays.

In case of CD tests, the volume changes are low in the initial stages for all ambient pressures. For higher values of σ_3, the volume change decreases which could be attributed to the dilation. Unlike CU tests, the stress-strain curves do not show a clear peak value which makes an estimation of shear strength, difficult.

The unconfined undrained tests showed that the samples grow stiffer with increase in σ_3 as shown by the values of initial tangent modulus.

Compaction studies were conducted on air dried samples of Cochin marine clay, which showed considerable improvement in compressibility and shear strength characteristics, due to aggregation of fines during drying. The optimum moisture content can be reached either by wetting the fully air dried sample (wetting process) or by drying the moist sample (drying process). CU tests conducted on specimens prepared by both processes showed that drying process gave better results. The angle of internal friction was found to be almost same for specimens prepared by both processes. But drying process gave higher values for unit cohesion. This could be due to the higher percentage of clay size fraction. Specimens prepared with higher compactive effort showed that the increase in shear strength is mainly due to increase in c value, while Ø remained unaltered.

The effect of lime content and curing period on the shear strength characteristics was studied in detail. In the initial stages of curing, the low shear strength could be measured only by laboratory vane shear tests. To compare these values with UCC strength obtained from specimens cured for longer periods, a correlation has been obtained between vane shear and UCC strengths. A statistical fit to the test results gave the following equations for Cochin marine clays.

For natural samples,

$\tau = 1.19\ c$, with a correlation coefficient $r = 0.998$.

For natural and treated clays combined,

$\tau = 1.45\ c$, $r = 0.92$.

As in the case of compressibility studies, lime contents lower than 3% did not have much influence on shear strength. The shear strength steadily increased with percentage of lime and curing period. But, the rate of gain in strength was found to be inversely proportional to lime content which was consistent with earlier findings.

A series of unconfined compressive strength tests showed that specimens tend to become stronger and more brittle as lime content and curing period increases. The residual strength remained almost constant irrespective of the curing period. It was found that the strains corresponding to peak values in the stress-strain curves remained unaltered with lime content. The time required to attain the maximum strength increases with lime percentage. A pair of curves - one between lime percent and $q_{u_{max}}$ and another between curing period and $q_{u_{max}}$ will be useful in the application of lime stabilisation in geotechnical problems.

Triaxial shear (CU) tests conducted on lime treated clays showed that the peak stress values in the stress-strain curves remained steady unlike the natural soils, where it showed a decreasing tendency. Tests showed that as curing period increases, there was considerable increase in c values, the changes in Ø were marginal.

The consolidated drained tests gave stress-strain plots without any peak values like natural soils, necessitating estimation of shear strength for particular strain values.

Model foundation beds stabilised with lime using rodding method and sand-lime piles method gave very encouraging results. The substantial gain in strength obtained in the laboratory studies cannot be made available in the field due to several constraints as shown by the model studies.

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