A STUDY OF THE PHYSICAL AND ENGINEERING BEHAVIOUR OF COCHIN MARINE CLAYS

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CERTIFICATE

This is to certify that the thesis entitled **"A Study of the Physical and Engineering Behaviour of Cochin Marine Clays"** is a report of the original work carried out by Shri Babu T.Jose, under my supervision and guidance in School of Technology. No part of the work reported in this thesis has been presented for any other degree from any other University.

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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.K.K.Punnoose, Professor of Civil Engineering, M.A.College of Engineering, Kothamangalam, in School of Technology. No part of this thesis has been presented for any other degree from any other institution.

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Chapter I

INTRODUCTION

The growth of Indian port cities over the last few decades has been phenomenal and Cochin is no exception. With the increase in population, housing and construction of various facilities have been a problem with urbanisation. Having exhausted all the trouble free hand, man is now on the look out for techniques to improve areas which were originally considered uninhabitable. Thus studies on the nature and engineering behaviour of soft clays covering long stretches of coastal line and methods to improve their geotechnical properties have been of great relevance, of late.

The peninsular region of India consists of nearly 6400 kms of coastal line. The coastal belt starting from Rann of Cutch on the western side to Calcutta on the eastern shore line is almost completely covered by soft highly compressible marine clays (Fig.1.1). Problems arising out of the high compressibility and poor shear strength have thrown considerable challenges to the geotechnical engineers.

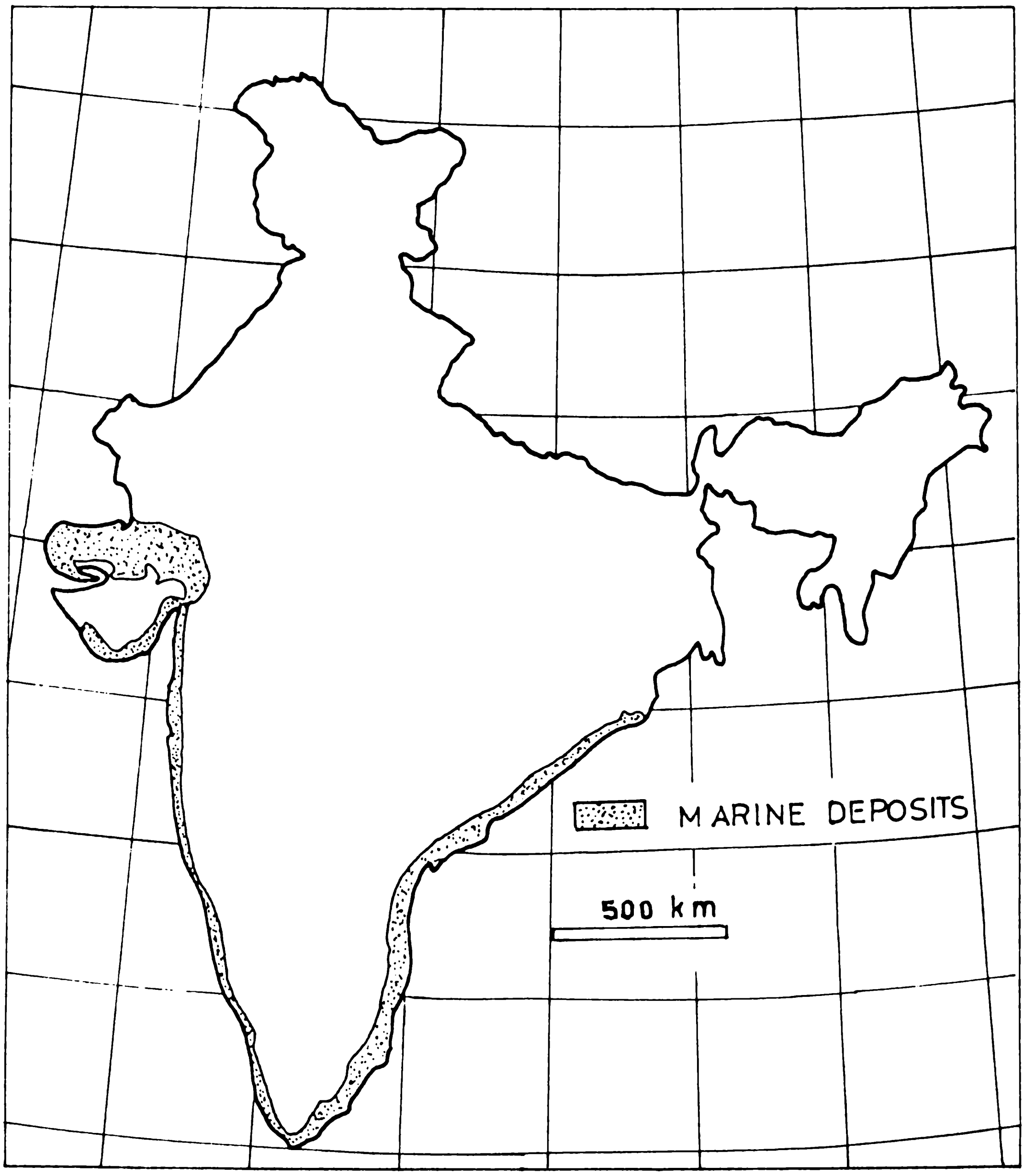


Fig. 1.1. DISTRIBUTION OF MARINE CLAYS IN INDIAN PENINSULA.

The aim of the present investigation is to study in detail the physical and engineering behaviour of the marine clays of Cochin. While it is well known that the marine clays have been posing numerous problems to foundation engineers all along, the relevant literature reveals that no systematic and comprehensive study has been attempted to date.

There have been innumerable problems connected with settlement of structures founded on these soils and with embankments and deep cuttings for both railways and highways. The Indian marine clays are pleistocene to recent in origin, considered to be young and were deposited in salt or brackish water environment (Narasimha Rao & Kodandarama Swamy, 1984; Mohan & Bhandari, 1977). It has been established that the physical characteristics of offshore deposits are essentially the same as those of comparable terrestrial deposits (Noorany & Gizieuski, 1970). Hence the knowledge gained and data generated could be extended to foundations of offshore structure also. With the present hectic activity in oil drilling in deep sea, this assumes importance. The present work envisages a systematic study of the engineering behaviour and electrical characteristics of Cochin marine clays together with their physical properties. The knowledge gained is suitably used to improve these properties with appropriate additives.

The contents of the various chapters are briefly described below.

Chapter II presents a detailed review of the relevant literature bringing out the limitations that exist in the present knowledge and focusses attention on the scope of the problem.

Chapter III deals with the materials and methods used in the investigation. It describes in detail the method of collection of samples from far flung locations in Greater Cochin area. The various tests carried out are described in detail with procedures followed.

Due to the peculiar nature of marine clays wherein the geotechnical properties are significantly altered, by drying in almost all tests for physical properties, special care has to be taken to arrive at meaningful values. The Indian Standard Specifications do not deal with the special attention required by marine clays especially during preparation of samples. I.S.Procedures, if followed as such, can yield highly erroneous results in case of marine clays. This aspect has been well brought out while discussing the procedures adopted for each property. Modifications or amendments required in case of each experiment has also been presented, highlighting the need for each.

It was noticed during the preliminary experiments that 'drying' and methods of drying considerably influence the physical and engineering properties of these clays. Hence care was taken to study the various characteristics under three initial conditions viz. moist, air dried and oven dried. How samples are prepared under these three initial conditions for each test is described in detail. Preparation of 'washed' samples and 'soaked' samples for different tests is also dealt with.

The significant role, the dispersing agent plays in case of marine clays while carrying out the hydrometer analysis has been brought out. While discussing the consolidation tests on marine clays, the improvements that can be made in conducting the test have been pointed out. A different loading pattern, which is simpler, has been adopted for the consolidation tests. The limitations of the existing log t and $\sqrt{t}$ methods for determination of coefficient of consolidation have been brought out. The rectangular hyperbola method suggested by Sridharan et al. (1987) has been discussed in detail.

It has already been pointed out that I.S. procedure for free swell index can yield misleading results even in case of ordinary soils. In case of marine clays the problem becomes more complex as drying of samples alters its swelling characteristics. But Indian Standards insist on a dry specimen for the test. The procedure suggeted by Sridharan et al. (1985) was found to be more appropriate for marine clays, which was followed throughout.

Attempts were made to improve the strength characteristics of marine clays with 18 different additives as listed in this chapter. Lime was found to be the most suitable stabilising agent. The procedure adopted for preparation of quick lime and of stabilised soil samples for different tests is also presented.

Chapter IV deals with the physical properties of Cochin marine clays. The results of the index properties vis-a-vis the initial condition of the samples are listed in detail for the first time. Tests on Atterberg limits and shrinkage limit have yielded good correlation between plasticity index and liquid limit, liquid limit and clay content and shrinkage limit and clay content.

How the initial conditions of samples can influence the grain size distribution is also discussed. As marine clays mostly exist in the form of flocs, dispersion of clay is very important in hydrometer analysis of marine clays. Through a series of tests using eight dispersing agents, it has been proved that the I.S.recommendation is also the most suitable one for marine clays.

Through the conventional activity chart, Cochin marine clays could be classified 'active' while moist and 'inactive' after drying. Free swell index could be correlated with clay content and liquid limit. The influence of pore fluids of different dielectric constants on the physical properties such as liquid limit and free swell index have been discussed in detail.

A significant finding reported in this chapter is the correlation between shear strength and water content ratio (which is the ratio of natural water content to liquid limit of the soil) which exist for the marine clays.

Excellent linear correlations could be obtained between water content ratio and logarithm of shear strength with a high correlation coefficient even with marine clays collected from locations, 50 kms apart, because they are of same geological origin. Similar correlations could be obtained for other soils also. More potential uses of these semilog plots between τ and WCR are indicated.

Numerous consolidation tests conducted on these clays could bring out many features of the compressibility characteristics of Cochin marine clays in particular and other soils in general as discussed in chapter V. The effect of air drying and oven drying on the consolidation behaviour has been brought out. The reliability of the compression index C_c in the determination of total settlement has been discussed pointing out that the C_c values, as suggested by Indian Standards yield very high settlement values which are very rarely realised in field. The slope of the e - log p curve in the region of consolidation pressure of $40T/m^2$ or $80T/m^2$ need not represent the compression of the clay layer in field. Highlighting these, a new method called segmental compression index method has been suggested for the calculation of consolidation settlements.

It has been shown in this work that duration of application of consolidation pressure influences the resistance to compressibility developed by marine clays considerably. The development of this resistance during secondary compression has been well substantiated. Rebound characteristics can be indicative of bond strength developed.

An important offshoot of the long series of consolidation tests is the development of a new technique called 'log-log method' for the determination of preconsolidation pressure. It has been shown that the value of p_c can be obtained with greater ease and accuracy from the point of intersection of two straight lines drawn through the initial and final points in a log e-log p plot. The advantages of this method over the existing Casagrande method could be brought out by comparing the p_c values obtained and the actual preconsolidation pressure at which the samples have been consolidated and unloaded to seating load. The prior knowledge of p_c of samples helped to establish the credentials of the new method.

The desirable changes brought in by air drying of marine clays can be taken advantage of in the construction of pavements and enbankments. It has been brought out that the compressibility and strength characteristics of air dried Cochin marine clays got vastly improved. This introduces the possibility of using this clay after air drying. Particle aggregation by oven drying is almost irreversible.

As part of basic studies, experiments were also conducted with four fluids of different dielectric constant. The results clearly indicate that the clay minerals in Cochin marine soils is of most montmorillonite type.

The engineering behaviour of lime-treated marine clays is discussed in chapter VI. Eventhough the chemistry of the lime stabilisation is not fully understood, the factors which can be the reason for the gain in strength are discussed along with the contradiction that exist in the findings of the different workers. Rather than going into the complex reactions, the investigations have been directed to result oriented study on the improvements in physical properties and compression behaviour.

After a preliminary study on the development of shear strength over a period of one month, on treatment with 18 different additives, lime was identified as the most beneficial one. Six percent by dry weight of lime was found quite adequate and a strength gain as high as eighteen times has been achieved.

Eventhough the effect on liquid limit has been erratic, plastic limit readily increased with lime treatment which brought down the plasticity index. The compressibility characteristics improved significantly with lime content and curing period. Deformation of treated soils reduced to 20-25% of compression of the untreated clay, for pressures lower than the bond strength (quite analogous to preconsolidation pressure) developed during curing. Lower percentages of lime were found to register a faster rate in gain of bond strength. While the value of de/d(log p) decreased with time of curing and lime content, coefficient of consolidation also was found to decrease.

Chapter VII presents the electrical characteristics of Cochin marine clays. Investigations have been carried out to measure electrical resistivity with variations in water content, density and physico-chemical environment. Since interpretations of field resistivity measurements are based on the absolute resistivity of the individual soil layers, the laboratory determination of these values are important. It was found that resistivity measurements are dependent on sample dimensions and hence a standardisation of the test is essential.

Polarisation of water molecules by the soil interferes with resistivity measurements to a great extent questioning the validity of measurements and measuring techniques. The need for taking into consoderation the dipole moment and dielectric constant of the pore fluid is brought out. Still the potential of resistivity measurements as an easy and rapid technique to give an insight into the geotechnical properties of soils is highlighted. The superiority of resistivity measurements with A.C. supply is discussed in this chapter.

Chapter VIII gives a detailed list of conclusions drawn from the various investigations carried out in this work.

Chapter II

REVIEW OF PAST WORK

2.1 COCHIN MARINE CLAYS

Most of the Greater Cochin area consists of marine clays which are known for its high compressibility and poor shear strength. The depth of the soft layer is considerable, often extending to 30 to 40 metres. Hence it is not economically feasible to replace them by better material.

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.

Failures of embankments and bridge foundations constructed on these soft marine deposits have been reported by Ramanatha Iyer (1966). While Chekkidikad bridge failed within an year of construction due to differential settlement between central pier and abutment; the Chanthathottam bridge failed by differential settlement of piers. Rotational failure of embankments in Cochin Bye Pass could be stopped only with extensive berms.

Marine deposits can be found all along the coastal belt of Indian Peninsula. Narasimha Rao and Kodandaramaswamy (1984), based on investigations on samples 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.

2.2 PHYSICAL PROPERTIES

Jagadish Narain and Ramanathan (1967) were perhaps the earliest to observe the physical properties of marine clays in 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.

The phenomenon of reduction in plasticity on drying has been attributed to dehydration of hydrated iron/ aluminium sesquioxides and of the halloysite mineral, by Frost (1967). He brings out the significant changes during oven drying and indicates the importance of carrying out tests on soil in its natural state. 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 in the exchange sites and presence of a high salt concentration (9.4 gm/ litre) which facilitates strong interparticle attraction and a close spacing of the particles. In their study they have ruled out the possibility of coulombic particle-cation-particle linkage. According to them, the aggregation of particles and variations in plasticity characteristics is due to cohesion from surface tension forces arising from capillary minisci formed between the particles at the boundaries of the soil specimen during drying.

Atterberg limits have been in wide use for preliminary soil classification. Seed et al. (1964) presented an exhaustive work dealing with the fundamental aspects of the empirically defined values. Sridharan and Rao (1975) discuss in detail the mechanisms controlling the liquid limit of clays. Atterberg limit tests are analogous to shear tests. The results obtained by Casagrande (1958) and Norman (1958) by the use of direct shear and vane shear apparatuses, respectively, indicate that the strength at liquid limit is of the range 15-30 gm/cm^2.

The shear strength must be due only to the net attractive forces of the particles. Seed (1964) suggested that the liquid limit depends on the interparticle forces and therefore on the surface characteristics of the soil. But 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.

In case of normally consolidated marine clays, Skempton and Bjerrum have found that there is a well defined correlation between Cu/p and plasticity index and any deviation from this relation can be considered as a test for sampling disturbances (Bishop & Henkel, 1962). A similar correlation has been suggested for effective angle of shearing resistance Ø against plasticity index where Ø reduces with plasticity index (Bjerrum & Simons, 1960).

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. The liquid limit considerably reduced on air drying. They established that air drying caused formation of aggregates and this was responsible for the irreversible reduction in plasticity. They also proved that correlation of the type proposed by Skempton and Bjerrum cannot be considered as valid for clays where changes in soil structure are predominant. They also showed that the mechanical properties are dependent on soil structure which is a function of interparticle forces as well as particle size and arrangement.

Most of the soil testing is of either an inferential or an environmental nature. An inferential testing program is one in which the properties are not directly measured but inferred through empirical correlations. The Atterberg tests, proposed by Atterberg (1911), was introduced into soil mechanics by Terzaghic (1925). Casagrande (1958) perfected and standardised it as a percussion test, which became the most inferential soil test with wide universal acceptance.

Soil with water contents at their liquid limits as discussed earlier, possess definite but small shearing resistance which was successfully measured by many (Youssef et al., 1965; Hajela & Bhatnagar, 1972). The flow index obtained from the liquid limit test can be used with advantage for determining other properties such as plastic limit. The one point method has received wide attention as a simple procedure. 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) and Clayton and Jukes (1978).

Based on the same reasoning, the possibility of developing the one point method for the cone penetration method of liquid limit has been successfully examined by Nagaraj and Jayadeva (1981). They have shown that if Casagrande percussion apparatus is used.

$$w_L = \frac{w}{1-K(\log N - \log 25)} \qquad (2.2.1)$$

Another expression for determination of liquid limit by one point method is of the form

$$\frac{w_L}{w} = \left(\frac{N}{25}\right)^{\alpha} \qquad (2.2.2)$$

Mohan and Goel (1959) have shown that the variation in α is less than 1% for the range of N between 20 and 30.

The drawbacks of the percussion test, such as difficulty in cutting a groove in soils of low plasticity, and the tendency of soils to slip, have been overcome in the cone penetrometer test. Nagaraj and Jayadeva (1981) have developed a one-point method which can be used for determining the liquid limit from cone penetration test

$$w_L = w/0.77 \log D \qquad (2.2.3a)$$

$$w_L = w/(0.65 + 0.0175D) \qquad (2.2.3b)$$

Several attempts have been made to link plasticity index with liquid limit through empirical relations (Casagrande, 1948; Dersseault & Scafe, 1979; Seed et al., 1964; Wroth & Wood, 1978). All these relations have been suggested on the presumption that plasticity index and liquid limit are fundamentally independent parameters. After examining the data of 520 samples and plotting on a plasticity index vs. liquid limit graph, Nagaraj and Jayadeva obtained a linear correlation

$$PI = 0.74(LL - 8) \qquad (2.2.4)$$

with a correlation coefficient of 0.957.

They have also shown that plasticity index and liquid limit are not independent parameters but the former is directly related to the liquid limit water content.

Attempts have been made to correlate Atterberg limits with surface area, cation exchange capacity, swelling behaviour, California Bearing Ratio, compressibility etc. and to use alternate test methods like the cone penetration method (Uppal & Aggarwal, 1958), the dye adsorption method (Ramachandran et al., 1963), the vane shear method (Darienzo et al., 1963) and the soil moisture tension method (Russel et al., 1970). But it remains a fact that investigations on the physical mechanism governing the liquid limit of a soil have been very limited.

Sridharan et al. (1975) have investigated in detail the possible mechanisms governing the liquid limit of kaolinite and montmorillonite types of clays. To alter the force fields governing the particulate system, six organic fluids and water with wide variations in their dielectric properties have been used in the investigation. The cone penetrometer method was chosen for the study as it was found convenient especially with organic pore fluids. Their investigations showed that for kaolinite, a non-expanding lattice type of clay, the contribution due to diffuse double layer is insignificant and the liquid limit is primarily governed by the shearing resistance at the particle level. Hence an increase in dielectric constant reduces the liquid limit. In case of montmorillonite, the liquid limit is governed by the shearing resistance. Being an expanding lattice type of clay, the contribution from the diffused double layer overrides and primarily governs the liquid limit.

Studies by Sridharan et al. (1988) on kaolinite clay showed that the liquid limit of these clays has no correlation with the percentage of clay size fraction. The liquid limit of kaolinite is not a function of diffuse double layer thickness.

The shear strength of fine grained soils is largely dependent on their water content, w_o. Wroth and Wood (1978) gave a linear relationship between w and the undrained shear strength C_u in the following equation

$$w + A \log C_u = \text{constant} \qquad (2.2.5)$$

where A is a constant.

But the result obtained by Skempton and Northey (1953) indicated that the relationship between the water content or liquidity index I_L and C_u is not a straight line on a semilogarithmic plot. But both agreed that the shear strength of a soil at plastic limit is 100 times that at its liquid limit. According to Wroth and Wood (1978) in the cone penetration apparatus the plastic limit value corresponds to the water content which permits a penetration of 2 mm, which is too low to be achieved. This has been overcome by making use of a plot between liquidity index and penetration wherein the point representing the plastic limit is obtained from the intersection of two lines (Harrison, 1988).

2.3 CONSOLIDATION

The most important bench mark in the studies on consolidation was set by Terzaghi in 1923, eventhough some aspects of consolidation such as pore pressure dissipation was understood even during Sumerian civilisation (Kerisel, 1985). In the last 60 years considerable work and progress have been achieved to improve our understanding of clay behaviour. But it has to be conceded that understanding the problem fully has not been achieved yet and soil engineers even now are wondering whether there is a combination of primary and secondary compression during primary consolidation.

The basic and most general equation for one dimensional consolidation can be written (Berry & Poskilt, 1972) as

$$\frac{d\epsilon_v}{dt} = \frac{1+e_o}{r_w} \frac{d}{dz} \left(\frac{k}{1+e} \frac{du'}{dz} \right) \tag{2.3.1}$$

where

ϵ_v = vertical strain
z = depth related to initial thickness
t = time
e_o = initial void ratio
e = present void ratio
k = permeability
γ_w = unit weight of water
u' = excess pore water pressure.

The marine clay of eastern Canada, called Leda clay, covers the broad, highly developed regions of Ottawa and St.Lawrence river valleys. 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. Hamilton and Crawford (1959) have investigated this clay in great detail and drawn many useful conclusions. The result showed that the size of load increments do not affect the e - log p curves. A method was developed to determine the end of primary consolidation under a load increment. A rapid method called controlled strain type of test for preconsolidation was perfected during the studies on Leda clay.

In 1979, Butterfield reported that for soft soils, the log f vs. log p relations could be represented by a straight line in the range of normal consolidation 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 natural 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 segments of log f - log p curves intersect agrees well with the consolidation yield stress which is obtained from the graphical method proposed by Casagrande.

Through a series of consolidation tests on Whangamarino clay, Newland and Alley (1960) could arrive at definite trends involving parameters like pressure increment ratio, thickness of sample etc. They showed that when time rate curves are plotted in terms of change in void ratio vs. the logarithm of time, the slope of the secondary tail is independent of a wide range of factors. For a given duration of increment, the ratio between the void ratio change occurring as secondary compression, and the total consolidation in the increment decreases, as higher pressure increment ratios are used. They have pointed out that considerable shortening of standard consolidation test, without loss in accuracy, may be obtained by using much larger pressure increment ratios and restricting the duration of increments to the time necessary for the completion of primary consolidation.

In case of marine clays, the electrolyte concentration of free pore water can vary. Mesri and Olson (1971) have shown that both the electrolyte concentration in the free pore water and the pH do not influence the consolidation curves of calcium montmorillonite. Consolidation tests in which organic fluids were used demonstrated that swelling of montmorillonite is caused by either adsorption of the pore fluid or formation of diffuse double layer.

2.3.1 Constant rate of strain tests

In the last two decades, numerous constant rate of strain tests (CRS tests) have been performed on natural soils at various strain rates. CRS studies on eastern Canada clays by Crawford (1965), Vaid et al. (1979) and Leroueil et al. (1983), on Swedish clay by Sallfors (1975), on Belfast clay by Graham et al. (1983) are well accepted. All the tests show that the strain rate influences the compression curves of clays. At a given strain, the higher the strain rate, the higher the effective stress.

Constant rate of loading tests (Jarret, 1967), stage loading tests (Larsson, 1981) and controlled gradient tests (Leroueil et al., 1983) confirm the important effects of strain. In stage loading tests, such as the conventional 24 hours oedometer test, it is possible to characterise two different phases primary and secondary. Eventhough various definitions have been given for these two phases (Bjerrum, 1967; Leonards, 1977), the definition of Crawford (1985) is most suitable. According to him "during the so called primary phase, the permeability of the soil is the predominant factor controlling the rate of volume change and during the secondary phase, it is the resistance of the structure that determines the rate".

In a constant rate of strain test, the stress-strain curve depends on the strain rate used. Data compiled by Leroueil et al. (1983) for eastern Canada clays show that the preconsolidation pressure dobtained in CRS tests is on average 28% higher than that obtained in conventional tests. Holtz et al. (1986) have found similar results on very stiff Montalto de Castro clay.

2.3.2 Compression index

Compression index C_c, which is the slope of the plot of e - log p curve, represents the compressibility of the soil. To have an independent method of estimation, many investigators in the past have empirically linked the compression index with the inferential and state parameters of the soil (Hough, 1957; Oswald, 1980; Koppula, 1981). Since the state of soil in nature is varying and complex, a unique C_c equation is not tenable. Further the empirical C_c equations which have no scientific basis can be used only in specific soil states.

Nagaraj and Srinivasa Murthy (1986) have critically examined the known empirical compression index equations to identify their possible scientific basis. They have also tried to define the soil state in which these equations can be used. For this they have used the rationally generated compressibility equation (Nagaraj & Srinivasa Murthy, 1983) which is based on the Gouy Chapman diffuse double layer theory and is applicable to normally consolidated saturated uncemented soils. Their study has indicated that the (e/e_L) - (R-A) relationship is more general and unique than the d - (R-A) relationship. The parameter e_L inherently accounts for the variation in physico-chemical environment in addition to the specific surface of the soil. Two compatible compression index equations ie., $C_c = 0.234\ e_L$ and $C_c = 0.39\ e$ have been rationally derived.

For saturated normally consolidated clays, the compression index, C_c is the slope of the straight line portion of the e - log p curve. Among the various correlations available in published literature relating compression index and liquid limit water contents, Skempton's (1944) relationship is the most widely accepted. He observed that for samples selected at random, from different parts of the world, where the initial moisture contents were at liquid limit water contents all points are located close to the line which is represented by the equation

$$C_c = 0.007(LL - 10) \qquad (2.3.2.1)$$

The classical Gouy-Chapman diffuse double layer theory, has been effectively used by Nagaraj and Srinivasa Murthy (1983) to evolve a generalised e - log p relationship

$$\frac{e}{e_L} = 1.099-0.2237 \log p \qquad (2.3.2.2)$$

The involved microparameters, specific surfaces and half-space distances between clay platelets are successfully replaced by easily measurable macroparameters such as liquid limit water contents and insitu void ratios in the development of the above equation.

The relationship between C_c and soil compressibility is not a direct one, and wrong conclusions are easily drawn about the relative compressibilities of soils by comparing values of C_c or liquid limit. The manner in whichs C_c is presented implies that soils with high values of C_c naturally have high compressibility. For a liquid limit of 200, $C_c = 1.33$ and for 100, the value of $C_c = 0.63$. Comparing these two C_c values one would think that the former will be twice as compressible as the latter.

The expressions for consolidation settlement is

$$S = \frac{C_c . H}{1+e_o} \log_{10} \frac{p_o + \Delta p}{p_o} \qquad (2.3.2.3)$$

where

S = settlement (m)

H = thickness of clay layer (m)

e_o = initial void ratio at pressure p_o

p_o = effective overburden pressure

Δp = the pressure increment.

It is evident from this expression that the soil parameter indicating compressibility is $\frac{C_c}{1+e_o}$ and not C_c alone. Saxena et al. (1978) define $C_c/1+e_o$ as compression ratio.

Wesley (1988) has examined the relative significance of C_c, CR and m_v as measures of compressibility and their relationship to Atterberg limits. The investigations were based on samples from three soil groups--sedimentary clays, tropical red clays and volcanic ash soils. The results showed that remoulded soils undergoing virgin consolidation, the compression ratio CR (or m_v) is a better indicator of soil compressibility. For undisturbed soils, the recompression ratio RR = $(C_s/1+e_o)$ is a better indicator of in situ compressibility than CR.

2.4 LIME STABILISATION OF SOFT CLAYS

Eventhough the use of lime dates back to Romans, the first attempt to improve soil by lime was perhaps made in 1924 in the state of Missouri as an experiment on construction of a road using hydrated lime (Uppal & Palit, 1964).

Clayey soils can be stabilised by the addition of a small percentage of lime. It enhances many of the engineering properties of the soil. Generally the amount of lime required to modify a clay soil varies from 1 to 3%, while that required for cementation varies from 2 to 8% (Bell, 1988). Montmorillonite clays respond more readily to lime treatment than kaolinites.

When lime is added to clayey soils, calcium ions are combined with clay minerals which leads to an improvement in soil workability. There is an immediate increase in plastic limit and a decrease in liquid limit. The optimum lime content which produces maximum plastic limit is called lime fixation point. Lime in excess of this is used for cementation process and increases the strength of the soil.

When lime is used to stabilise clay, it forms **a calcium silicate** gel which coats and binds the lumps of clay together and occupies the pores in the soil. The gel then crystallises to form an interlocking structure.

The quantity of lime needed is related to the clay mineral content. Ingles and Metcalf (1972) suggested that the addition of upto 3% lime would modify silty clays, heavy clays and very heavy clays. While 2 to 4% was required for the stabilisation of silty clay and 2 to 8% was proposed for stabilisation of heavy and very heavy clays.

In most cases the effects of lime on plasticity of clay is more or less instantaneous (Bell, 1988). As explained by Davidson and Handy (1960) calcium ions from lime causes a reduction in the plasticity of cohesive soils, so that they become more friable and more easily worked. Very small quantities of lime only are required to bring about these changes in plasticity. Tests carried out by U.S.Bureau of Reclamation indicated that the addition of 4% lime reduced plasticity index of clay from 47 to 12% (Anon, 1975). However in case of kaolinite clays, lime may sometimes increase the plasticity index.

Mateous (1964) showed that the minimum amount of hydrated lime required to be added to montmorillonite clays for maximum increase in plastic limit is given by

$$PL_{(max)} = \frac{\%\ 2\ \text{microns clay}}{35} + 1.25 \qquad (2.4.1)$$

The addition of lime to clayey soils reduces their potential for swelling. With small additions high calcium hydrated lime is more effective than other hydrated limes. Quick limes appear to be more effective than hydrated limes as far as improvement of shrinkage characteristics are concerned (Anon, 1975).

Compared to cement stabilised soils, compaction of lime treated soils is more tolerant. Not only is delay less critical, so also is compaction moisture content. In other words, lime treatment flattens the compaction curves. This ensures that a given percentage of prescribed density can be achieved 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 (Bell, 1988). However the strength gain in the soil normally will more than compensate for changes in compaction optima, and they should not be regarded as disadvantageous.

The strength of lime soil mixtures 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 (Ingles & Metcalf, 1972). 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 occurring in soils with higher clay fractions. This decrease is because lime itself has neither appreciable friction nor cohesion.

Of late, lime column technique has been introduced as a foundation improvement technique in soft clays. Lime column can be used to support light to medium structures in soft clays. Somayazulu et al. (1984) carried out detailed model studies on lime columns in test tanks with piles of 50 mm diameter and 300 mm depth, 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 seeps from the columns into surrounding soil. There is considerable reduction in plasticity index and significant increase in particle size due to aggregation.

The reactivity of Japanese marine soils to lime treatment is quite high as evaluated by unconfined compressive strength q_u (Terashi et al., 1977). Deformation modulus, E_{50}, which is the secant modulus at $q_u/2$ and strain at failure show that the residual strength at failure is very low and the strain at failure is small. These indicate the brittle nature of the treated Kawasaki marine clay.

The treated marine clay behaves like an overconsolidated soil due to the development of shear strength by bonding.

Engineering properties of lime treated soils have been studied in detail by highway engineers for base or sub-base material. For the application of lime in foundation engineering, development of deep in situ mechanical mixing method has been very helpful. This method is called the deep lime mixing method or D.L.M. method or D.M. method. The machine consists of one steel pipe and several shafts for mixing blades. Chemical agents in slurry form are supplied into the ground by pressure through small flexible pipe.

Another machine has been developed in Japan (Yamanouchi, 1978) for lime stabilisation upto a depth of 1.3 metre while travelling 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. A test road stabilised with lime using the above machine was 375 m long with six kinds of sections, heavily instrumented with settlement meters, inclinometers, earth pressure cells, and pore water pressure meters. The road was subjected to repeated truck running reaching 30,000 repetitions. The stabilised bed withstood the test well and provided useful data for analytical studies.

Since the use of lime stabilisation in foundation engineering is closely linked with machinery for deep lime mixing, the development of such machinery is equally important.

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 sand wicks 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.

2.5 ELECTRICAL RESISTIVITY OF SOILS

One area where the electrical resistivity of soil is of paramount importance is in the interpretation of results from electrical resistivity ground surveys (Higginbotham, 1976). Other areas of relevance include applications to non-destructive testing of concrete piles (McCarter, 1981); the grounding characteristics of transmission tower foundations: electro-osmotic draining and consolidation of clays (Butterfield & Johnson, 1980).

Variation of electrical resistivity of soil with moisture content has been the subject of little investigation (McCarter, 1984). At low moisture contents, resistivity decreases rapidly with increasing moisture content. Studies on remoulded Cheshire clay showed that resistivity is a function of

a) Moisture content
b) Air-void ratio
c) Degree of saturation
d) The fractional volume of water in the soil which is defined as the ratio of the volume of water in the sample to the total volume of sample.

The results indicated that resistivity vs. fractional volume of water relationships have been established for two different clays over a practical range of moisture contents, and a unique relationship exists between these parameters for each clay.

The Offset Wenner System of vertical electrical sounding has been developed to reduce the spurious effects on sounding curves which are produced by near surface variations in ground resistivity. Barker (1988) demonstrated how high quality Wenner apparent resistivity curves can be easily obtained in engineering site investigations. The new system eliminates the electrode spacing errors and increases productivity.

One of the problems faced by the geophysical engineers is the selection of the appropriate choice of interpretation method. There are a number of techniques available but no single method can be considered fully satisfactory. Kate and Khichchu Mal (1983) have shown that Moore's cumulative plot method is more reliable compared to other methods.

Murthy and Chandra (1981) have investigated the use of electrical resistivity method for foundation analysis. Eventhough it is a quick method, they are of the opinion that only the general character of the soil can be estimated and it has to be supplemented by subsoil borings.

From the electrical resistivity survey of an unstable slope at Simla, Malhotra (1981) showed that resistivity results match very well with other geophysical surveys.

When an impulse of alternating current is applied to a clay-water-electrolyte system, its response can be measured in terms of a resistance R and a capacitance C. Capacitance can be expressed in terms of dielectric constant, which is the ratio of the measured capacitance to the capacitance of vacuum. The dielectric constant in the radio frequency range of 10^6 to 10^8 Hz is referred to as apparent dielectric constant

$$\epsilon' = \frac{Cd}{A\epsilon_o} \qquad (2.5.1)$$

where

d = the length of sample

A = cross sectional area

ϵ_o = dielectric constant of vaccum (8.85×10^{-14} farad cm^{-1})

Arulanandan and Smith (1978) have shown that ϵ' is a function of frequency. The difference of ϵ' between the frequencies 3×10^6 Hz to 75×10^6 Hz is referred to as dielectric dispersion $\Delta\epsilon'$ (Fernando et al., 1977). $\Delta\epsilon'$ can be made use to identify the clay mineral, estimate clay content and water content, cation exchange capacity etc. (Fernando & Doshi, 1984). According to Arulanandan et al. (1973) the value of dielectric dispersion can be used to characterise clays without destroying or separating the soil mass into different sizes.

Chapter III

MATERIALS AND METHODS

3.1 INTRODUCTION

The aim of the present study has been to investigate in detail the physical properties and engineering behaviour of the deep marine clay deposits covering extensive areas in Greater Cochin. Since a comprehensive study was intended, the results will be more reliable and findings will have wider acceptability if sampling and testing cover an area as extensive as possible. Further in an investigation as envisaged herein, the test results can crystallise and converge to well-substantiated conclusions only if the soils collected have the same geological origin and identical physico-chemical environment which controlled the sedimentation process. When the former calls for as extensive an area as possible, the latter tends to confine to narrow limits. The ideal extent lies in between where sampling is done over an area as large as possible without causing serious losses in uniformity of samples, leading to dispersion of results. Hence the significance in the choice of locations from which samples are to be collected cannot be overemphasized.

Samples were collected from seven locations spread over Greater Cochin area. The sites are:

1. Munambam
2. Parur
3. Cheranelloor
4. Elamkulam
5. Maradu
6. Nettoor
7. Kumbalam

The locations of these seven sites are shown in Fig.3.1.1. While Munambam is at the northern boundary, Kumbalam is the southern tip of Greater Cochin with a distance of about 50 km between them.

The sampling locations were chosen after scrutinising as many soil investigation reports as possible and after conducting a detailed reconnaissance survey. In almost all locations thick uniform layers of marine clays could be obtained only after 2 to 3 metres. Hence undisturbed and representative samples could be collected only from bore holes advanced by shell and auger method. Undisturbed samples were collected from different depths to study the engineering properties. At Nettoor, thick layers of marine

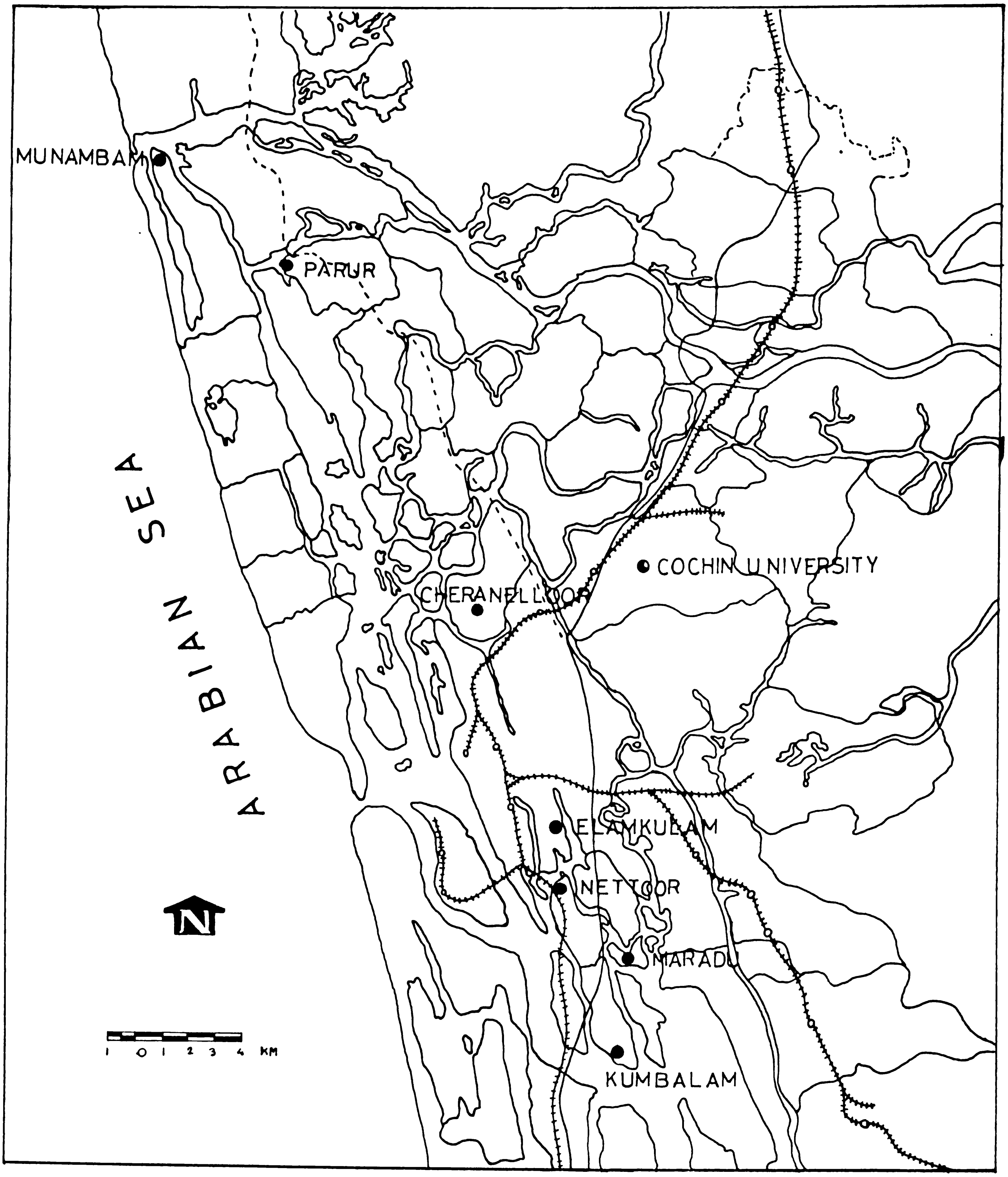


Fig. 3.1.1 MAP OF GREATER COCHIN SHOWING THE LOCATIONS.

clay were available at shallow depths. Hence sampling at this site was done by open excavation.

It can be seen that sampling locations at Munambam, Parur, Kumbalam and Maradu were within a few metres of the saline backwaters. Nettoor and Elamkulam were a little interior and Cheranelloor was almost a kilometre away from the backwaters. These locations helped to cover a wider range of soil samples.

Soil profiles

Fig.3.1.2a and b show the soil profiles of the six locations where samples were collected from bore holes. At Munambam the marine clay layer was encountered from 8.0 to 14.5 metres from ground level. In Parur, the clay layers were met with between 5.5 and 13.5 metres. The clay layer at Cheranelloor was a thin seam just about a metre thick at a depth of 5 metres.

At Elamkulam, the marine clay layers were extremely soft as indicated by the zero N value. The collection of undisturbed samples met with some difficulties here as the soft clays often failed to hold on to the sides of the sampling tubes. The soil profile at Maradu shows medium stiff

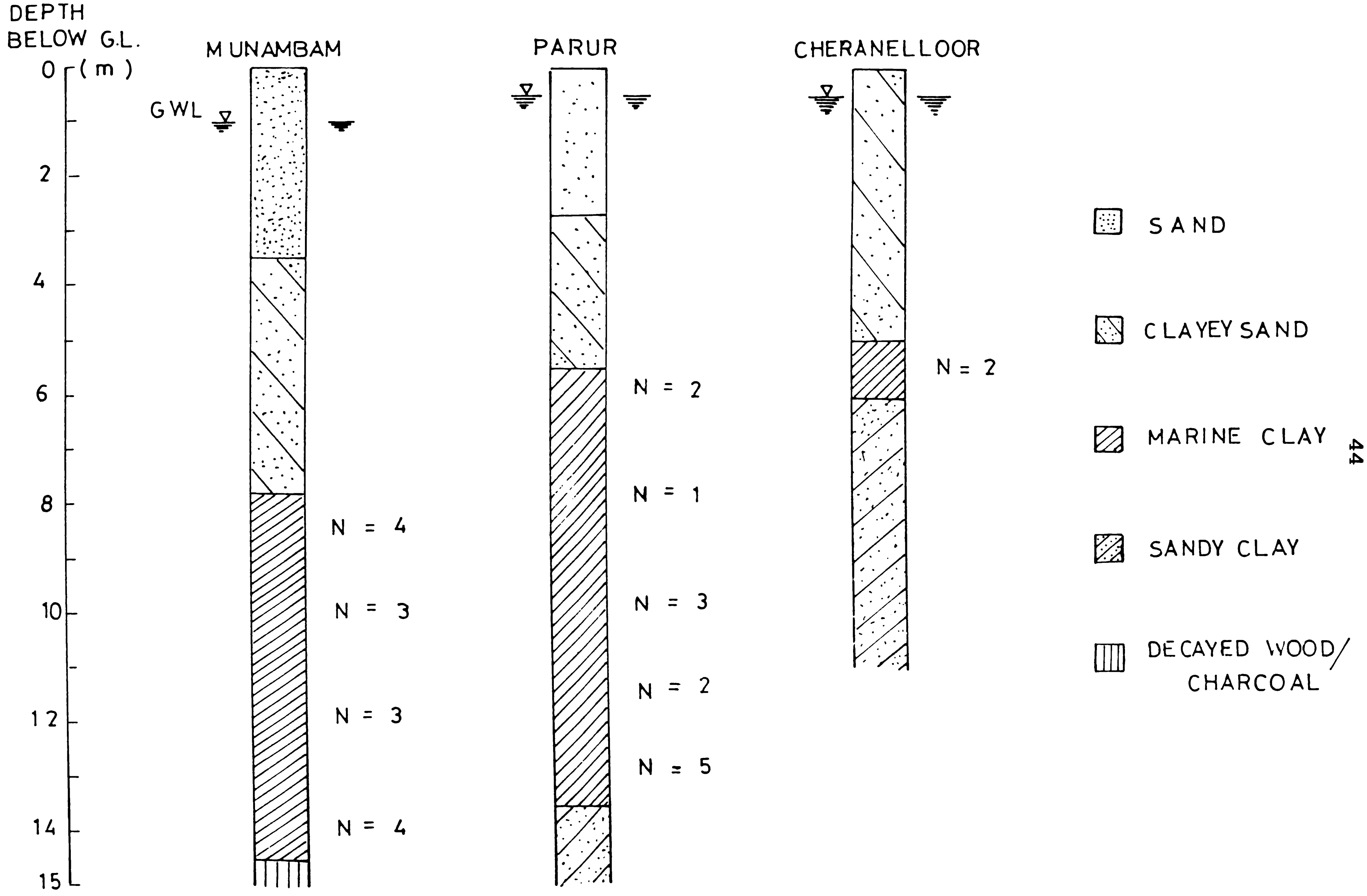


Fig. 3.1.2a TYPICAL SOIL PROFILES.

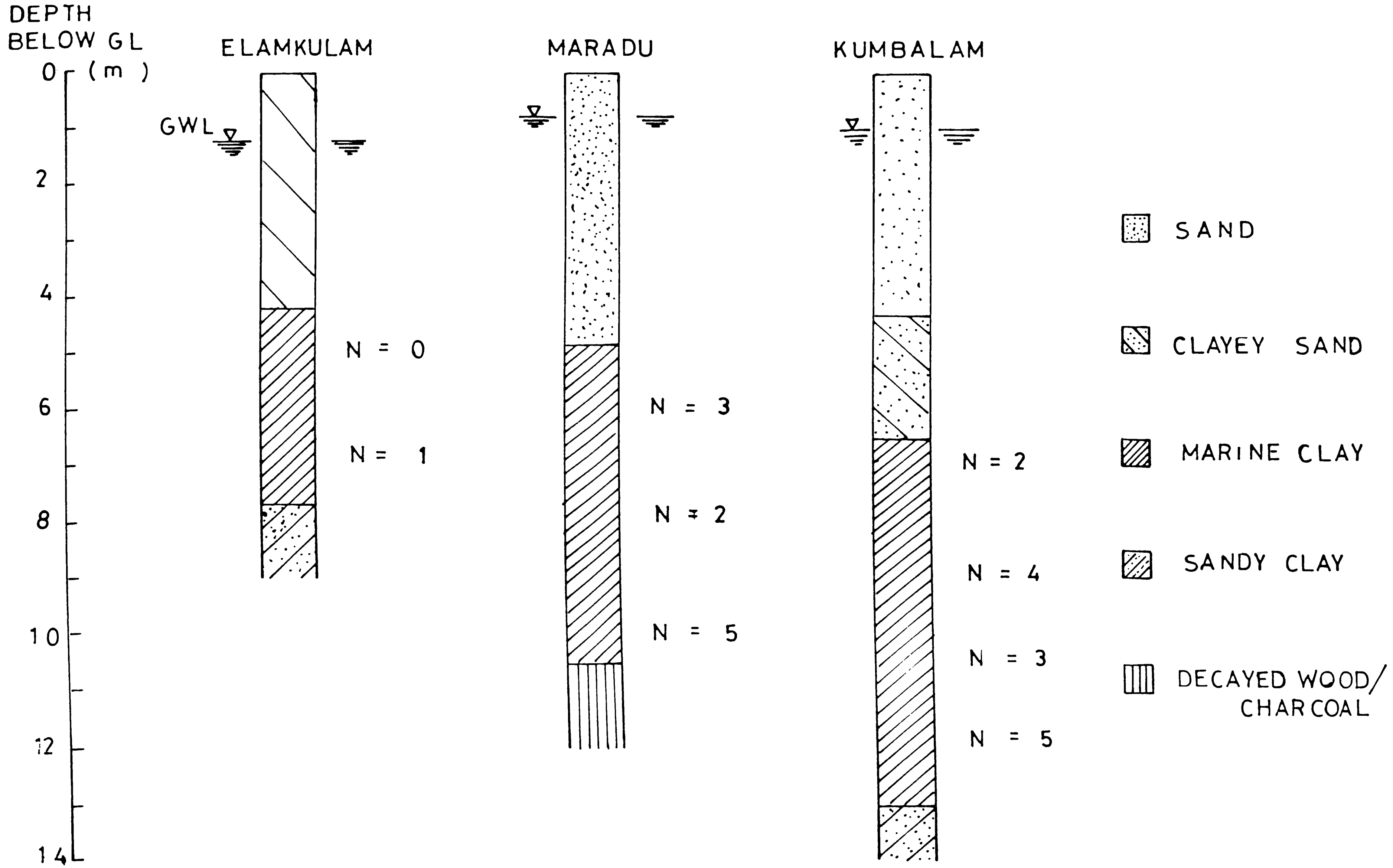


Fig. 3.1.2b : TYPICAL SOIL PROFILES.

marine clay between 5.0 and 10.7 metres. At Kumbalam the marine clay deposit started at 6.5 metres below ground level.

3.2 FIELD COLLECTION OF SAMPLES

Considerable quantities of samples were required for the numerous tests carried out on soil samples collected from seven sites as listed above. Undisturbed samples were collected for determination of engineering properties and representative samples were collected for index properties. The procedure adopted for collection of samples is described in detail below.

Since the top layers in almost all locations except Nettoor consisted of sandy soils, thick layers of marine clay deposits were available only at depths varying from 3 to 9 metres. Bore holes were taken to the clay layers for collection of samples. Shell and auger method was used for advancing bore holes. The bore holes had a diameter of 150 mm. Casing pipes 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 Surface Investigation for Foundations.

Representative samples were collected using an 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 two percent 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.

Since the marine clay samples were collected from soft clay formations, an area ratio of 7.5 percent was adopted for the cutting edge. This is sufficiently less than the maximum value of ten percent recommended for soft and sensitive clays.

The samples collected in sampling tubes were trimmed on both ends and immediately sealed with molten wax to sufficient thickness so that the natural water content, the field density and the soil structure are kept intact. They were kept in sample boxes and transferred to laboratory on the same day and preserved under humid conditions.

Bentonite slurry was never used during the boring operations as it might contaminate the soil samples.

In case of the site at Nettoor, thick deposits of marine clay were available at shallow depths. Therefore in this case alone, samples were collected by open excavation.

The method of sampling from open pits was more advantageous as it was cheaper and faster. More samples could be collected from open pits than from bore holes. But since the water table in Cochin is very high (less than a metre in Nettoor area) dewatering had to be arranged during sampling.

Since the top layers consisted of sandy clay deposits upto 1 m, top soil was removed upto this depth. The sample collection was done in three stages. At first, samples were collected from a depth of 1 to 1.5 m. A number of 100 mm diameter sampling tubes were pushed into this layer and samples collected and sealed immediately. Representative samples were collected in polythene bags. The debris was then removed upto 1.5 m and sampling was continued. The process was repeated for the third layer from 2 m.

For carrying out some tests, bore hole water was required so that the salinity in the pore water is not diminished by the addition of distilled water. The following procedure was adopted for collection of bore hole water. Tight-fitting casing pipes were introduced to the middle of the clay layer during boring. The loose material in the bore hole was removed by augers and the water was baled out completely using a shell. The mouth of the bore hole was covered and left for overnight. Pore water from the clay stratum which oozed out during the night was collected the next day and preserved in polythene bottles.

3.3 PREPARATION OF SAMPLES FOR LABORATORY TESTS

Based on the results of some preliminary tests for the index properties of Cochin marine clays, it was seen that drying influences these values significantly. For a comprehensive study of the geotechnical properties of Cochin marine clays, samples with four different initial conditions were required for the laboratory investigations. They are:

1. Moist samples
2. Air dried samples
3. Oven dried samples, and
4. Washed samples

When large quantities of soft sediments are required, the main problem faced is the lack of uniformity even among the samples collected from the same depth in different bore holes. Lack of uniformity in soil 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 is designated as the 'moist' sample of Cochin marine clay. In order to study the properties of these clays without the marine pore water and excess salts, representative samples were repeatedly washed in plastic buckets with distilled water taking extreme care not to lose any fines. Since the entire clay fraction in case of Cochin marine clays exist in the form of flocs, the supernatant water above the soil being washed in plastic buckets, was drained out by siphoning without any disturbance to the soil suspension. This was repeated a number of times. After a few washings it was observed at times that the colour of water turned light yellow and sedimentation took more time. Samples washed like this were allowed to lose their water content by exposure to air till the moisture content became slightly above the liquid limit of moist sample. These samples are designated as 'washed' samples and are preserved in polythene bags in humid atmosphere.

To prepare 'air dried' specimen, representative samples were spread in large trays and dried under room temperature (30±4°C) and humidity (60±10%) till equilibrium is reached which took about 15 days. The lumps formed during drying were broken by a wooden mallet. The sample, towards the end of drying were pulverised using a heavy hammer, and passed through 425 micron sieve without any loss of material.

'Oven dried' samsples were prepared by drying air dried samples in an oven at 105-110°C.

3.4 ATTERBERG LIMITS

IS 2720 (Part I) 1972, 'Preparation of Dry Soil Samples for Various Tests' lays down the procedure for preparation of samples for various tests. It also gives the approximate quantity of dry sample required for each test. As per the code, the specimen required for liquid limit is 270 gm of air dried sample passing through 425 micron sieve.

Since air drying brings in significant changes in the plasticity of marine clays, the liquid limit from air dried samples will show considerably low values. Hence air dried samples cannot be used for determination of liquid limit. IS 2720 (Part V) 1970 indicates that wet soil also can be used for the liquid limit tests. To prepare sample passing through 425 µm sieve, the code recommends rubbing the wet soil through a 425 µm IS sieve until a sufficient quantity is obtained. The code observes that the results from the two procedures for preparation of sample may differ. It accepts both values, perhaps on the presumption that the values may not differ significantly. But in the case of marine clays this difference is considerable. The liquid limit of marine clays reduces by 20-30% on air drying and 40-60% on oven drying. Hence only 'moist' soil can be used for the determination of liquid limit in case of marine soils.

To obtain the flow curve, the water content of the soil sample is varied usually by addition of distilled water. In the case of marine clays the pore water is saline and the salinity can vary with the addition of distilled water. The salinity can be kept unaltered if bore hole water is used for the tests. To examine the influence of this parameter on the liquid limit values, tests were conducted using both distilled water and bore hole water. The results are presented in Table 3.4.1.

Sample of marine clay from Parur in the north and Kumbalam in south of Greater Cochin situated at a distance of around 40 km, were tested and results presented. They show that the values of liquid limit are essentially the same whether bore water is used or not. Hence the tests can be conducted using distilled water without any discernible error.

In the case of plastic limit, IS 2720 (Part I) 1972 recommends 50 gm of air dried soil passing through

Table 3.4.1

EFFECT OF BORE HOLE WATER

Sl.No.	Location	Type of samples	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)	Shrinkage limit (%)	Free swell index (cc/g)
1.	Parur	a) Moist soil + distilled water	99.5	40.5	59.0	21.4	4.44
		b) Moist soil + bore hole water	102.0	41.5	60.5	22.1	4.85
2.	Kumbalam	a) Moist soil + distilled water	96.5	36.5	60.0	19.8	4.75
		b) Moist soil + bore hole water	96.5	39.0	57.5	19.0	4.40

425 μm sieve. For reasons mentioned above, 'moist' soil has to be used for the test for plastic limit. The results of tests carried out with bore hole water are also presented in Table 3.4.1.

Tests for Atterberg limits were conducted on air dried and oven dried samples also, dried as discussed earlier. The dried samples were mixed with water to a water content higher than liquid limit and preserved as such without loss of moisture content for 24 hours to arrive at equilibrium. The test for liquid limit was started 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.

3.4.1 Shrinkage limit

Aggregation of clay fraction into coarser particles during drying influences the shrinkage limit also. Since a coarser soil fabric tend to increase the shrinkage limit values, the shrinkage limit of marine clays increases on drying unlike the Atterberg limits. The code for preparation of dry soil samples prescribes 100 gm of air dried soil passing through 425 micron IS sieve for this test. The effect of bore hole water on the test results is negligible as shown in Table 3.4.1.

The above discussions focus the attention on the need for a specific mention in the Indian Standard Codes for determination of Atterberg limits and shrinkage limits making it mandatory to use 'moist' samples for these tests in case of marine clays.

Tests were also carried out on air dried and oven dried samples for shrinkage limit. The dried samples were mixed with distilled water to a water content higher than liquid limit and kept in a glass plate covered by a china dish for more than 24 hours to arrive at a uniform paste.

3.4.2 Soaked samples

Both Atterberg limit tests and shrinkage limit tests were conducted on soaked samples also. Samples of marine clay which were initially air dried or oven dried were kept in buckets filled with water to the top of the soil. They were kept in buckets for over two years to check whether the aggregation will show any tendency to disintegrate. If so, this will be reflected in the values of Atterberg limits. Hence tests were conducted on soaked air dried and soaked oven dried samples.

3.5 GRAIN SIZE ANALYSIS

Grain size distribution is the most obviously affected index property of Cochin marine clay 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. Hence grain size distribution tests on dried samples give anomalous results.

The code for preparation of dry soil samples recommends air dried samples for wet sieving and for dry sieving and hydrometer analysis specification warrants the use of soil sample oven dried for 24 hours at 105-110°C. For the grain size analysis of marine clays, only wet analysis on 'moist' sample alone gives the correct results.

During the preparation of washed samples and soaked samples, it was noticed that the supernatant water over the soil was always clear. With ordinary clayey soils, this portion was always filled with colloids which will never settle irrespective of the time allowed for sedimentation. But in the case of soil suspensions of Cochin marine clay, the supernatant water was always clear. This absence of colloids and very fine clay fraction indicates that in marine clays, the clay fraction existed in the form of flocs. Thus compared to the normally encountered clayey soils, the need of a deflocculating agent was more keenly felt for marine soils.

Since the role of dispersing agent was more conspicuous in the hydrometer analysis of marine clays, a series of tests were conducted to arrive at the most suitable dispersing agent and the optimum quantity of the same. The different dispersing agents tried were

1. Sodium chloride
2. Sodium phosphate
3. Sodium carbonate
4. Sodium nitrate
5. Sodium sulphate
6. Sodium bicarbonate
7. Sodium hexametaphosphate
8. Sodium hexametaphosphate + Sodium carbonate.

The results and observations on these hydrometer analysis are presented in section 4.4.1 in chapter IV. It was found that the standard dispersing agent, 100 ml of 33 gm of sodium hexametaphosphate + 7 gm of sodium carbonate 1000 ml of distilled water is the most ideal one for Cochin marine clays also.

3.6 CONSOLIDATION TESTS ON COCHIN MARINE CLAYS

Consolidation tests on undisturbed samples were conducted as per IS: 2720 (Part XV) 1965: Determination of Consolidation Properties. Undisturbed 'moist' sample was extracted from 100 mm diameter stainless steel sampling tubes. The first 3 to 4 cm length of sample is cut off and discarded as this might have been disturbed during sealing with molten wax or during preservation by loss of moisture content. 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 is then trimmed in level with the edges of the oedometer ring. The initial water content of soil sample is determined from these trimmings.

The oedometer rings are 60 mm in diameter and 20 mm in height. 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 0.0625 kg/cm^2. The sample was then inundated with water from a reservoir with a head of 50 cms.

Standard consolidation tests

Several series of consolidation tests were carried out on samples of Cochin marine clays varying different parameters like duration of loading, effect of pore fluid, effect of washing, etc. All these test results need to be compared with the results of a standard consolidation test, which had to be defined.

In the consolidation test which served as the reference test, the load increment ratio was kept at 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 of the standard test for each load increment was kept at two days.

IS: 2720 (Part XV) 1965 recommends a loading sequence of 0.1, 0.2, 0.4, 0.8, 1.6, 3.2 and 6.4 kg/cm^2 starting with a seating load of 0.05 kg/cm^2. Eventhough this pattern has a load increment ratio of one the loading stages are a little odd figures especially when the e - log p curve is plotted. Further, a loading of 6.4 kg/cm^2 (64T/m^2) is rarely realised as a pressure increment in a clay layer in field. Considering these two aspects of the problem, a loading sequence of 0.0625, 0.125, 0.25, 0.5, 1.0, 2.0 and 4.0 kg/cm^2 was adopted for the numerous consolidation tests performed on untreated and treated specimens of Cochin 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 rectangular hyperbola method was used to determine the coefficient of consolidation C_v throughout this work. These timings for recording the deflections were chosen to suit the rectangular hyperbola method. A detailed discussion on the method and its advantages over the existing two methods viz. $\sqrt{t}$ method and log t method are discussed later in this chapter.

Consolidation tests on remoulded samples

A number of consolidation tests were conducted on remoulded samples. Leonards and Ramaiah (1959), based on a series of consolidation tests in clays remoulded at water contents intermediate between the plastic and liquid limits, had observed that a reduction in remoulding water content produces an effect similar to preconsolidation pressure in e - log p relationship. Hence the initial moisture content in all the remoulded samples was kept equal to liquid limit. Sample for remoulding was taken from the same sampling tube close to the undisturbed specimen. After thoroughly working on the sample, it was hand remoulded into the oedometer ring, taking care to prevent any entrapment of air in the specimen. Consolidation test was then conducted starting with a seating load of 0.0625 kg/cm^2.

Consolidation tests on air dried and oven dried samples

Obviously consolidation tests could be carried out only on remoulded specimens of air dried and oven dried samples of Cochin marine clays. The initial moisture content was kept close to the respective liquid limit values of air dried and oven dried marine clay samples which were considerably lower than the liquid limit values of 'moist' samples.

One of the parameters considered while running long series of the tests on untreated samples was the effect of duration of loading. While the duration given for each load increment for the standard consolidation test was two days, consolidation tests were carried out giving 1 day, 2 days, 4 days, 8 days, 15 days and 30 days for each load increment to study its effect especially on the development of higher cohesive strength during secondary compression at each load increment.

3.6.1 Rectangular hyperbola method for determination of C_v

Since settlement is one of the design criteria for structures founded on cohesive soils, the consolidation test is essential to ensure satisfactory performance of the structure. One of the important aspects of laboratory consolidation tests is the determination of coefficient of consolidation C_v, for predicting the rate of settlement eventhough its applicability to field conditions is questionable.

Of the various methods for the determination of C_v, Taylor's square root time fitting method and Casagrande's logarithm of time fitting method are the most widely used methods. All the methods for determination of C_v are developments of a comparative study of time t and deformation δ with the theoretical relationship between time factor, T and degree of consolidation U. A new method called rectangular hyperbola method developed by Sridharan et al.(1987) which was found simple and versatile has been used in this work for the determination of C_v. The data available from the numerous consolidation tests was taken advantage of for further study of this method vis-a-vis Cochin marine clays.

The rectangular hyperbola method assumes that the Terzaghi's U-T relationship can be taken as a rectangular hyperbola over a wide range of T. Hence a plot of $\frac{T}{U}$ vs. T gives a curve upto U = 60% and a straight line for $60\% \leq U < 90\%$. The linear portion of the curve is given by

$$T/U = 8.208 \times 10^{-3}T + 2.44 \times 10^{-3} \qquad (3.6.1.1)$$

and it has a correlation coefficient of 0.9999.

Fig.3.6.1.1 shows a T/U vs. T plot for the theoretical curve. It shows that the ratios of the slopes M_1 and M_2 of the lines joining the origin to the points corresponding

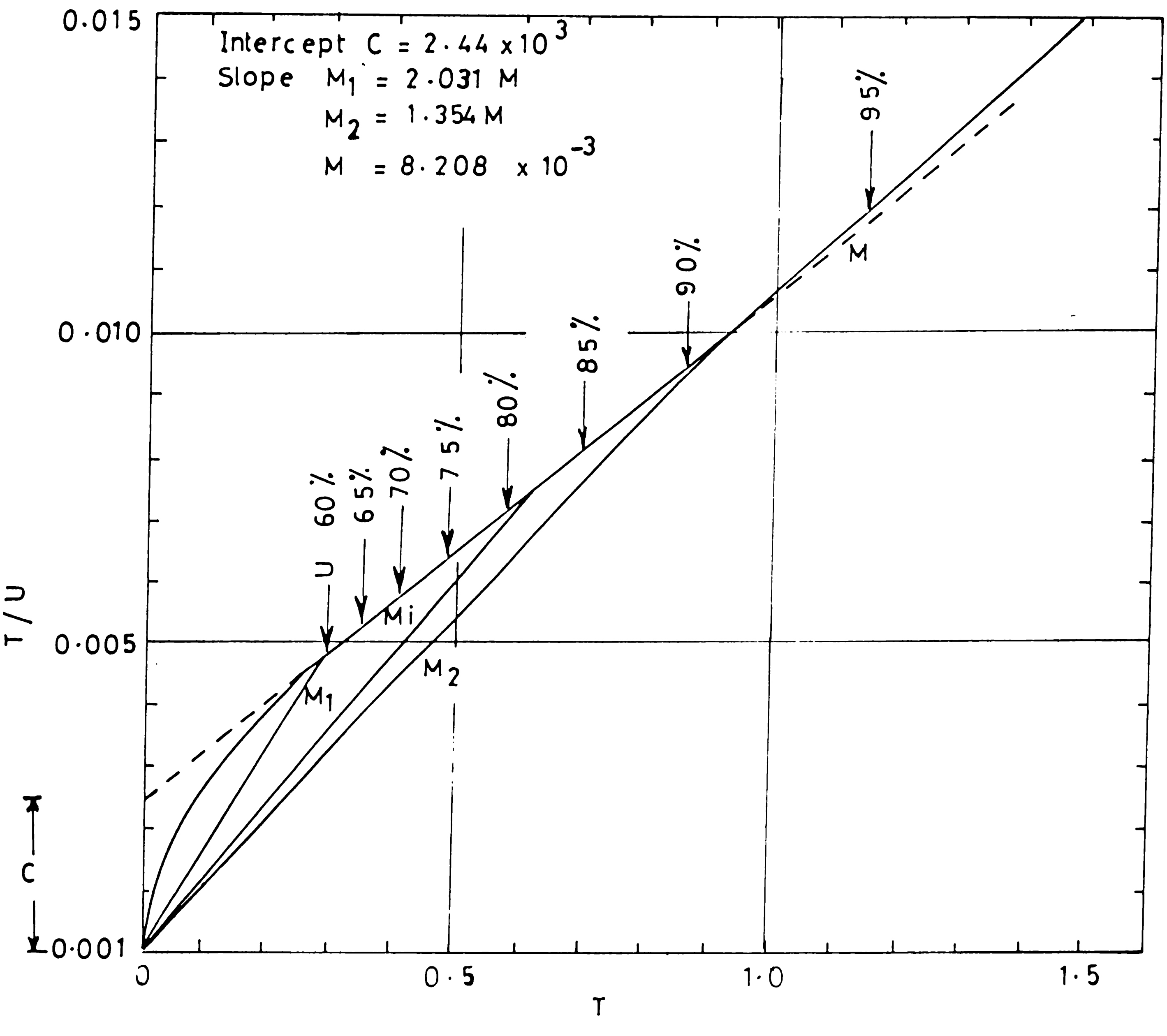


Fig. 3.6.1.1 THEORETICAL T/U-T RELATIONSHIP FROM TERZAGHI'S CONSOLIDATION (Rectangular/linear pore water pressure distribution)

to 60% and 90% consolidation respectively to the slope M of the linear portion of the plot are 2.031 and 1.354 respectively. This property can be used to locate t_{60} and t_{90} points on an experimental curve. Similarly points corresponding to other degrees of consolidation can be found from the ratio M_i/M which is termed the 'A' value. From the figure 3.6.1.1

$$T/U = M_i . T \qquad (3.6.1.2)$$

$$1/U = M_i$$

$$A = M_i/M$$

$$A = \frac{1}{MU} \qquad (3.6.1.3)$$

For an experimental data plotted as $t/(\delta - \delta_i)$ vs. t,

$$\frac{t}{(\delta - \delta_i)} = m't + c' \qquad (3.6.1.4)$$

where δ_i is the initial compression, m' is the slope of the linear portion of the curve and c' is the intercept on the $t/(\delta - \delta_i)$ axis.

The equation of the line joining the origin to any point on the linear portion of $t/(\delta - \delta_i)$ vs. t plot is given by

$$\frac{t}{(\delta - \delta_i)} = Am't \qquad (3.6.1.5)$$

where A is a dimensionless quantity and a function of degree of consolidation U.

$$t = \frac{c'}{(A-1)m'} \qquad (3.6.1.6)$$

This gives the time for any specific degree of consolidation.

$$C_v = \frac{TH^2}{t}$$

$$= T(A-1)\frac{M'H^2}{c'} \qquad (3.6.1.7)$$

ie. $$C_v = Bm'H^2/c' \qquad (3.6.1.8)$$

where $B = T(A-1) = C/M = 0.297$. Thus C_v is computed.

The rectangular hyperbola method for C_v can claim many advantages over the other methods. Some of the instances where $\sqrt{t}$ method fails are:

1. When the $\sqrt{t}$ curve does not have an initial straight line but has a continuous curvature preventing us from drawing the line leading to the t_{90} point.

2. When the compression is low initially and rapid after a certain stage.

3. When there is very rapid compression initially and almost a flat curve thereafter.

Similarly Casagrande's logarithm of time method also becomes ineffective in certain cases such as

1. When secondary compression is very large and does not yield a straight line failing to give the point t_{100}

2. In partially saturated soils, the initial portion may not be parabolic and it becomes difficult to determine t_o

3. A longer duration of consolidation test may be required so that the secondary compression portion is well defined. This slows down the tests.

The rectangular hyperbola method will yield results in any of the situation listed above. This is illustrated with some of the test results on Cochin marine clay.

Fig.3.6.1.2 shows the δ vs. log t plot for the test data on an undisturbed sample from Maradu at 9 m depth for the pressure increment from 0.5 to 1.0 kg/cm^2. Obviously it is difficult to get the point of 100% consolidation from the curve. The same data has been plotted in Fig.3.6.1.3 by Taylor's $\sqrt{t}$ method. It can be seen from the curves that it is difficult to draw a straight line through a set of points at the initial portion as very few points fall in a straight line. But by rectangular hyperbola method the value of C_v can be easily arrived at without any ambiguity as shown in Fig.3.6.1.4. The value of intercept c is 0.068 and slope m is 0.00188. These give a value of C_v as 1.25×10^{-4} cm^2/sec.

When the method was first proposed by Sridharan and Sripada Rao (1981a), it did not consider values of the initial compression. Later Sridharan et al. (1987) perfected the technique to include initial compression also. If c' and m' are the vertical intercept and slope of the straight line $t/(\delta-\delta_i)$ vs. t taking into consideration the initial compression also, it can be shown from a wide range of experimental results that

$$m'/c' = 0.809\ m/c \qquad (3.6.1.9)$$

with a correlation coefficient of 0.9944.

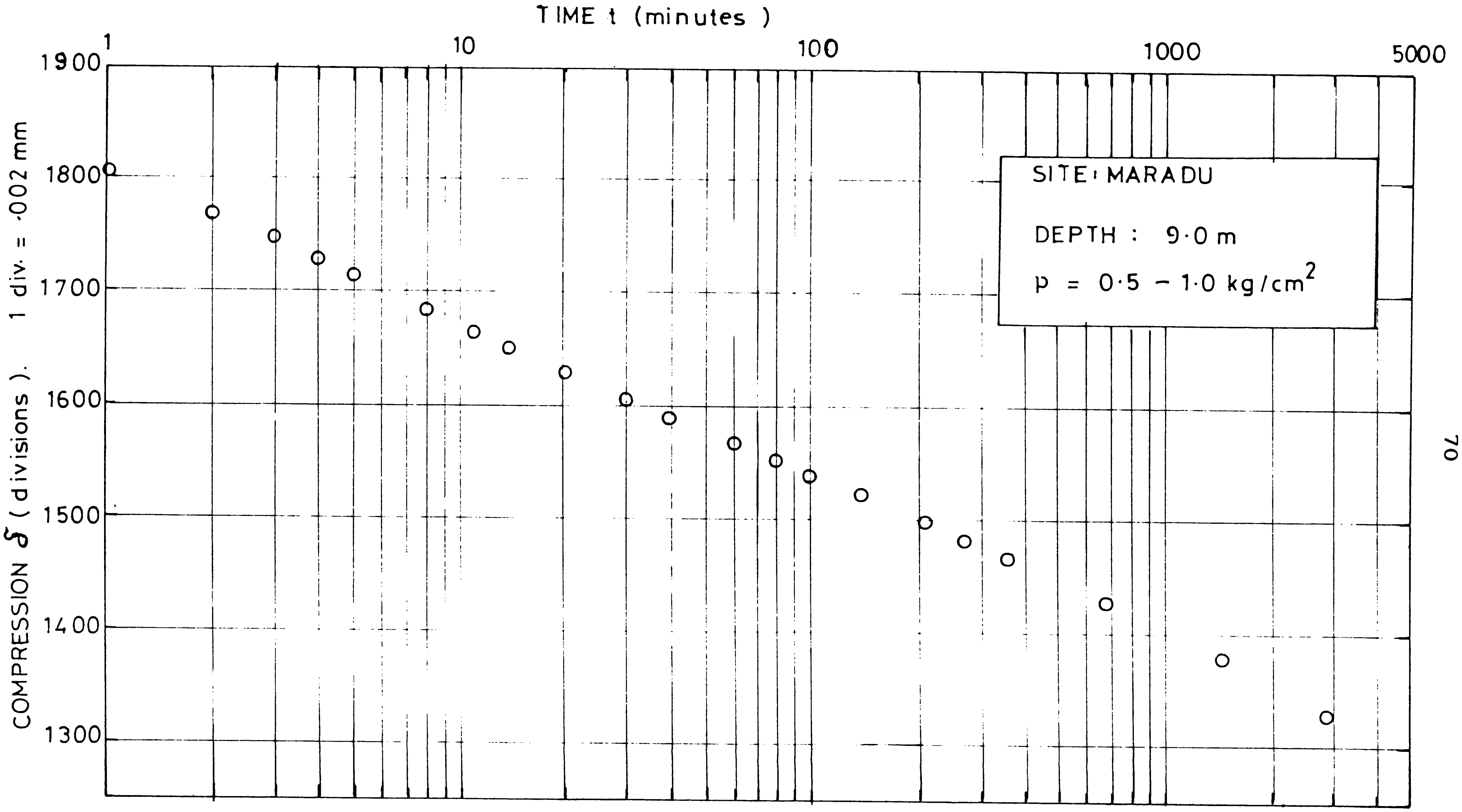


Fig. 3.6.1.2 : δ-log t CURVE

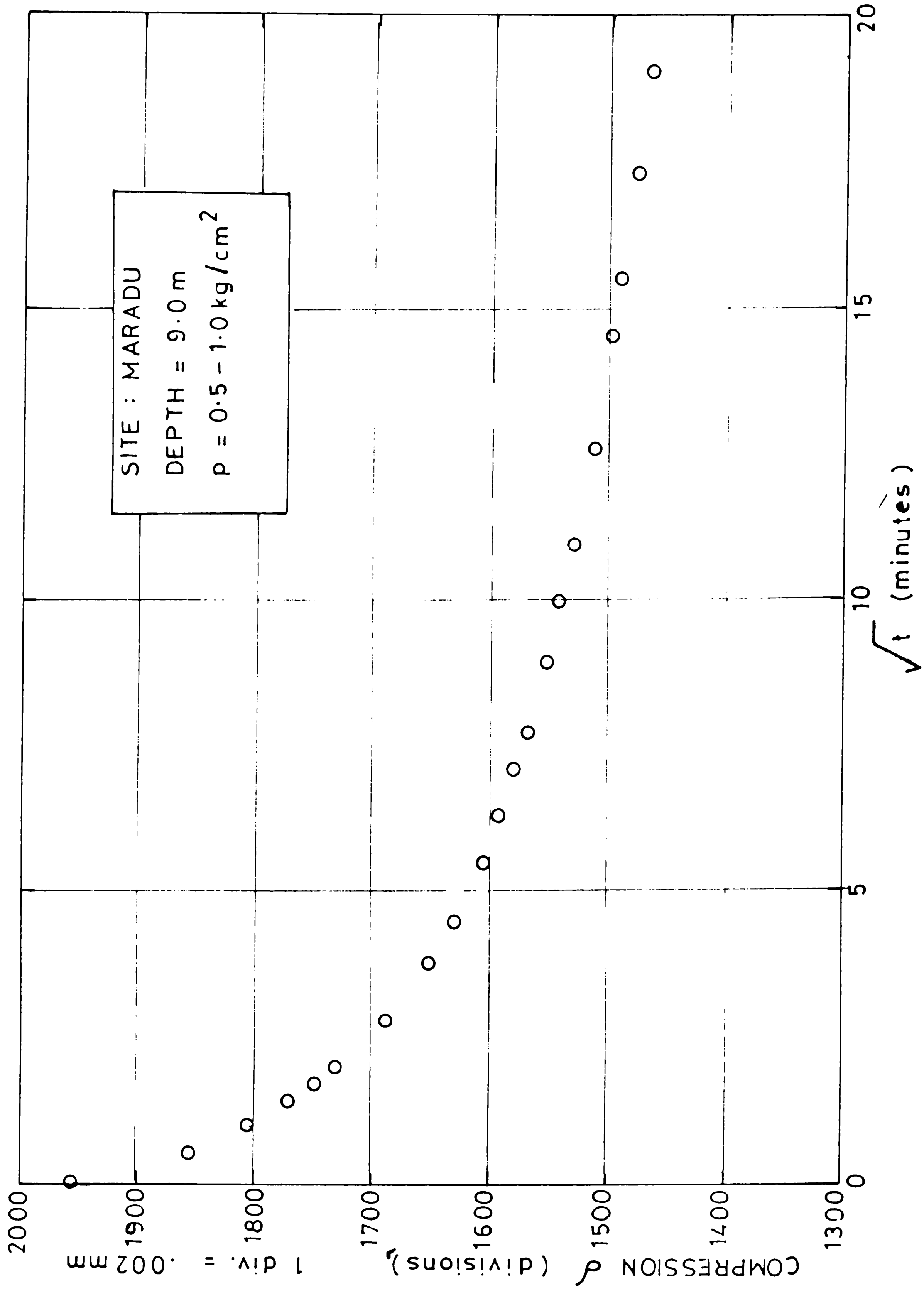


Fig. 3·6·1·3 : δ – √t CURVE.

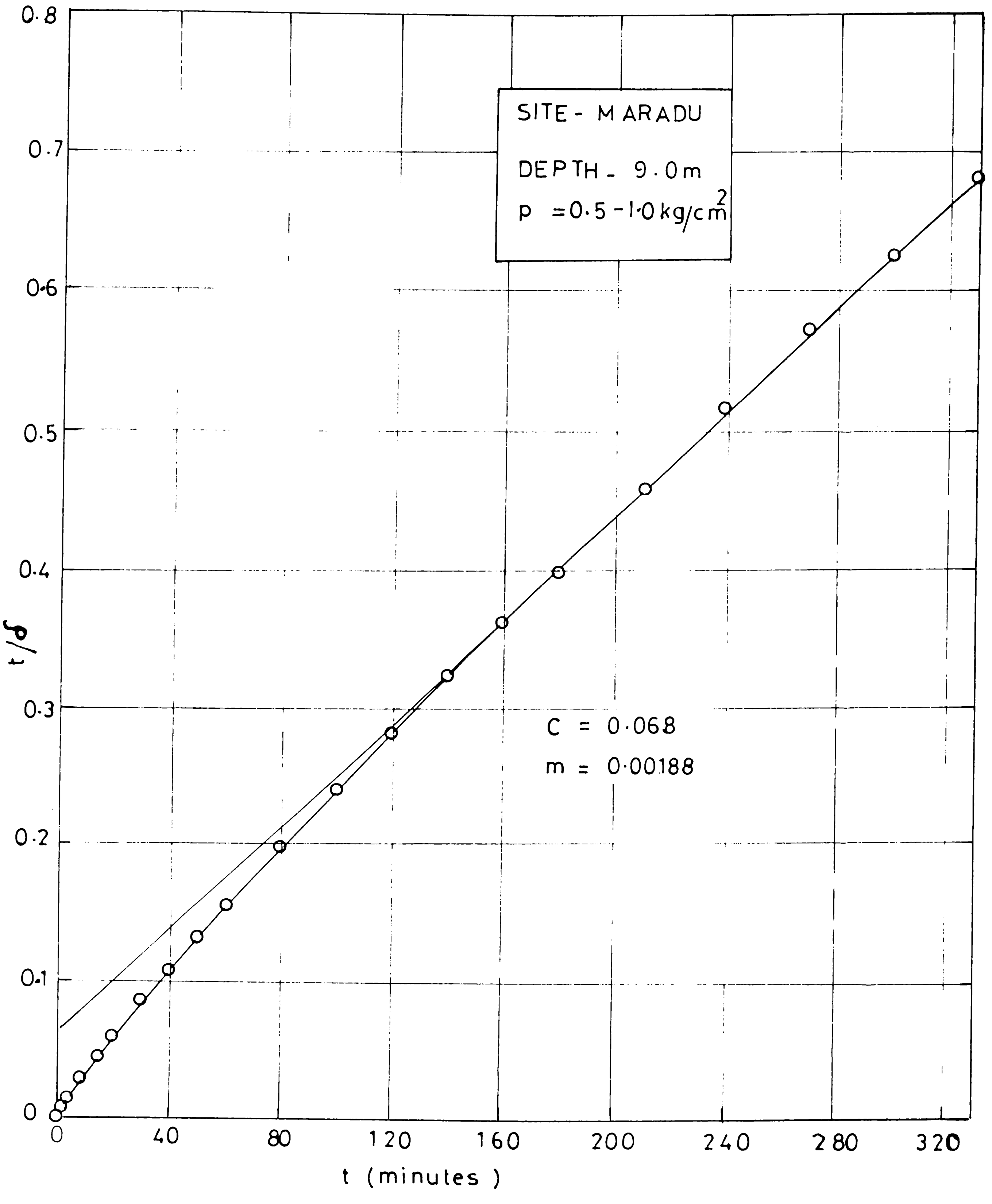


Fig. 3.6.1.4 t/δ - t CURVE.

The equation 3.6.1.6 gets modified to

$$t = \frac{c}{K'(A-1)m} \qquad (3.6.1.10)$$

where K' = 0.809.

The value of C_v can now be obtained from

$$C_{vR} = B'mH^2/c \qquad (3.6.1.11)$$

where B' = BK' = 0.241.

For a comparative study of the results given by the three methods the consolidation test data has been analysed by all the three methods for undisturbed and remoulded samples. The results are presented in Table 3.6.1.1.

Due to the advantages the present method has, in comparison to the Taylor's and Casagrande's methods, the rectangular hyperbola has been used throughout this study. However, for quite a number of cases the δ - log t method also has been included especially for the determination of secondary consolidation.

3.6.2 Preconsolidation pressure

The importance of an accurate determination of preconsolidation pressure of clay layers can hardly be

Table 3.6.1.1

COMPARISON OF C_V VALUES BY THE THREE METHODS

Site and Depth of sampling	Pressure p (kg/cm^2)	Undisturbed			Remoulded		
		$\sqrt{t}$ method	log t method	R.H.method	$\sqrt{t}$ method	log t method	R.H.method
Nettoor - 1.00 m	0.25-0.50	1.79	1.45	1.37	0.76	0.79	1.01
	0.50-1.0	1.43	1.29	1.14	0.84	0.70	0.87
	1.0-2.0	1.10	1.15	0.93	0.77	0.67	0.77
	2.0-4.0	1.47	1.11	0.98	0.76	0.62	0.64
Parur - 13.5 m	0.25-0.50	1.05	0.73	0.96	4.16	3.48	4.94
	0.50-1.0	1.38	0.94	0.92	1.51	1.06	3.00
	1.0-2.0	1.43	1.05	1.21	1.67	1.51	1.24
	2.0-4.0	1.26	1.23	1.32	1.89	0.43	1.71
Parur - 6.5 m	0.25-0.50	1.38	1.21	1.28	1.22	1.04	1.19
	0.50-1.0	1.46	1.37	1.34	1.45	1.22	1.36
	1.0-2.0	1.78	1.40	1.48	1.71	1.30	1.74
	2.0-4.0	2.24	1.69	1.65	1.93	1.50	1.32
Maradu - 9 m	0.25-0.50	--	--	--	0.96	0.62	0.76
	0.50-1.0	1.68	--	1.25	0.92	0.73	0.84
	1.0-2.0	2.32	1.47	1.12	1.03	0.84	0.98
	2.0-4.0	2.20	1.55	1.05	1.10	0.88	0.97

overemphasized as it helps to classify them into under consolidated, normally consolidated or over consolidated clays. The engineering behaviour of clays is closely linked to the state of consolidation. As an offshoot of the several series of consolidation tests carried out during this investigation, a new method called log-log method, has been developed for the determination of preconsolidation pressure.

The main problem with attempts to develop methods for the determination of p_c from undisturbed samples was the non-availability of samples whose preconsolidation pressures are known beforehand. Eventhough there are different procedures proposed by various workers, their reliability could not be verified for want of a prior knowledge of p_c. In this context, the attempt to prepare laboratory samples for specific values of preconsolidation pressure has been a success.

In log-log method, p_c is obtained as the point of intersection of two straight lines extended from the linear portions on either end of the compression curve plotted to log scale on both axes. In order to check the validity of the proposed method, consolidation tests were conducted on samples which were consolidated to known pressures earlier. To prepare such specimens, remoulded samples of marine clay were loaded in consolidation cells with the initial water content equal to the liquid limit values, with a seating load of 0.0625 kg/cm^2. The pressure was increased in steps to 1 kg/cm^2 and the load was retained for long durations such that the samples were fully consolidated at their known pressures. The load was then released in steps to 0.0625 kg/cm^2 with the sample fully inundated so that the sample was always fully saturated. A full consolidation test with a maximum pressure of 4 kg/cm^2 was then conducted on this sample preconsolidated to 1 kg/cm^2. The consolidation test data was plotted to get the e - log p curve and p_c was determined by the popular Casagrande method. The data was also plotted with both e and p on log scales; p_c was found out by the intersection of two straight lines fitted to the initial and final points on the log e - log p plot. Since the actual preconsolidation pressure was known, the results obtained from both methods could be verified. In all the cases it was found that log e - log p plots gave better values. Even the development of quasi preconsolidation pressure could be detected better by the log-log method.

Since the method was successful, it was verified with marine clays from different sites and other types of soils also. The soils tried are:

1. Marine clay from Nettoor
2. Marine clay from Parur
3. Red earth
4. Black cotton soil
5. Kaolinite
6. Marine clay treated with 3% lime
7. Undisturbed samples from Maradu, Kumbalam and Elamkulam.

Tests with these soils having wide and varied properties helped to confirm the versatality and accuracy of the new log-log method. This is further discussed in section 5.6.2.

Constant rate of strain tests

Constant rate of strain tests or CRS tests have the advantage of completing the consolidation test at a much shorter duration. The success of this test lies in the selection of the rate of strain which will allow sufficient time for dissipation of pore pressure but at the same time will help to complete the test at a shorter duration.

About ten CRS tests were conducted on samples from Parur with three strain rates. The strain rates adopted were 0.00192 mm/minute, 0.001414 mm/minute and 0.00065 mm/minute. This has been further discussed in section 5.7.

3.6.3 Compaction characteristics

The changes that are caused by air drying and oven drying of marine clays improve their compaction characteristics also. Since these changes are accompanied by considerable development in shear strength, detailed investigations were carried out on the compaction characteristics of dried samples of Cochin marine clay. Since the use of oven dried samples in field application did not have any scope, the studies were restricted to air dried samples only.

The water content dry density relationship using light compaction was obtained as per IS: 2720 (Part VII) 1980. Air dried sample was compacted in the mould in three layers, each layer being compacted with 25 blows from a 2.6 kg rammer dropped from a height of 310 mm, distributed uniformly over the surface of the soil. The water content-dry density plot from the test gave the optimum moisture content and the maximum dry density that can be achieved in field.

Since the shear strength of these air dried compacted marine clays showed considerable gain apart from improvement in compressibility characteristics, triaxial shear tests were carried out on samples prepared at maximum dry density and optimum moisture content. These results can be useful in the analysis of rotational base failure and stability of slopes.

The water content-dry density relationship was also obtained as per IS: 2720 (Part VIII) 1983 for heavy compaction. The soil was compacted in five layers, each layer being compacted by 25 blows from a 4.9 kg rammer dropped from 450 mm height.

The marine clay samples were first air dried and then water was added to vary the water content to obtain the compaction curve. The water content can be brought down by exposing the specimen to atmosphere starting from natural water content. Compaction tests were done on such samples also.

3.7 DETERMINATION OF FREE SWELL INDEX

Free swell, as defined by IS: 2720 (Part XL) 1977 is the increase in the volume of a soil, without any external constraints, on submergence in water. The test procedure consists of weighing two 10 gm specimen of oven dry soil passing through 425 micron IS sieve. Each soil specimen is poured into each of two graduated glass cylinders of 100 ml capacity. One cylinder is then filled with kerosene oil and the other with distilled water upto 100 ml mark. After the removal of the entrapped air by stirring with glass rod or shaking, the soils in both the cylinders shall be allowed to settle for a minimum of 24 hours, to attain the equilibrium state of volume without any further change in the volume of the soils. The final volume of soil in each cylinder is read. Kerosene, being a nonpolar liquid, does not cause swelling of the soil. The free swell index is then calculated by

$$\text{Free swell index} = \frac{V_d - V_k}{V_k} \times 100 \qquad (3.7.1)$$

V_d = volume of soil specimen read from the jar with distilled water

V_k = the volume of soil specimen read from the jar containing kerosene.

The present method or definition has some inherent drawbacks even in case of ordinary clayey soils as pointed out by Sridharan et al. (1985). After a detailed study of nine natural soils, they reported that this definition can give negative swelling index in certain cases. The details of the samples and their sediment volume in kerosene and water are given in Table 3.7.1. With the exception of sample collected from Gas Turbine Research Establishment (G.T.R.E.) Campus which occupied a high sediment volume in kerosene than in water, all other soils gave a positive free swell value. X-ray diffraction spectra indicated that GTRE soil contained only kaolinite. The free swell index test conducted with pure kaolinite mineral gave a higher sediment volume in kerosene than in water similar to the observation in GTRE Campus soil. Thus it was proved that pure kaolinite mineral occupies a higher sediment volume in a nonpolar solvent like kerosene than in water.

The above results point out the assumption that the level of a soil in kerosene graduated cylinder may be read as its original volume used in calculating the percent free swell index does not hold good for soils containing kaolinite alone. Further, the presence of kaolinite in soils results in their occupying a markedly higher sediment volume in kerosene in comparison to other soils without kaolinite, suppressing their percent free swell values. Hence a redefinition of free swell index is warranted.

Table 3.7.1

SEDIMENT VOLUMES OF CLAYS AND SOILS IN KEROSENE AND WATER AND THEIR FREE SWELL VALUES

(After Sridharan et al. 1985)

Sl. No.	Soil	Principal clay mineral	Liquid limit (%)	Sediment volume in kerosene (cm^3)	Sediment volume in water (cm^3)	Free swell index as per IS code (%)	Free swell index as per new definition (cc/gm)
1.	Kaolinite	Kaolinite	47	28.0	14.5	48.21	1.45
2.	G.T.R.E. Campus	Kaolinite	53	19	14.0	26.32	1.40
3.	Mangalore marine	Kaolinite and montmorillionite	72.6	17.0	19.75	16.17	1.975
4.	I.I.Sc.Campus	-do-	75.0	14.0	17.75	26.78	1.775
5.	Domalur	-do-	75.0	14.5	19.0	31.03	1.90
6.	Narayanpur	Montmorillonite	100.0	10.5	21.0	100.0	2.10
7.	Benihalla-I	-do-	84.0	11.0	21.0	90.9	2.10
8.	Benihalla-II	-do-	106.4	10.5	33.0	214.28	3.30
9.	Renigunta	-do-	124.2	11.0	80.0	627.27	8.00

During the discussions on preparation of moist, air dried and oven dried soils, it has already been pointed out that the clay content or the activity of a marine clay sample is considerably reduced by drying, due to aggregation of finer particles to coarser grains. Swelling being a result of a surface activity, a reduction in clay fraction or total surface area can considerably bring down the swelling potential of a clayey soil. Since drying is inevitably accompanied by aggregation in marine clays, use of dry soil specimen as stipulated by I.S. Codes will lead to erroneous results. Table 3.7.2 gives the free swell index values of marine clay samples from different sites in Greater Cochin area. The need for a new method was thus keenly felt in case of marine clay samples as in the case of kaolinite clays. In this context the method proposed by Sridharan et al. (1985) as given below is more acceptable.

$$\text{Free Swell Index} = \frac{V_d}{10} \text{ cc/gm} \qquad (3.7.2)$$

where V_d = the volume of 10 gm of soil specimen read from the 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.

Table 3.7.2

SEDIMENT VOLUMES OF DRIED SAMPLES OF COCHIN MARINE CLAY OBTAINED AS PER I.S. SPECIFICATIONS

Sl. No.	Location of	Depth (m)	Air dried		Oven dried	
			Volume in distilled water (cc)	Volume in kerosene (cc)	Volume in distilled water (cc)	Volume in kerosene (cc)
1.	Nettoor	1.0	17	10	12	10
		1.5	14	11	15	12
		2.0	16.5	11	14	11.5
2.	Munambam	10.40	--	--	14.50	14
		11.40	--	--	13	10
		12.10	--	--	14	14
		14.40	20	13.5	14	10
3.	Cheranelloor	4.5	21.5	12	16	13
4.	Parur	11.50	20	11	15	13
		11.50 (washed)	21	14	17	14
		7.50	19	12.5	17	14.5
		7.50 (<75 um)	24	16	20	15.5
5.	Maradu	9.0	19	11	18	16
6.	Kumbalam	10.0	20.75	15	16.5	13

In the case of kaolinite soils, 10 gm of dry soil can be weighted and then transferred to the graduated cylinder. But since drying alters the swelling properties of marine clays considerably, only a known quantity of 'moist' soil can be transferred to the jar. The dry weight of the soil specimen can later be determined by finding out the natural moisture content of the 'moist' sample used for the test.

Thus due to the peculiar behaviour of marine clays upon drying, the Indian standard procedure cannot be adopted for the determination of free swell index of Cochin marine clays. The definition proposed by Sridharan et al. (1985) only can be adopted. This has the added advantage that measurement in kerosene is eliminated. All the values of free swell index reported here are based on the definition suggested by Sridharan et al.

3.8 OTHER TESTS

The methods adopted for determination of organic matter, calcium carbonate, cation exchange capacity and pH value are given below.

3.8.1 Determination of organic matter

As per the procedure laid down by IS: 2720 (Part XXII) 1972 for determination of organic matter, the specimen required is air dried soil sample. The actual weight used is determined by an equivalent weight of soil oven dried and weighed. Out of the two procedures recommended by the Code, the procedure where potassium dichromate was used to oxidise the organic matter was used in the present study. The percentage of organic content was determined by the following equations.

$$V = 10.5(1-y/x) \tag{3.8.1.1}$$

where

V = volume (ml) of potassium dichromate

y = total volume of ferrous sulphate used in the test

x = total volume of ferrous sulphate used in the standardisation test.

Percentage of organic matter is given by

$$\text{OM (\% by weight)} = \frac{0.67 w_2 V}{w_1 w_2} \tag{3.8.1.2}$$

where w_2 = weight on oven dry basis of the soil samples passing 10 mm IS sieve

w_1 = weight on oven dry basis of total soil sample taken for the test before sieving.

w_3 = weight on oven dry basis of soil specimen used in the test.

Tests were carried out with both air dried soil and moist soil. The difference in values obtained was negligible.

3.8.2 Determination of calcium carbonate

The procedure for determination of calcium carbonate is given by IS: 2720 (Part XXIII) 1976. The carbonate content is determined by treating the soil with hydrochloric acid. The carbonate present in soil, percent by mass, is given by

$$\begin{array}{l}\text{Volume of 1N hydrochloric acid}\\ \text{used for 5 gm of soil}\end{array} \times 0.05 \times \frac{100}{5} \qquad (3.8.2.1)$$

Here also the specimen recommended by code is oven dried soil sample. But moist sample whose equivalent dry weight was accurately measured was used for the test.

3.8.3 Determination of cation exchange capacity

IS: 2720 (Part XXIV) 1976 give two procedures to determine the cation exchange capacity of soil samples. The method in clause 5 for the determination of cation exchange capacity treating metallic and hydrogen ions together has been used for the determination of the value listed in Table 4.1. Here also moist sample was used for the tests. But tests with air dried and oven dried samples indicated that the change in CEC values are negligible.

3.8.4 Determination of pH value

The pH value of the soil was determined using a Systronics digital pH meter. The equipment was standardised using buffer solution of pH equal to 9.2 and the pH value was measured as per IS: 2720 (Part XXVI) 1973.

3.9 IDENTIFICATION OF CLAY MINERAL BY X-RAY DIFFRACTION

Concepts about clay structure took a new turn in 1923 when Rinne in Sweden and Hadding in Germany first succeeded in obtaining X-ray diffraction patterns of clay minerals (Sudhakar Rao, 1982). This work was rapidly followed by other studies which established that a great majority of clays have well organised crystal structures.

The Bragg condition for production of a significant diffracted beam is

$$n \lambda = 2d \sin \theta$$

where n is the order of reflection, λ is the wavelength of the X-rays employed and θ is the angle the incident beam makes with the set of planes that have the interplanar spacing d.

The first order basal reflections of the clay minerals are frequently their most important reflections. From the geometry of the diffraction pattern of a single crystal, information on the symmetry of atomic arrangements as well as on size and shape of unit cell, can be gained. From the intensities of the diffracted beams, the positions of atoms averaged over many unit cells can be calculated.

The main structural difference between the different clay mineral species is the repeat distance perpendicular to the alumino silicate layers, often loosely referred to as the 'layer thickness'. Different clay mineral groups have different layer thicknesses. This forms the basis for identification of clay minerals in soil. However, the diffraction characteristics of clay minerals have considerable similarities so that identification based on diffraction data must be complimented by other studies.

3.9.1 Preparation of specimen for X-ray diffraction

For identification of clay minerals present in Cochin marine clays, the clay size fraction was given standard chemical treatment and analysed by X-ray diffractions as detailed below (Jackson, 1964).

Soil specimen from the Parur site was first washed with distilled water so as to remove the free salts. The washing was persisted till the electrical conductance of the latter was close to that of distilled water. Next the washed soil specimen was dispersed by agitation in an aqueous medium containing 0.4% of sodium hexametaphosphate (by weight) as the dispersing agent (ASTM) 1972. The clay fraction was separated by sedimentation of coarser particles under gravity for 3.5 hrs at 25°C (Jackson, 1964). The supernatant containing the dispersed clay fraction was pippetted out and suspended in 50 ml solution of 1N sodoum acetate - acetic acid buffer of pH 5 boiled gently for 5 minutes. Then 10 ml of 1N magnesium chloride was added. The suspension was thoroughly mixed and centrifuged and the supernatant liquid discarded. The clay fraction was washed once with

1N magnesium acetate and twice with 1N magnesium chloride to remove acetates which are difficult to wash out with alcohol. The clay was next washed, once with a small volume (10 ml) of water and then once in 95% alcohol. To solvate the clay sample, 10% glycerol (by volume in water) was added and mixed thoroughly to obtain a smooth suspension. The latter was spread on a glass slide, allowed to air dry and its X-ray diffraction spectra recorded with Philips X-ray diffractometer using C_u-K_α radiation (λ = 1.54Å) and Ni filter. The untreated silt and sand separates were also analysed by X-ray diffraction.

3.9.2 X-ray diffraction analysis of Cochin marine clays

X-ray diffraction pattern (Fig.3.9.2) of the treated Parur clay fraction shows strong reflection around 17.65Å which is characteristic of montmorillonite, relatively weaker reflection was observed around 7.25Å typical of kaolinite clay (Jackson, 1964). Additional reflections noted around 3.58Å and 3.35Å can be attributed to kaolinite and montmorillonite respectively.

It may be of interest to note that the method of sample preparation has a profound influence on the resultant X-ray diffractogram. For example, recording the X-ray

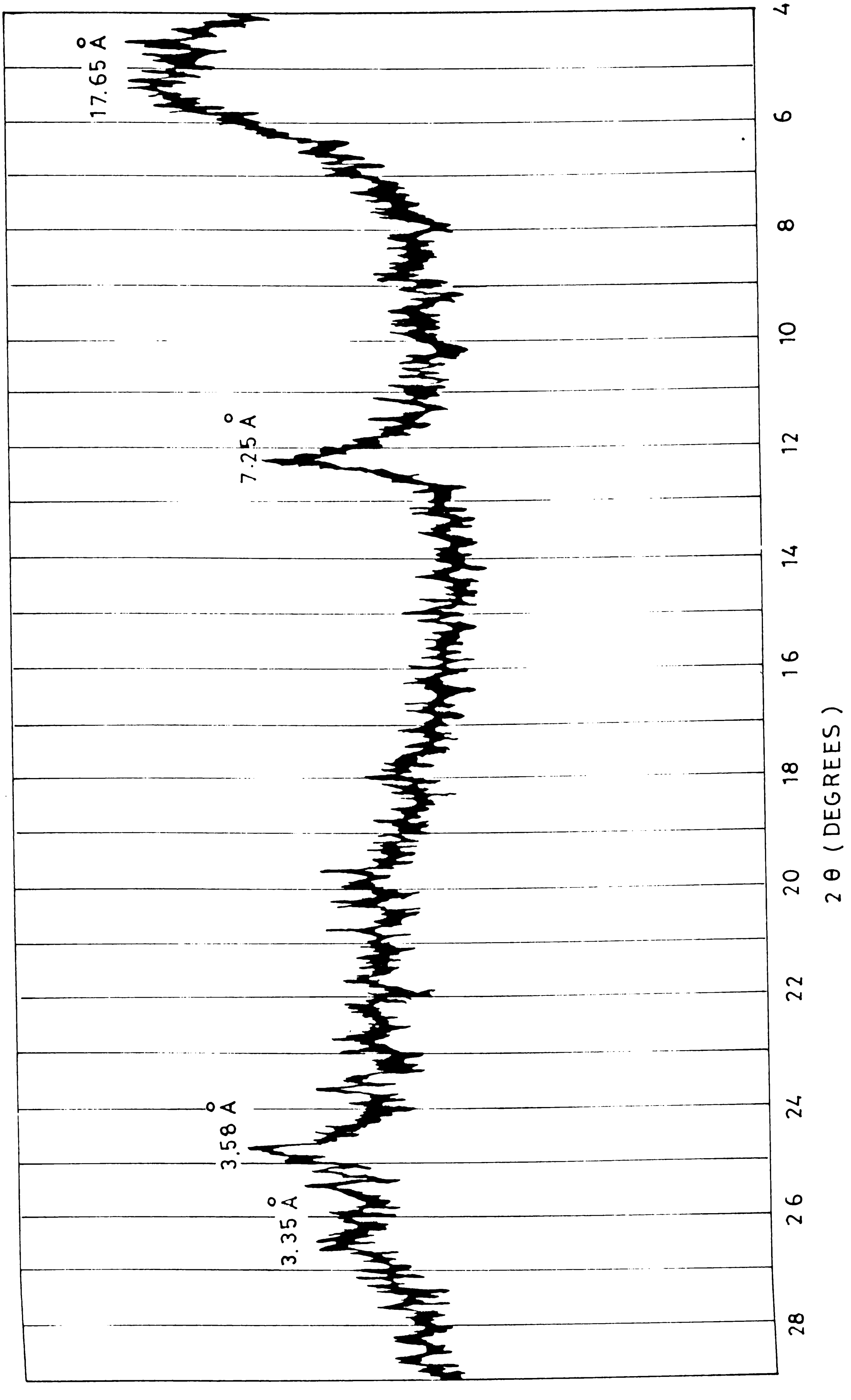


Fig 3.9.2.1 X-RAY DIFFRACTION PATTERN.

diffraction pattern of the oven dried and pulverised marine soil specimen without prior fractionisation and chemical treatment led to diffused nature of the X-ray spectrum. In another trial the marine soil specimens was sieved through a B.S. standard 200 mesh sieve eliminating the sand fraction and the residual silt and clay fraction was subjected to magnesium saturation and glycerol solvation as described previously. The chemically treated silt and clay fraction deposited on a watch glass was oven dried and pulverised. On recording the X-ray diffraction pattern of the above pulverised fraction, major peaks around 17Å and 7.2Å corresponding to montmorillonite and kaolinite respectively were missing. However the secondary peaks corresponding to montmorillonite and kaolinite were resolved.

From the various trials, it is concluded that fractionisation chemical treatment, followed by deposition and air drying of the treated specimen on a glass slide, is imperative to obtain reliable analysis of the clay minerals present in the marine clays. The diffused nature of the X-ray pattern in trial I and the absence of montmorillonite and kaolinite peaks in trial II may be attributed to the presence of the coarser particles in the specimen which would dilute the overall clay percentage in the sample. Consequently as the X-ray diffraction method is not sensitive enough to determine small amounts of clay when mixed with larger amounts of other coarser materials in the soil, failure to resolve the diagnostic peaks of the clay minerals results. Another possibility is that trials I and II involved oven drying of the specimens. It has been observed that on oven drying the marine clay undergoes aggregation, resulting in formation of coarser particles, which are less sensitive to X-ray diffraction analyses.

3.10 STABILISATION OF MARINE SOILS

As one of the main objectives of the study was improvement of shear strength and compressibility characteristics of Cochin marine clays, attempts were made to identify a suitable stabilising agent for the clay and study the improvement in the engineering properties of the treated soil. Twenty additives, as listed below, were tried on the marine clay samples from Parur.

1. Sodium hydroxide
2. Sodium chloride
3. Sodium silicate
4. Potassium hydroxide
5. Potassium chloride
6. Potassium dichromate
7. Ferric chloride
8. Calcium hydroxide

9. Calcium chloride
10. Calcium carbonate
11. Calcium sulphate (gypsum)
12. Sodium carbonate
13. Sodium hexametaphosphate
14. Potassium permanganate
15. Magnesium chloride
16. Aluminium chloride
17. Industrial waste No.I
18. Industrial waste No.II
19. Lime
20. Cement.

It has been found that lime is the most suitable stabilising agent. The details of the various tests conducted and the results obtained from various additives are presented in chapter VI. Industrial wastes I and II did not show any promise.

3.10.1 Lime treated marine clays

'Moist' marine clay samples were treated with 6% by weight of each of the above additives. The treated soil was remoulded into the moulds for laboratory vane shear test. Six samples of marine clay treated with each additive was prepared. Three were tested on the seventh day and the other three were tested after a month in a laboratory vane shear test apparatus. The average values of 7-day and 30-day strength are presented in Table 6.2.1. Two industrial wastes from neighbouring factories were also tried. A comparative study of all the results indicated that lime was the most suitable stabilising agent for marine clays. The increase in shear strength after a month of curing was about twenty times that of the strength of the untreated moist sample. For purposes of comparison the additives were tried on red earth also.

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 air tight multilayer polythene bags. Just sufficient water was sprinkled over the lime shells taken from these bags on each day of preparation of lime treated samples, till all the shells crumble to fine power 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.

Once the most suitable stabilising agent was identified as lime, a detailed study of lime stabilisation was taken up. Several series of consolidation tests were run on lime treated specimens with varying lime contents and curing periods. The lime contents selected were 3%, 6%, 9% and 12%. Lime was added to moist soil and was thoroughly mixed. The sample was then remoulded into stainless steel rings of 100 mm diameter and 40 mm height taking care to see that air is not entrapped during hand moulding the soil into the ring. The rings are then covered with polythene bags. They are then covered with thin circular G.I.sheets with diameter equal to the outer diameter of the rings. This assembly is again covered with polythene bags and tied with rubber bands. The specimens are then kept for curing for various durations.

Samples for consolidation tests are then taken into the oedometer rings by pushing it slowly into the ring after removing the polythene covers and G.I.plates. Consolidation tests are conducted on these samples cured for various periods.

The improvement in shear strength after 7 days and 30 days of curing was measured by conducting laboratory vane shear tests on treated soil samples. 'Moist' samples of marine clay were mixed thoroughly with different additives without changing the initial water content and filled in the moulds of laboratory vane shear apparatus of size 3.8 cm in diameter and 7.6 cm in height. The specimens were kept for curing in air tight polythene bags in humid atmosphere. During the test, the vanes were gently introduced into the soil mass and kept for five minutes to allow for dissipation of pore pressure, if any, before the test was performed. IS: 2720 (Part XXX) 1980 was strictly followed for the conduct of the test.

Chapter IV

PHYSICAL PROPERTIES

4.1 INTRODUCTION

Marine clays formed by the sedimentation of clay soils in marine environments exhibit many unusual physical properties. They possess very high liquid limit and their natural moisture content is close to the liquid limit values. The most striking feature of the physical properties is the phenomenal changes that are caused by drying of the sample. In this chapter a detailed study of the physical properties of Cochin marine clays has been presented.

Table 4.1 presents the physical properties of samples of marine clay deposits in their natural state and when they are dried by exposure to air or dried in an oven at a temperature of 105-110°C. From the data given in the table, it can be seen that both air drying and oven drying significantly reduce liquid limit and free swell index considerably. The oven dried samples have liquid limits in the range of 40-60% of the natural moist sample and air drying reduces liquid limit by 20-30%. The main reason attributed to this striking behaviour is the aggregation of particles on drying (Ramanatha Iyer, 1966). A similar reduction

Table 4.1

TYPICAL PHYSICAL PROPERTIES OF COCHIN MARINE CLAYS

Sl. No.	Location	Munambam	Nettoor	Cheranelloor	Parur	Maradu	Kumbalam
	Depth of sampling (m)	9.40	1.0	5.50	11.50	9.0	10.0
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1.	Specific gravity	2.66	2.64	2.66	2.62	2.65	2.60
2.	Natural density (gm/cc)	1.50	1.42	1.45	1.49	1.49	1.46
3.	Natural moisture content (%)	92.0	124.0	87.0	98.0	77.0	81.0
4.	Liquid limit (%)						
	(a) moist soil	116.5	137.5	118.5	108.0	137.0	96.0
	(b) air dried	--	87.5 (63.6)	96.5 (81.4)	73.0 (67.6)	105.0 (76.6)	79.5 (82.4)
	(c) oven dried	57.0 (48.9)*	55.0 (40.0)	57.0 (48.1)	56.5 (52.3)	80.0 (58.4)	56.0 (58.0)
5.	Plastic limit (%)						
	(a) moist soil	45.5	47.5	45.3	42.8	55.2	36.5
	(b) air dried	--	38.7 (81.5)	41.5 (91.6)	35.3 (82.5)	45.2 (81.9)	35.0 (95.9)
	(c) oven dried	38.0 (83.5)	33.6 (70.7)	34.5 (76.1)	33.7 (78.7)	40.5 (73.4)	32.0 (87.7)

* As a percent of its own moist value

(Contd.)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
6.	Plasticity index (%)						
	(a) moist soil	71.0	90.0	73.2	65.2	81.8	60.0
	(b) air dried	--	48.8 (54.2)	55.0 (75.1)	37.7 (57.8)	59.8 (73.1)	44.50 (74.2)
	(c) oven dried	19.0 (26.8)	21.4 (23.8)	25.5 (30.7)	22.8 (35.0)	39.5 (48.2)	24.0 (40.0)
7.	Liquidity index	0.65	0.84	0.57	0.85	0.27	0.74
8.	Shrinkage limit (%)						
	(a) moist soil	22.5	17.8	19.3	20.3	21.8	19.8
	(b) air dried	--	19.5 (109.5)	18.5 (95.9)	20.2 (99.5)	18.5 (84.9)	19.1 (96.5)
	(c) oven dried	27.0 (120.0)	19.5 (109.5)	20.3 (105.2)	21.4 (105.4)	19.4 (89.0)	18.5 (93.4)
9.	Grain size distribution						
	(a) moist soil						
	(i) Clay size (%) (<0.002 mm)	42	48	36	42	50	36
	(ii) Silt size (%) ($>0.002<0.075$ mm)	40	31	43	39	40	43
	(iii) Sand size (%) (>0.075 mm)	18	21	21	19	10	21

(Contd.)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
	(b) Air dried						
	(i) Clay size (%)	--	32	33	30	41	30
	(ii) Silt size (%)	--	43	46	44	44	49
	(iii) Sand size (%)	--	25	21	26	15	21
	(c) Oven dried						
	(i) Clay size (%)	23	25	23	23	30	16
	(ii) Silt size (%)	59	48	56	49	54	57
	(iii) Sand size (%)	18	27	21	28	16	27
10.	Activity						
	(a) moist soil	1.69	1.88	2.03	1.55	1.64	1.64
	(b) air dried	--	1.53 (81.4)	1.67 (81.4)	1.26 (81.3)	1.46 (89.0)	1.48 (88.6)
	(c) oven dried	0.83 (49.1)	0.44 (23.4)	0.98 (48.3)	0.99 (63.9)	1.32 (80.5)	1.30 (77.8)
11.	Free swell index (cc/gm)						
	(a) moist soil	4.30	5.20	4.50	4.44	6.50	4.70
	(b) air dried	--	1.70 (32.7)	2.42 (53.8)	2.26 (50.9)	2.52 (38.8)	2.28 (48.5)
	(c) oven dried	1.40 (32.6)	1.20 (23.1)	1.66 (36.9)	1.54 (34.7)	1.80 (27.6)	1.65 (35.1)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
12.	Salinity (g/l)	7.30	6.10	5.90	6.40	6.70	7.20
13.	Organic matter (%)	6.30	7.64	7.61	9.80	6.50	5.60
14.	pH value	7.72	7.83	7.60	8.23	7.80	7.55
15.	Calcium carbonate (%)	--	14.0	23.50	13.95	7.30	10.5

can be seen in the case of plastic limit as well as free swell index. Nambiar et al. (1985) also have reported reduction in liquid limit of fine grained carbonate soil from off west coast, though not significantly. Jagadish Narain and Iyer (1967) reported significant reduction in the liquid limit of Kuttanad clays on air drying.

But so far no systematic study has been carried out on the physical properties of Cochin marine clays with particular reference to initial testing conditions of the soil viz. moist, air and oven dried. Further experiments are also conducted to investigate the effect of washing, soaking, addition of salts and other additives, organic and carbonate content and grain size distribution on the physical properties of Cochin marine clays.

In this chapter an attempt has also been made to use liquid limit values to estimate the undrained strength of Cochin marine clays having different moisture contents.

4.2 ATTERBERG LIMITS

Typical values of liquid limit and plastic limit of samples from different locations are presented in Table 4.1. It can be seen from the table that eventhough the locations from where samples were collected are separated by even more than 50 kms at times, the typical values of physical properties are quite consistent with one another. For moist samples the liquid limit varies from 83.5 to 175% depending upon the composition of the soil together with the physico-chemical constituents. The values of the plastic limit vary from 33.8 to 60% for all the clays tested.

Fig. 4.2.la presents the liquid limit vs. plasticity index for the numerous soil samples tested in this investigation. Most of the points are close to the A-line.

But a statistical fit for the values obtained from moist samples gives the relation

$$PI = 0.62(LL - 2.0) \qquad (4.2.1a)$$

with a correlation coefficient of 0.92.

Table 4.1 also gives the liquid limit and plastic limit values for air dried and oven dried samples of Cochin marine clays. As explained earlier, due to aggregation of finer particles to coarser ones during drying, the clay content reduces and this in turn brings down the Atterberg

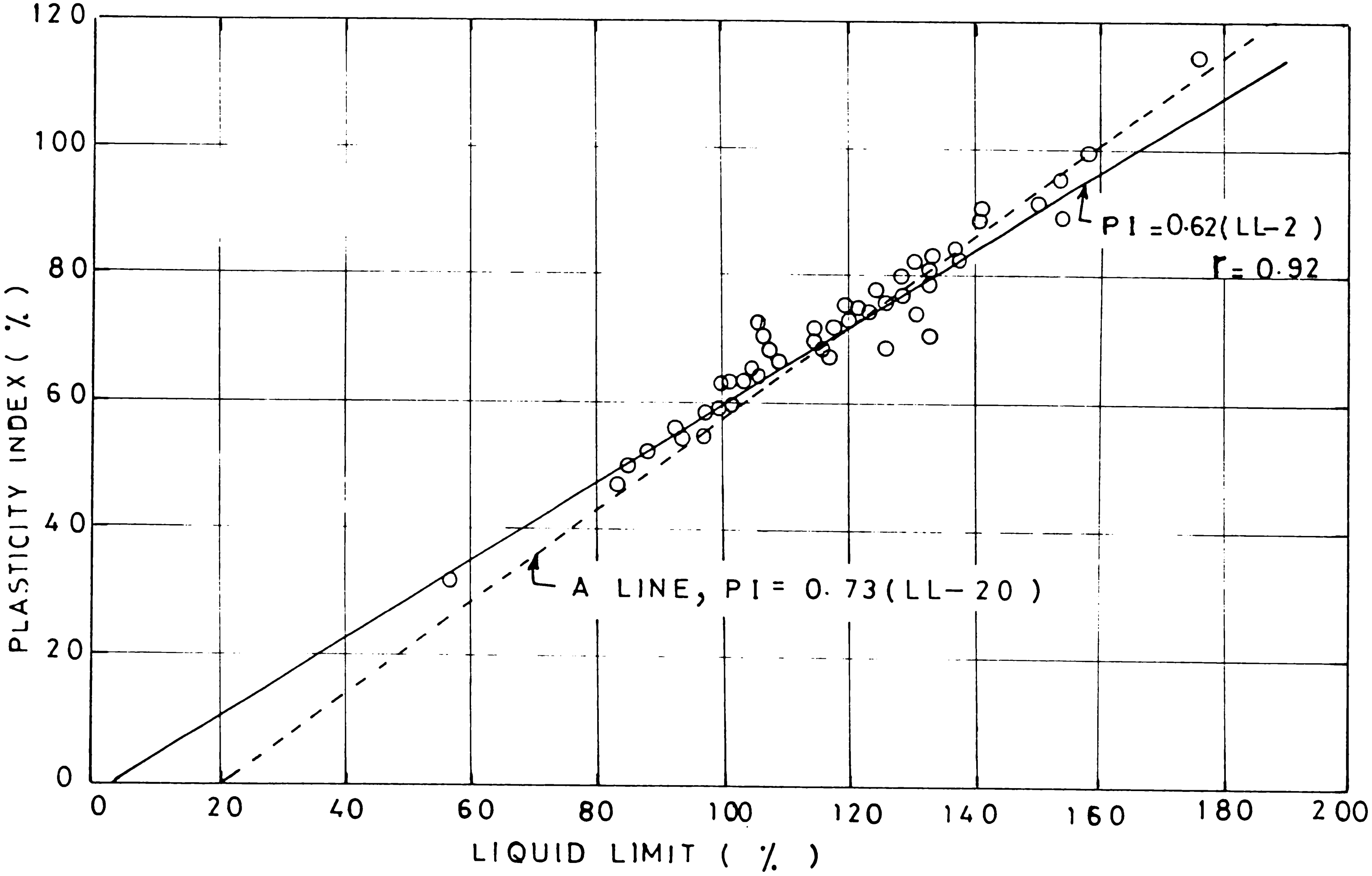


Fig. 4·2·1a· LIQUID LIMIT vs PLASTICITY INDEX FOR MOIST SOIL·

limits. Fig.4.2.1b shows the plasticity chart for air dried samples and Fig.4.2.1c is the plot for oven dried specimens. The plots give statistical fit as shown below:

$$PI = 0.56(LL-1.6) \qquad (4.2.1b)$$

with a correlation coefficient of 0.96 for air dried samples

$$PI = 0.67(LL-23.5) \qquad (4.2.1c)$$

with a correlation coefficient of 0.89 for oven dried samples.

Fig.4.2.1d shows the relation between plasticity index and liquid limit for all the three types of samples put together giving rise to the statistical fit

$$PI = 0.74(LL-23.5) \qquad (4.2.1d)$$

with a high correlation coefficient of 0.98.

Bjerrum (1954) reports the values of Atterberg limits for Norwegian marine clays. The liquid limit of these clays varies over a range of 59.4 to 21%. A statistical

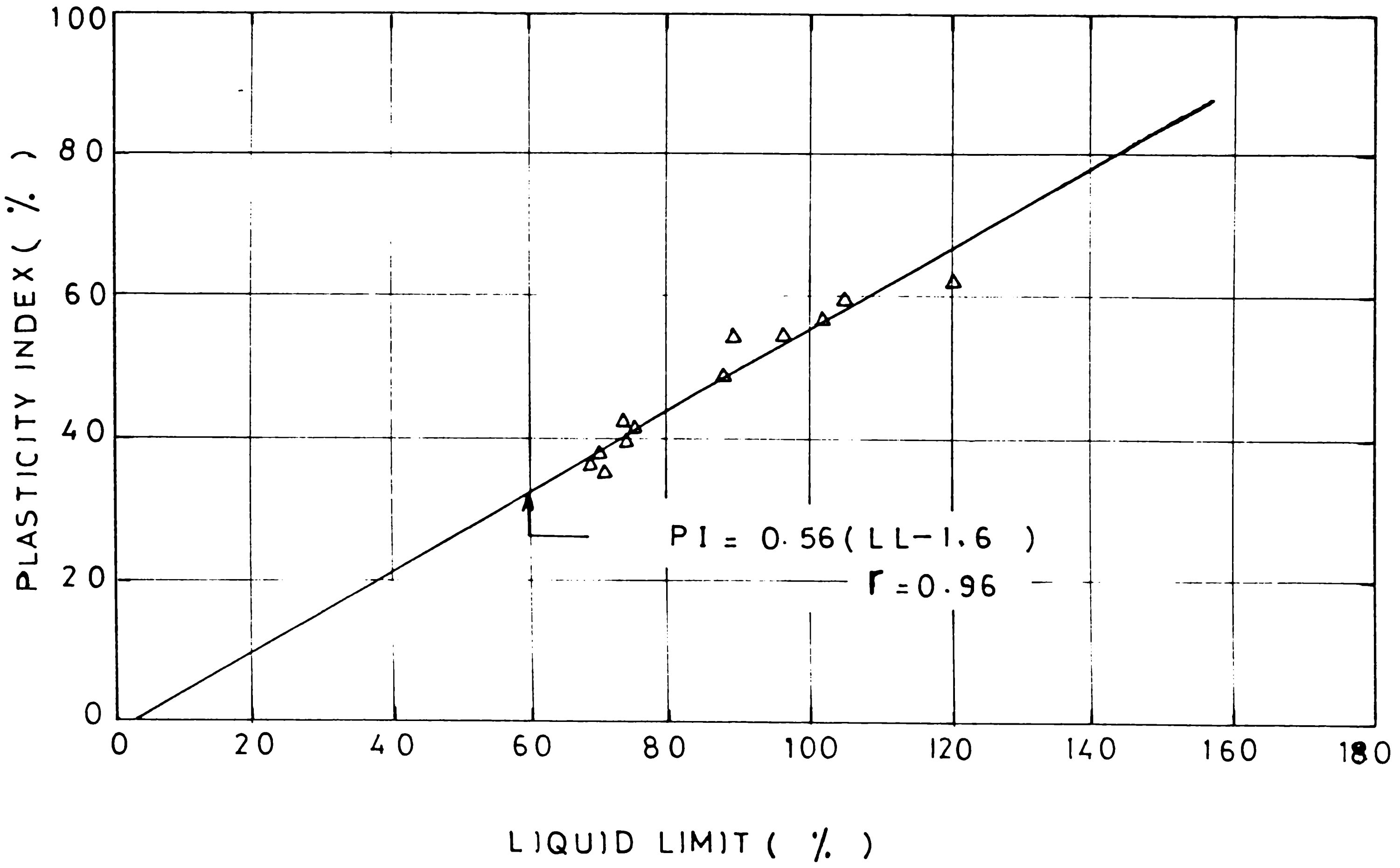


Fig. 4.2.1b. LIQUID LIMIT vs PLASTICITY INDEX FOR AIR DRIED SOIL.

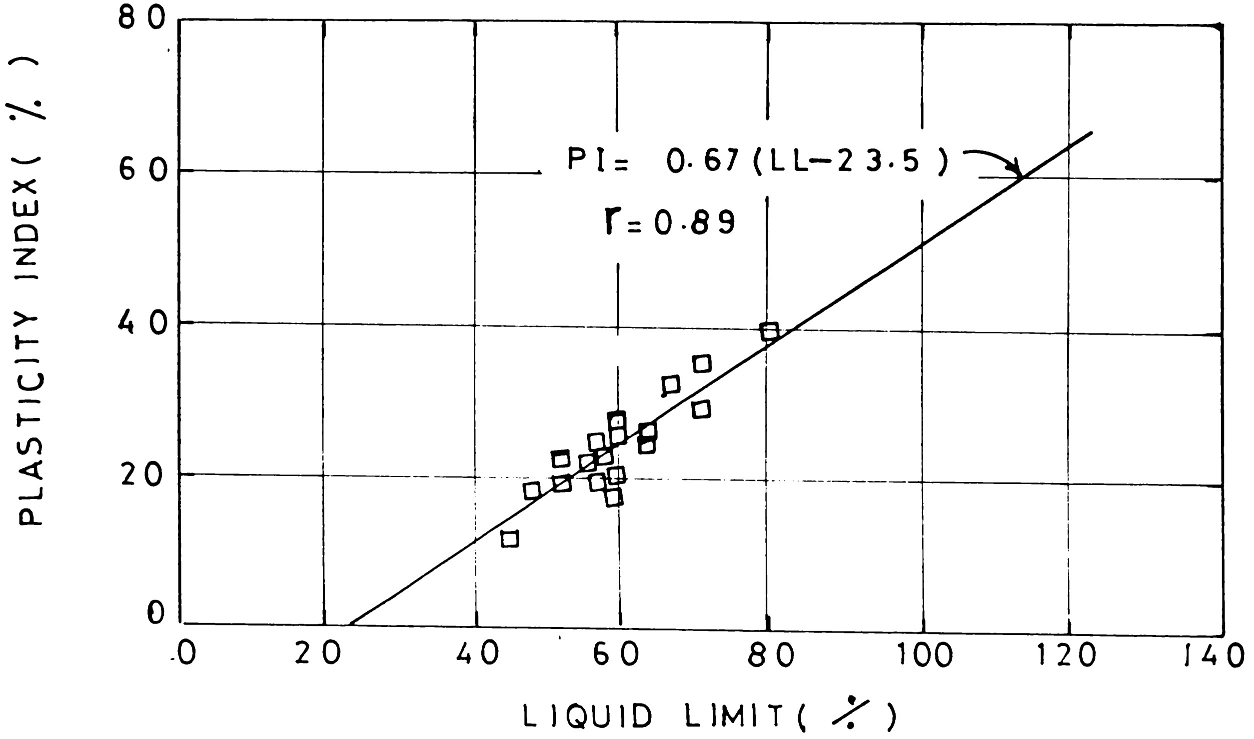


Fig. 4.2.1c. LIQUID LIMIT vs PLASTICITY INDEX FOR OVEN DRIED SOIL.

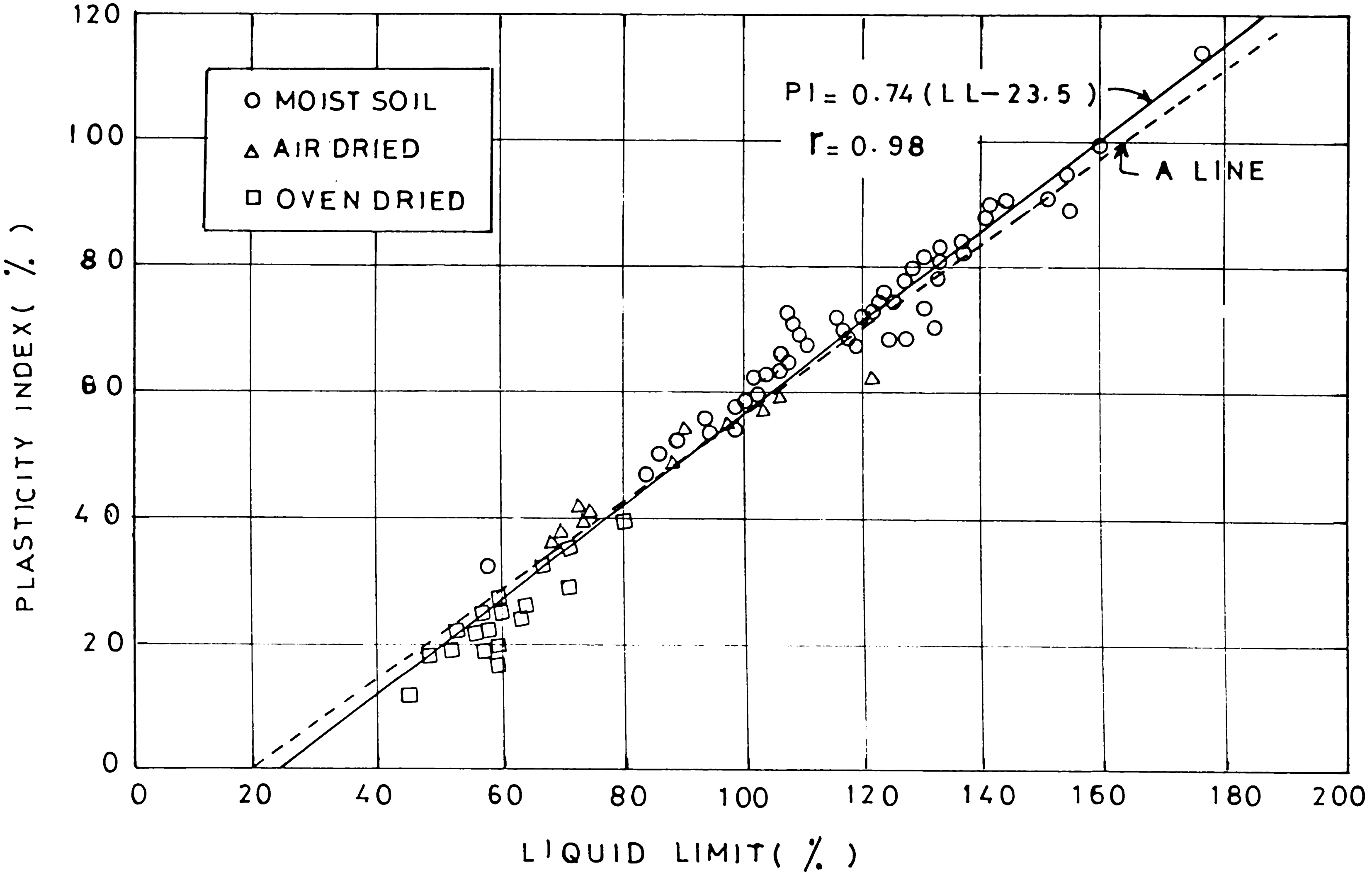


Fig. 4.2.1d. LIQUID LIMIT vs PLASTICITY INDEX FOR MOIST, AIR DRIED AND OVEN DRIED SOILS.

fit of the liquid limit vs. plasticity index gives the relation

$$PI = 0.795(LL-17.2) \qquad (4.2.2)$$

It can be taken that the slopes for both Cochin marine clays and Norwegian marine clays are of the same order. For inorganic soils, the relationship of Casagrande plasticity chart for inorganic soils lies above that of the relationship obtained for Cochin marine clays and that of Norwegian clays. This behaviour is possibly due to the presence of organic matter in these clays.

Several attempts have been made to link plasticity index and liquid limit treating them as independent parameters (Nagaraj et al., 1983; Seed et al., 1964; Casagrande, 1948). Apart from Casagrande's A-line which represents an empirical boundary between inorganic and organic soils there have been other attempts to establish relations between plasticity index and liquid limit.

The popular relations are

$$PI = 0.74(LL-8) \qquad (4.2.3a)$$

(Nagaraj and Jayadeva, 1983)

$$PI = 0.98(LL-27.5) \qquad (4.2.3b)$$

(Seed et al., 1964)

$$PI = 0.615(LL-9) \qquad (4.2.3c)$$

(Schofield and Wroth, 1968)

These relations assumed importance as many engineering properties have been correlated to Atterberg limits, in particular liquid limit and plasticity index. It also permits field engineers to check the suitability of any borrow pit for appropriate construction material. With proper judgement Atterberg limits can be effectively used in many geotechnical engineering problems. In view of its usefulness this aspect of geotechnical engineering is actively pursued even now as research topic (Sridharan et al., 1986 and 1988; Harrison, 1988).

The relation between flow index and plastic limit is shown in Fig.4.2.2a which is represented by a straight line,

$$PL = 0.473 (I_f + 65.9)$$

with a correlation coefficient $r = 0.80$.

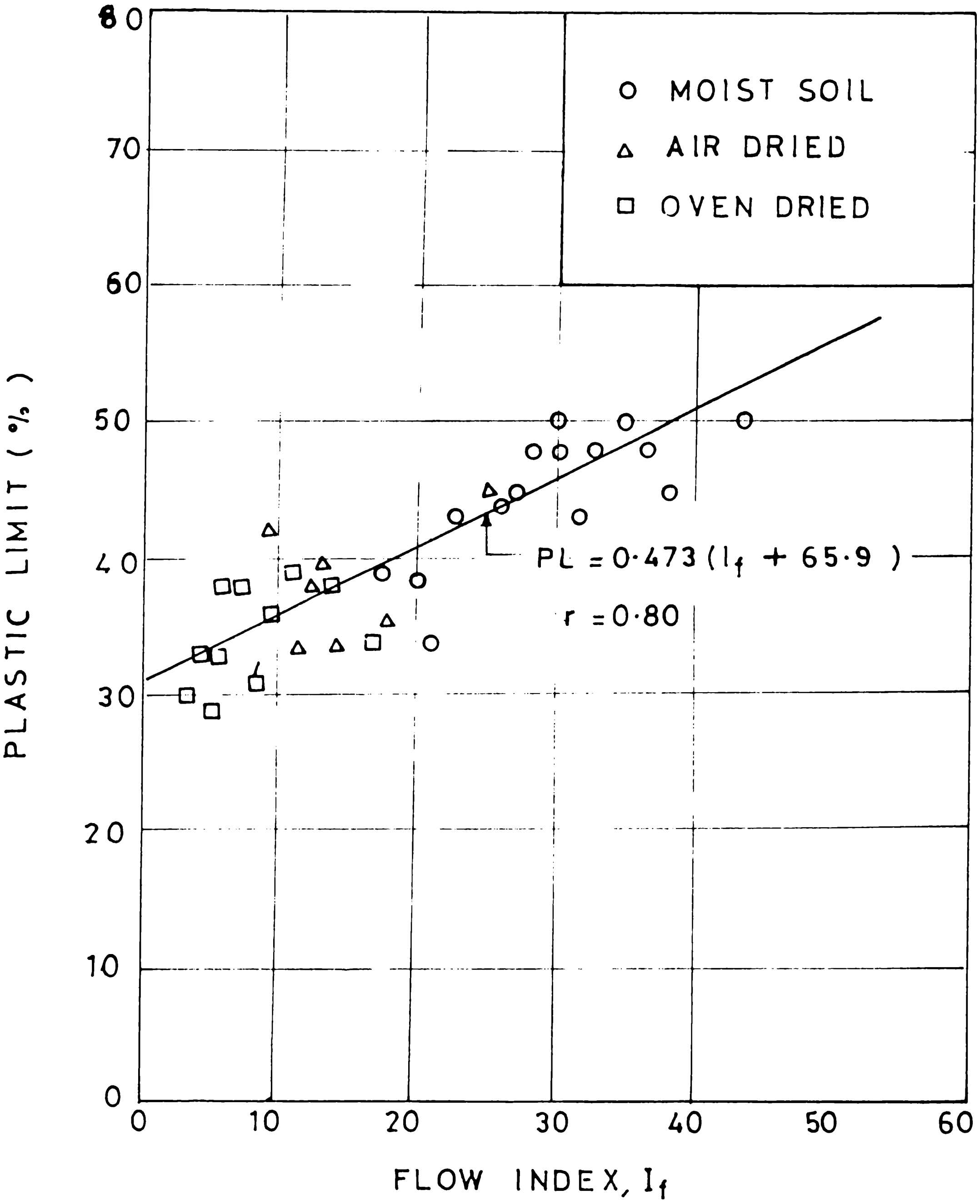


Fig. 4·2.2a· PLASTIC LIMIT vs FLOW INDEX·

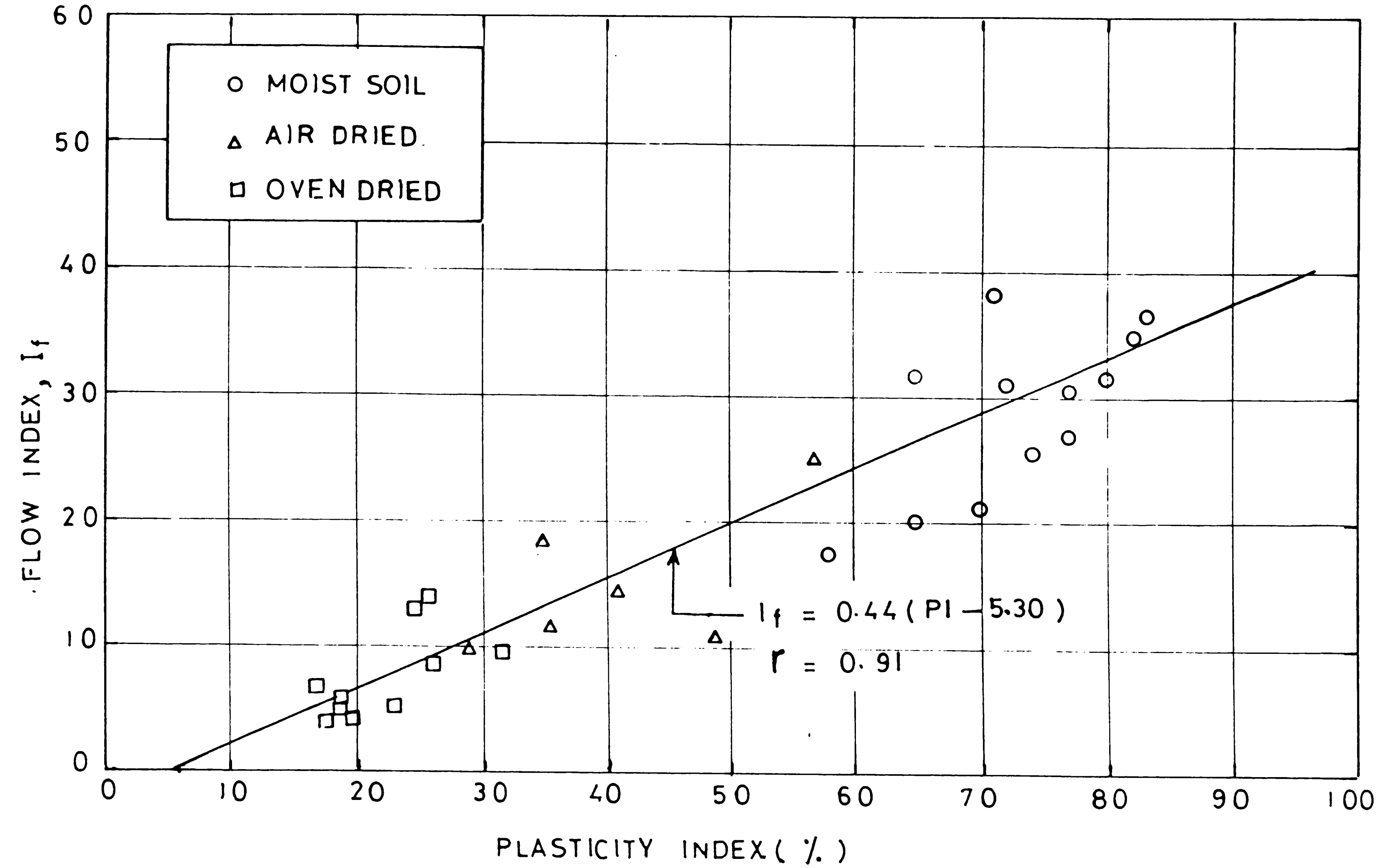


Fig 4·2·2b. PLASTICITY INDEX vs FLOW INDEX.

Fig.4.2.2b shows the relation between flow index and plasticity index for moist, air dried and oven dried samples. These values give rise to a linear relationship

$$I_f = 0.44(PI-5.30) \tag{4.2.4b}$$

with correlation coefficient equal to 0.91.

Tests to determine the liquid limit of a soil are analogous to shear tests. The number of blows N of the cup in percussion testing is a measure of the shear strength of the soil, at that water content. Nagaraj and Jayadeva (1981) have shown that the flow curves of different soils can be generated using their respective liquid limit water contents to obtain relationships of the form

$$\frac{W}{W_L} = C-d \log N \tag{4.2.5}$$

where W = water content

W_L = liquid limit

N = number of blows

c and d are constants.

Fig.4.2.3a shows the flow curves for moist, air dried and oven dried samples from Parur. When the water contents are divided by their respective liquid limits, in almost all cases, the values fall into a straight line or a narrow band.

Fig.4.2.3b gives the plot between $\frac{W}{W_L}$ vs. log number of blows for moist samples from various sites. A statistical fit to the data generate a straight line,

$$\frac{W}{W_L} = 1.42-0.305 \log N \qquad (4.2.6)$$

This has a high correlation coefficient of 0.97.

Thus it can be seen that equation 4.2.6 can be effectively used for finding out the liquid limit as one point method,

$$W_L = \frac{W}{c-d \log N}$$

The values of c and d for Cochin marine clays are 1.42 and 0.305 respectively.

Since the Atterberg limits are a measure of soil behaviour characterised by the surface forces of clay minerals,

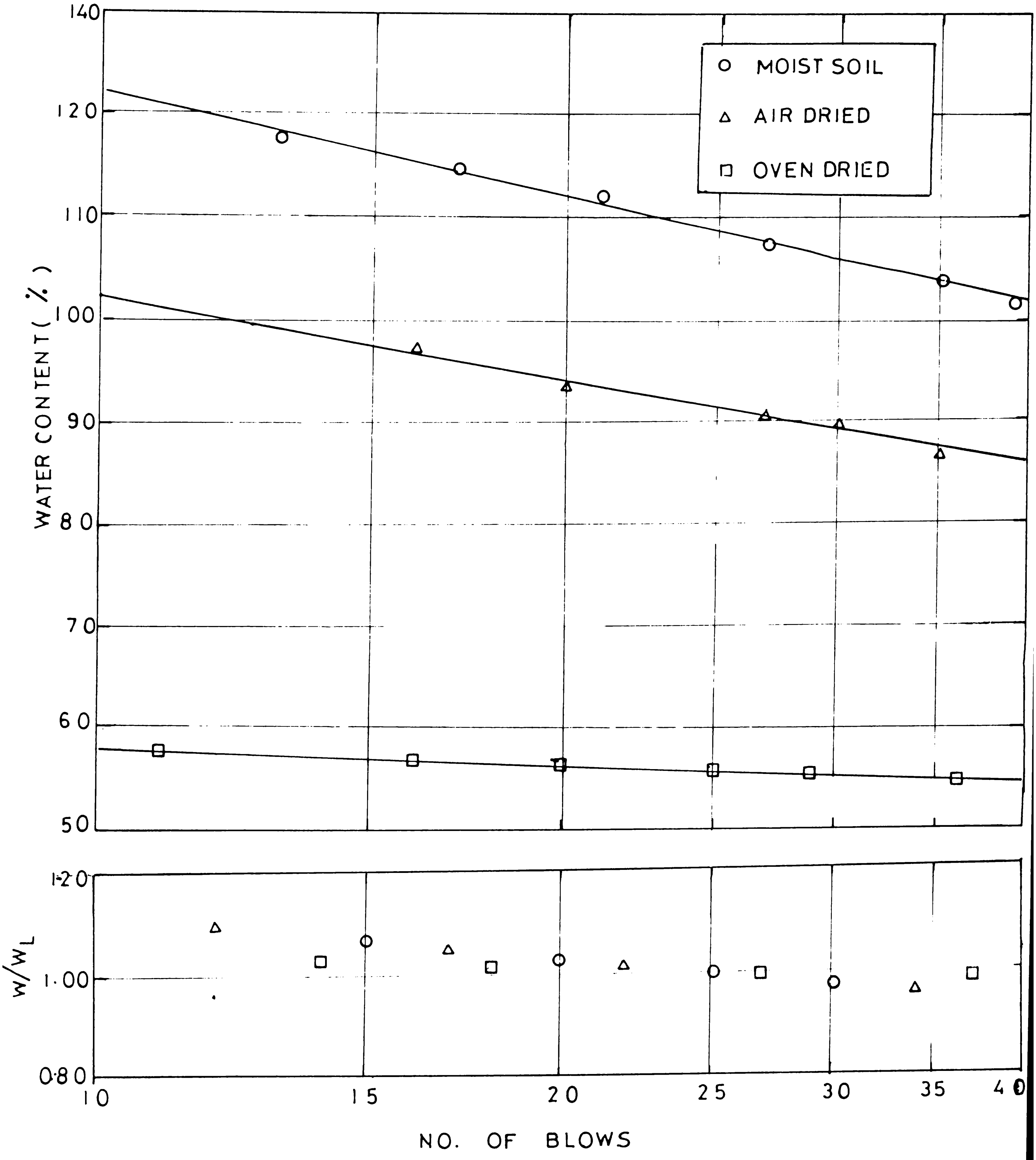


Fig. 4.2.3a. FLOW CURVES FOR SAMPLES FROM PARUR.

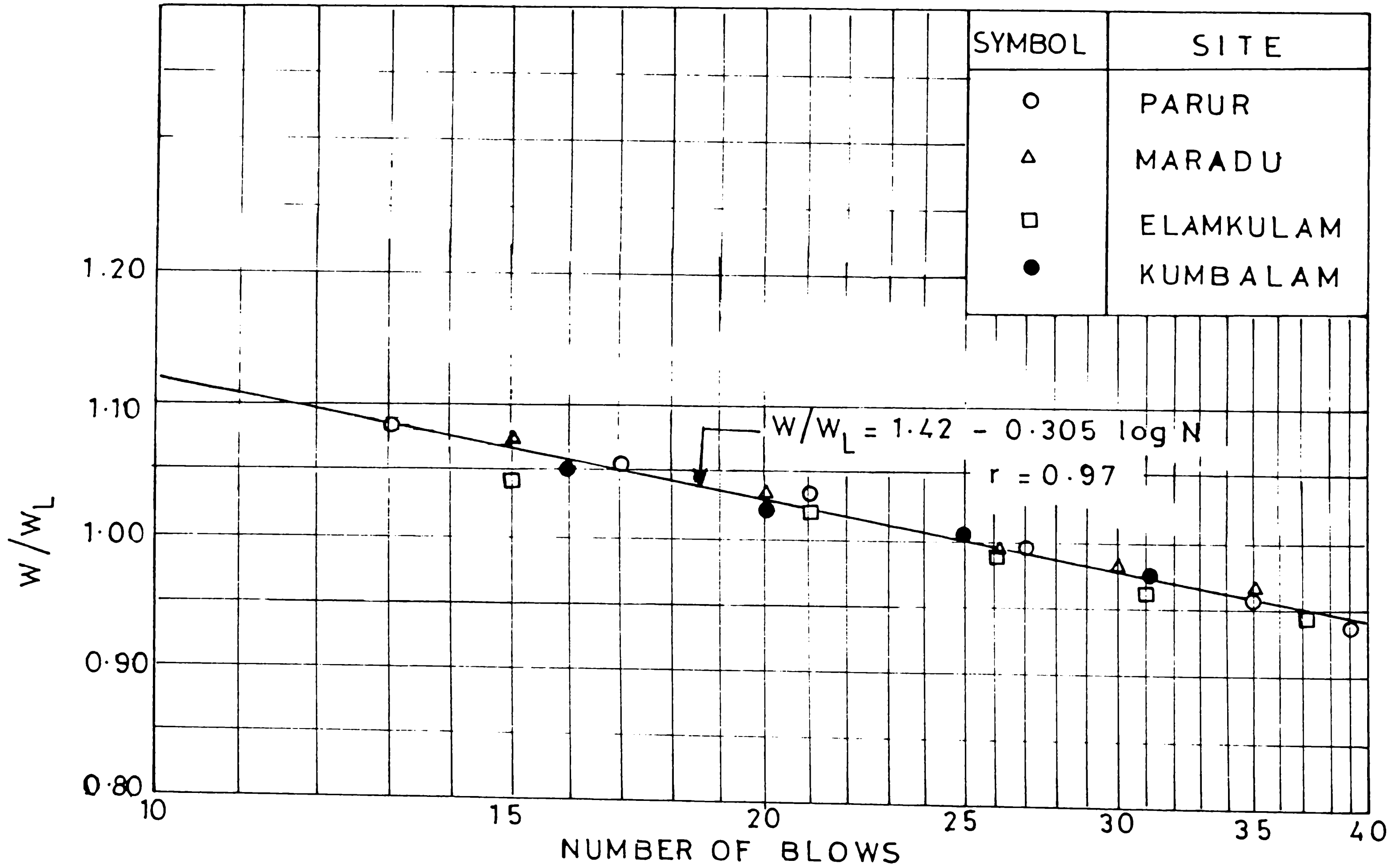


Fig. 4·2.3b. FLOW CURVE FOR MOIST SAMPLES FROM VARIOUS SITES.

they should show a correlation with the percentage of clay fraction. Plots between liquid limit and percent clay fraction are shown in Fig.4.2.4a,b,c and 4.2.5 for moist, air dried, oven dried and all samples respectively. They give the following linear relations with clay percent c.

For moist samples

$$LL = 3.4(C-10) \qquad (4.2.7a)$$

$$r = 0.83$$

For air dried samples

$$LL = 2.50(C+1.20) \qquad (4.2.7b)$$

$$r = 0.83$$

For oven dried samples

$$LL = 1.19(C-24.7) \qquad (4.2.7a)$$

$$r = 0.68$$

For moist, air dried and oven dried samples put together, the statistical fit gives the equation

$$LL = 2.94(C-4.0) \qquad (4.2.8)$$

with a correlation coefficient of 0.95.

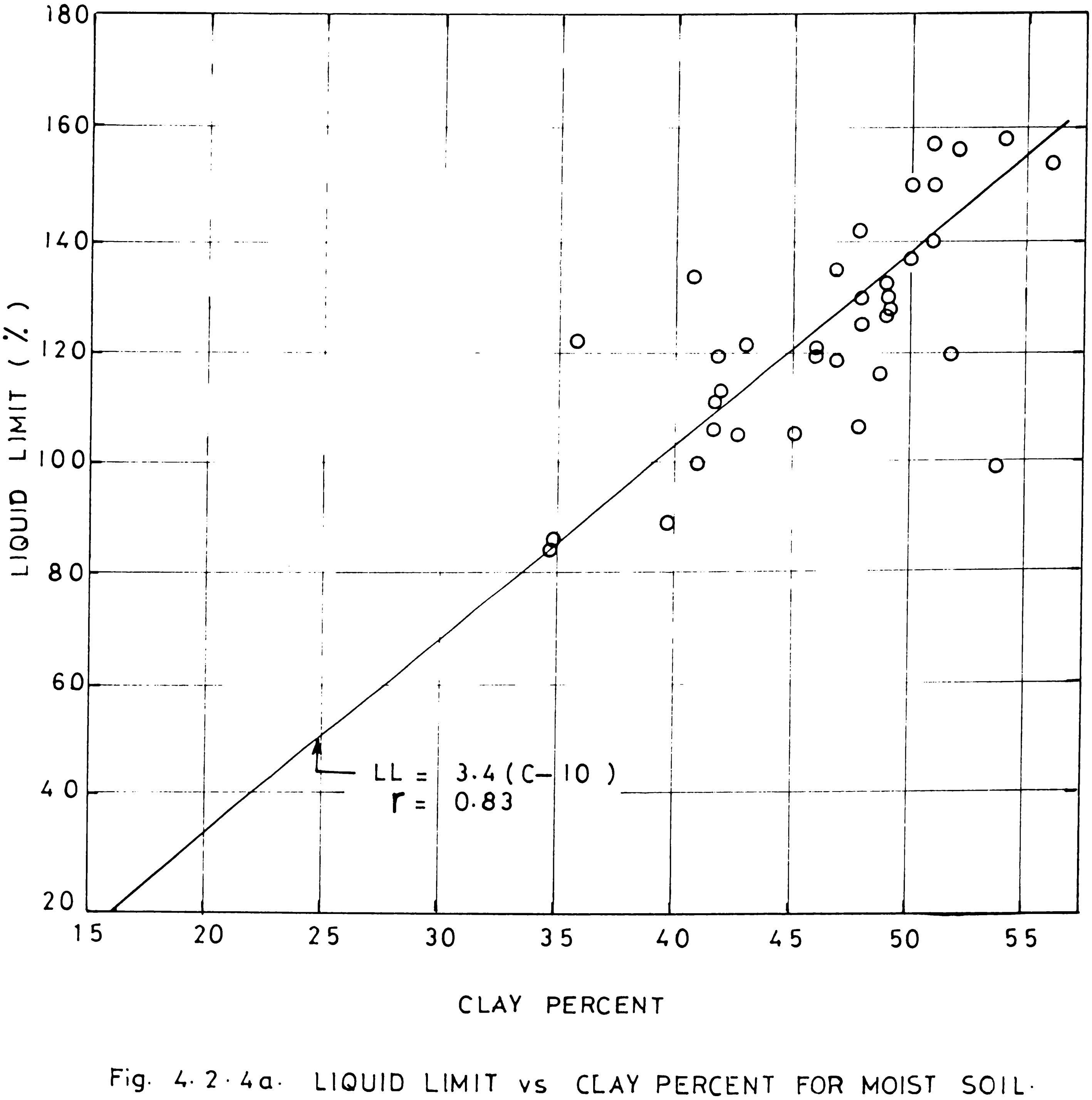


Fig. 4.2.4a. LIQUID LIMIT vs CLAY PERCENT FOR MOIST SOIL.

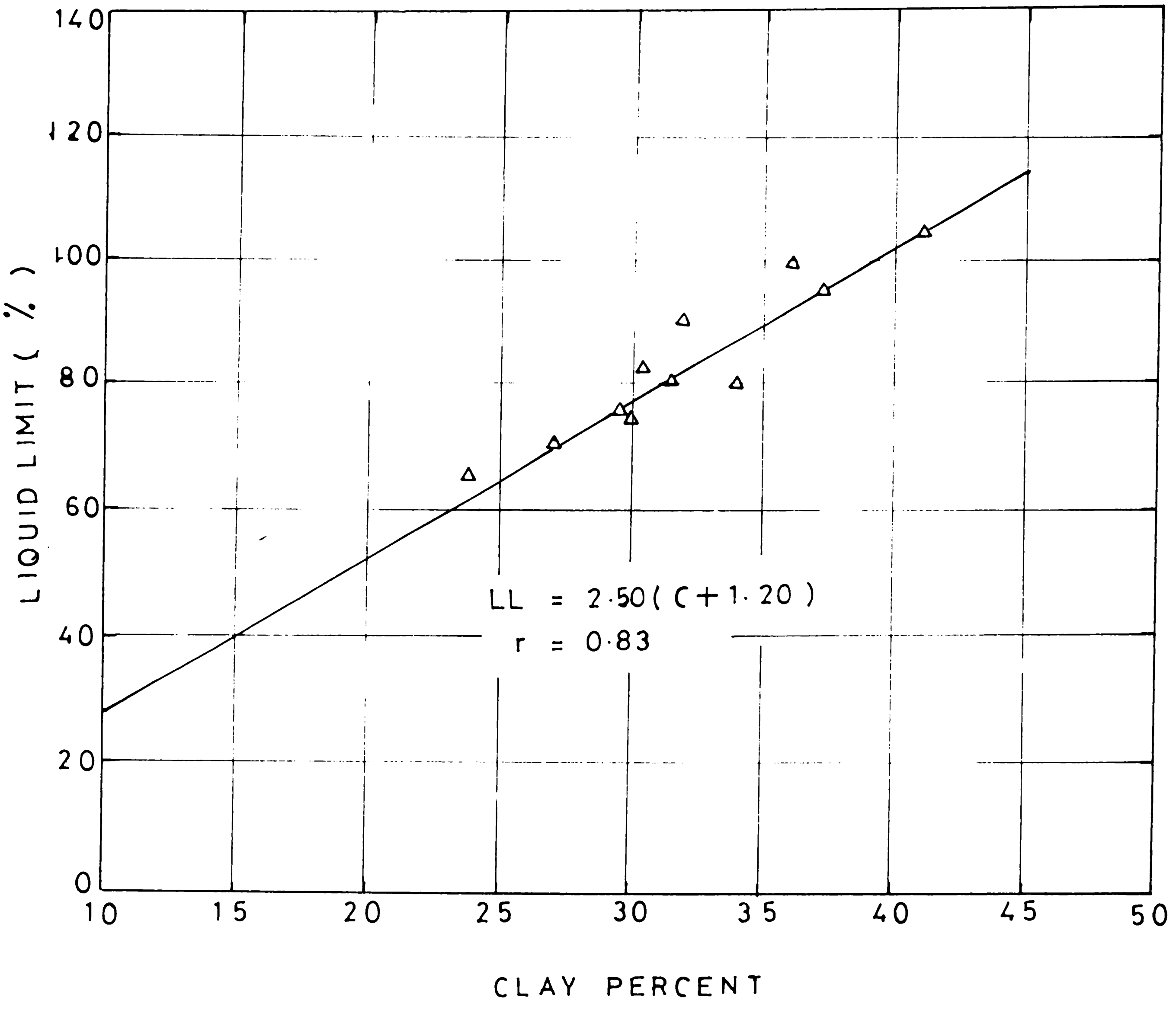


Fig. 4. 2. 4b. LIQUID LIMIT vs CLAY PERCENT FOR AIR DRIED SOIL.

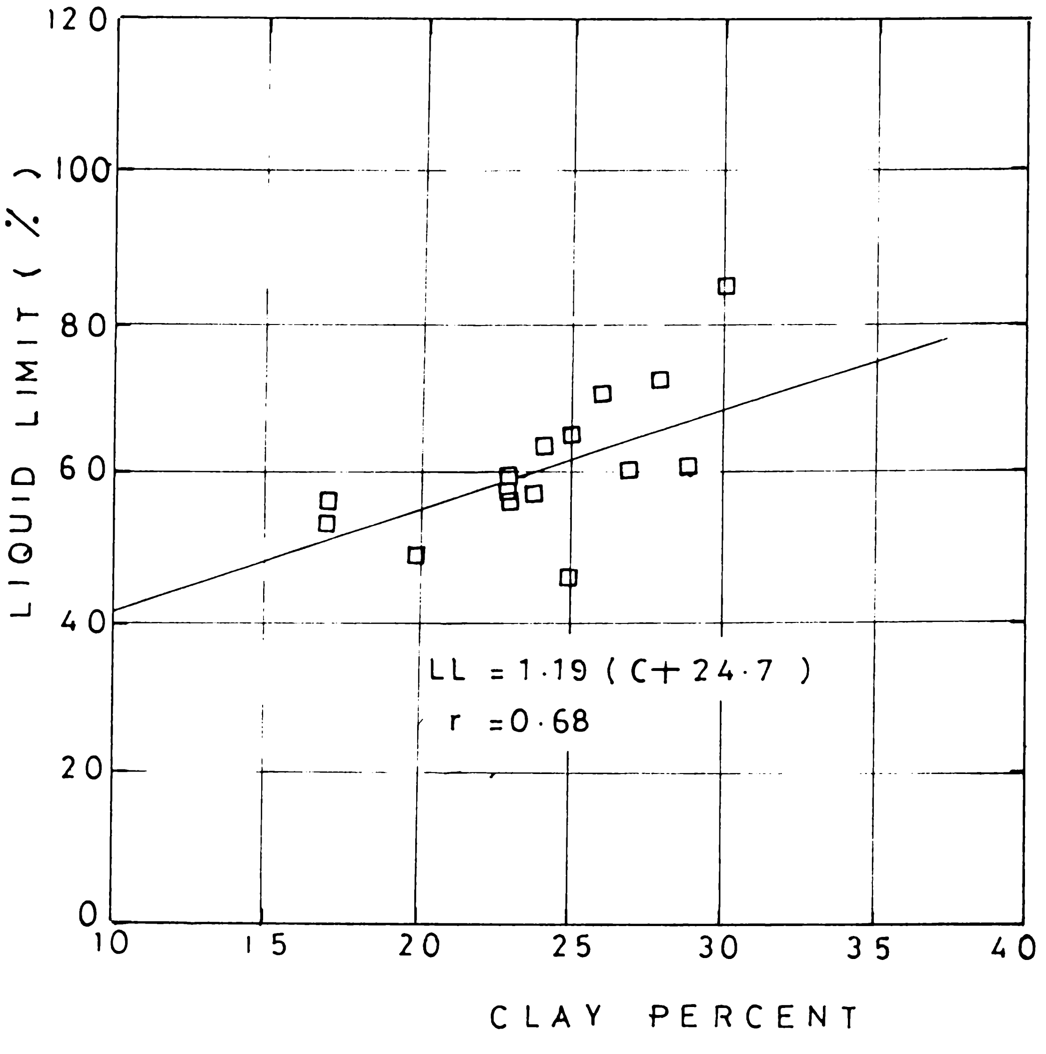


Fig. 4 2.4c. LIQUID LIMIT vs CLAY PERCENT FOR OVEN DRIED SAMPLES.

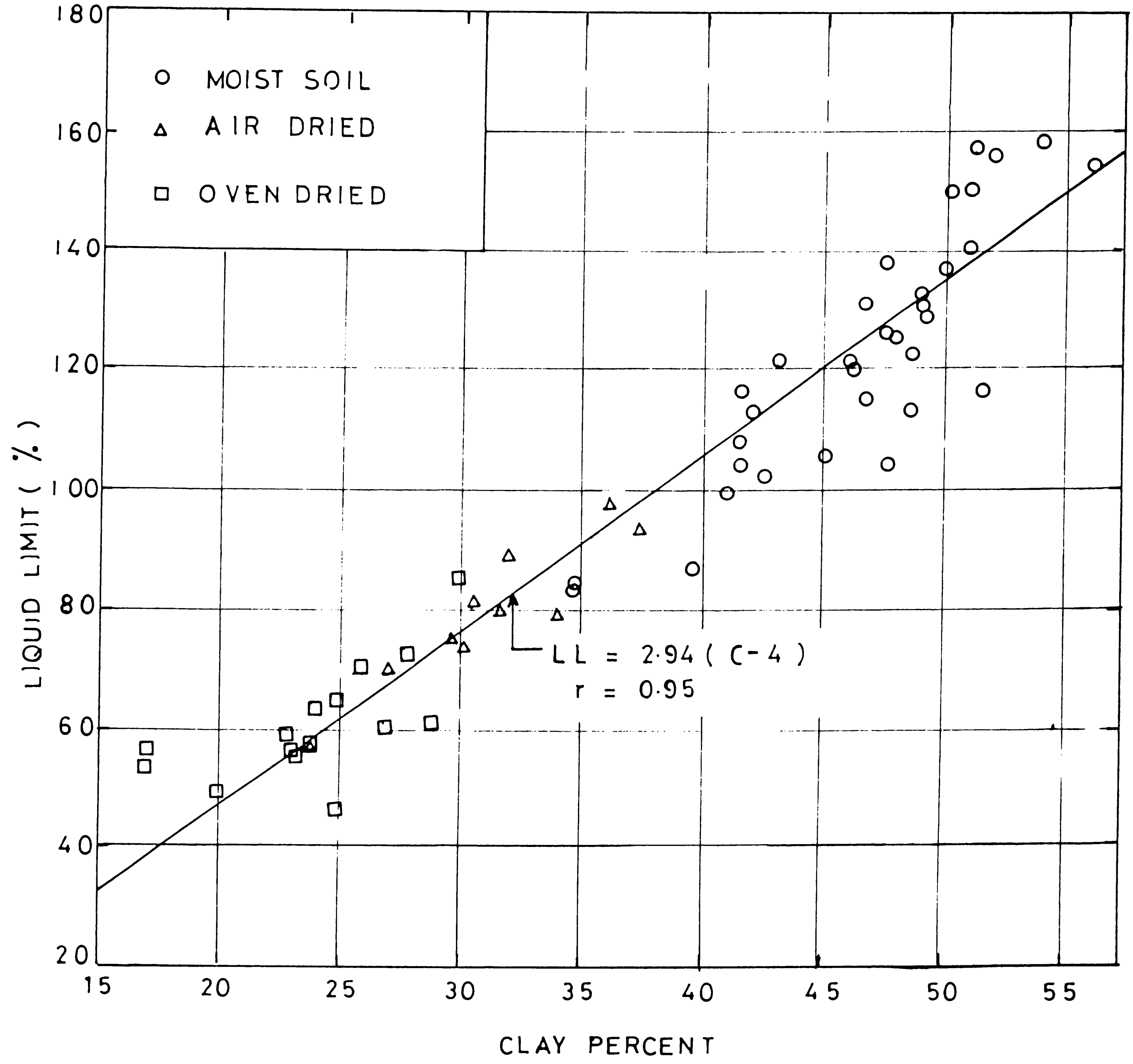


Fig. 4.2.5. LIQUID LIMIT vs CLAY PERCENT FOR MOIST, AIR DRIED AND OVEN DRIED SAMPLES.

While it can be noted that the correlation coefficient is very good for all the soils put together, for oven dried samples the relationship obtained is not that satisfactory, since the r obtained is only 0.68. However, for moist soil one can get very good correlation between liquid limit and clay percent, with more data. Since it is tedious and time consuming to obtain clay percent through hydrometer analysis, correlation with liquid limit will be of great use since we will always be determining liquid limit in any case.

4.3 SHRINKAGE LIMIT

The shrinkage limit is quite high for moist samples when compared to their liquid limits and plastic limits from which one can infer that the moist clay consists of flocs. It has been reported (Lambe, 1958) that higher shrinkage limit indicates flocculant fabric. Fig.4.3.1 gives a plot between the shrinkage limit and clay size fraction.

A statistical fit gives the relation

$$SL = 31.10-0.276c \qquad (4.3.1)$$

$$r = 0.83$$

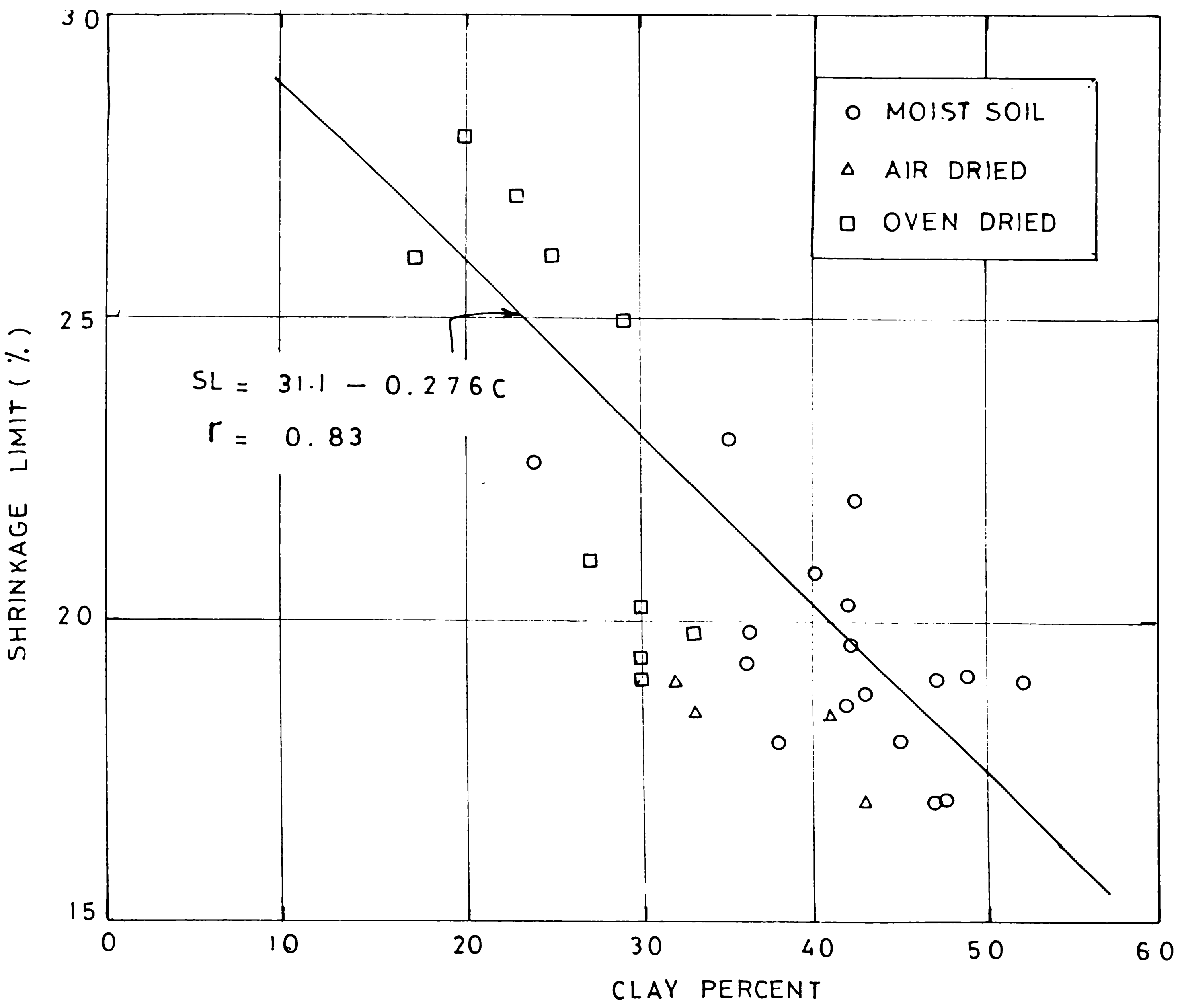


Fig. 4.3.1. SHRINKAGE LIMIT vs CLAY PERCENT.

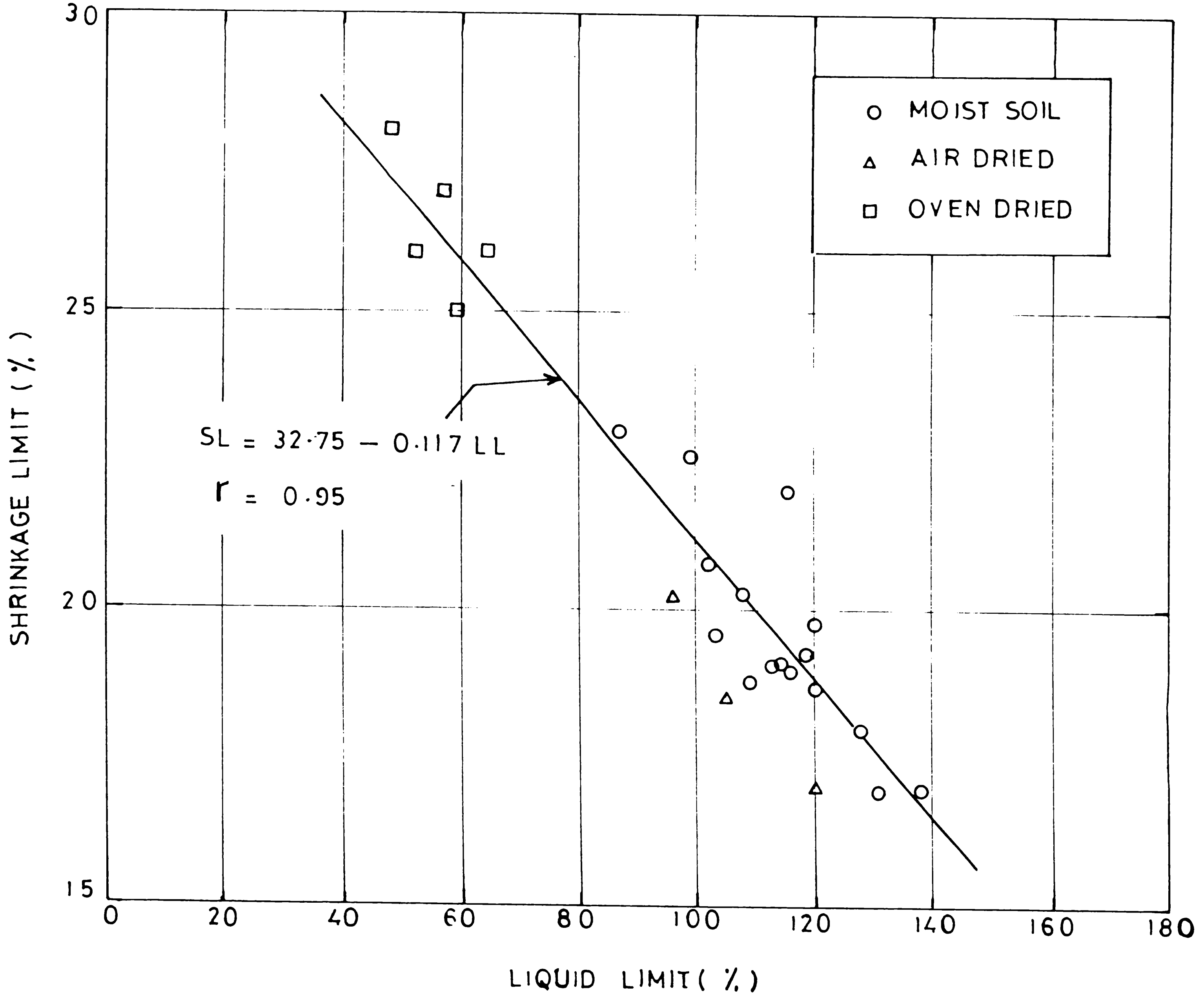


Fig. 4·3·2. SHRINKAGE LIMIT vs LIQUID LIMIT.

By and large, the shrinkage limit got increased for oven dried samples. It is as ought to be since coarser particles formed by aggregation result in higher shrinkage limit.

Fig.4.3.2 shows the plot between shrinkage limit and liquid limit for moist, air dried and oven dried samples. Since aggregation during drying causes a reduction in liquid limit, by the same reason the shrinkage limit should increase. Both these are well reflected in the figure. The equation for a statistical fit for the data is

$$SL = 32.75-0.117\ LL \tag{4.3.2}$$

which has a correlation coefficient of 0.95.

4.4 GRAIN SIZE DISTRIBUTION

Apart from the Atterberg limits, the effect of drying of marine clay specimens is most conspicuous on grain size distribution as can be seen from a number of figures provided in this section. It may be seen that aggregation takes place at all size levels. Fig.4.4.1a and b show typical grain size distribution curves for 4 locations from which samples were collected for the study. It may be mentioned here that while carrying out the hydrometer analysis dispersing agent was used as per IS:2720 (Part IV) 1975. In spite of using dispersing agent and mixing the soil slurry thoroughly with stirrers, the aggregation resulting from air drying and oven drying could not be broken to finer particles.

In order to study the grain size distribution of the clays with and without dispersing agents, a series of tests were conducted from samples from all the sites. The results of a typical test are presented in Fig.4.4.1c for moist, air dried and oven dried soil from Parur site. It can be seen from the figure that the percentage of clay size fraction increases considerably with the use of dispersing agents. In case of Parur soil the clay percent increased from 36 to 44% for moist sample and for air dried sample the value increased from 7 to 29%. Similar results have been obtained for other samples. These results clearly indicate that under natural conditions, a major portion of the clay content is in the form of flocs whose size is larger than clay size. These flocs are not broken by the conventional use of stirrers. Thus the dispersing

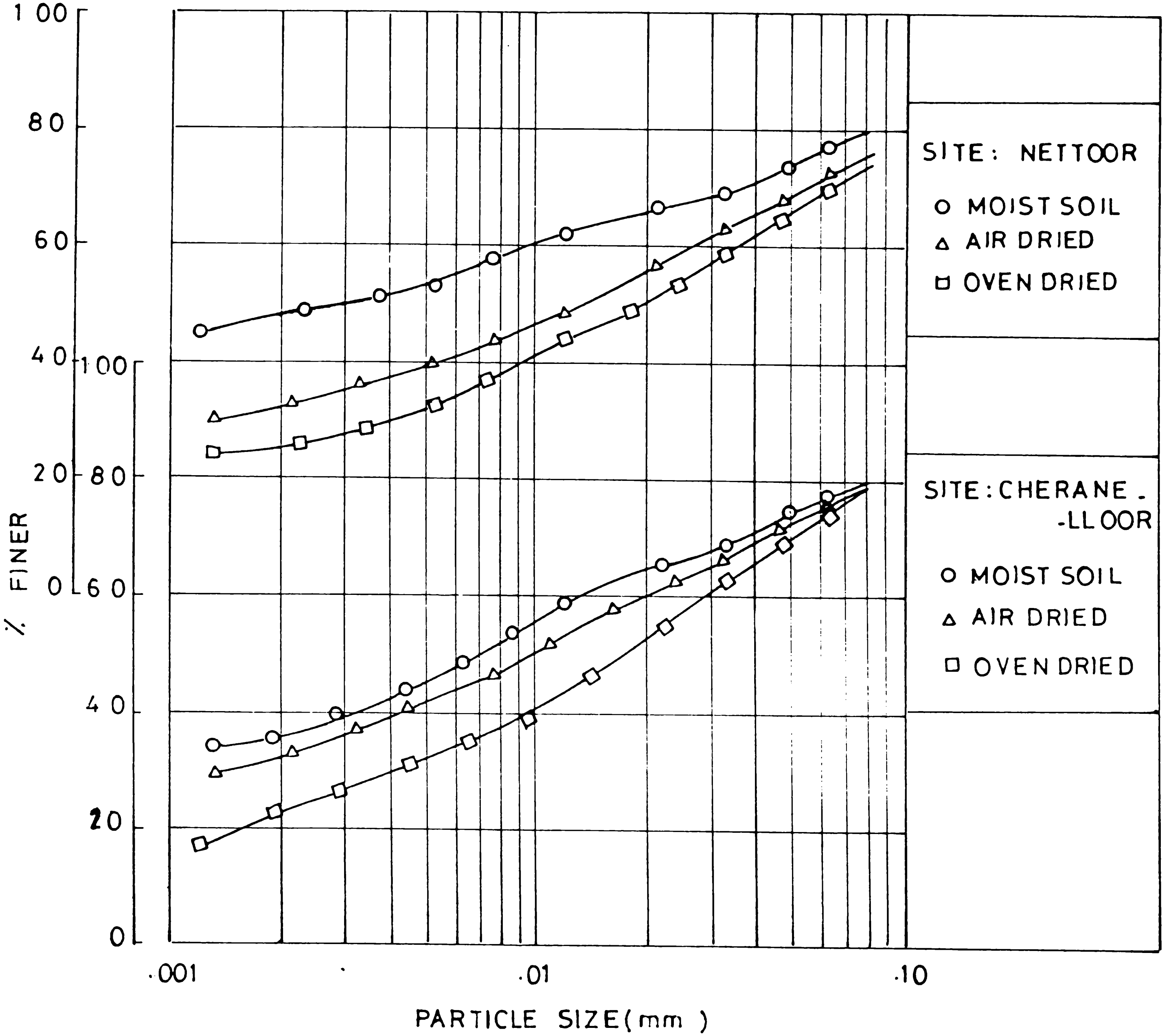


Fig. 4.4.1a. GRAIN SIZE DISTRIBUTION CURVES.

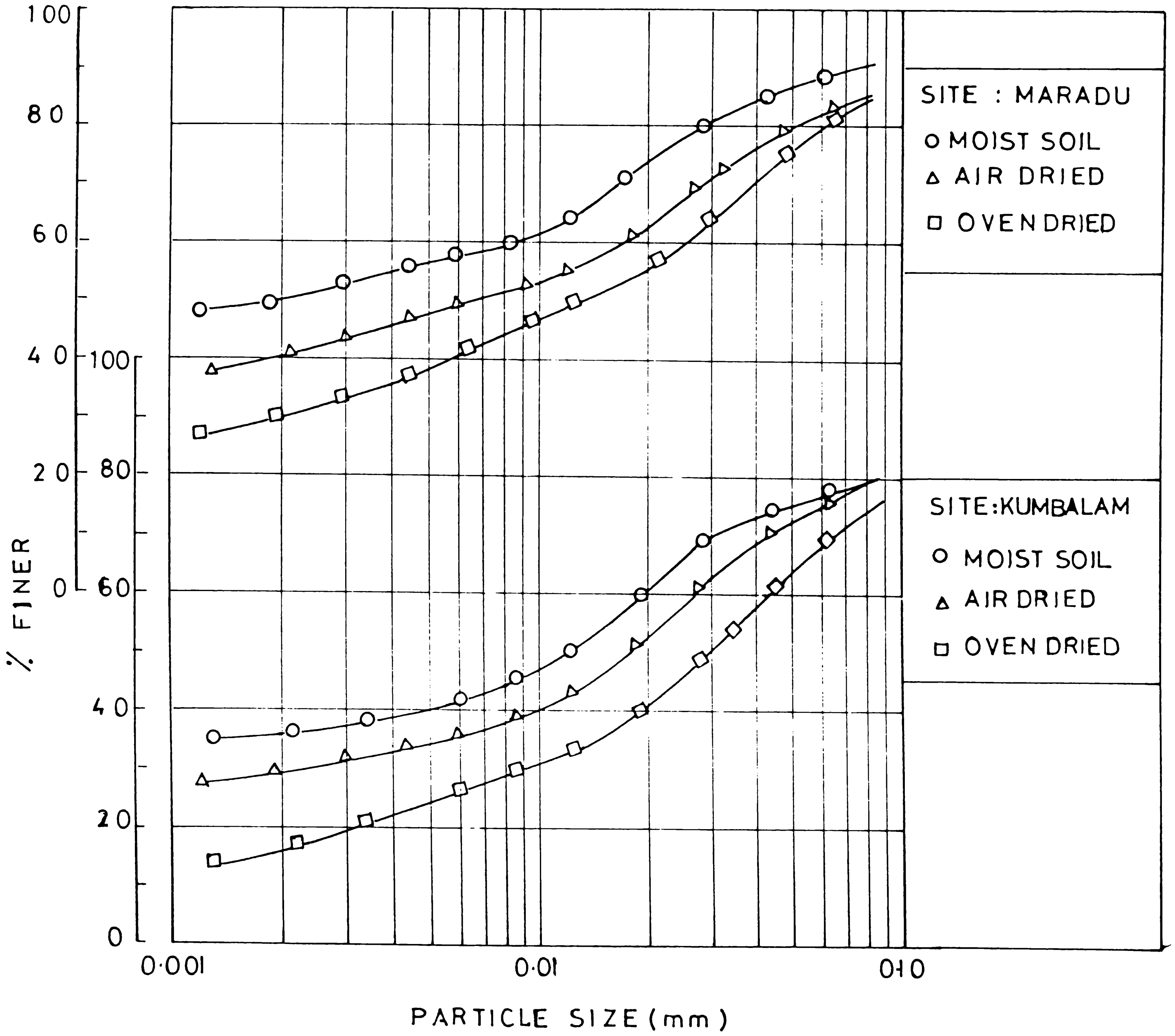


Fig. 4.4.1b. GRAIN SIZE DISTRIBUTION CURVES.

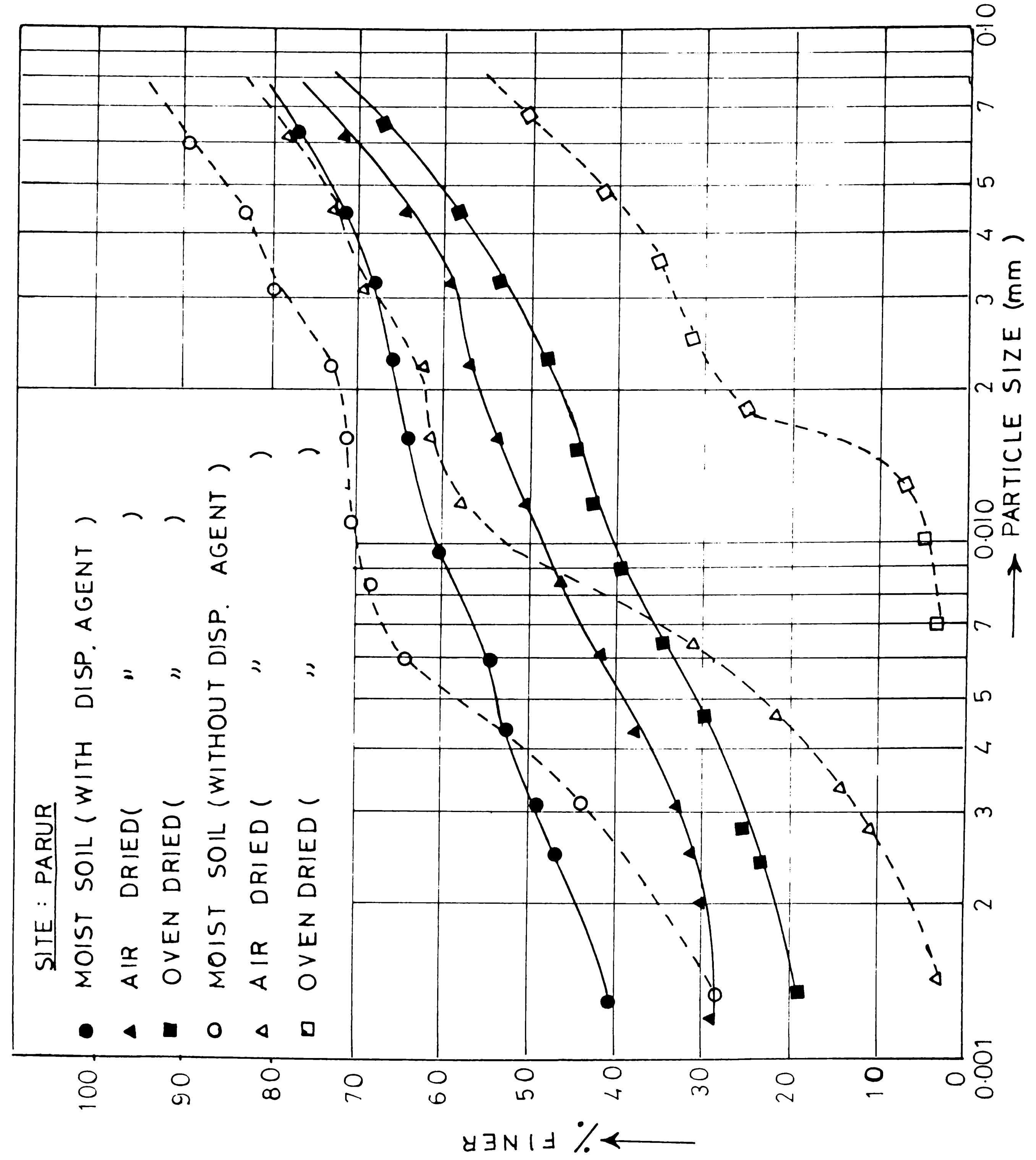


Fig. 4.4.1c. GRAIN SIZE DISTRIBUTION CURVES.

agents have a greater role to play in the grain size analysis by hydrometer method in case of marine clays.

4.4.1 Role of dispersing agents

Table 4.1 shows the wide variations in index properties between initially moist and initially air and oven dried samples. These results indicate that marine sediments require special attention with regard to test procedures for index properties, as the initial state of the sample - moist or dry - yields results that are significantly different. This prompted further investigations regarding the suitability of the normally accepted test procedures for the determination of index properties of marine clays. Aggregation being the primary cause for all these changes during drying, studies on grain size distribution of marine clays merit special attention. Fig.4.4.2 which shows the grain size distribution curves for the initially 'moist' and initially 'oven dried' samples underscores the necessity for the present investigation. They have been obtained by following strictly the Indian Standard Code of practice, which is the same as unified classification procedure, using soil samples whose equivalent dry weight is around 50 g. with 100 ml

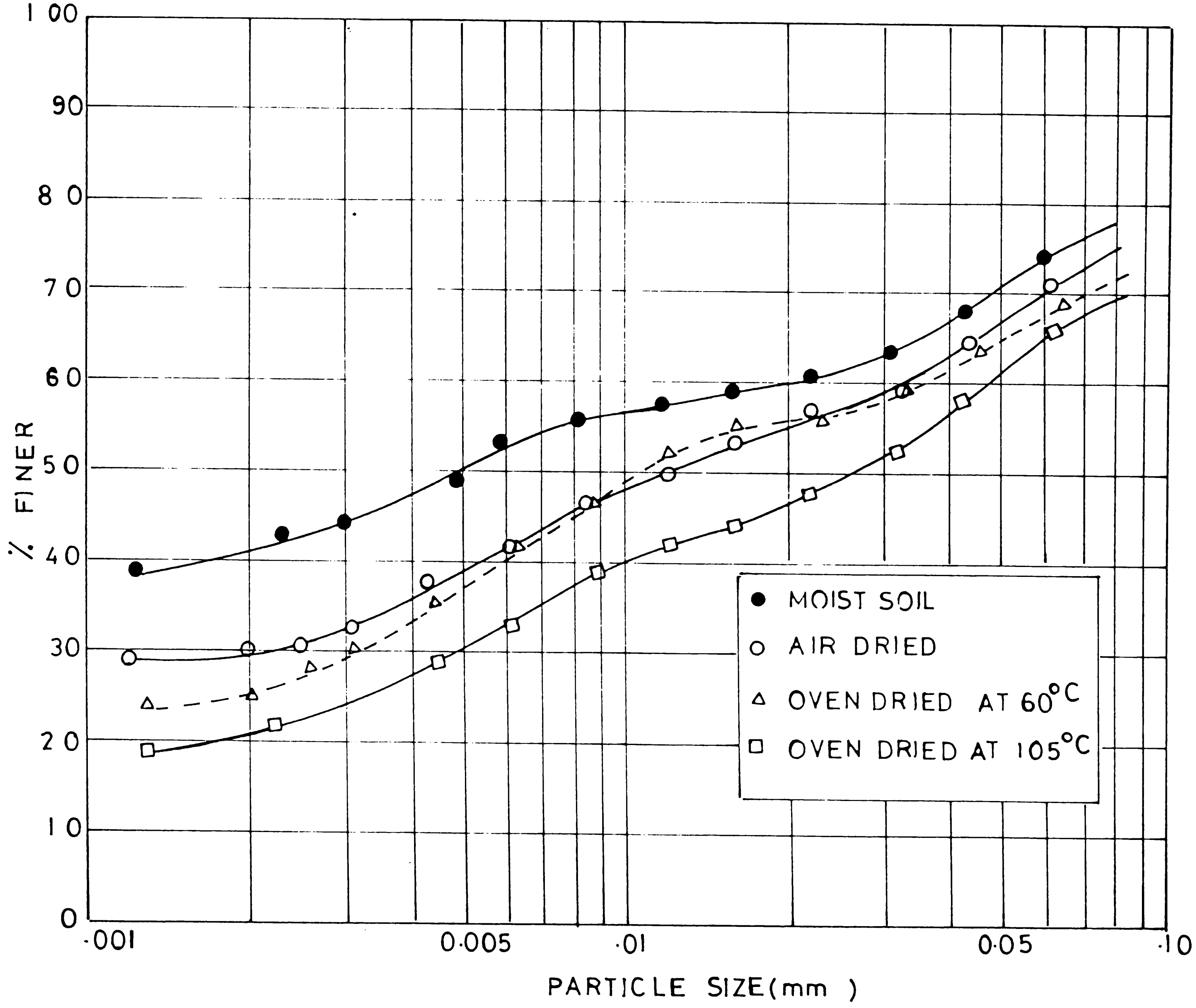


Fig. 4. 4. 2. EFFECT OF DRYING ON GRAIN SIZE DISTRIBUTION.

of standard dispersing agent (33 g. of sodium hexametaphosphate and 7 g. of sodium carbonate in 1 litre of distilled water). These curves are at considerable variance with each other and the grain size distribution curves of marine clay samples dried at lower temperatures (air dried and dried at 60°C) fall in between these two curves.

Since the entire clay fraction exists in the form of flocs, use of the type and quantity of dispersing agents plays a decisive role in obtaining the percent of clay size. In this work the most suitable dispersing agent has been identified and its optimum quantity has been arrived at.

Dispersing agents

A series of hydrometer analyses on Cochin marine clays were conducted with different dispersing agents and quantities. A deflocculating agent can either act as a protective colloid on the soil particle or alter the electrical charge on the particle to prevent the formation of soil flocs (Lambe, 1961). According to Lambe, two commonly used deflocculants are sodium silicate (water glass) and Daxad No.23 (polymerised sodium salts of substituted benzoid alkyl sulfonic acid). Lambe indicates that 2 cc of a freshly prepared 10% solution of Daxad No.23 has been found satisfactory for many clays. Likewise one-half to one cc of Baumae' sodium silicate has been found satisfactory for most clay types. Sodium pyrophosphate has also been used successfully. According to Parcher and Means (1965) the same dispersing agent will not work for all clays, but sodium silicate can be used successfully with many clays and is quite commonly used. Sodium hexametaphosphate is also frequently used as a dispersing agent.

More recently, Head (1980) states that numerous substances have been tried for use as dispersing agents, many of them very successfully for most soils. According to Head, sodium hexametaphosphate is one of the most suitable and convenient dispersants. The British standard recommends 35 g. of sodium hexametaphosphate with 7 g. of sodium carbonate along with distilled water to make 1 litre of standard solution. The Indian Standard code of practice recommends 33 g. of sodium hexametaphosphate with 7 g. of sodium carbonate in distilled water to make 1 litre of standard solution. Both codes recommend 100 ml of this standard solution for each experiment.

In this investigation eight dispersing agents each 40 g. in one litre of distilled water were tried. They are:

1. Sodium chloride
2. Sodium phosphate
3. Sodium carbonate
4. Sodium nitrate
5. Sodium sulphate
6. Sodium bicarbonate
7. Sodium hexametaphosphate
8. Sodium hexametaphosphate + Sodium carbonate

Fig.4.4.3a and b shows the grain size distribution curves along with percent clay size obtained using different dispersing agents. It can be seen that the percent clay size varies from about 45% to 4% depending upon the type of the dispersing agent used. To obtain percent clay size, material finer than 2 μm size (.002 mm) has been designated as clay. Further, it can be seen that sodium hexametaphosphate + sodium carbonate, which has been recommended by many, has given the maximum percent clay size.

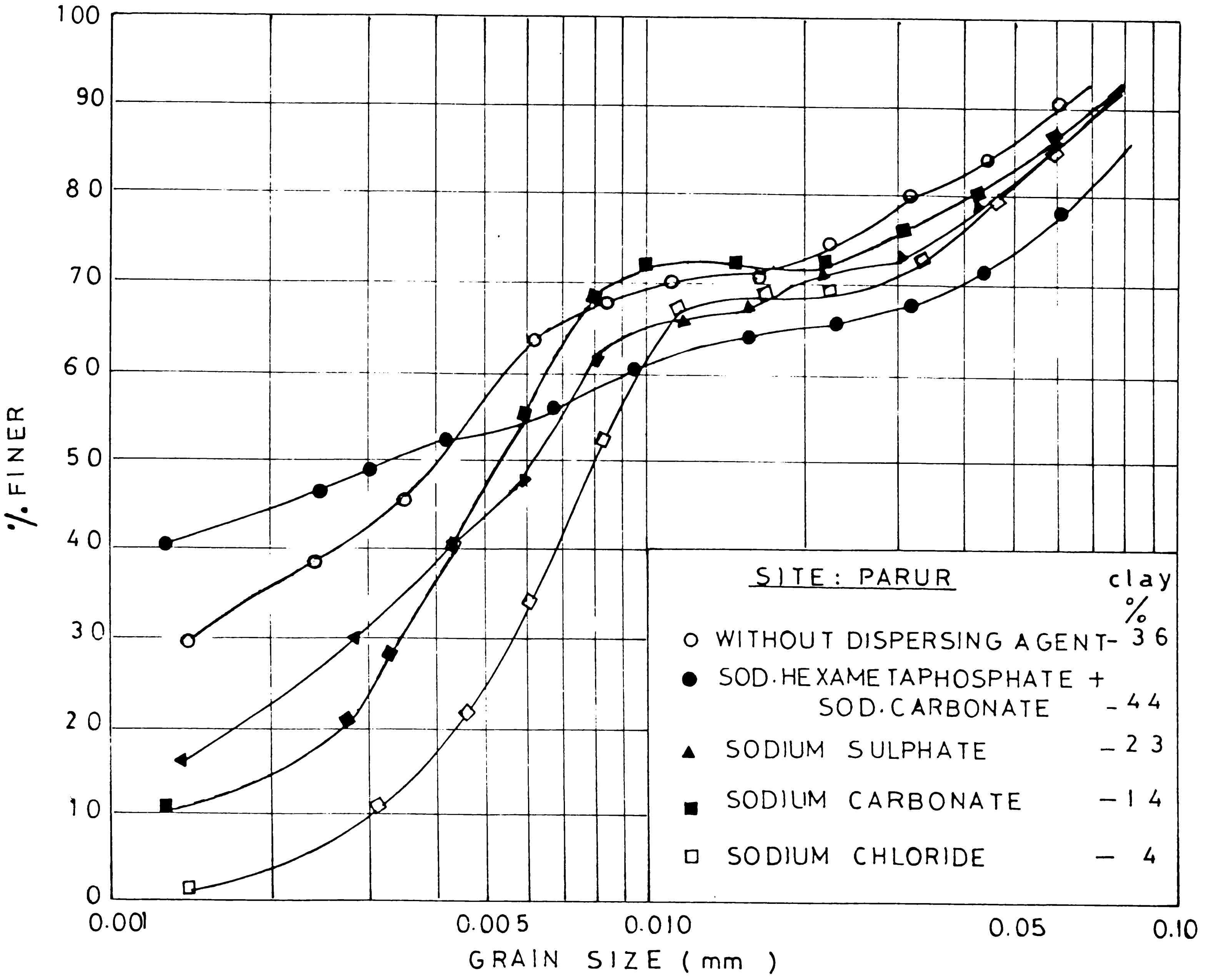
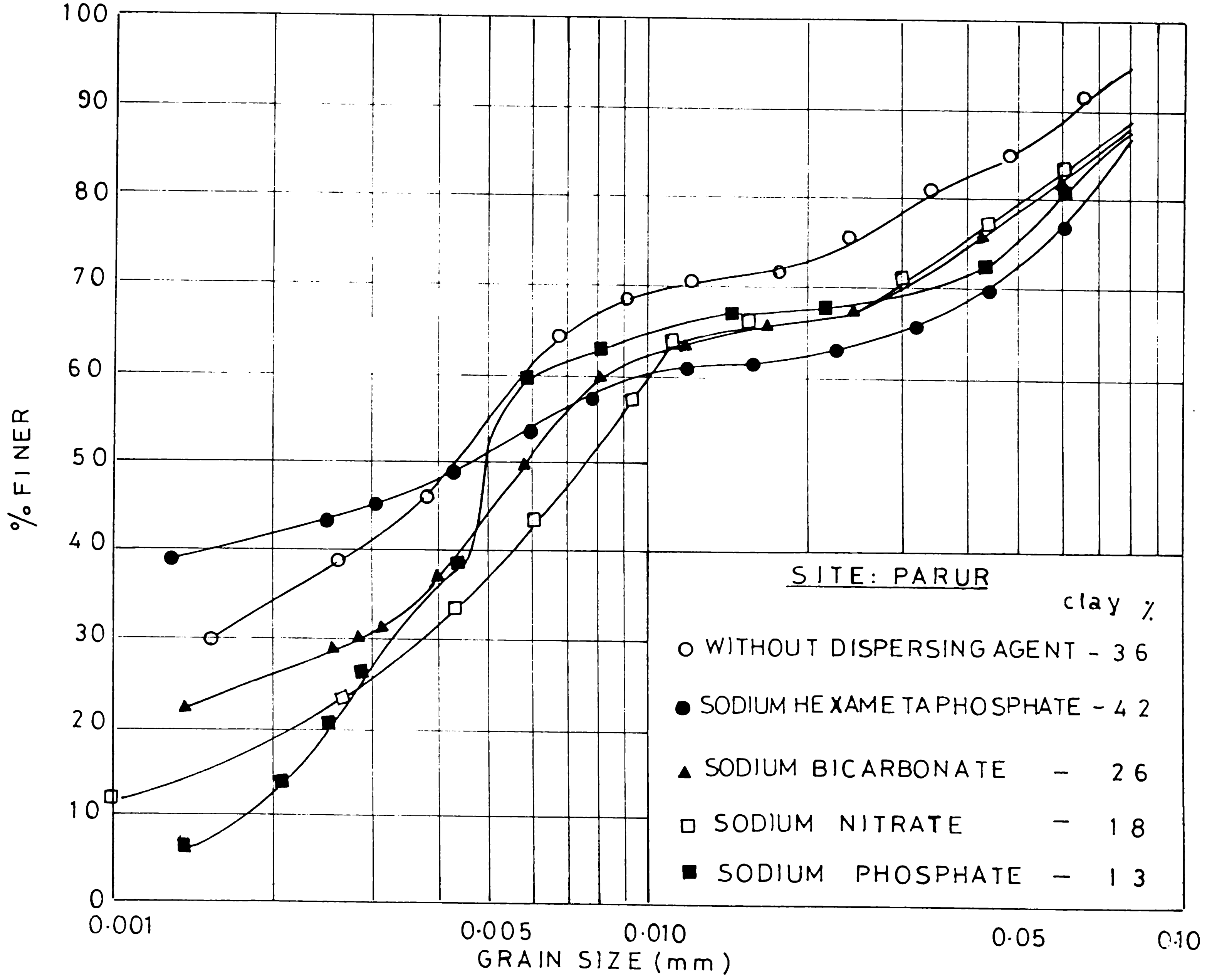


Fig. 4.4.3a. EFFECT OF DISPERSING AGENTS.


100
90
80
70
60
50
40
30
20
10
0
% FINER
0·001
0·005
0·010
0·05
0·10
GRAIN SIZE (mm)
SITE: PARUR
clay %
○ WITHOUT DISPERSING AGENT - 36
● SODIUM HEXAMETAPHOSPHATE - 42
▲ SODIUM BICARBONATE - 26
□ SODIUM NITRATE - 18
■ SODIUM PHOSPHATE - 13

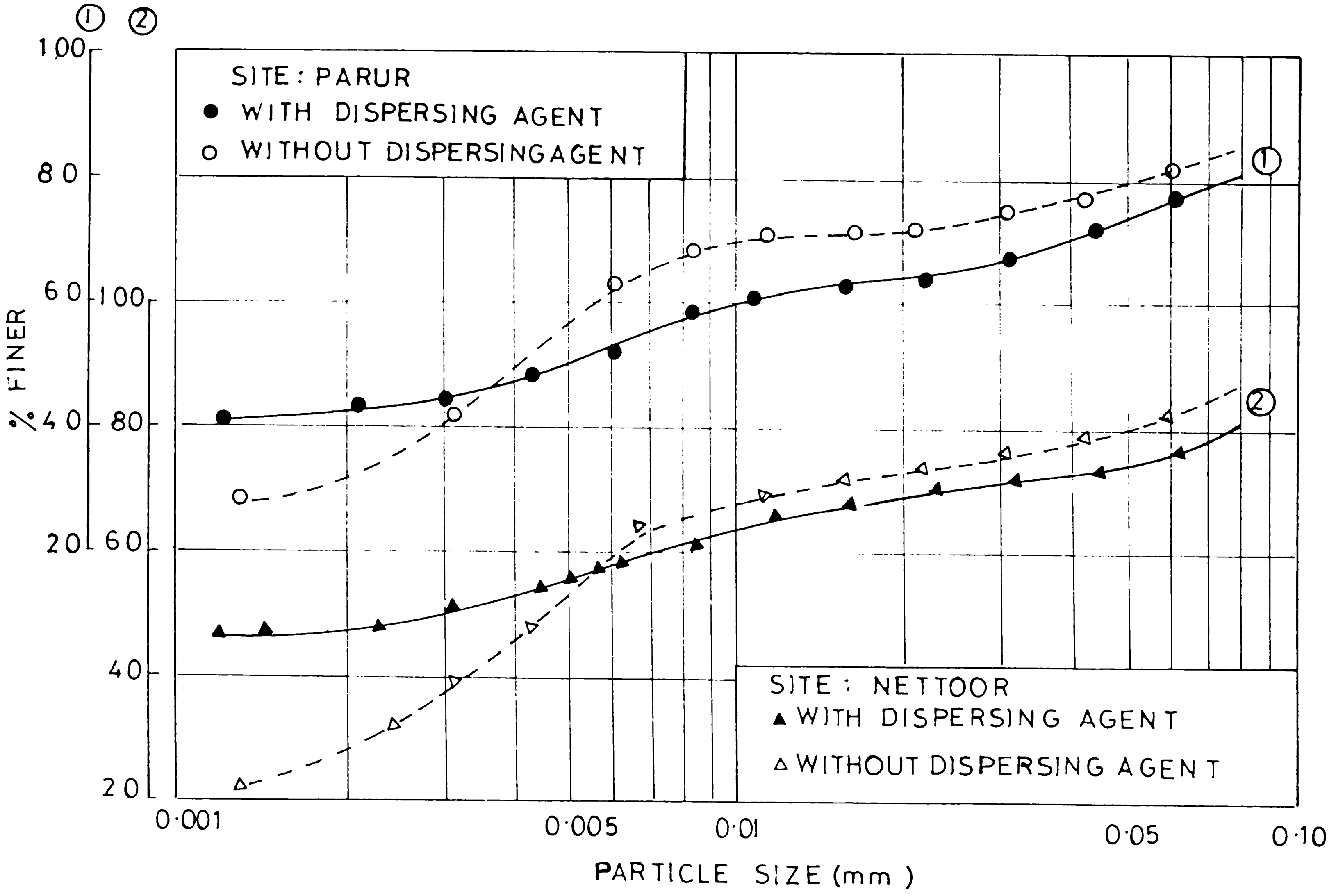


Fig. 4.4.4 EFFECT OF STANDARD DISPERSING AGENT.

In order to study whether the quantity of the dispersing agent has any effect on the percent clay size, experiments were conducted with different quantities of this standard dispersing agent.

Experiments without dispersing agents

In order to obtain more insight into the problem, experiments were also carried out without any dispersing agent. Fig.4.4.4 shows some typical grain distribution curves with and without the use of dispersing agent for the marine clays from two sites of Cochin.

The test without dispersing agent showed that after two or three hours, the top portion was completely clear and devoid of soil particles indicating a near total absence of colloids. There was a clear horizontal surface dividing the clean water above and soil suspension below, slowly descending with time. In contrast, experiments on kaolinite and bentonite clays without dispersing agents show presence of colloids throughout the suspension. There was no clear supernatant water as was noticed for the marine clay.

Fig.4.4.5 shows the different stages of the soil suspension in a hydrometer test on Cochin marine clay. Fig.4.4.5(b) shows the photograph of a hydrometer test on marine clay using standard dispersing agent, after 4 hours. Eventhough there is sedimentation of grains, the soil suspension covers the entire volume even after the elapse of days, since the colloids will not settle. Fig.4.4.5(c) and (d) show the stages after 4 and 8 hours respectively for the same clay where no dispersing agent was used. In both cases, it can be seen that the supernatant water is absolutely clear of soil grains including colloids. Fig. 4.4.5(a) shows the photograph of a hydrometer test on a bentonite sample without the use of dispersing agent after 4 hours. A comparison between bentonite and marine clay clearly shows that the entire clay fraction in marine clays exists in the form of flocs.

It can be clearly seen from Fig.4.4.4 that the grain size distribution curves are entirely different for samples with and without dispersing agents. This showed that dispersing agents have a greater role to play in case of marine clays. Since marine clays required more deflocculation or dispersion than other clays, it was felt that there is a need to determine the optimum quantity of the dispersing agent to be used.

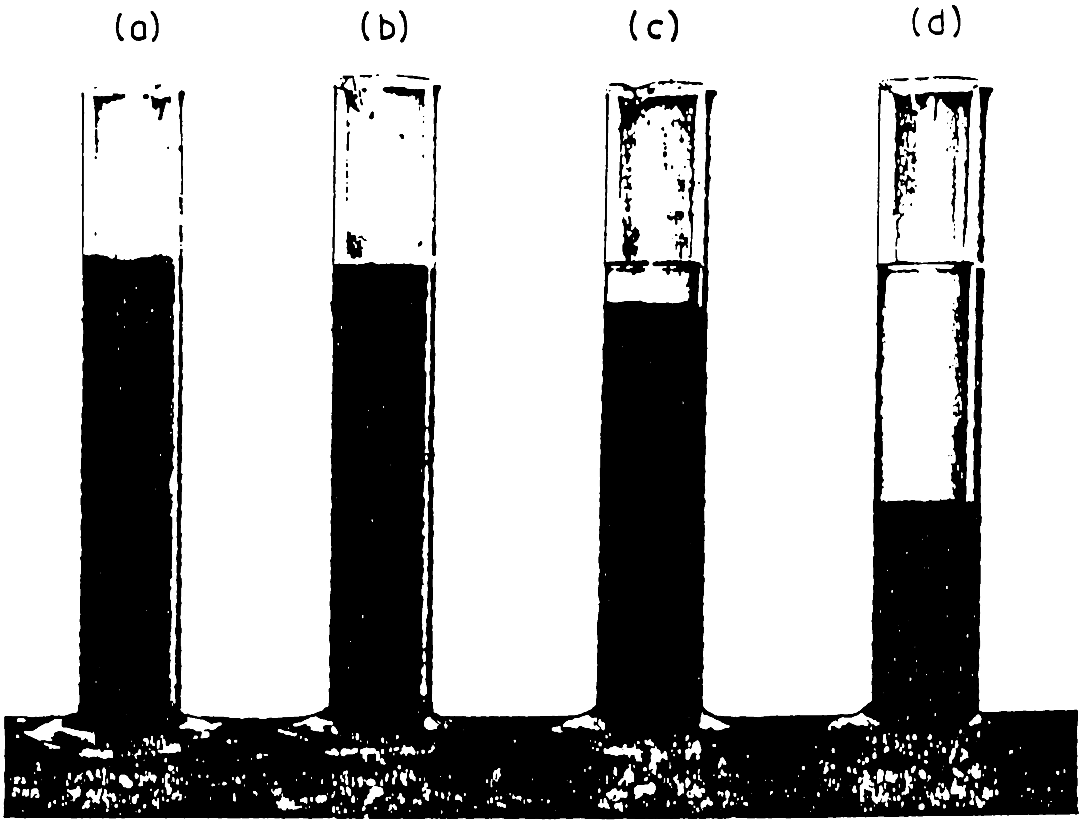


Fig 4·4·5 : EFFECT OF DISPERSING AGENT·

(a) Bentonite without dispersing agent – after 4 hours

(b) Marine clay with dispersing agent – after 4 hours

(c) Marine clay without dispersing agent – after 4 hours

(d) Marine clay without dispersing agent – after 8 hours

Optimum quantity of dispersing agent

Hydrometer tests were conducted with different quantities of standard dispersing agent (33 g. of sodium hexametaphosphate + 7 g. of sodium carbonate in 1000 ml of distilled water) varying from 0 to 250 ml, using moist marine clay whose equivalent dry weight was about 50 gms. Fig.4.4.6 shows the plot between quantity of dispersing agent and clay size fraction. At or slightly above 100 ml, maximum dispersion of the soil particles as indicated by the highest clay percent was achieved. It may be inferred that beyond the optimum value, aggregation of clay particles may take place resulting in a reduction of clay fraction. Similar experiments for a montmorillonitic clay (black cotton soil) showed that the changes in clay percent beyond 100 ml of standard dispersing agent is marginal (Fig.4.4.6).

Addition of dispersing agent is essential for obtaining the clay percent for marine clays in which clay particles exist in the form of flocs. They have to be tested in the natural moist condition without initially air/oven drying. Tests with different dispersing agents clearly indicate that the percent clay size can vary significantly depending upon the type of dispersing agent. The popular

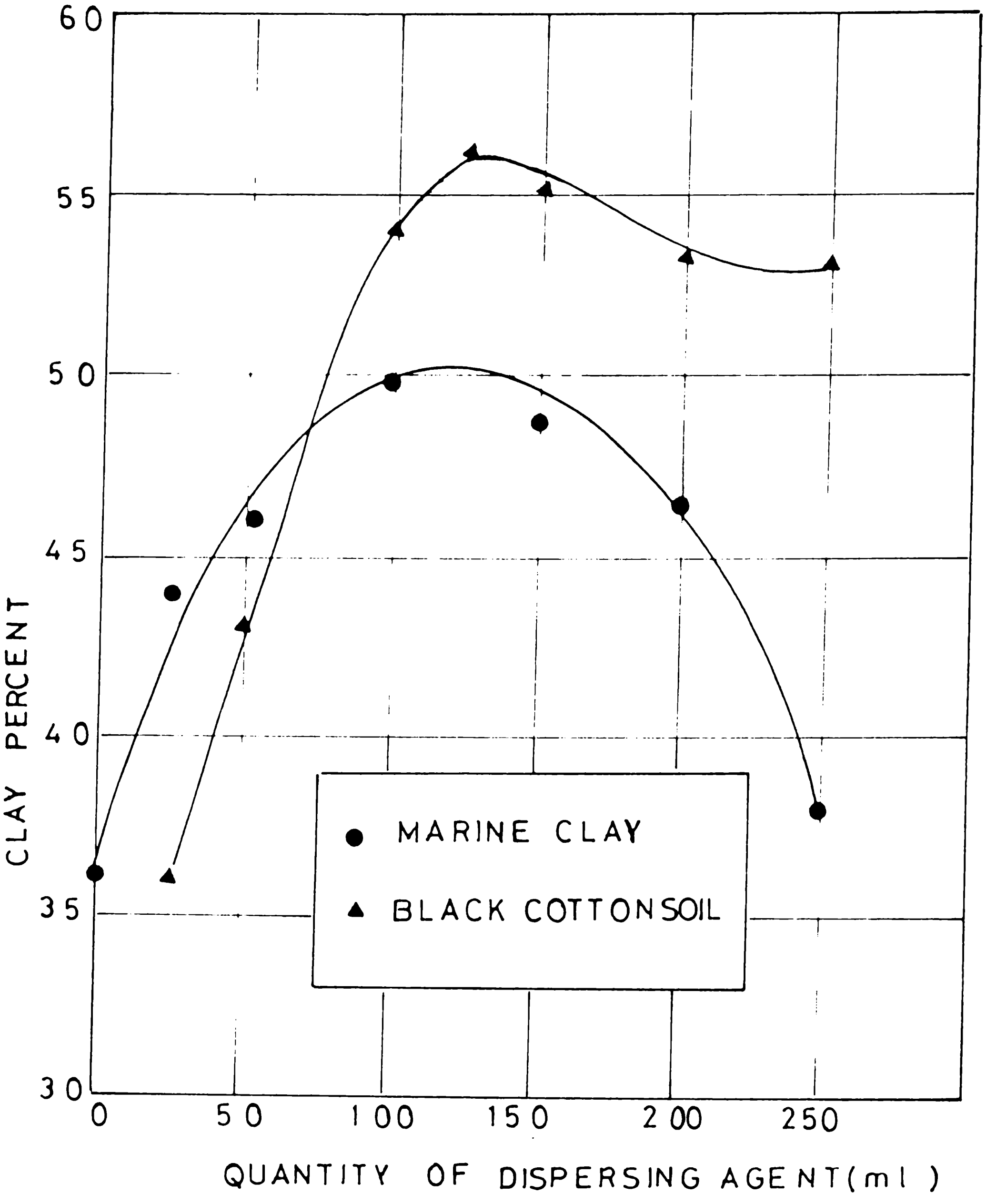


Fig. 4.4.6 : EFFECT OF QUANTITY OF DISPERSING AGENT.

dispersing agent viz., the standard solution of 33 g. of sodium hexametaphosphate and 7 g. of sodium carbonate in one litre of distilled water, is found to be the most efficient one to determine the clay size fraction. 100 ml of this solution is found to be the optimum quantity of dispersing agent which gave the highest percent of clay size.

4.4.2 Effect of increase of coarser fraction on physical properties

All the tests for physical properties so far were done on samples obtained from field. These soils had a range for the percentages of sand, silt and clay as shown in Table 4.1. It is possible that marine clays with higher percentages of sand or clay may be encountered during soil explorations. In order to get an idea of the engineering behaviour of Cochin marine clays with higher sand or clay contents, four soil samples were prepared in the laboratory from moist samples. Water in large quantities were added to 'moist' soil in a bucket A and mixed thoroughly. The mixture was permitted to sediment for a few minutes so that the coarser particles settled at the bottom of the bucket. The supernatant solution was poured to another bucket B.

Table 4.4.2.1

PHYSICAL PROPERTIES OF SAMPLES PREPARED BY SEDIMENTATION

Sl. No.	Description of samples	Grain size distribution			Liquid limit (%)	Plastic limit (%)	Shrinkage limit (%)	Plasticity index (%)	Activity	Free swell index (cc/gm)	Cation exchange capacity meq/100g.
		Clay (%)	Silt (%)	Sand (%)							
1.	Natural soil										
	(a) moist	35	29	36	82.0	35.6	20.4	46.4	1.33	3.25	44.1
	(b) air dried	27	37	36	69.5	33.4	18.6	36.1	1.33	1.70	--
	(c) oven dried	18	43	39	52.0	29.1	19.8	22.9	1.27	1.45	--
2.	Sedimented soil A	24	20	56	56.0	24.7	22.6	31.3	1.25	2.74	28.3
3.	Sedimented soil B	40	28	32	86.7	34.4	20.8	52.3	1.30	3.87	48.7
4.	Sedimented soil C	42	38	20	103.5	38.8	19.5	64.7	1.50	4.56	52.2
5.	Sedimented soil D	49	36	15	113.0	44.0	18.1	69.0	1.38	4.85	55.0

The process was repeated with the soil-water mixture in bucket B and the solution above was poured to bucket C. The process was repeated once more and the finest soil was collected in bucket D. After allowing sufficient time for sedimentation, four different soil specimens were obtained in buckets A,B,C and D, by pouring out the supernatant clear water without any loss of fines.

The index properties of the four samples are presented in Table 4.4.2.1 The results clearly underscore the soil behaviour discussed so far. The liquid limit increases with clay content. In soil A the clay content is only 24% and liquid limit is 56%. In soil D the clay content is 49% and this increases the liquid limit value to 113%. Corresponding variations in plastic limit and plasticity index can be observed in the Table.

The shrinkage limit decreases with increase in clay content. Hence the argument that higher shrinkage limit indicates greater fraction of coarser particles is fully substantiated. The free swell index increase from 2.74 to 4.85 cc/gm. The cation exchange capacity significantly improves with increasing clay size fraction.

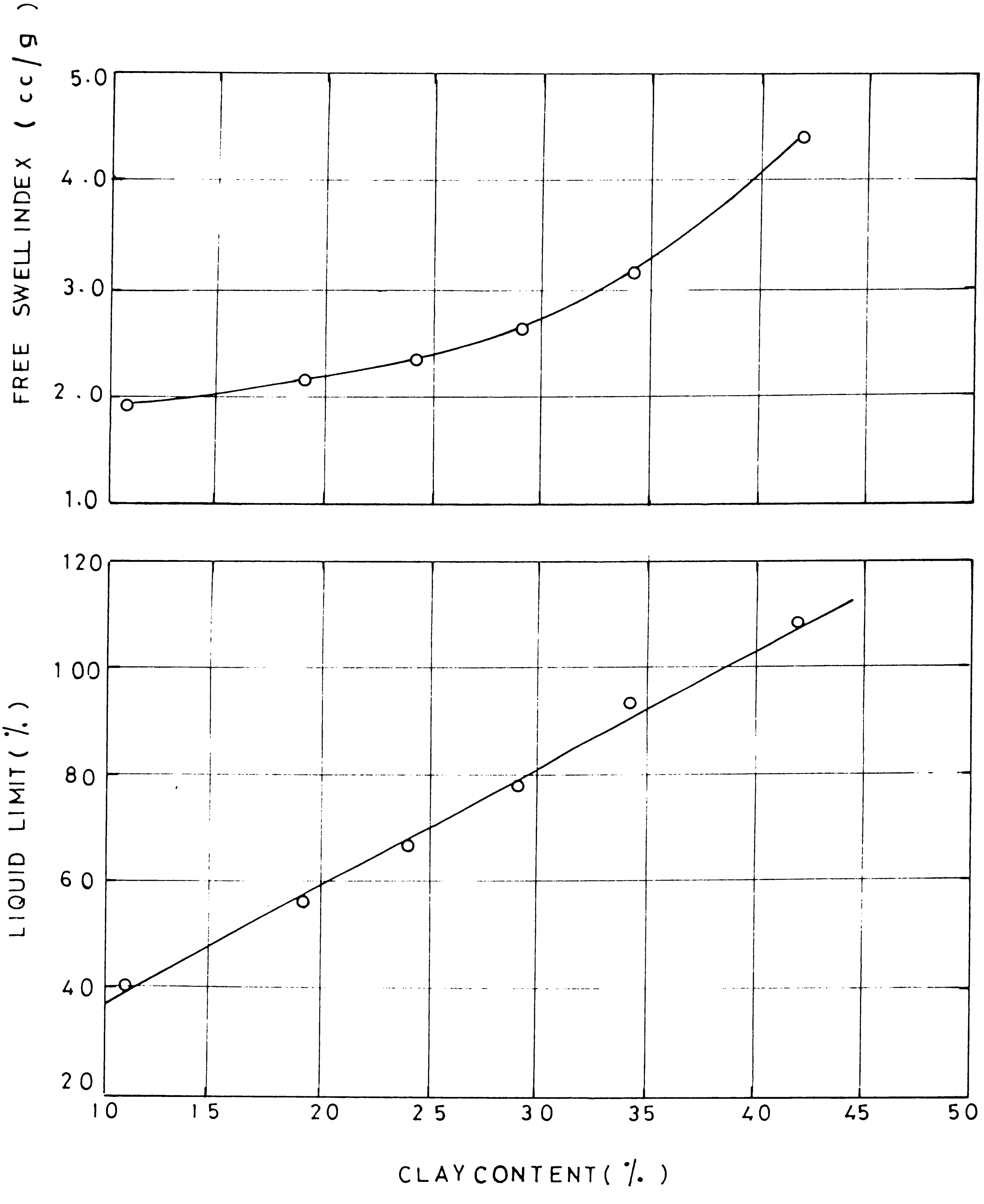


Fig. 4. 4. 2.1. EFFECT OF INCREASE IN COARSER FRACTION.

The content of coarser particles could also be increased by adding fine sand in small quantities. Table 4.4.2.1 gives the variation in physical properties on addition of fine sand. As expected the Atterberg limits and free swell index decreases with decreasing clay content.

Fig.4.4.2.1 gives the variation of liquid limit and free swell index with clay content.

4.5 ACTIVITY OF COCHIN MARINE CLAYS

To compare the clay fraction of different clay minerals Skempton (1953) introduced the term activity which is the ratio of plasticity index and clay fraction. It is found that with increasing activity, the cohesive component of shear resistance increases whereas the frictional component is reduced (Narain and Ramanathan, 1970). Gibson has studied the variation of cohesive and frictional components with activity (Trollope, 1960).

Bjerrum (1954) showed through his studies on Norwegian marine clays the usefulness of the average activity for correlation with index properties. Fig.4.5.1a shows the plot of plasticity index vs. clay percent for moist soil. If the plasticity index and clay size fraction is plotted for a number of marine clay samples, a straight line can be drawn through these points and the origin. The slope of the line gives the average activity of these clays. The average activity of moist soils is 1.63. Fig. 4.5.1b shows the average activity lines for air dried and oven dried soils.

Fig.4.5.2 shows the activity chart for moist, air dried and oven dried soils. The activity chart has been divided into 'active', 'normal' and 'inactive' zones as shown in the figure.

The activity of the moist samples range over 1.37 to 2.47 which could be classified as active clays (Skempton, 1953). Cochin marine clays become normal or inactive on oven drying. It is needless to state that the colloidal activity gets diminished on air and oven drying.

As it has been noticed in the case of liquid limit vs. clay percent, one can get a unique relationship between plasticity index and clay percent for moist, air dried and oven dried samples altogether. Fig.4.5.2 gives

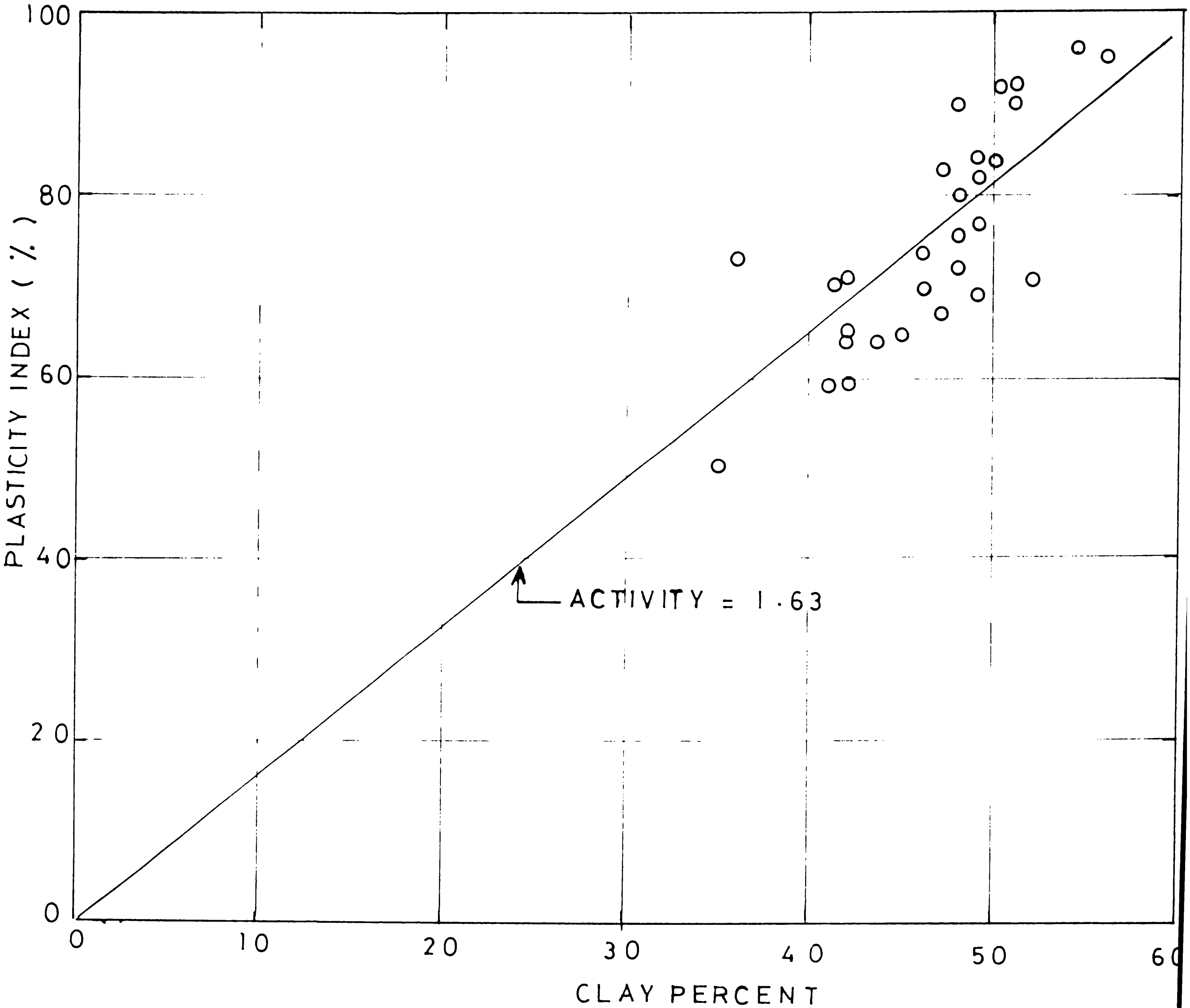


Fig. 4.5.1a: ACTIVITY CHART FOR MOIST SOIL.

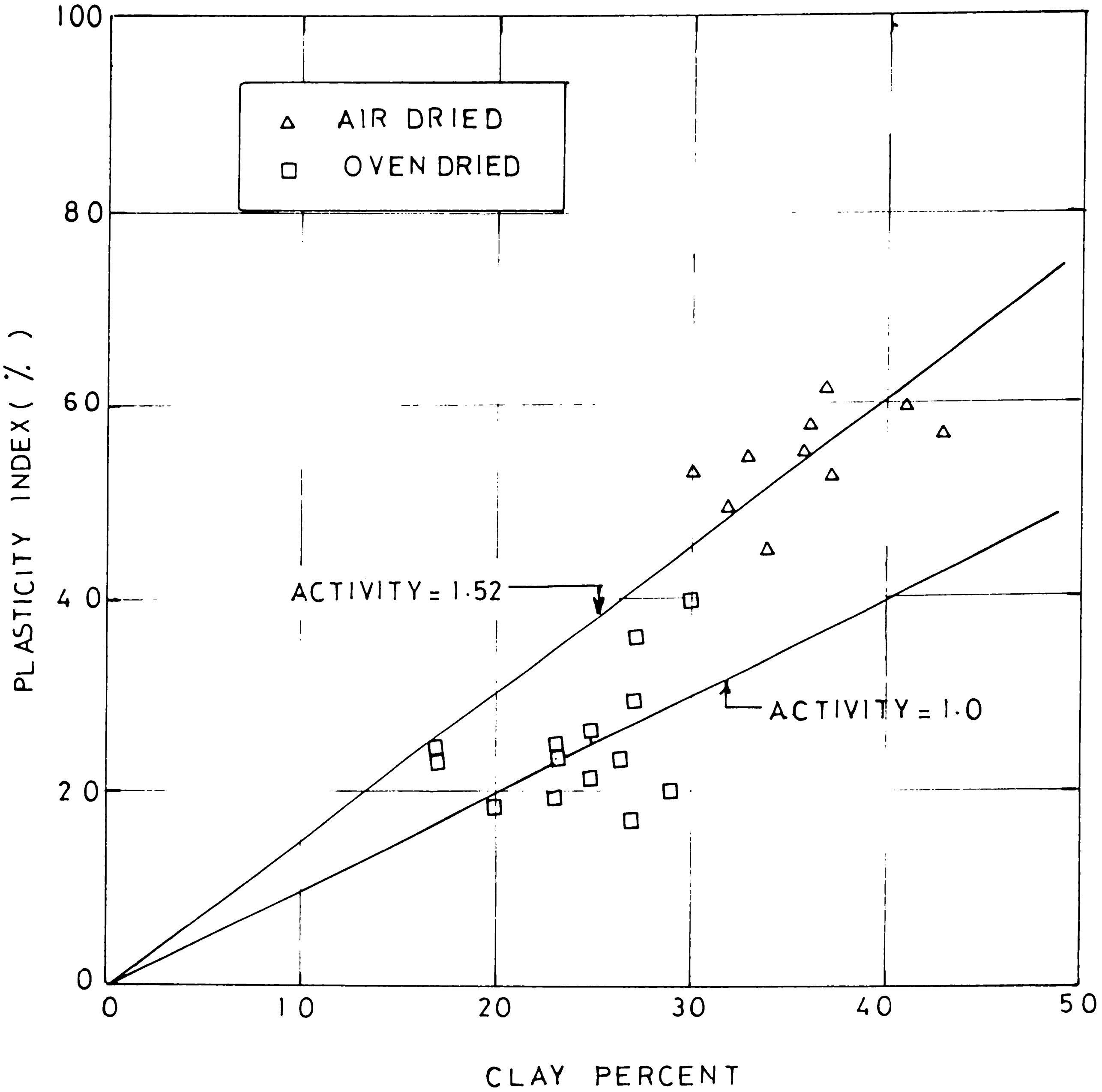


Fig. 4.5.1b : ACTIVITY CHART FOR AIR DRIED AND OVEN DRIED SOILS

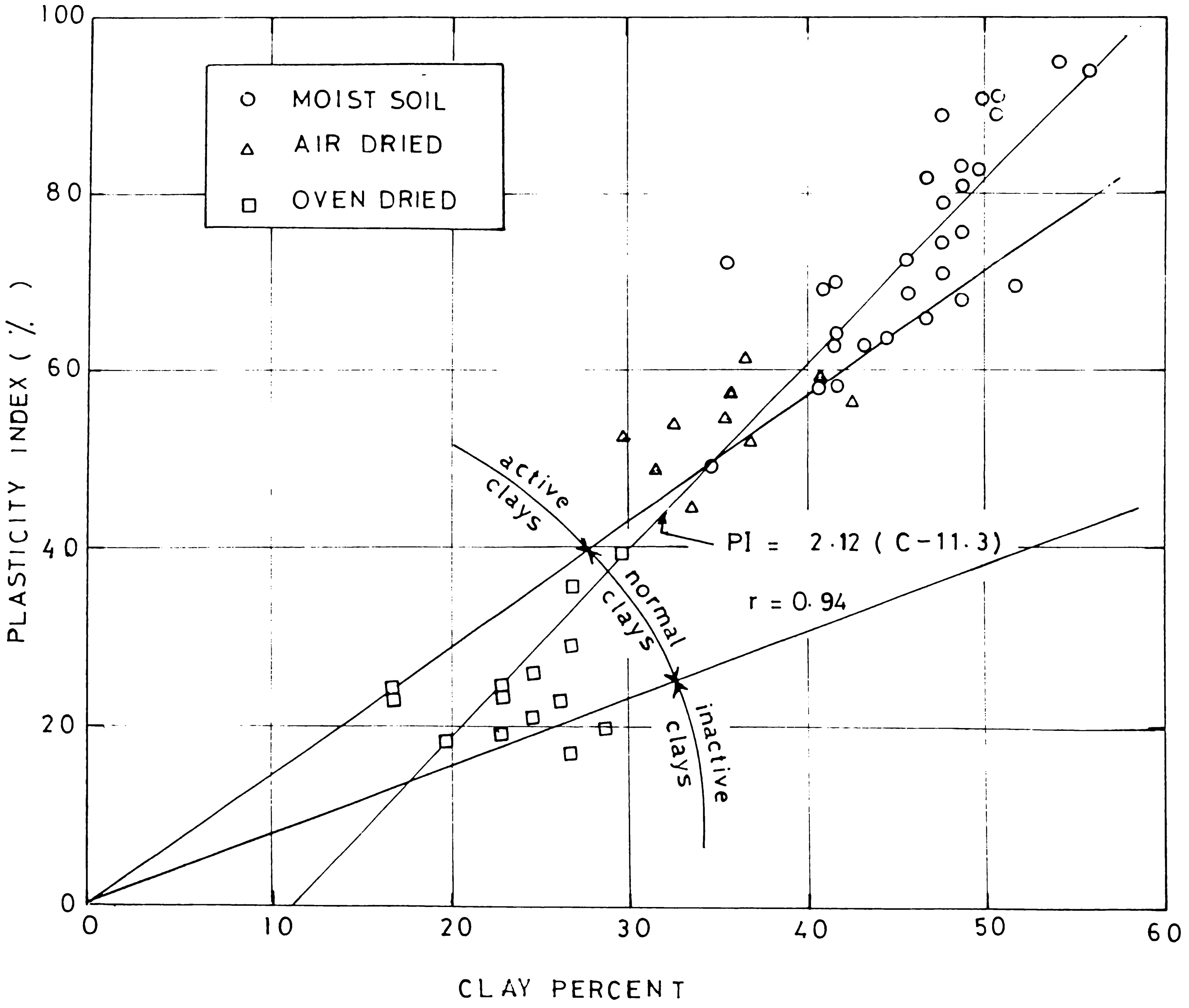


Fig. 4.5·2 ACTIVITY CHART.

this plot. The statistical fit for the data is

$$PI = 2.12(C-11.3) \qquad (4.5.1)$$

with a correlation coefficient of 0.94.

4.6 FREE SWELL INDEX

Free swell of a soil is defined (Indian Standard 2720 (Part XL) 1977) as the increase in volume of a soil without any external constraint, on submergence in water. The specimen for the test, as detailed by the code, is 10 g. of oven dry soil passing through 425 micron IS sieve. The disadvantages of this experimental procedure and its results have been discussed in detail by Sridharan et al., (1985). The problems are much more complex in case of marine clays as drying of the soil specimens would completely alter the grain size distribution and activity of the soil samples. Hence the values of swelling index of the marine clay samples reported in this work have been obtained by the procedure suggested by Sridharan et al. (1985) where

$$\text{Free Swell Index} = \frac{V_d}{10} \text{ cc/gm} \qquad (4.6.1)$$

where V_d = the volume of 10 gm of soil specimen read from the graduated cylinder containing distilled water.

Table 4.1 reports the free swell index values as defined by Sridharan et al. for samples from various locations. The equilibrium swollen volume of the soil without any restraint in distilled water was reached in about 7 days and the dry weight was determined later as the experiment had to be started with a 'moist' specimen whose equivalent dry weight was 10 gm approximately.

The value was found to vary from 3.0 to 5.9 cc/gm for moist samples of Cochin marine clays. Similar tests, carried out for kaolinite, and bentonite resulted in free swell index values of 1.45 and 6.7 cc/gm respectively. As per their classification (Sridharan et al., 1985) Cochin marine clays could be termed as moderately swelling.

Drying reduces the value of free swell index considerably. On air drying the value for Nettoor clay reduces from 5.9 to 2.2 cc/gm (62%) and to 1.5 cc/gm (74.5%) upon oven drying. These values show that the swelling characteristics in case of marine clays can be brought down significantly by drying. This fact can be taken advantage of in constructions on and with marine clay soils which has been discussed later.

Fig.4.6.1a and b give the relation between the percentage of clay fraction and free swell of moist soils and free swell of all specimens respectively.

The correlation between liquid limit and free swell is shown in Fig.4.6.2a and b.

Statistical fits can be obtained for all the four curves and they are given below.

For moist soil

$$\text{Free Swell} = 0.12(C-1.2) \qquad (4.6.1a)$$

$r = 0.89$

For moist, air dried and oven dried soils

$$\text{Free Swell} = 0.13(C-9) \qquad (4.6.1b)$$

$r = 0.86$

In terms of liquid limit values,

For moist soil

$$\text{Free Swell} = 0.043(LL-1.7)$$

with $r = 0.89$

For moist, air dried and oven dried soils together

$$\text{Free Swell} = 0.05(LL-24.4)$$

The correlation coefficient is 0.89.

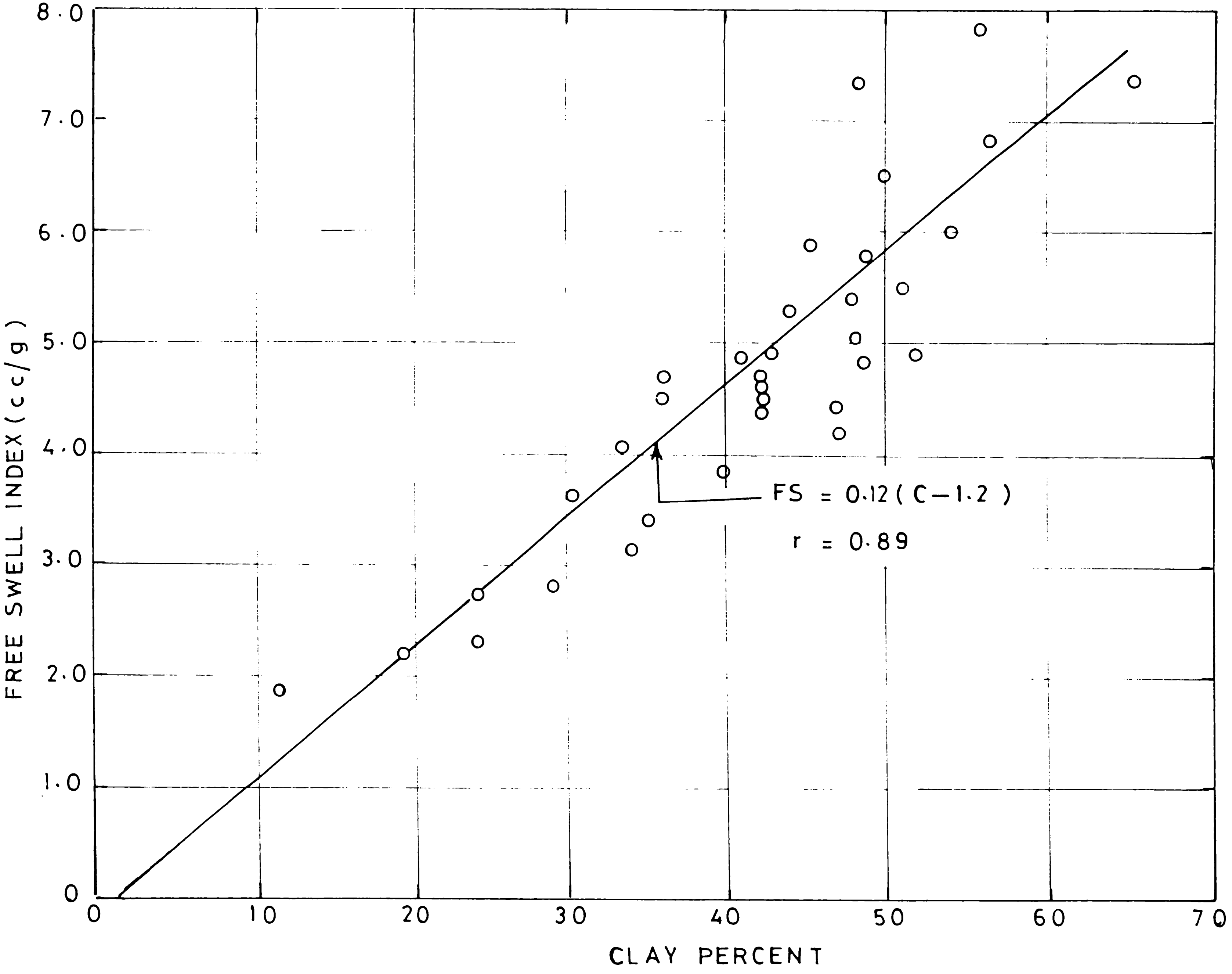


Fig. 4.6.1a: FREE SWELL INDEX vs CLAY PERCENT FOR MOIST SAMPLES.

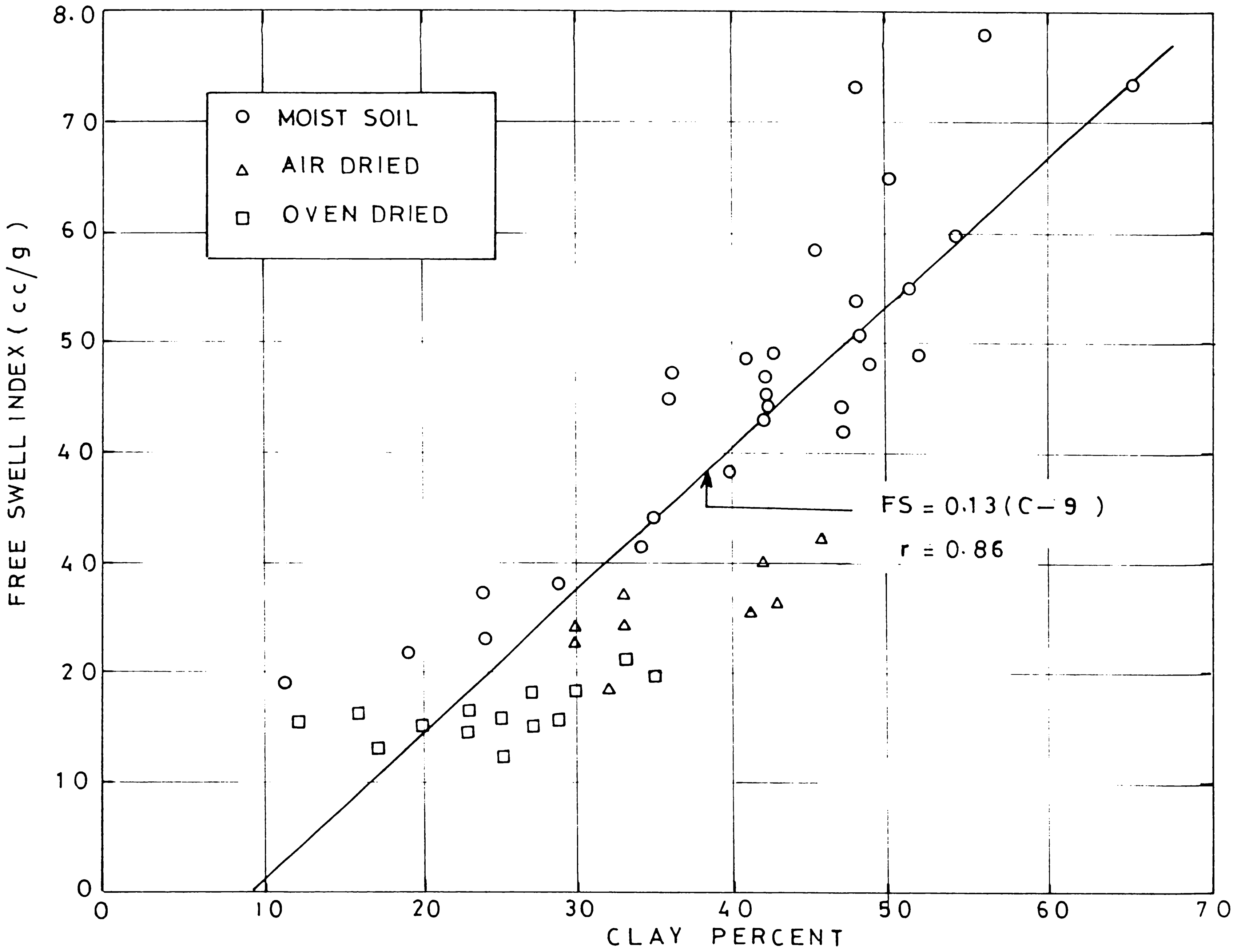


Fig 4.6.1b : FREE SWELL INDEX vs CLAY PERCENT.

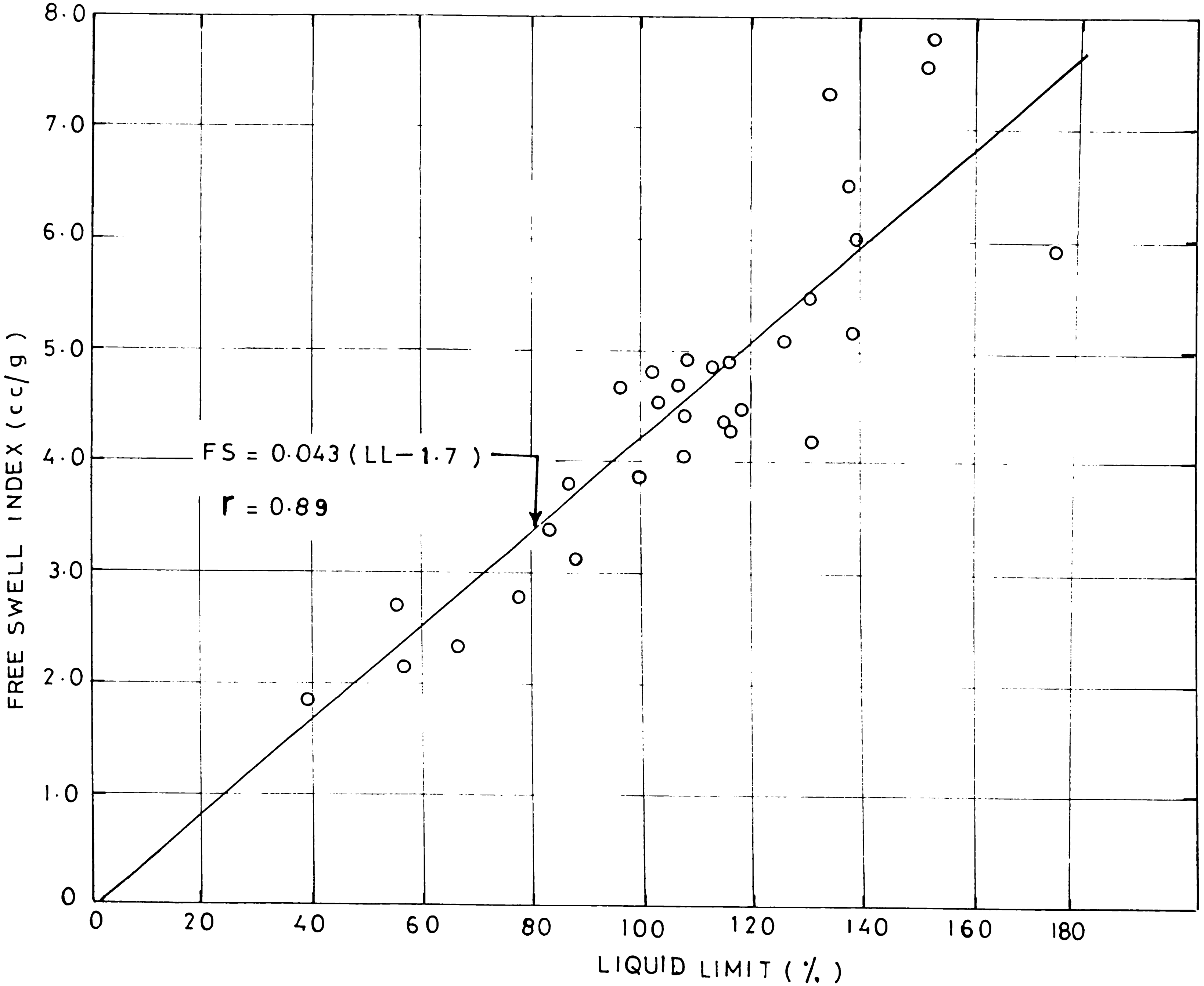


Fig. 4.6.2a: FREE SWELL INDEX vs LIQUID LIMIT FOR MOIST SOIL.

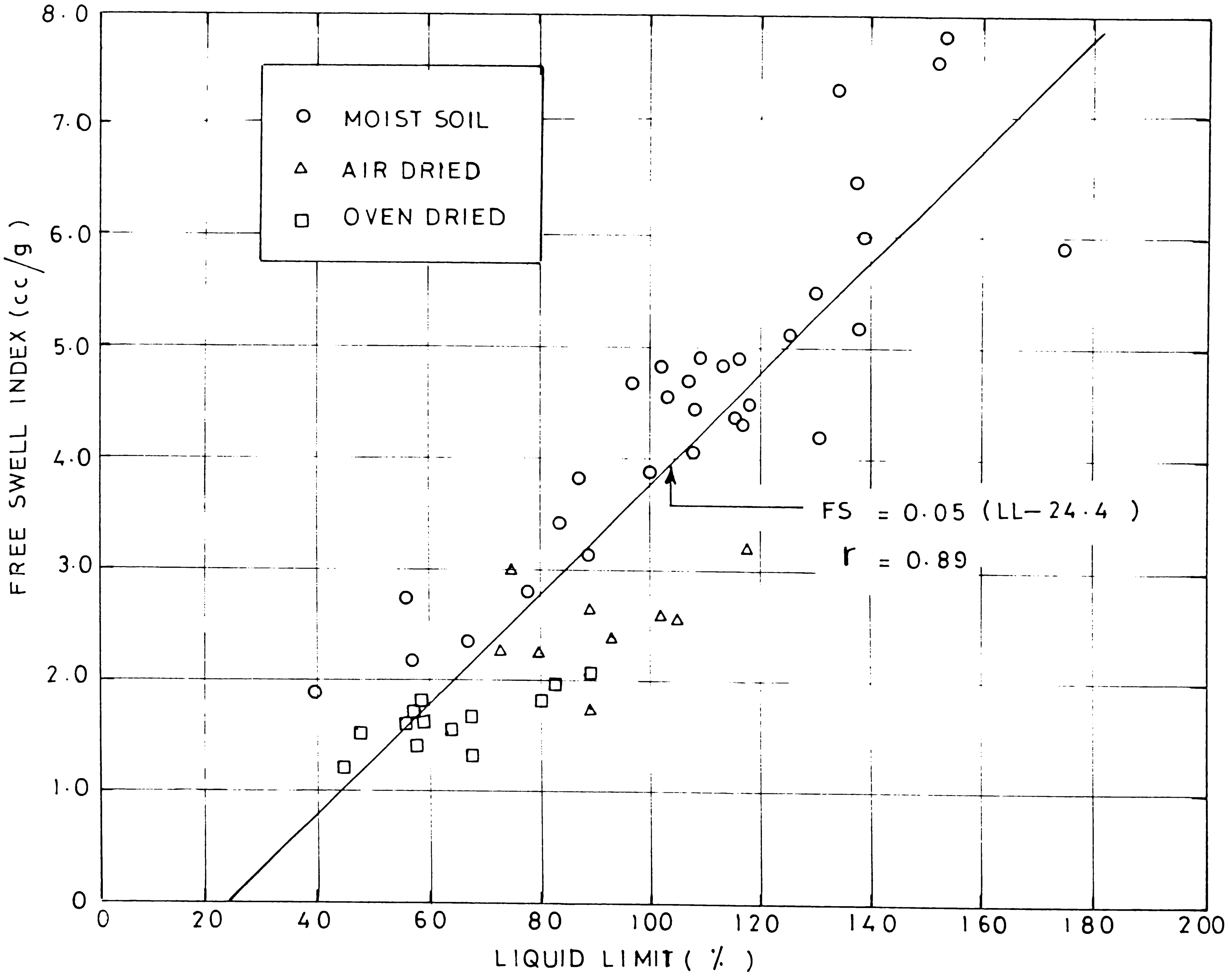


Fig. 4.6.2b : FREE SWELL INDEX vs LIQUID LIMIT.

4.7 ORGANIC CONTENT AND CARBONATE CONTENT IN COCHIN MARINE CLAYS

Studies on marine clay sediments from Santa Barbara Basin (off Southern California) by Booth and Dahl (1986) showed that good correlation exists between organic content and Atterberg limits. The Atterberg limits increased with organic content.

Nambiar et al. (1985) reported the aggregation phenomenon of marine clays from investigations carried out on fine grained carbonate soils from off the west coast of India. The calcium carbonate content of this marine clay were very high ranging over 47% to 55%. Since the above factors--high organic and carbonate contents--could be the cause for aggregation, a series of tests were taken up to find out whether these influenced the changes in the physical properties of Cochin marine clays on drying.

The organic content of the soil can be removed by treating with hydrogen peroxide and the carbonates can be removed by hydrochloric acid. The moist clay samples were treated with H_2O_2 and HCl and the treated samples were oven dried and the Atterberg limits and free swell index determined. The results of these tests are presented in Table 4.7.1.

Table 4.7.1

EFFECT OF ORGANIC AND CARBONATE CONTENT

Sl.	Description of sample	LL (%)	PL (%)	SL (%)	PI (%)	Free Swell index (cc/gm)	Organic matter (%)	Calcium carbo-nate (%)
	Untreated	108	42.5	20.5	65.5	4.44	9.01	21.8
1.	Soil treated with H_2O_2							
	(a) moist soil	100	44.3	20.7	55.7	4.03	0.0	18.8
	(b) air dried	89	37.7	18.7	51.3	3.68	--	--
	(c) oven dried	66	33.2	18.9	32.8	2.55	--	--
2.	Soil treated with HCl							
	(a) moist soil	98	35.8	19.8	62.2	4.57	8.8	6.3
	(b) oven dried	46	28.2	19.5	17.8	1.70	--	--
3.	Soil treated with H_2O_2 and HCl							
	(a) moist soil	97.5	40.8	23.3	56.7	4.52	0.0	7.1
	(b) oven dried	46	34.7	--	11.3	1.59	--	--

It can be seen from the results presented in the table that these two either do not influence the aggregation at all or they fail to make any discernible difference.

4.8 EFFECT OF WASHING

In order to study the effect of salt content already present in the natural moist soil, series of tests were carried out on repeatedly washed moist soil. For this purpose, large quantities of moist soil were taken in a bucket of 20 litre capacity and mixed thoroughly with large quantities of distilled water with a mechanical stirrer. After thorough mixing, the samples were allowed to settle down for a number of days. The clear supernatant liquid was carefully siphoned out taking extreme care not to permit any loss of fines. In the initial few washings, the water at the top was clear without the presence of any colloids. As the salt content got reduced by washing, more and more colloids were released by deflocculation, increasing the time for sedimentation. The washing was repeated till the supernatant water is free of salts. The process took more than 3 months. Series of tests for physical properties of such prepared clays were carried out.

Table 4.8.1 presents the test results on washed clay samples from Nettoor. It is seen that washing results in an increase of liquid limit though not significantly. The plastic and shrinkage limits also get increased marginally. The primary reason for increase is attributed to the change in fabric from flocculant to a relatively less flocculant fabric. While discussing the mechanism controlling the liquid limit of soils, Sridharan and Rao (1975) have brought out that relatively more flocculant fabric shows higher liquid limit. Bjerrum (1954) reports reduction in liquid limit values with reduction in salt content for Norwegian marine clays.

The free swell index of washed sample is less than that of natural moist sample, supporting the view that the washed samples possess less flocculant fabric than natural moist samples. The marginal changes in the properties can be attributed to less salt content of the Cochin marine clay. It can be seen from Table 4.1 that the salt content varies from 5.9 to 7.3 gm/litre. In comparison the Norwegian clays are significantly affected by washing because of the high salt content in its natural state (Bjerrum, 1954).

Table 4.8.1

EFFECT OF WASHING (PARUR SOIL)

Sl. No.	Description of soil	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)	Shrinkage limit (%)	Grain size distribution			Activity	Free swell index (cc/gm)
						Clay size (%)	Silt size (%)	Sand size (%)		
1.	Natural soil									
	(a) moist soil	108.0	42.8	65.2	20.3	42	39	19	1.55	4.44
	(b) air dried	73.0	35.3	37.7	20.2	30	44	26	1.20	2.26
	(c) oven dried	56.5	33.7	22.8	21.4	23	49	28	0.99	1.54
2.	Washed soil									
	(a) moist soil	109.0	43.7	65.3	21.3	43	41	16	1.62	4.72
	(b) air dried	82.5	35.6	46.9	19.3	30	54	16	1.56	2.10
	(c) oven dried	65.5	34.3	31.2	20.4	25	57	18	1.25	1.70

It may be noted from Table 4.8.1 that in spite of repeated washing, the reduction in liquid limit of air dried and oven dried samsples are of the same order; thus revealing that presence of excess salts is not the primary reason for aggregation.

4.9 EFFECT OF SOAKING ON DRIED SAMPLES

To explore the possibility of the initially air/ oven dried soil regaining their original properties prior to drying, the samples were tested after soaking them in water for different periods. The results are presented in Figs.4.9.1 and 4.9.2. Although the liquid limit steadily increased with duration of soaking, it could not reach its value corresponding to that of original moist samples. It could attain only about 80% of its original value for initially air dried samples and 62% for initially oven dried samples even after soaking for 2 years. Correspondingly, the plastic limit of initially air dried sample increased from 35.3 to 39.5% on soaking. In case of initially oven dried samples, the value increased from 33.7 to 36.5%.

It can be seen from Fig.4.9.1 that the bulk of the increase in the value of liquid limit and plastic limit takes place within the first 3 to 4 months. The increase

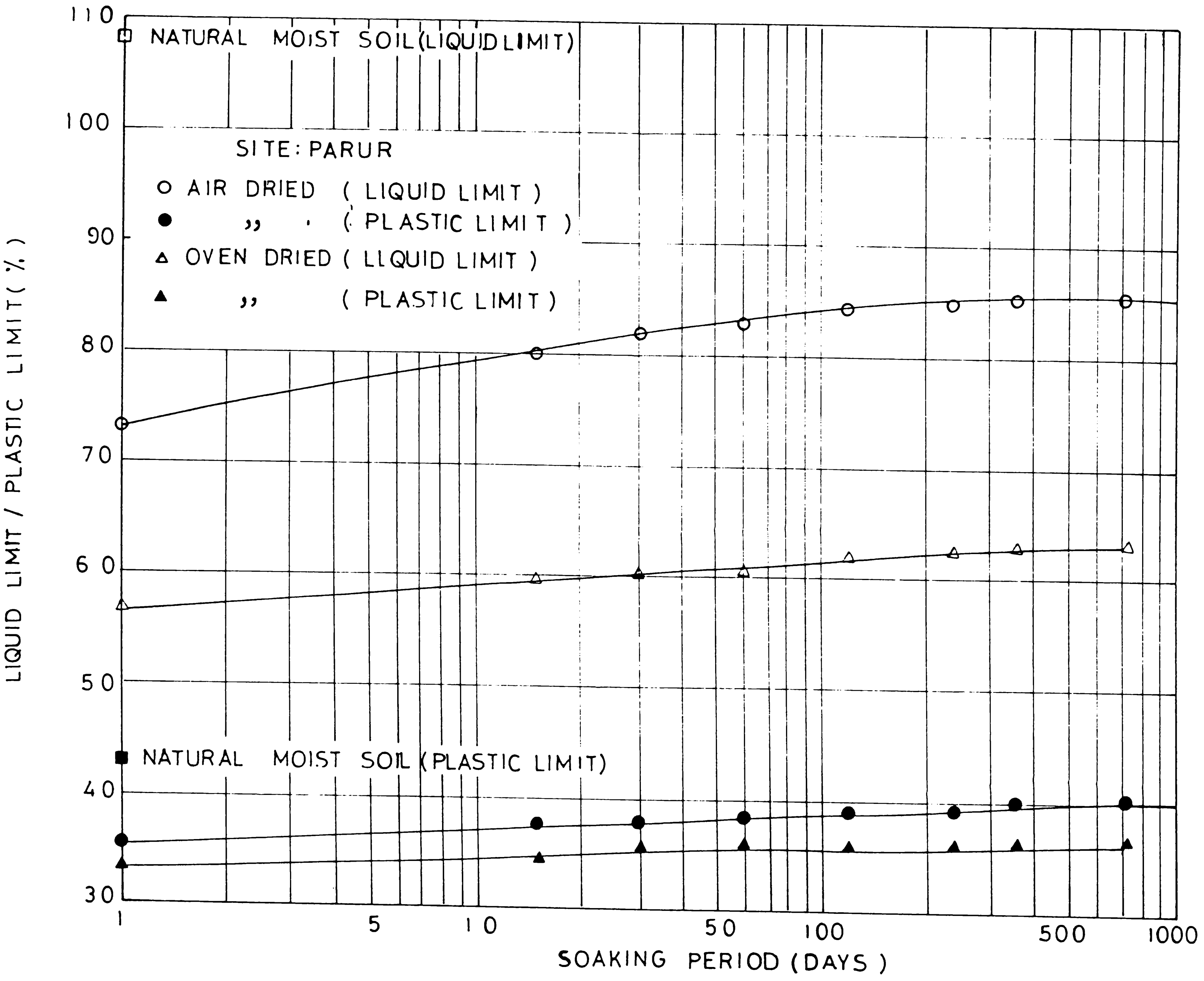


Fig. 4.9.1: EFFECT OF SOAKING ON LIQUID LIMIT & PLASTIC LIMIT.

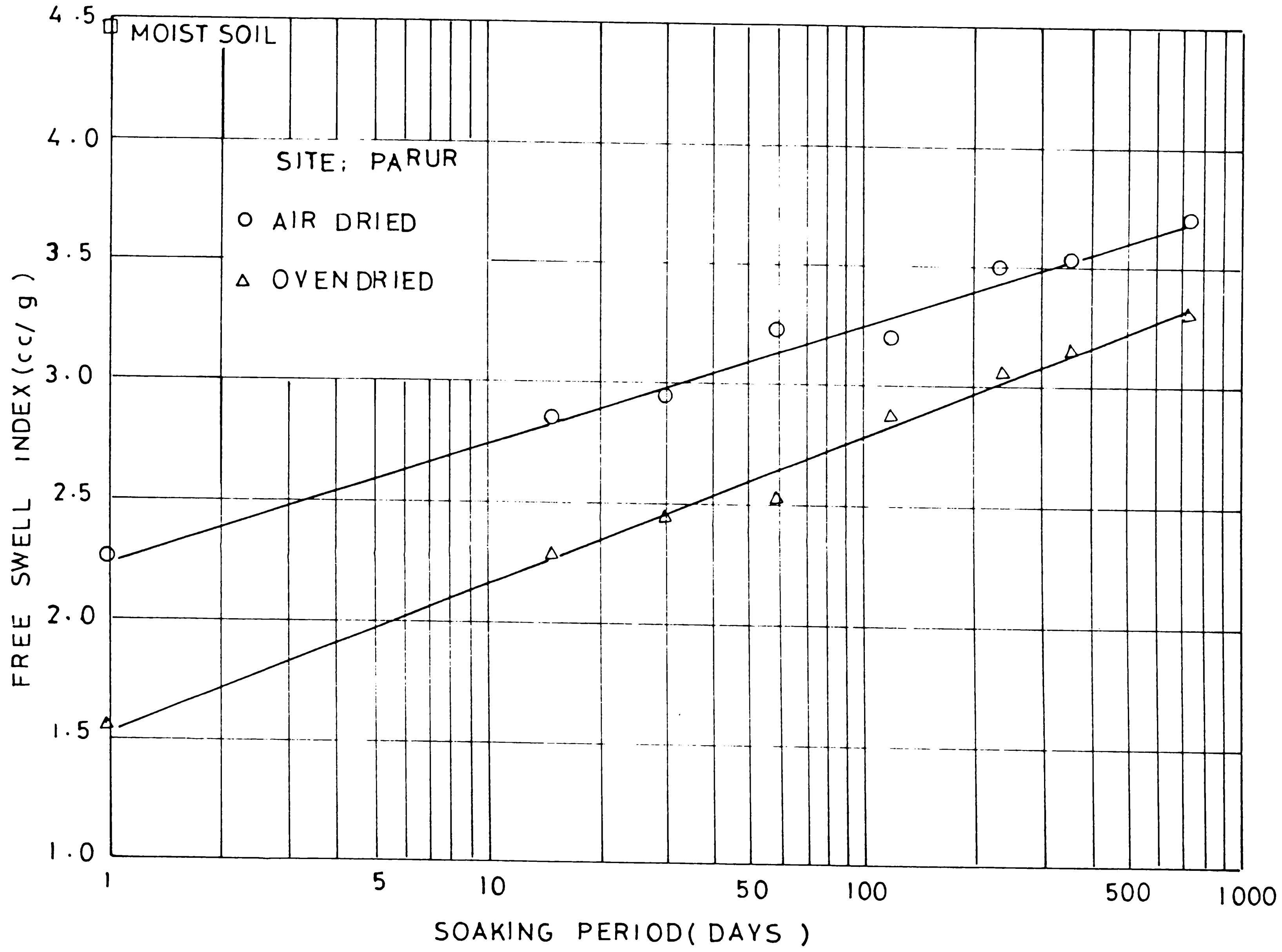


Fig. 4.9.2 EFFECT OF SOAKING ON FREE SWELL INDEX.

thereafter is marginal. Hence it could perhaps be surmised that aggregation is almost an irreversible process.

The variation in the free swell index value for 24 months duration of soaking is shown in Fig.4.9.2. Free swell index increases with soaking period and the rate of change is higher than that of Atterberg limits.

4.10 EFFECT OF SALTS

It is possible that addition of salts like sodium chloride and calcium chloride can have marked changes in the physical properties of these marine clays. In order to get a preliminary information on this, index and other properties were determined with different percentages of sodium chloride and calcium chloride salts. For these, the moist soil was mixed thoroughly with 5 to 20% by dry weight of sodium chloride and calcium chloride salts and the physical properties were determined. It is seen from Table 4.10.1 that addition of salts reduces the liquid and plastic limits. While the reduction in liquid limit is marginal (138 to 122%) for the addition of 20% sodium chloride, the reduction is quite considerable (138 to 102%) for the addition of 20% calcium chloride. The effect of salt concentration has been explained by depression of electrical

Table 4.10.1

EFFECTS OF SALTS ON PHYSICAL PROPERTIES (NETTOOR SOIL)

Sl. Description of the test	Specific gravity	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)	Shrinkage (%)	Grain size distribution Clay size (%)	Silt size (%)	Sand size (%)	Free swell index (cc/gm)
1. Moist Soil	2.72	137.5	47.5	90.0	17.80	48	31	21	5.20
2. Moist soil + Sodium Chloride		134.0	43.4	90.0	18.5	44	32	24	4.55
1) 5% NaCl	2.70	134.0	43.4	90.0	18.5	44	32	24	4.55
2) 10% NaCl	2.80	129.0	42.0	87.0	25.4	--	--	--	4.35
3) 20% NaCl	2.82	122.5	39.0	83.5	30.1	--	--	--	2.70
3. Moist soil + Calcium Chloride									
1) 5% $CaCl_2$	2.55	131.0	47.0	84.0	14.4	36	45	19	4.16
2) 10% $CaCl_2$	2.63	122.0	40.0	82.0	14.0	--	--	--	3.12
3) 20% $CaCl_2$	2.46	102.0	34.0	68.0	12.6	--	--	--	3.80

double layer (Olsen & Mesri, 1970; Mesri & Olsen, 1971; Sridharan & Jayadeva, 1982). In this case also the reduction in liquid limit could be attributed to the depression of double layer because of increase in salt concentration (Sridharan & Rao, 1975).

Table 4.10.1 also presents data on shrinkage limit and free swell index. The shrinkage limit with sodium chloride as additive has shown significant increase, while calcium chloride as additive does not show much variation. Further there is a marked reduction in free swell index with the increase in salt content. This can be attributed to deflocculation and aggregation of particles. The slight increase in silt size and sand size content supports the marginal formation of aggregation due to addition of salts.

4.11 SHEAR STRENGTH AS RELATED TO WATER CONTENT

In Greater Cochin area the layers at shallow depths have their natural moisture contents quite close to their liquid limits. The clay deposits below vary widely in consistency upto 40 to 50 m where good load bearing stratum can be met with. While large high rise buildings are necessarily founded on these strata over 40 to 50 m below, structures of lesser height are supported by friction piles. In order to design these friction piles, it is essential to get an accurate assessment of shear strength of these marine clay deposits. Since the consistency of the clay layers varies widely, the estimation of shear strength can be done only through shear strength tests on numerous undisturbed samples. Since this is a tedious and time consuming process, the need for a faster and easier technique for estimation of shear strength is keenly felt by geotechnical engineers handling foundation problems in soft marine sediments all over the coastal belt in Indian peninsula.

Even during the soil exploration, it was found that collection of undisturbed samples of extremely soft clays was a difficult task, as the sampling tubes could not hold the samples due to its poor adhesive strength. At the same time it was found far easier to collect representative samples, though disturbed, with the help of mechanical means such as augers. In this study on Cochin marine clays efforts were made to develop correlations between the index properties of clayey soils and shear strength. The results of the investigations to predict shear strength of Cochin marine clays based on their natural moisture contents are presented in this section. The applicability of the results were tested for other soft fine grained soils of high liquid limit.

4.11.1 Water content ratio

The shear strength of clayey soils is dependent on its water content. Any increase in water content is necessarily accompanied by a reduction in cohesion or shear strength. But at the same water content, different soils have different shear strength, as shown in Table 4.11.1. The table gives the shear strength values of five different soils whose natural water contents vary from 100 to 106%. Eventhough the natural water contents are in the same narrow range, the marine clay from Maradu in Cochin has the lowest shear strength of 0.0175 kg/cm^2, while bentonite has a shear strength of 0.115 kg/cm^2. Thus if the natural moisture content alone is taken as the basis, the shear strength behaviour looks highly erratic.

Eventhough a comparison between natural moisture content and shear strength does not indicate any possibility for a correlation between the two, incorporation of the liquid limit values into the analysis shows some definite

Table 4.11.1

NATURAL WATER CONTENT AND SHEAR STRENGTH OF CLAYEY SOILS

Sl. No.	Soil	Natural water content W (%)	Shear strength (kg/cm^2)	Liquid limit W_L(%)	Water content ratio W/W_L
1.	Cochin marine clay (Parur)	105.9	0.021	118.3	0.90
2.	Cochin marine clay (Maradu)	105.7	0.0175	105.7	1.0
3.	Cochin marine clay (Elamkulam)	105.8	0.0236	119.1	0.89
4.	Bentonite	104	0.115	307	0.34
5.	Black cotton soil	101	0.021	103	1.0

trends. Table 4.11.1 also shows the liquid limit values for the soils listed. It could be clearly seen that for the same water content, the shear strength is directly proportional to the liquid limit values. Thus it is obvious that shear strength values can show regular and consistent tendencies only if it is correlated to natural moisture content along with the respective liquid limits. A new factor called 'Water content ratio' defined as the ratio of natural moisture content to liquid limit is introduced here to obtain a regular correlation with shear strength. Obviously shear strength is inversely proporational to water content ratio (WCR) though not linearly.

Significance of water content ratio

The utility and versatility of the proposed water content ratio is well brought out by figures 4.11.1.1 and 4.11.1.2. The shear strength at different water contents obtained from laboratory vane shear tests on remoulded representative samples of Cochin marine clays collected from two sites are given in Fig.4.11.1.1. Eventhough the two sites are within Greater Cochin, the sampling locations are separated by 35 km. But the clays are of the same geological origin. Samples from Kumbalam had a liquid limit of 98% while Parur sample had a liquid of 118%. The samples give two separate curves as shown in Fig.4.11.1.1. These two curves do not help us to establish any correlation between shear strength and water content.

Fig.4.11.1.2 gives a plot between shear strength and WCR for the two samples. It can be seen from the figure that the two curves merge into one single curve indicating that good correlation is possible between shear strength and water content ratio.

The liquid limit value of any clay is dependent on the clay mineral and the clay content of the sample. As the clay content increases, liquid limit also increases. Similarly clays containing montmorillonite clay mineral will have a higher liquid limit than kaolinite clays. Both these have significant influence on the shear strength of clayey soils. But the influence of both are conveniently incorporated by normalising the water content by expressing it as a ratio of natural moisture content and liquid limit.

A semilogarithmic plot between shear strength and water content ratio can give a better correlation between the two as shown in Fig.4.11.1.3. The data from Fig.4.11.1.2

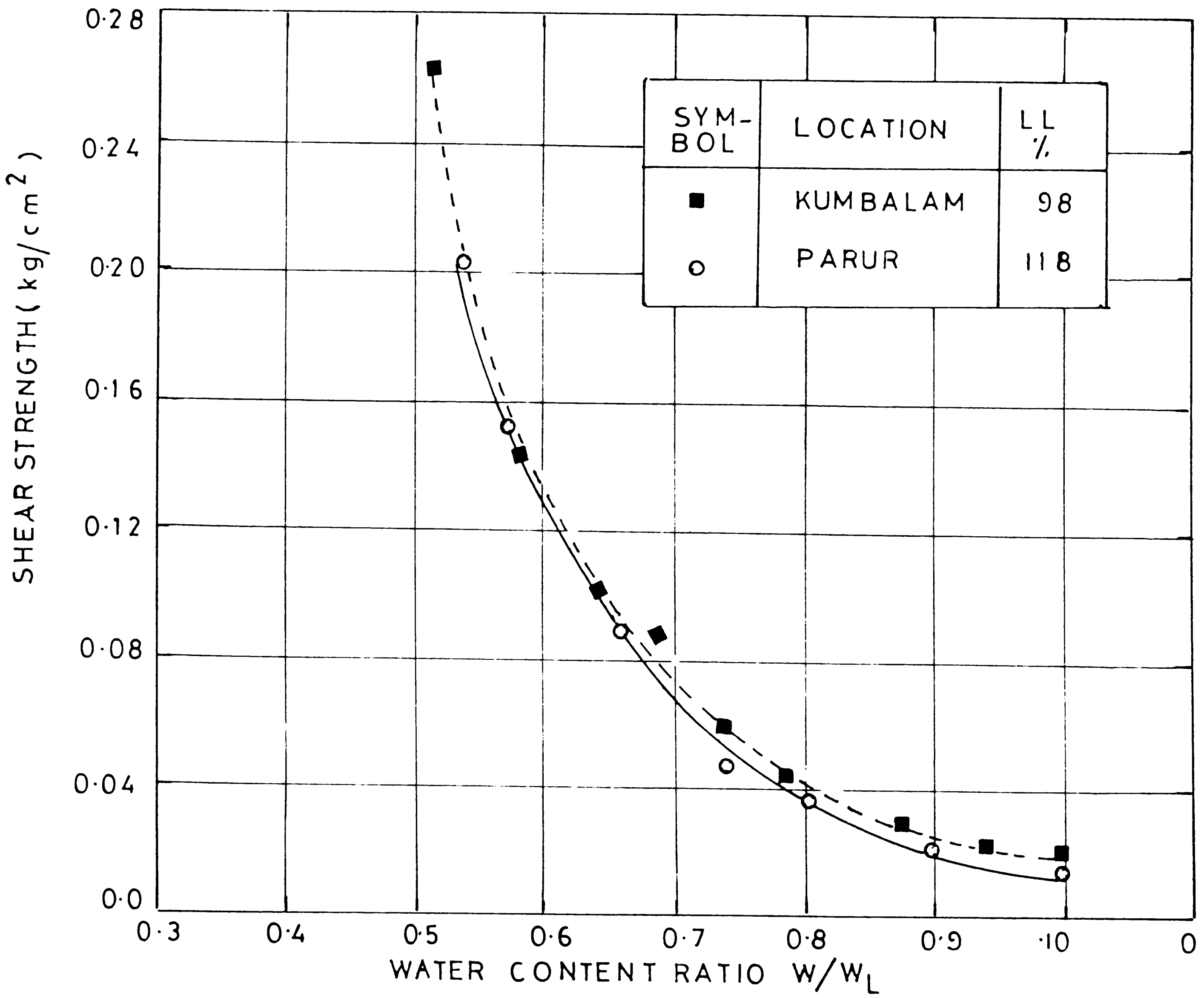


Fig. 4.11.1.2 SHEAR STRENGTH vs WATER CONTENT RATIO.

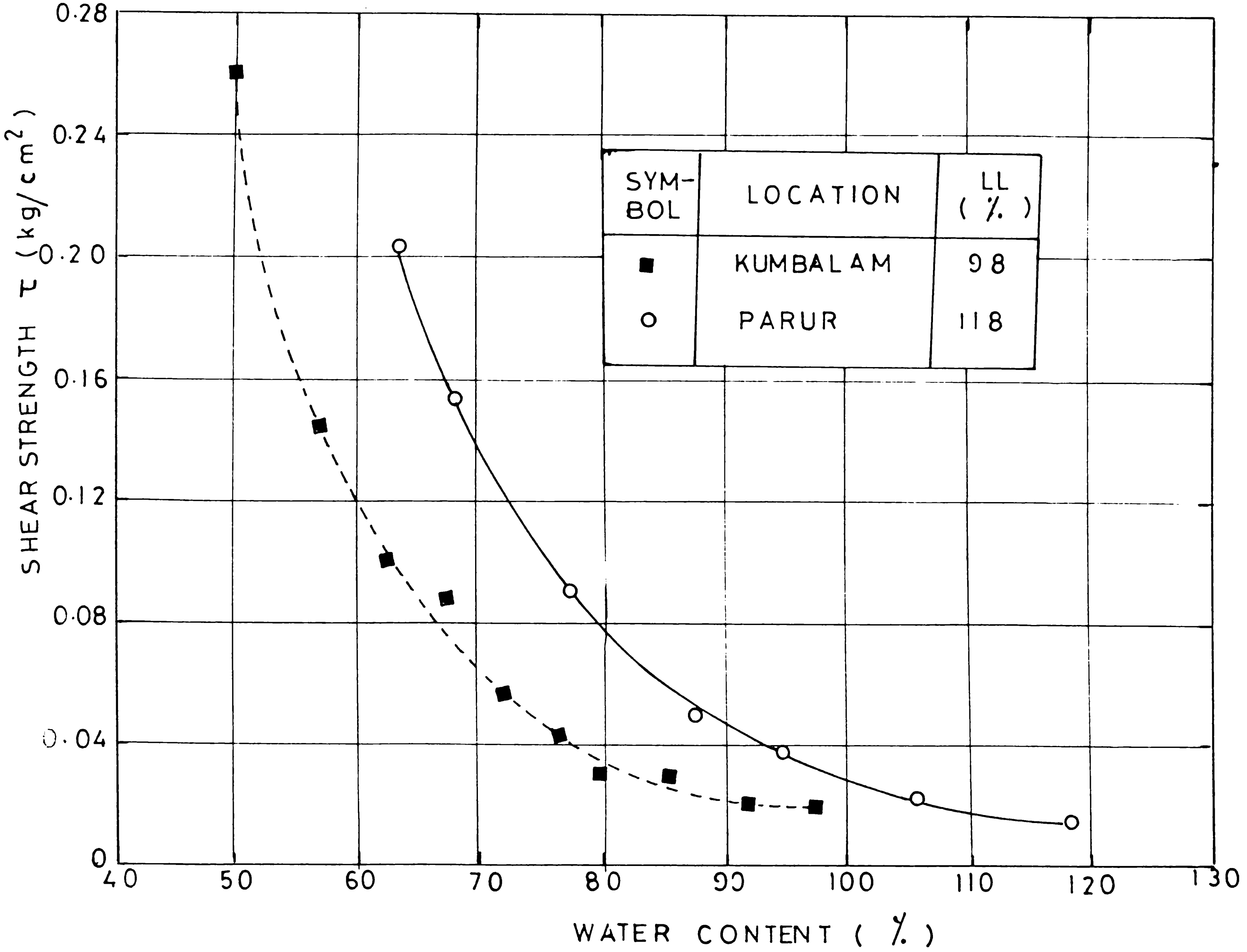


Fig. 4.11.1.1 SHEAR STRENGTH vs WATER CONTENT.

are plotted in Fig.4.11.1.3 with shear strength on logarithmic scale and water content ratio on natural scale. Excellent linear relationships have been obtained for the two soils. They also have almost the same slope. Experimental data on two more samples of Cochin marine clays collected from two other sites viz. Maradu and Elamkulam, are also plotted in Fig.11.1.3 which gave the same slope.

Fig.4.11.1.4 shows the semilog plot between water content ratio and shear strength values of the same four samples of Cochin marine clays. It can be seen from the plot that a linear correlation could be easily obtained between water content ratio and shear strength for Cochin marine clays, though the clay percent and liquid limit values of the individual samples are different. A statistical fit to the experimental data gave the following expression.

$$\mathrm{WCR} = 0.231 - 0.414 \log \tau \qquad (4.11.1.1)$$

with a high correlation coefficient of 0.987.

Correlation between liquidity index and shear strength

Attempts have been made by earlier workers to correlate shear strength with liquidity index of clays. Plots between these two for different clays collected from

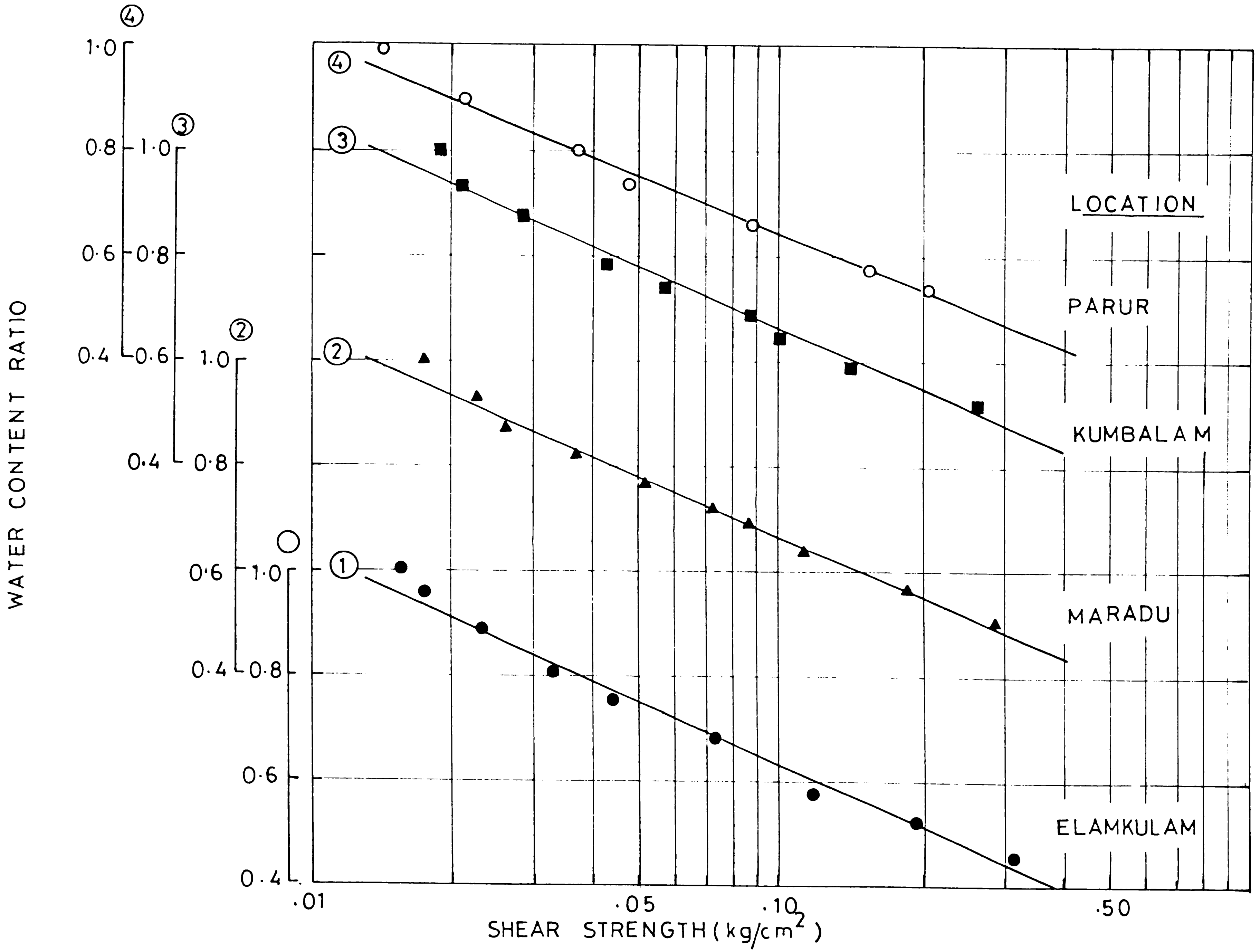


Fig. 4. 11.1.3 SHEAR STRENGTH vs WATER CONTENT RATIO.

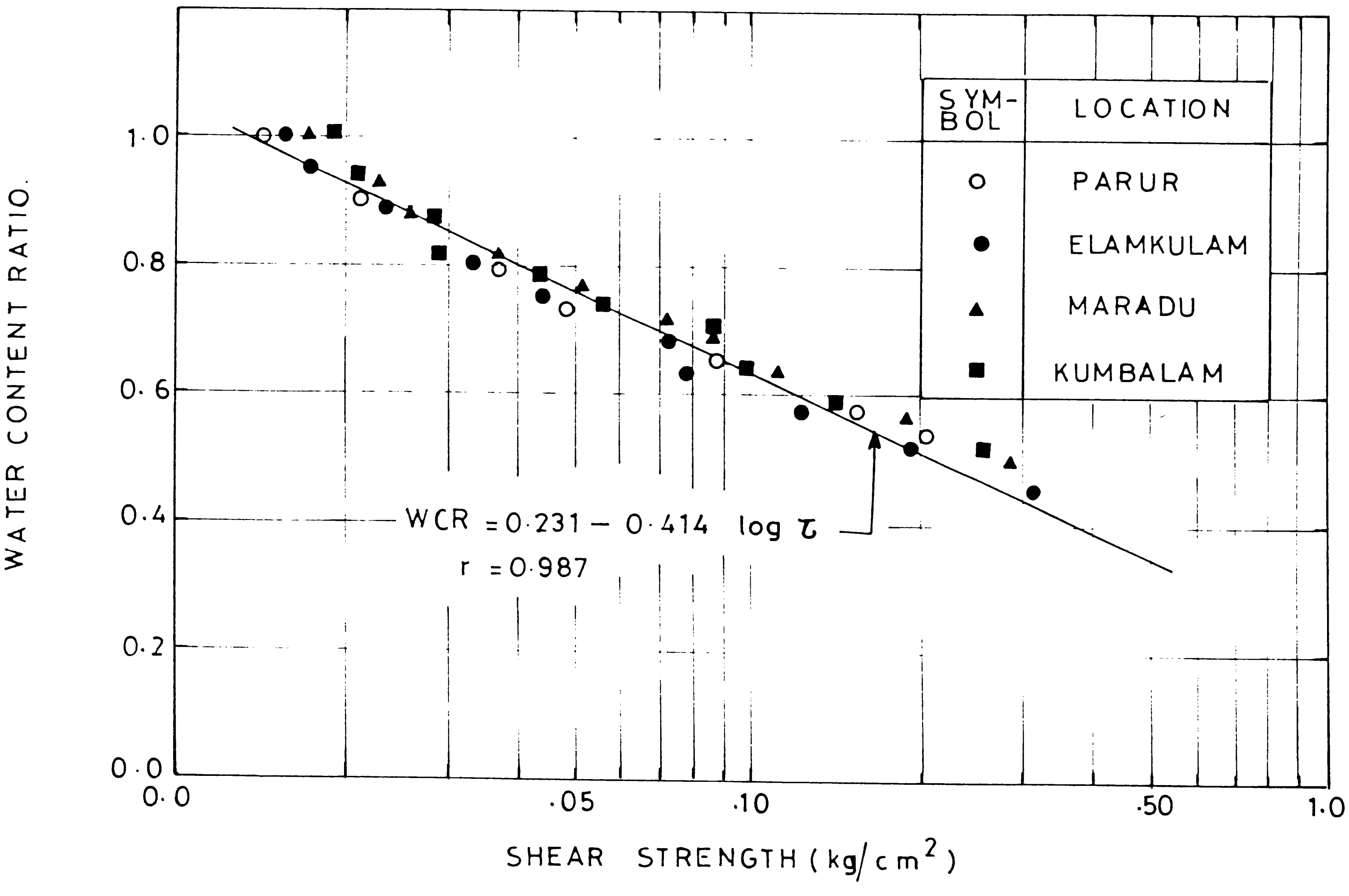


Fig. 4. 11. 1.4 SHEAR STRENGTH vs WCR FOR COCHIN MARINE CLAYS.

literature are given in Fig.4.11.1.5. Eventhough Wroth and Wood (1978) proposed a linear relationship between these two; plots in Fig.4.11.1.5 clearly indicate that this is far from reality. Leroueil et al. (1983) gave the following expression

$$(C_u)_r = \frac{1}{(I_L - 0.21)^2} \qquad (4.11.1.2)$$

which is not a simple linear expression as claimed by them. Further, correlations with liquidity index involve the determination of plastic limit in addition to liquid limit. Yet the liquidity indices for the various samples of Cochin marine clays are plotted in Fig.4.11.1.6 from which it can be concluded that the relation between liquidity index and shear strength is not linear. Thus the method of correlating shear strength with water content ratio is simpler and has great potential for design applications.

4.11.2 Correlations for other soils

The method proposed can be acceptable only if the correlation suggested is valid for other clayey soils also. Laboratory vane shear strength was found out for the following four soils by varying their water contents.

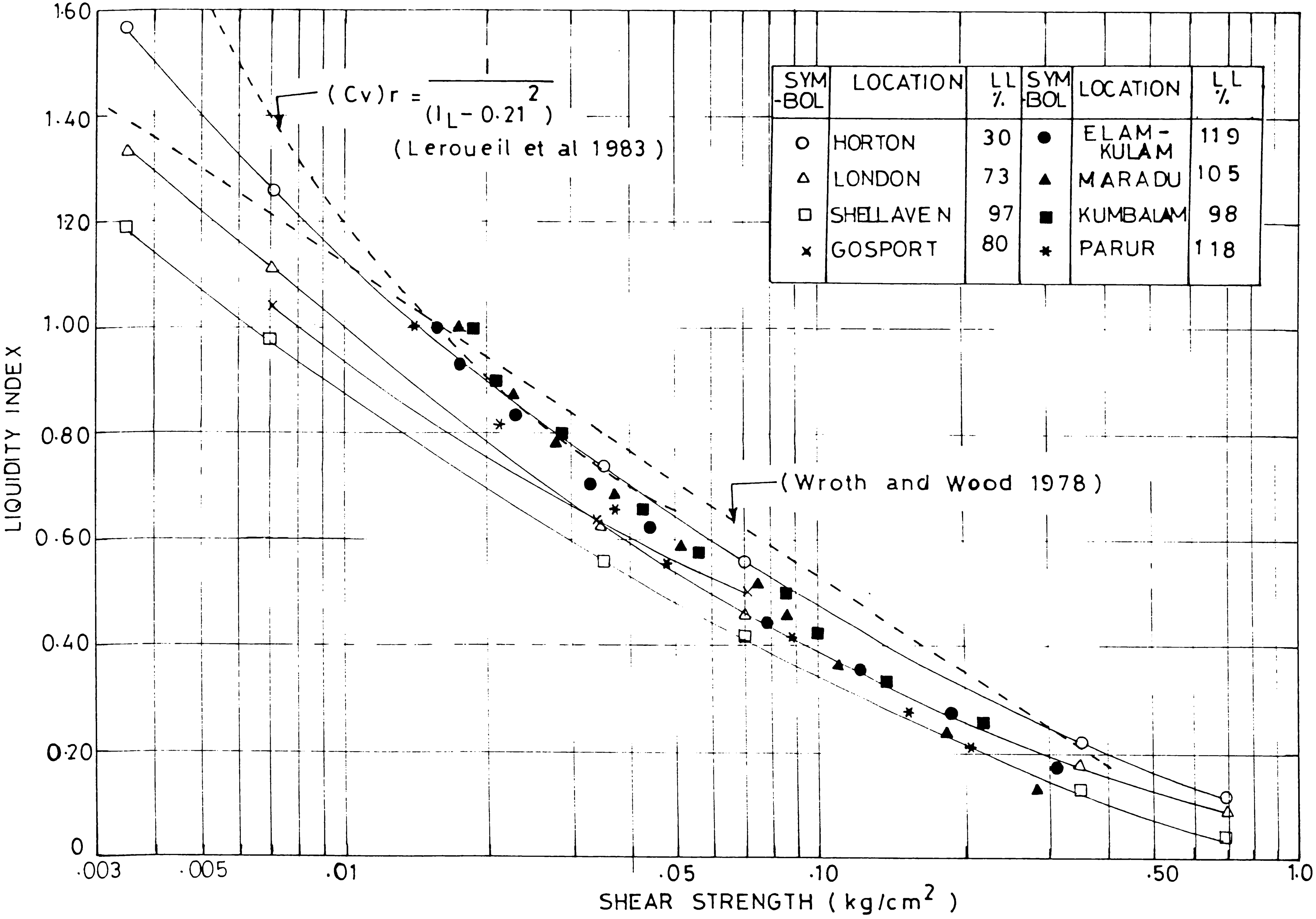


Fig. 4.11.1.5 SHEAR STRENGTH vs LIQUIDITY INDEX.

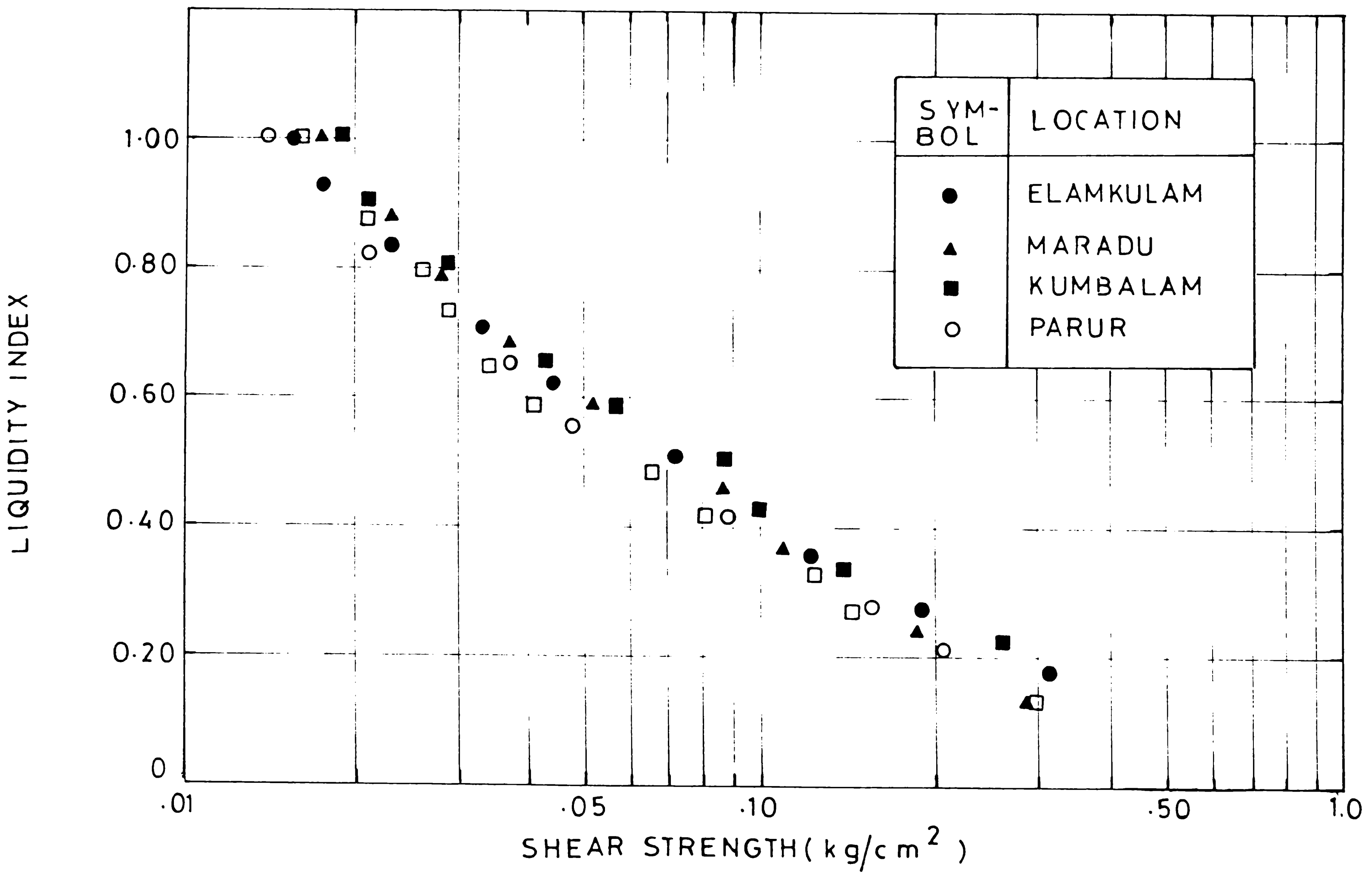


Fig. 4.11.1.6 LIQUIDITY INDEX vs SHEAR STRENGTH - COCHIN MARINE CLAYS.

1. Bentonite
2. Black cotton soil
3. Red earth (passing through 425 microns sieve)
4. Kaolinite

The results are plotted in Fig.4.11.2.1. In these cases also, linear relationships could be obtained between shear strength and WCR. Fig.4.11.2.2 shows the liquidity index vs. shear strength relations for the same four soils showing no uniqueness. A comparative study of these two figures underscores the superiority of the proposed correlation for shear strength.

The plots in Fig.4.11.2.1 indicate some more possible uses for these correlations. While kaolinite clay has the least slope for the τ - WCR plot, the bentonite has the maximum slope with other soils falling in between. The variation in slope indicates the possibility of identifying the clays of similar geological origin and similar geotechnical behaviour using these plots. Clays with kaolinite mineral (such as red earth) will fall in the region with flatter slope, while soils containing montmorillonite mineral (such as Cochin marine clays) show a steeper slope.

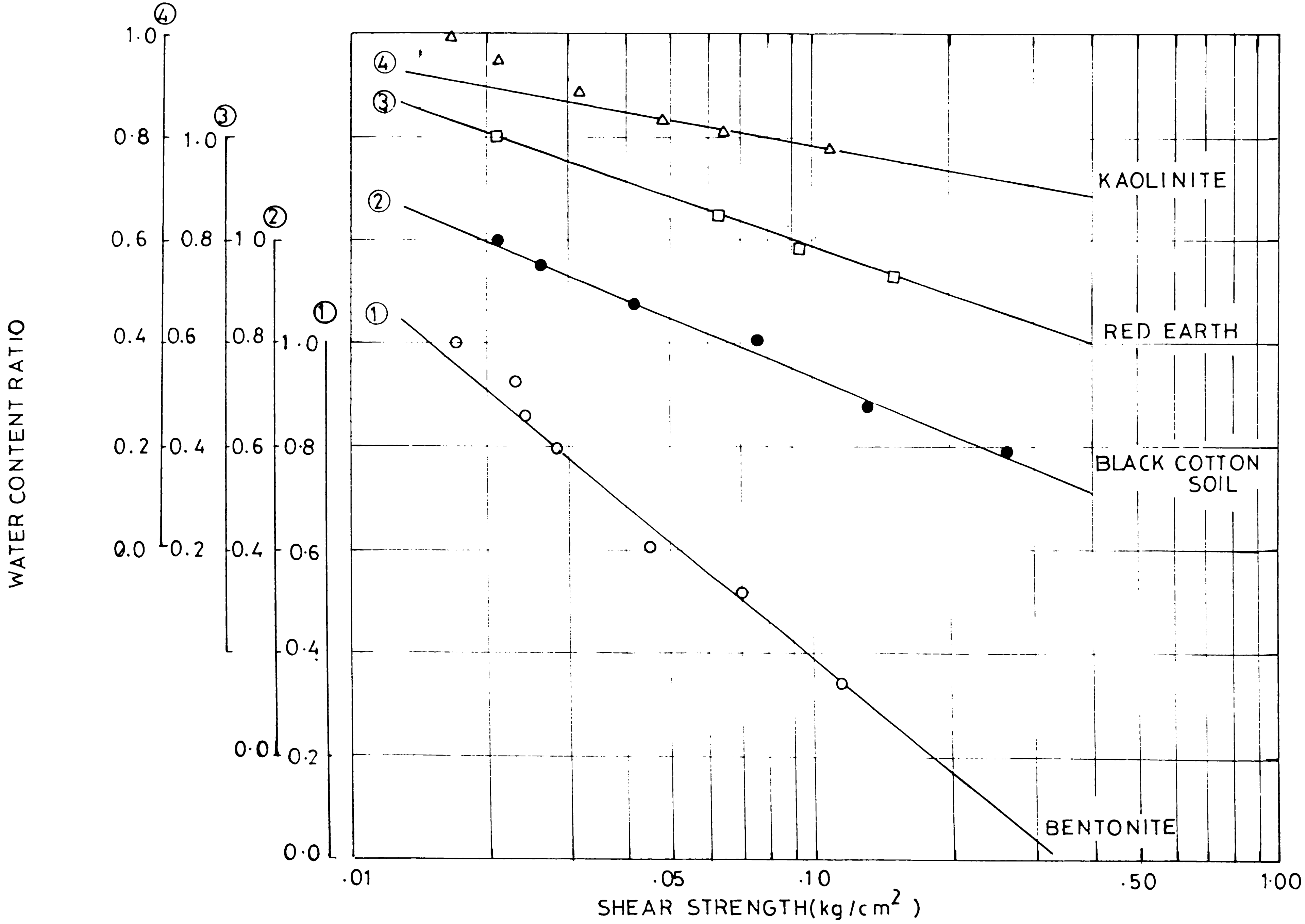


Fig. 4.11.2.1 : τ vs WCR FOR DIFFERENT SOILS.

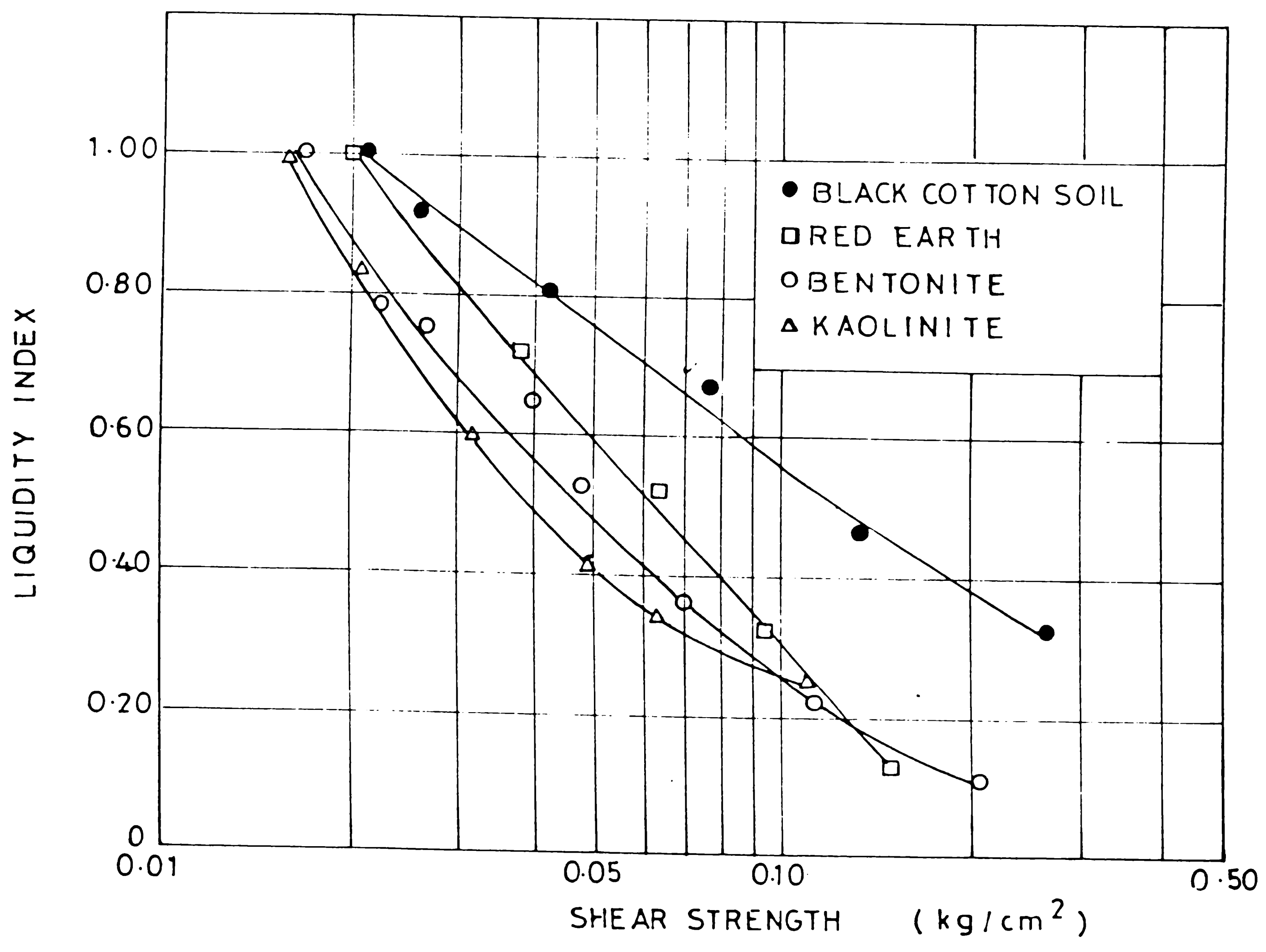


Fig 4.11.22 : τ vs LIQUIDITY INDEX FOR DIFFERENT SOILS

The possibility of correlating water content ratios with compressibility has been investigated (Nagaraj & Murthy, 1983 and 1986).

Table 4.11.2 shows the shear strength of different soils at their liquid limits and plastic limits. The value of shear strength at liquid limit is reported to vary in the range of 15 to 30 g/cm^2 (Sridharan & Rao, 1975). The present investigations show that it varies from 10 to 21 g/cm^2. At plastic limits, shear strength varies from 180 to 600 g/cm^2. In case of Cochin marine clays, the shear strength ranges over 14 to 19 g/cm^2 at liquid limit and 380 to 540 g/cm^2 at plastic limit.

The above discussions show that a good correlation can be established between the natural water content and shear strength of the clay. Water content ratio provides the relation between the two. Good linear correlation could be established through semilogarithmic plots between water content ratio and shear strength. The slope of the straight lines can help to identify the clay mineral. Thus water content ratio can be a simple and useful factor to assess the shear strength of soft layers.

Table 4.11.2

SHEAR STRENGTH OF DIFFERENT SAMPLES OF CLAYS

Sl. No.	Soil type	L.L. (%)	Shear strength (kg/cm^2)	
			AT L.L.	at P.L.
1.	Bentonite	429	0.013	0.26
2.	Black cotton soil	100	0.021	0.60
3.	Kaolinite	47	0.0105	0.18
4.	Red earth	51	0.021	0.21
5.	Cochin marine clay			
	(a) Parur	118	0.014	0.43
	(b) Elamkulam	119	0.0157	0.54
	(c) Maradu	106	0.0175	0.41
	(d) Kumbalam	98	0.019	0.38

4.12 DIELECTRIC CONSTANT OF PORE FLUID AND INDEX PROPERTIES

Eventhough Atterberg limits were originally devised for classification, its utility and significance in geotechnical engineering steadily increased as more and more soil properties such as surface area, cation exchange capacity, swelling behaviour, California Bearing Ratio, compressibility, shear strength, compaction etc. were correlated to them. Since the dielectric constant of the pore medium exercises considerable influence on the shear strength of the soil through variations in the thickness of electrical double layer, the consistency limits of fine grained soils should be influenced by their values. Eventhough Kinsky (1971) reported that there is a tendency for the liquid limit values to increase with dielectric constant, the values reported has considerable scatter. Judging from the behavioural pattern in other properties, this correlation should have been more regular. Studies on Cochin marine clay using different dielectric liquids helped to confirm the correlation between liquid limit and values of dielectric constant.

Liquid limit tests were carried out in Casagrande apparatus or cone penetrometer using the following six fluids.

1. Carbon tetrachloride
2. Ethyl acetate
3. Acetone
4. Methanol
5. Dimethyl sulphoxide
6. Water

To obtain the influence of the above organic fluids on liquid limit value of Cochin marine clays, 'moist' samples should have been used. But since it was found difficult to obtain 'moist' samples in which the marine pore water could be completely displaced with these organic fluids by circulation or permeation, oven dried marine clay samples were used for carrying out the tests for liquid limit. Since oven drying considerably influences the physical properties of the soil as discussed earlier, this change is bound to influence the liquid limit values and correlations could be different from what they really ought to be. But since X-ray diffraction studies have already indicated the type of the basic clay mineral in Cochin marine clay to be montmorillonite this variation could be considered while interpreting the results.

Table 4.12.1 shows the properties of the organic fluids used for the test and the values of liquid limit. Since it has been reported that the mechanisms controlling shear strength are different for montmorillonite and kaolinite clays, tests were carried out on those clays also to obtain a comparative study with Cochin marine clays.

Since some of the pore fluids used are volatile, it was difficult to find out the liquid limit value using Casagrande's apparatus. The liquid limit tests using

Table 4.12.1

RELATION BETWEEN DIELECTRIC CONSTANT OF PORE FLUIDS AND PHYSICAL PROPERTIES OF SOILS

Sl. No.	Pore fluid	Density* (gm/cc)	Dipole* moment* (debyes)	Dielectric constant*	Kaolinite			Cochin marine clay			Bentonite		
					Liquid limit (wt. basis) (%)	Liquid limit (vol. basis) (%)	Free swell index (cc/gm)	Liquid limit (wt. basis (%)	Liquid limit (vol. basis (%)	Free swell index (cc/gm)	Liquid limit (wt. basis) (%)	Liquid limit (vol. basis) (%)	Free swell index (cc/gm)
1.	Carbon tetra-chloride	1.585	0	2.284	105.0	176.32	1.70	86.50	145.25	1.20	115.0	193.11	2.60
2.	Ethyl acetate	0.895	1.81	6.02	--	--	-	--	--	-	--	--	--
3.	Acetone	0.785	2.72	20.70	46.0	155.66	1.30	45.0	152.2	1.30	62.0	209.80	2.10
4.	Methanol	.787	1.664	32.63	42.50	143.64	1.27	46.0	155.46	1.30	68.0	229.50	1.85
5.	Di-methyl sulphoxide	1.100	3.90	48.9	--	--	--	--	--	--	--	--	2.10
6.	Water	0.999	1.84	80.4	49.0	130.83	1.15	63.0	173.55	1.55	345.0	921.15	19.0

* The fluid properties are collected from Washburn (1928); Weast (1968) and Weissberger (1955).

pore fluids other than water were carried out by cone penetrometer. The water content that will permit a penetration between 20 mm and 30 mm was determined and liquid limit was found out from the equation

$$W_L = w_d + 0.01(25-x)(w_d+15) \qquad (6.8.1)$$

where W_L = liquid limit of the soil

w_d = moisture content corresponding to penetration x

x = depth of penetration of cone.

Fig 4.12.1 gives the plot between the liquid limit of oven dried Cochin marine clays and dielectric constant of the pore medium. The liquid limit values were calculated by volume basis as the unit weight of the pore fluid used was different from one another. The water content by volume basis is defined as the ratio of the volume of fluid to the volume of solids expressed as a percentage.

The figure shows that while the liquid limit of bentonite increases significantly with the dielectric constant of the pore fluid, that of kaolinite shows a decreasing tendency. The liquid limit of oven dried Cochin marine clay shows a tendency to increase with increase in liquid limit but considerably lower than the increase shown by montmorillonite.

The behaviour of bentonite and kaolinite which represent two different types of soil structure has been explained very well by Sridharan and Rao (1975).

Mechanisms controlling liquid limit

Both attractive and repulsive forces of an electrical nature exist between clay particles (Kruyt, 1949; Lambe, 1958; Rosenquist, 1955). The primary parameters affecting the electrical forces are the distance of separation of the particles and the dielectric constant of the medium (Kruyt, 1949; Davies, 1965; Lambe, 1958). The electrical attractive forces vary inversely with the dielectric constant of the pore medium and the repulsive forces increase directly with dielectric constant (Sridharan, 1968; Sridharan & Rao, 1971, 1973, 1975). An increase in attractive forces increases the shear strength while an increase in repulsive forces decreases the shear strength. Consequently, an increase in the value of dielectric constant should be accompanied by a decrease in shear strength.

Liquid limit is essentially a test for shear strength. It is the water content or fluid content at which the shear strength is around 15-30 gm/cm^2. Therefore an increase in dielectric constant should bring down the liquid limit value. But from Fig.4.12.1 only kaolinite follows this dictum. In case of montmorillonite and oven dried marine clay, the liquid limit increases with dielectric constant.

The peculiar behaviour of montmorillonite has been well explained by Sridharan and Rao (1975). Other factors remaining same, the thickness of diffused double layer varies directly with dielectric constant of pore medium (Lambe, 1958). Eventhough there is a reduction in shear strength with ϵ , the increase in thickness of diffused double layer is more predominant. Thus the liquid limit of clays of the montmorillonite type increases significantly with

It can be seen from the figure that oven dried marine clay occupies a position in between kaolinite and montmorillonite. The liquid limit of oven dried Cochin marine clay tends to increase with dielectric constant, but the variation is not significant. Eventhough Cochin marine clays are montmorillonite clays, because of its formation in saline environment it undergoes considerable changes on air or oven drying. Due to aggregation of the finer particles, the thickness of the double layer of oven dried sample is not as large as the moist marine clay. Thus, eventhough, the plot for oven dried marine clay shows an inclination for higher values

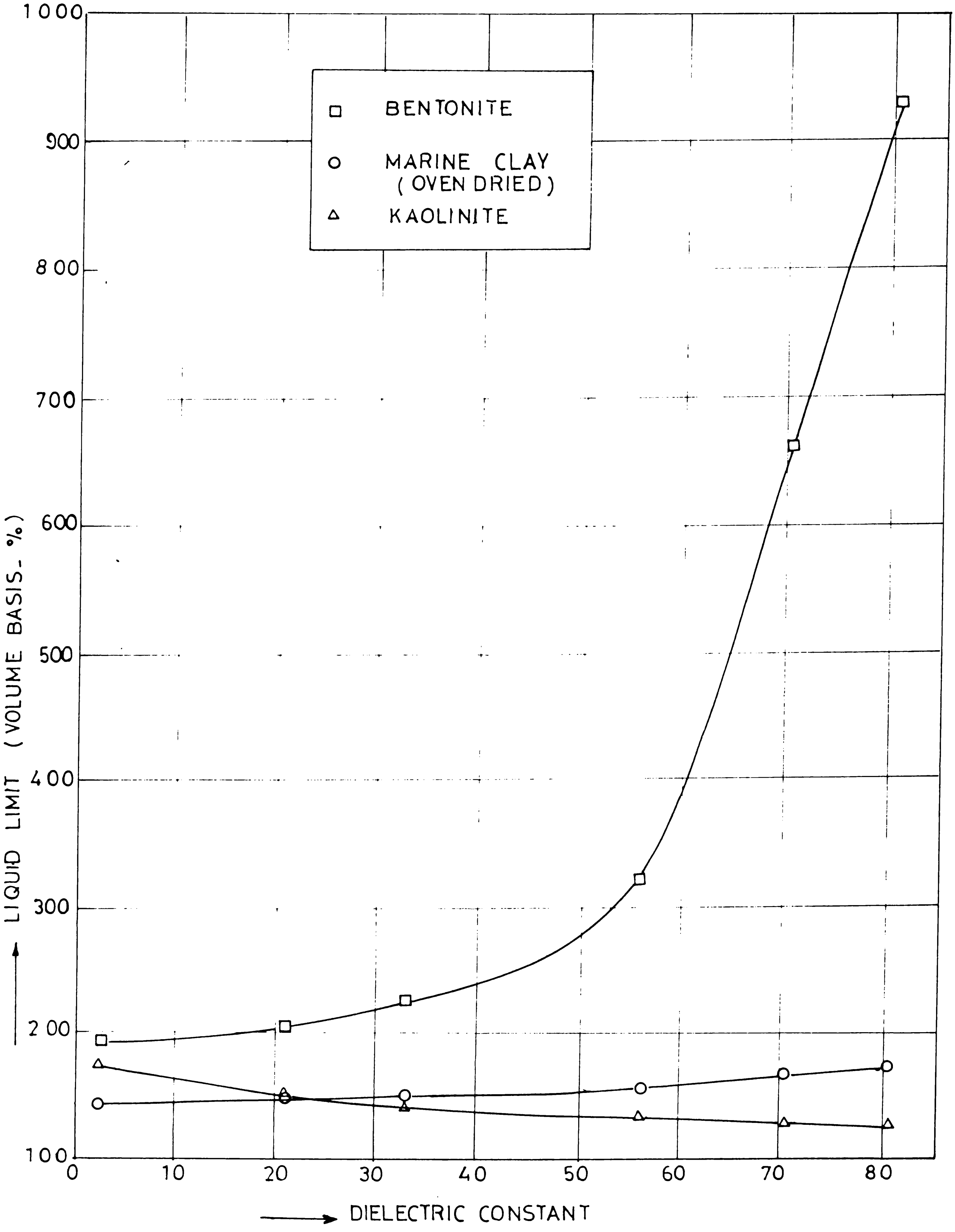


Fig. 4.12.1 LIQUID LIMIT vs DIELECTRIC CONSTANT

for liquid limit with increasing values of ϵ , the variation is quite subdued.

Fig.4.12.2 shows the free swell index for bentonite, kaolinite and oven dried Cochin marine clay, in different organic fluids with varying dielectric constants. Bentonite shows a free swell index as high as 19.0 in water and as low as 2.6 in carbon tetrachloride. The variation in kaolinite is very low. Oven dried marine clay shows similar tendencies as bentonite but the variation is far below the free swell index shown by bentonite.

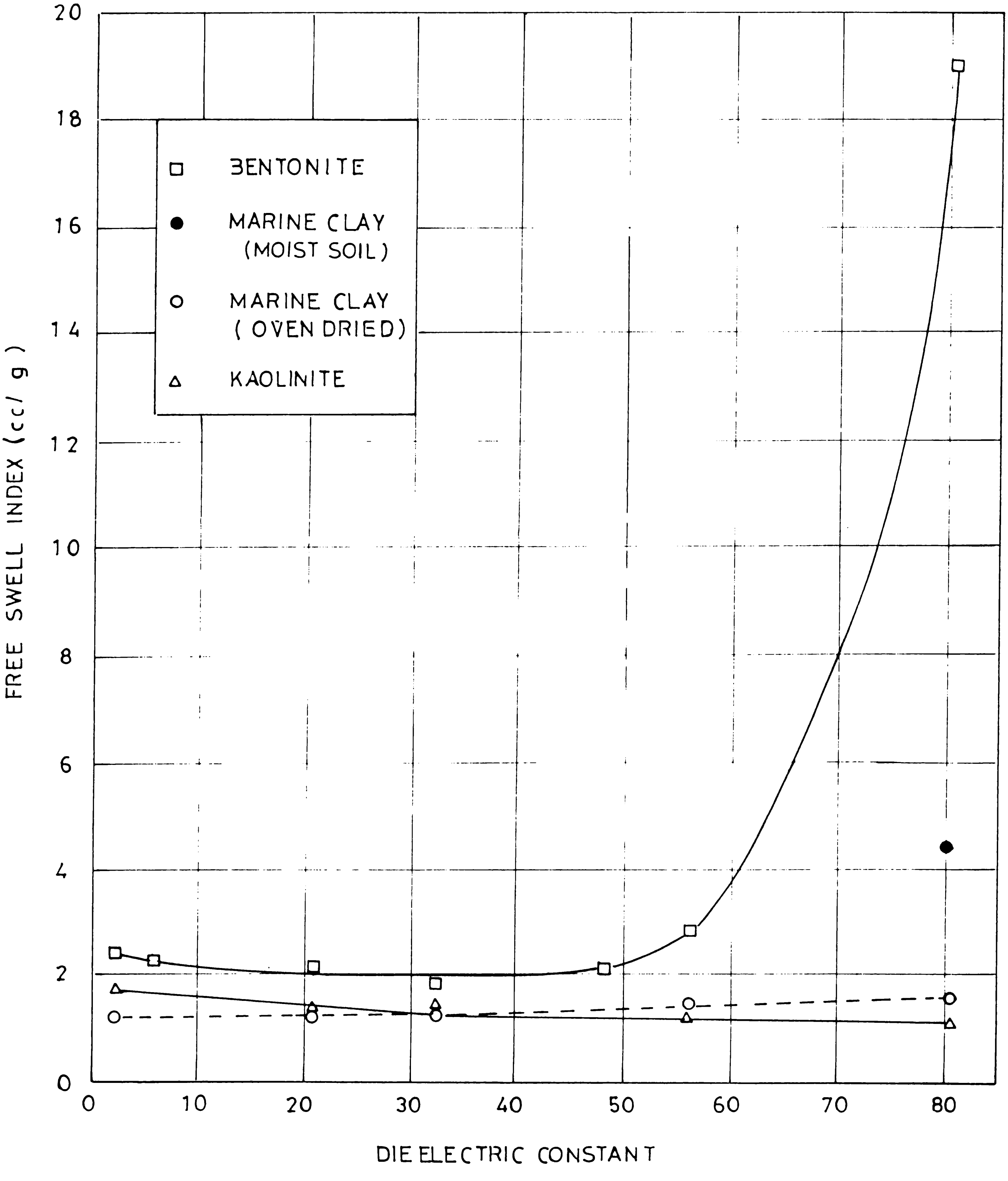


Fig. 4.12.2 FREE SWELL INDEX vs DIELECTRIC CONSTANT.

Chapter V

CONSOLIDATION CHARACTERISTICS OF COCHIN MARINE CLAYS

5.1 INTRODUCTION

Consolidation has been a major concern to builders for a very long time. Kerisel (1985) reported that 6000 years ago, Sumerians built Ziggourats - embankments to support temples - on the soft soils of the Euphrate valley. One of the temples, the white temple of Eridou, has now its foundation 12 m below the street. Since that time, a great number of structures settled and suffered damage. With the rapid industrialisation seen during the last few decades and consequent urbanisation, it has become necessary to occupy lands that were earlier avoided and marshes considered unsuitable for habitation. Thus the spate of buildings on soft soils multiplied.

Another consequence of industrialisation is the increasing demand for water, which is often taken from the subsoil. The consolidation of marine deposits in coastal cities like Tokyo, Stockholm, Shanghai, Venice etc. has caused some settlement problems (Leroueil, 1988). Yamamote (1977) estimates that in Japan 1200 square kilometers of valuable land has submerged below the sea level. Vast stretches of areas in the coastal belt of Kerala, with deep formations of marine clays, pose serious problems to the geotechnical engineers. Hence the compressibility characteristics of marine clays is of great significance.

The consolidation test provides basic information on the compressibility characteristics of soil for use in computation of the magnitude and time rate of settlement resulting from one dimensional consolidation. To improve the applicability of the test results, the series of tests planned should simulate as closely as possible the conditions which exist for an element of the prototype in the field. To achieve this objective, the series of consolidation tests planned should take into consideration as many parameters as possible which influence the volume change behaviour of fine grained soils.

5.2 AN OUTLINE OF THE TEST SERIES

In an extensive study of the compressibility characteristics of these marine deposits, several series of consolidation tests have been carried out to accommodate the implication of various parameters like the effect of drying of marine clays, duration of load increments, accurate assessment of preconsolidation pressure, etc. Since the engineering behaviour of montmorillonite soils is governed by the physico-chemical environment than the soil fabric (Sridharan & Rao, 1973, 1975; Sridharan & Jayadeva, 1982; Kinsky et al. 1971; Mesri & Olson, 1971) the dielectric properties of the pore fluids exercise significant influence on the compressibility characteristics of these clays. Leaching of marine clays can alter the physical and engineering properties of marine deposits (Bjerrum, 1954). An attempt has been made to study the geotechnical behaviour of Cochin marine clays in the background of the above aspects.

As indicated earlier, several series of consolidation tests have been carried out to study the consolidation properties of Cochin marine clays. A number of tests have been carried out on untreated marine clay samples and the remaining were conducted on clays treated with additives.

5.3 COMPRESSIBILITY OF MARINE CLAYS

Several series of consolidation tests were carried out on saturated undisturbed samples from Parur, Nettoor, Maradu, Kumbalam, and Munambam as per standard procedure. The procedure for determining the one dimensional consolidation behaviour has been described in detail in chapter III. In order to obtain "remoulded" samples, soil taken out from the same sampling tube, was worked on thoroughly and hand remoulded into the consolidation ring without permitting any change in natural moisture content.

For the present series, a load increment ratio of one has adopted and the samples were inundated with bore hole water. A duration of two days was given to all samples for each load increment unless mentioned otherwise.

Fig.5.3.1 shows typical e - log p curves for three undisturbed samples from Kumbalam, Elamkulam and Maradu. The samples were collected from depths of 9.5, 4.5 and 9 metres respectively. The curves show preconsolidation pressures consistent with the depths. Fig.5.3.2 shows the e - log p curves for two undisturbed samples from Nettoor. The upper curve has a load increment ratio of one. For the second curve the sample was loaded with smaller load increments to arrive at the preconsolidation pressure more accurately. The curves are more or less parallel indicating that the load increment ratio of one can describe the compression of marine clays satisfactorily.

The e - log p curve for the remoulded sample is also shown in the figure. A comparative study of the curves shows that remoulding destroys the bond strength completely. At higher pressures both undisturbed and remoulded samples almost merge at a void ratio nearly equal to 0.4 e_o which agrees well with Schmertmann's (1955) observations.

Fig.5.3.3 shows some typical e - log p curves of undisturbed samples obtained from three different locations having different liquid limits. The ratio of initial water content (W_i) to liquid limit (W_L) is approximately 0.9 for these curves. It may be noted that the curves are placed in the order of their liquid limits and they tend to merge at higher pressures. Fig.5.3.4 shows another pair of e - log p curves with W_i/W_L nearly equal to 0.7. In both cases higher plasticity curves are placed above those with lower plasticity (Lambe, 1958).

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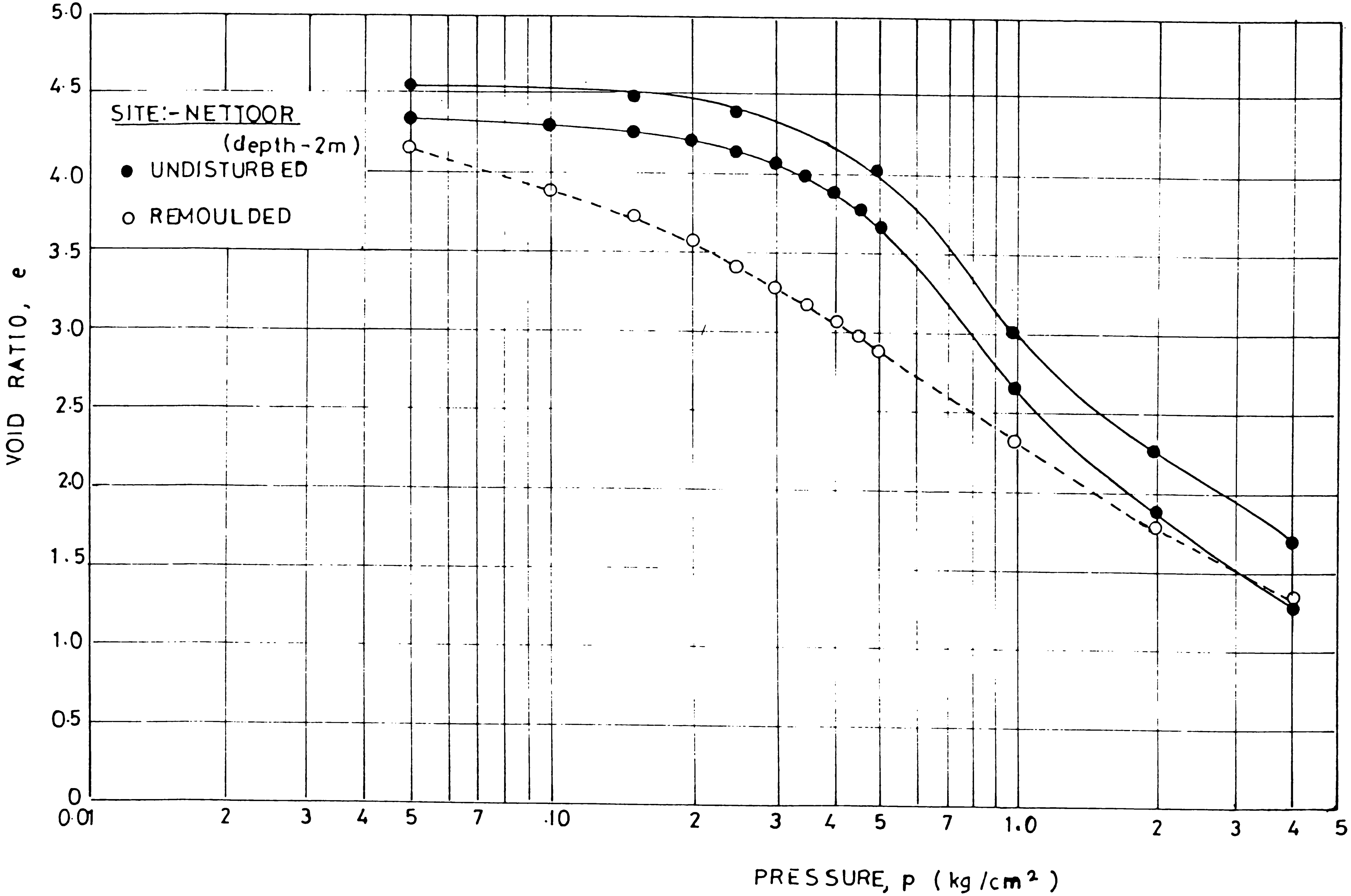

Fig. 5.3.2 : e - log p CURVES.

smaller load increments to arrive at the preconsolidation pressure more accurately. The curves are more or less parallel indicating that the load increment ratio of one can describe the compression of marine clays satisfactorily.

The e - log p curve for the remoulded sample is also shown in the figure. A comparative study of the curves shows that remoulding destroys the bond strength completely. At higher pressures both undisturbed and remoulded samples almost merge at a void ratio nearly equal to 0.4 e_o which agrees well with Schmertmann's (1955) observations.

Fig.5.3.3a shows some typical e - log p curves of undisturbed samples obtained from three different locations having different liquid limits. The ratio of initial water content (w_i) to liquid limit (w_L) is approximately 0.9 for these curves. It may be noted that the curves are placed in the order of their liquid limits and they tend to merge at higher pressures. Fig.5.3.3b shows another pair of e - log p curves with w_i/w_L nearly equal to 0.7. In both cases higher plasticity curves are placed above those with lower plasticity (Lambe, 1958).

IS: 2720 (Part XV) 1965: Methods of test for soils - determination of consolidation properties, gives the method

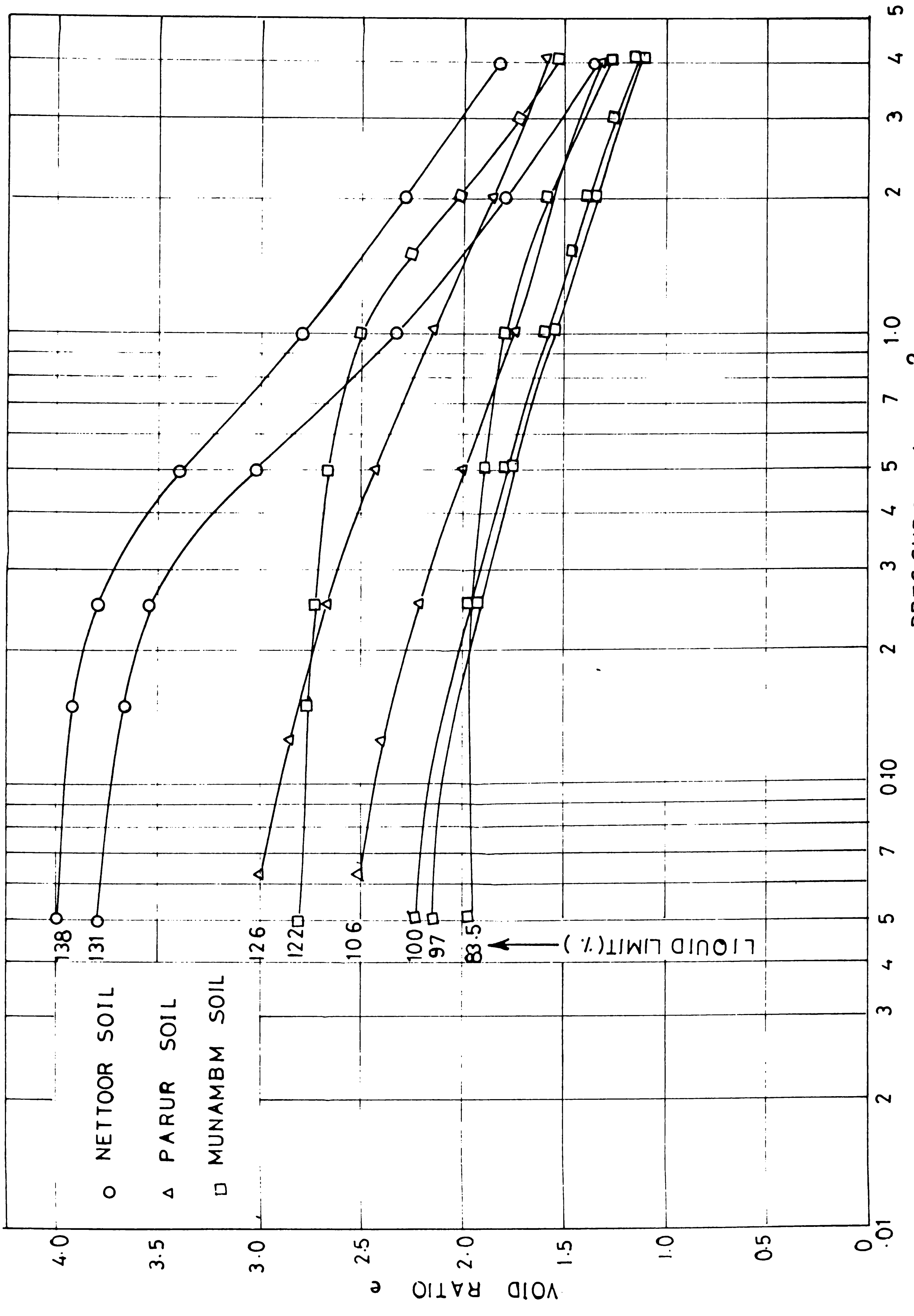

Fig. 5.3.3a TYPICAL e-log p CURVES.

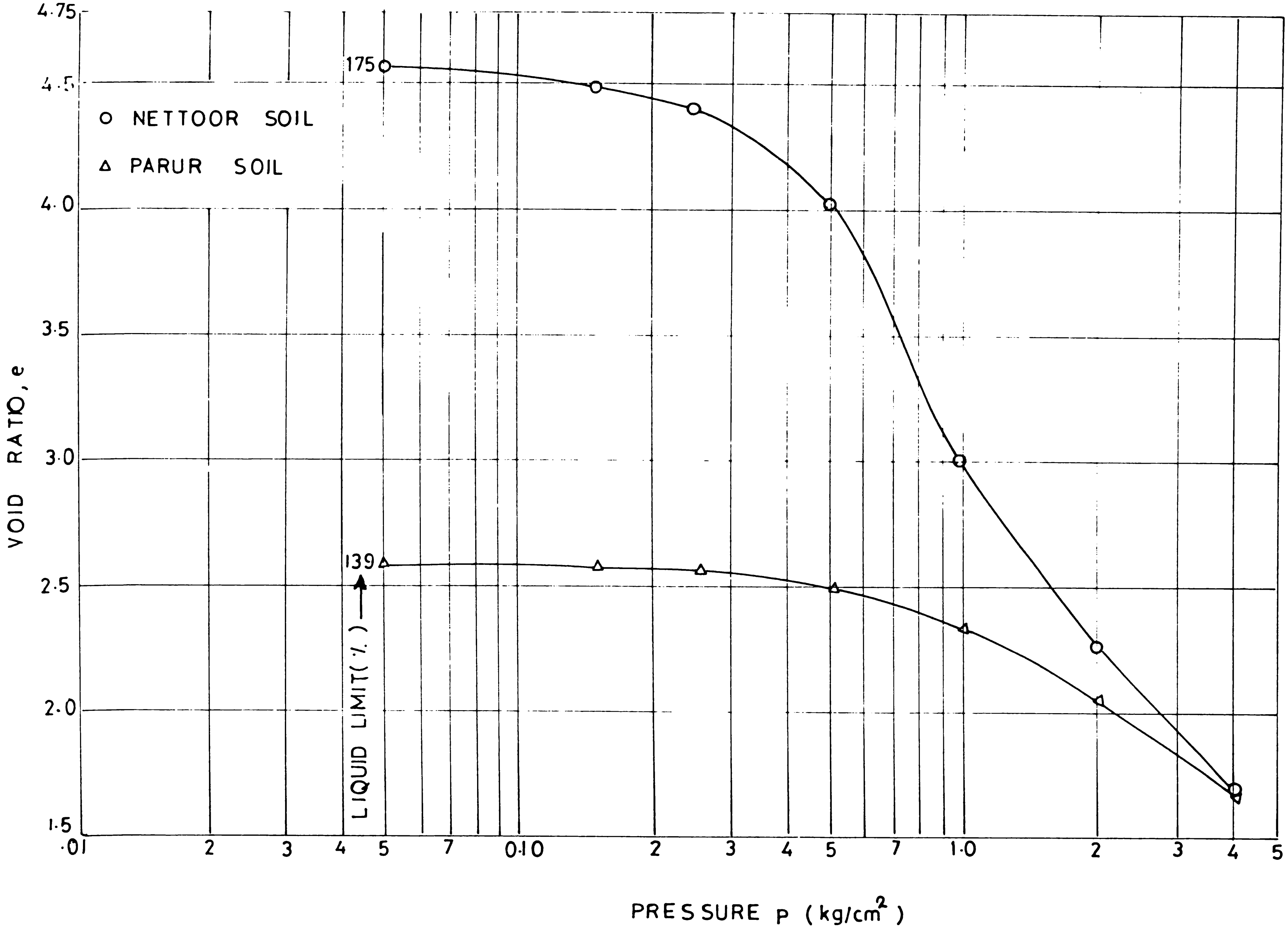

Fig. 5.3.3b TYPICAL e-log p CURVES.

to determine C_c. According to the code, "the initial compression curve would be found to be a straight line at pressures above a certain value. The slope of this straight line portion of the curve shall be reported as the compression index C_c".

The code tacitly assumes that the portion of the e - log p curve after the preconsolidation pressure, p_c is a straight line. But when one considers the numerous consolidation curves obtained in the present investigation, the e - log p curves very rarely is linear after p_c. The slope of the curve varies and quite often there will be a reversal of curvature also. Thus by the value of compression index, as defined by the code, one cannot describe the rate of change of void ratio with increase in pressure. To overcome this anomaly, the compressibility characteristics can better be described by the slope of the e - log p curve for different pressure increments. This slope is given by de/d(log p) for the pressure increment. Throughout this work, the nature of compression curve has been described by the new factor de/d(log p) rather than C_c as it described the compressibility characteristics of marine clays and other soils better than the conventional compression index, C_c.

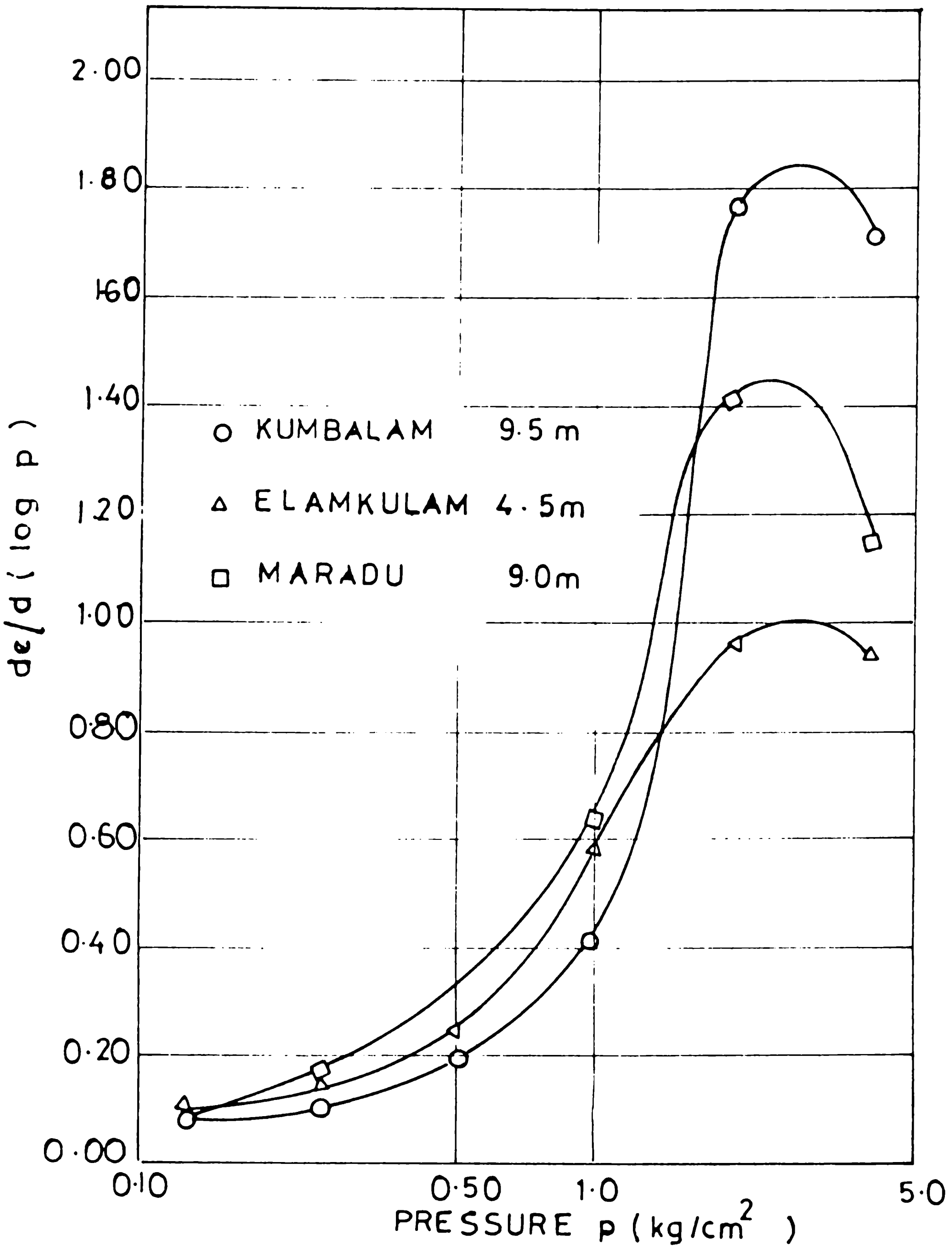

Fig. 5.3.4a de / d (log p) vs PRESSURE.

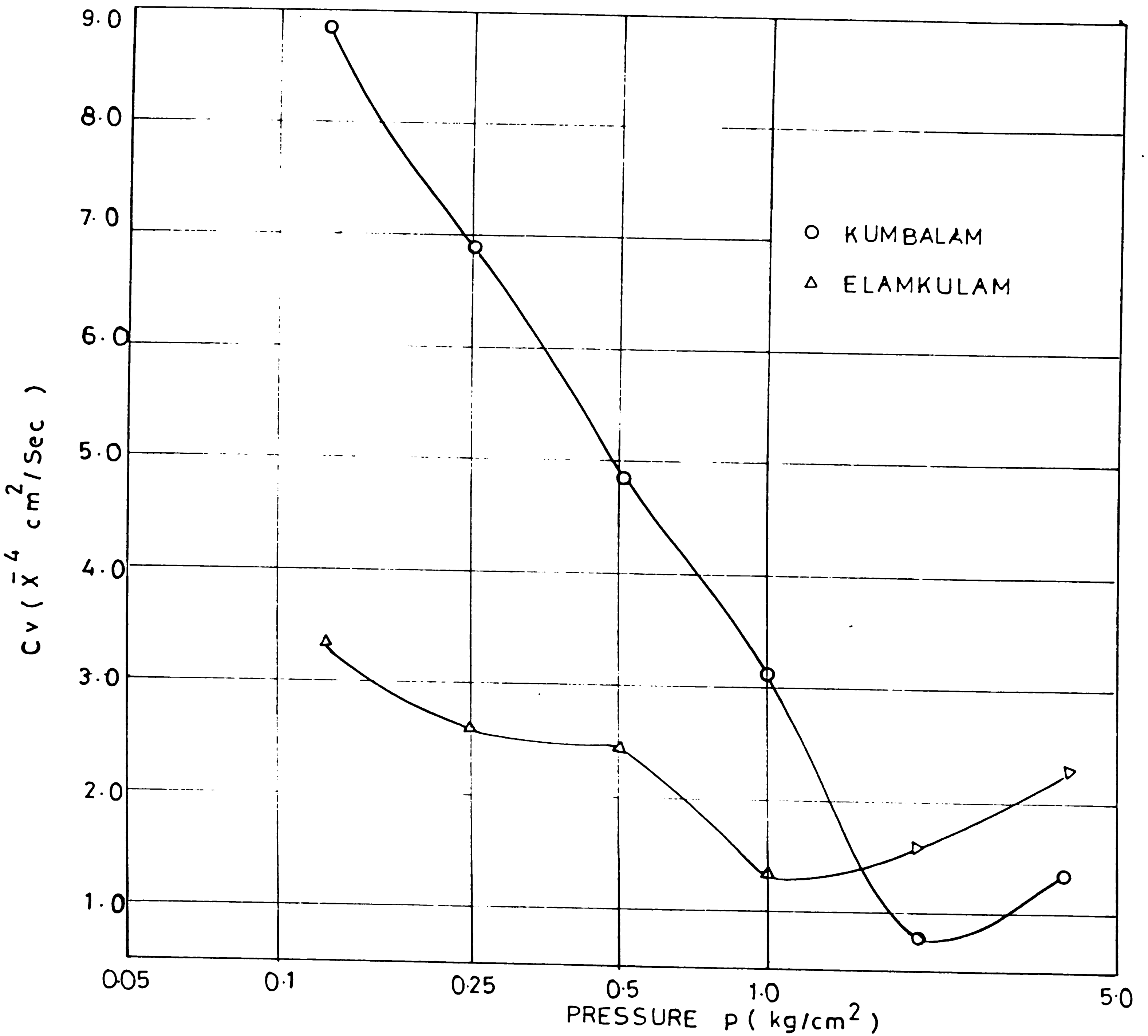

Fig. 5.3.4b : C_v vs PRESSURE.

The ratios of de/d(log p) were calculated for different pressure ranges starting from the lowest value and are reported in Fig.5.3.4a. Although by convention, the compression index C_c is meant to represent the straight line portion of the e - log p curve, here the de/d(log p) values have been calculated for lower pressure ranges also in order to study this behaviour. It can be seen that for undisturbed samples, for pressure ranges below the preconsolidation pressure, the de/d(log p) values are low. The undisturbed samples have yielded larger values of de/d(log p) at higher pressure ranges. Its implications have been brought out in section 5.4. Fig. 5.3.4b shows the variation of C_v values with respect to pressure increments for undisturbed samples from Kumbalam and Elamkulam. The curves show higher values at lower pressure ranges and a minimum between 1-2 kg/cm^2. The values pick up thereafter. But, in general, the behaviour of C_v with pressure has been erratic.

5.3.1 Effect of drying of compressibility

It has already been discussed at length in Chapter IV that air drying or oven drying has significant influence on the grain size distribution of marine clays due to the aggregation of finer particles into coarser grains. The value of liquid limit reduces considerably. The aggregation of particles due to air drying and oven drying brings in significant improvement in the compressibility characteristics of air dried or oven dried marine clays. Through a series of consolidation tests, it has been shown that the value of compression index C_c is brought down by drying.

Fig.5.3.1.1a shows the e - log p curves for undisturbed, remoulded, air dried and oven dried marine clay samples from Nettoor collected from a depth of 1.5 m. While the remoulded sample was subjected to consolidation at the natural moisture content, the initial moisture content of air dried and oven dried samples was kept below, but close to, their respective liquid limit values. It can be seen from the figure that the e - log p curves for air dried and oven dried specimens are considerably below the curves for undisturbed and remoulded samples. Fig.5.3.1.1b shows the same set of curves for specimens from Parur from a depth of 13.5 m. The pattern is quite consistent with the previous figure.

The improvement in compressibility characteristics is fully reflected in the values of de/d(log p). Fig.5.3.1.2 shows the variations of de/d(log p) for the four specimens.

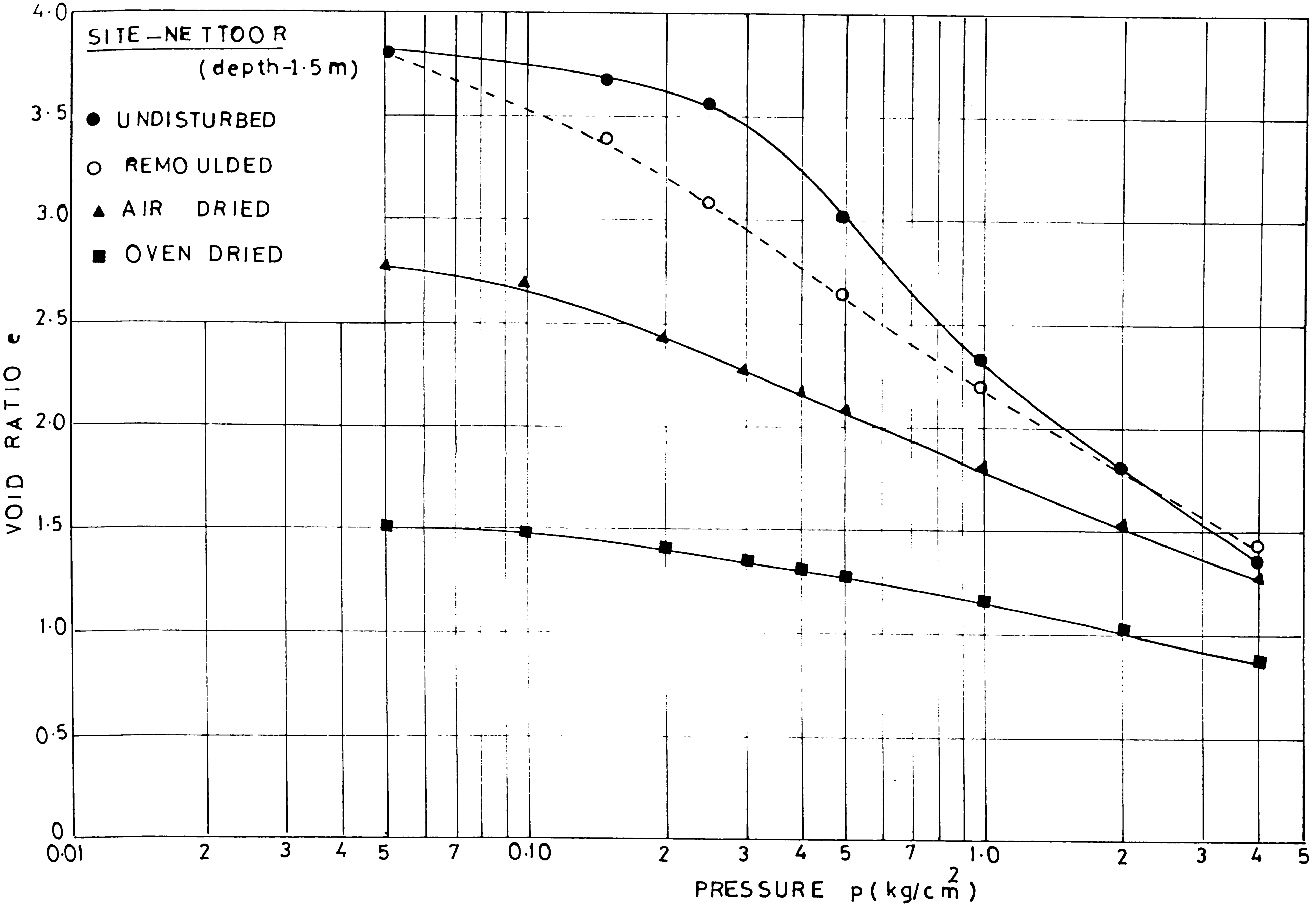

Fig. 5·3·1·1a TYPICAL e – log p CURVES.

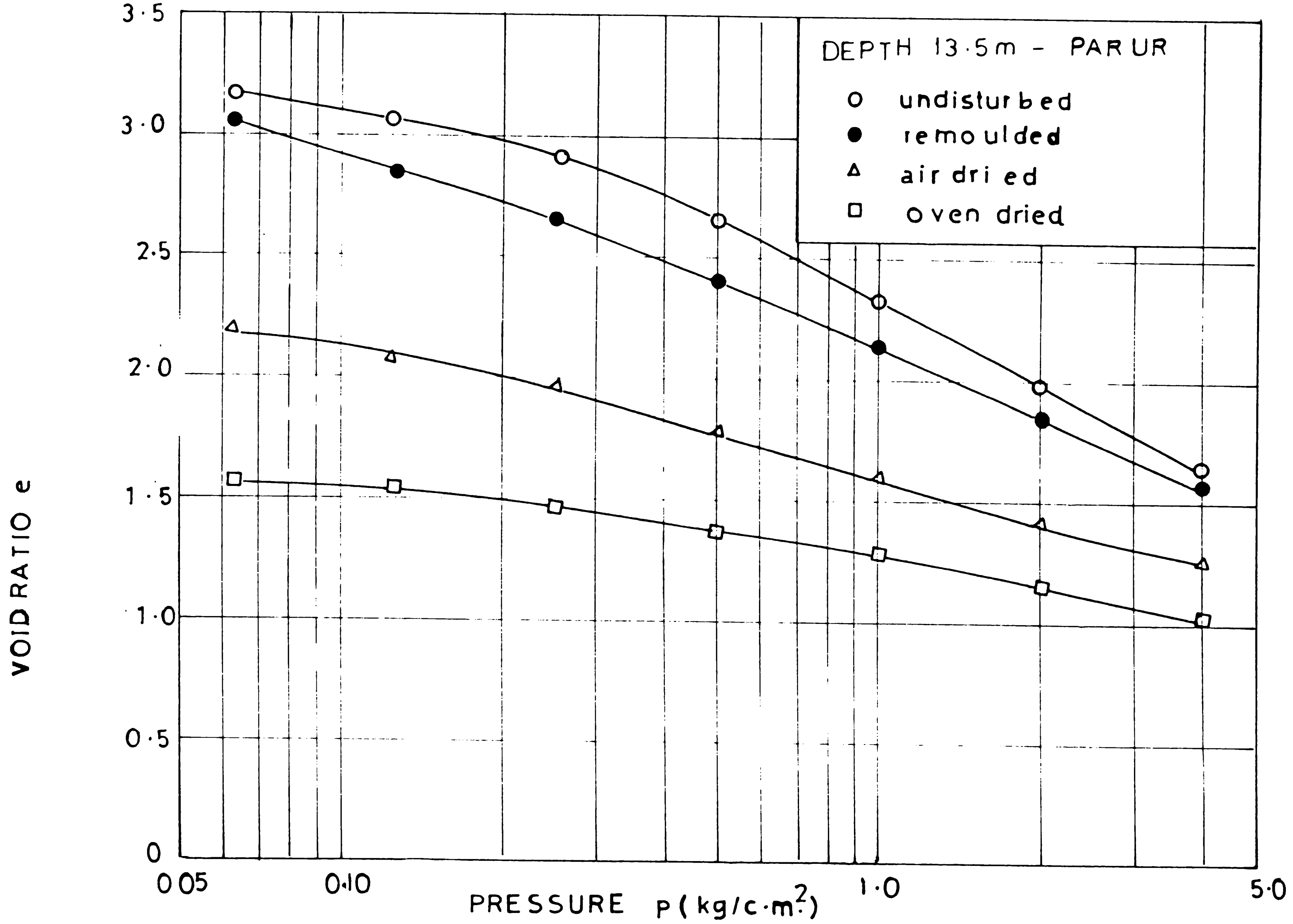

Fig. 5.3.1.1b : TYPICAL e – log p CURVES.

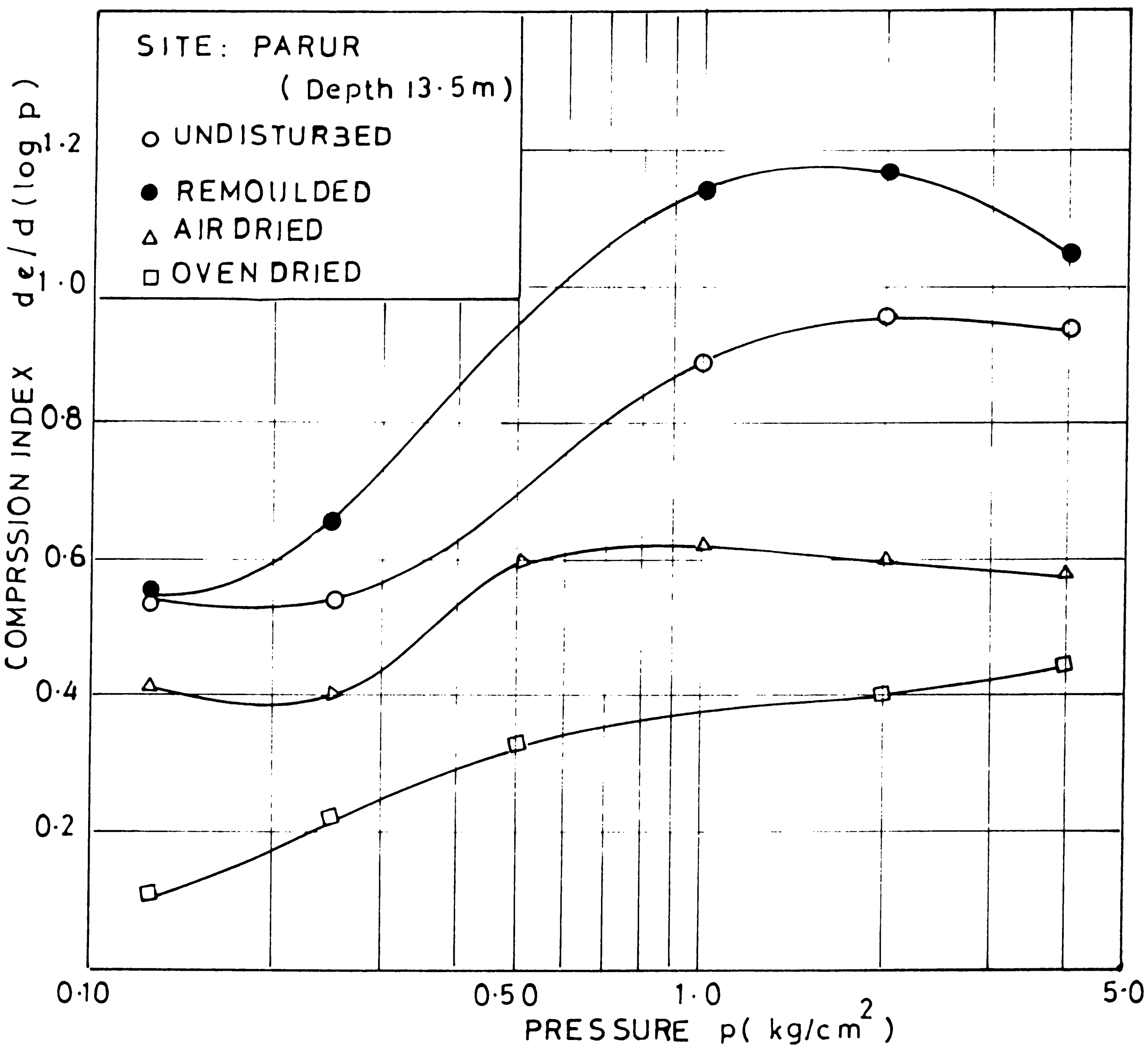

Fig. 5.3.1.2 : de/d(log p) vs PRESSURE.

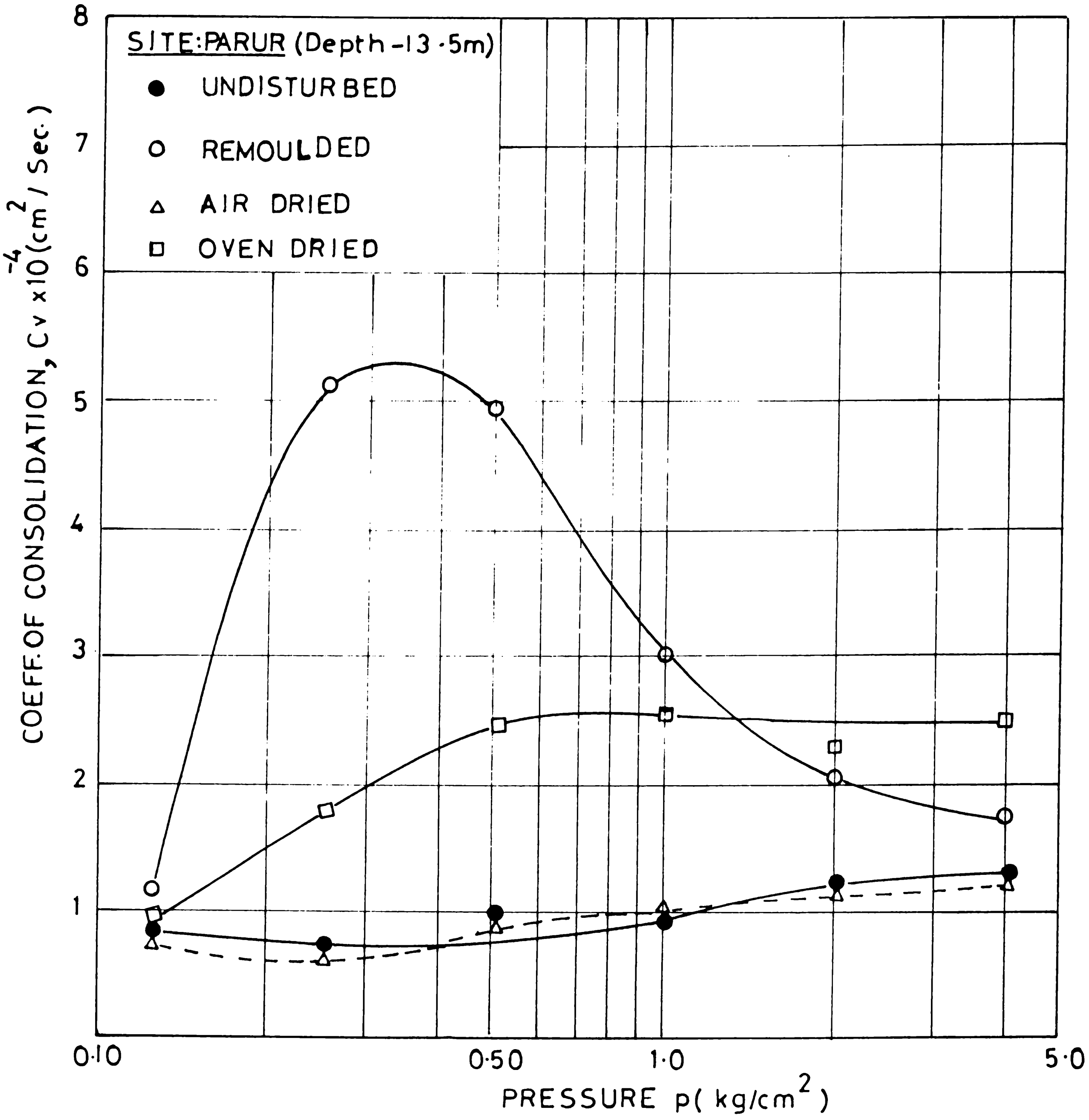

Fig. 5.3.1.3 C_v vs PRESSURE.

While maximum value of de/d(log p) for undisturbed sample is 1.17 for the stress range 1.0-2.0 kg/cm^2, the same for air and oven dried samples are 0.615 (0.5-1.0 kg/cm^2) and 0.438 (2.0-4.0 kg/cm^2). These factors have been taken full advantage of in the studies on compaction of Cochin marine clays which will be discussed later in this chapter. The variation in coefficient of consolidation for the same set of samples is brought out in Fig.5.3.1.3.

Consolidation tests on samples from other locations gave similar e - log p curves.

5.3.2 Compression index

It is well known that as liquid limit increases the compression index increases. Higher plasticity curves are placed above those with lower plasticity (Lambe, 1958). The relationship between C_c and the liquid limit (LL)

$$C_c = 0.007(LL - 10) \qquad (5.3.2.1)$$

for remoulded soils is one of the most widely quoted relationships in soil mechanics literature.

But there are quite a few workers (Saxena et al., 1978; Oikawa, 1987; Wesley, 1988) who feel that $\frac{C_c}{1+e_o}$ represents the compressibility characteristics of fine grained soils better.

Fig.5.3.2.1 gives a plot between de/d(log p) of undisturbed Cochin marine clays and the corresponding liquid limit values of moist samples for the pressure increment of 1 to 2 kg/cm^2. A statistical fit for the values plotted give the equation,

$$de/d(\log p) = 0.011\ (LL - 24.8) \qquad (5.3.2.2)$$

with a correlation coefficient of 0.61.

Fig.5.3.2.2 shows the relation between de/d(log p) and LL for the pressure increment 2 to 4 kg/cm^2

$$de/d(\log p) = 0.011(LL - 20.6) \qquad (5.3.2.3)$$

with a correlation coefficient = 0.65.

To examine the superiority of the factor $\frac{de/d(\log p)}{1+ e_o}$ compared to de/d(log p) alone, to represent the compressibility characteristics of Cochin marine clays, the corresponding

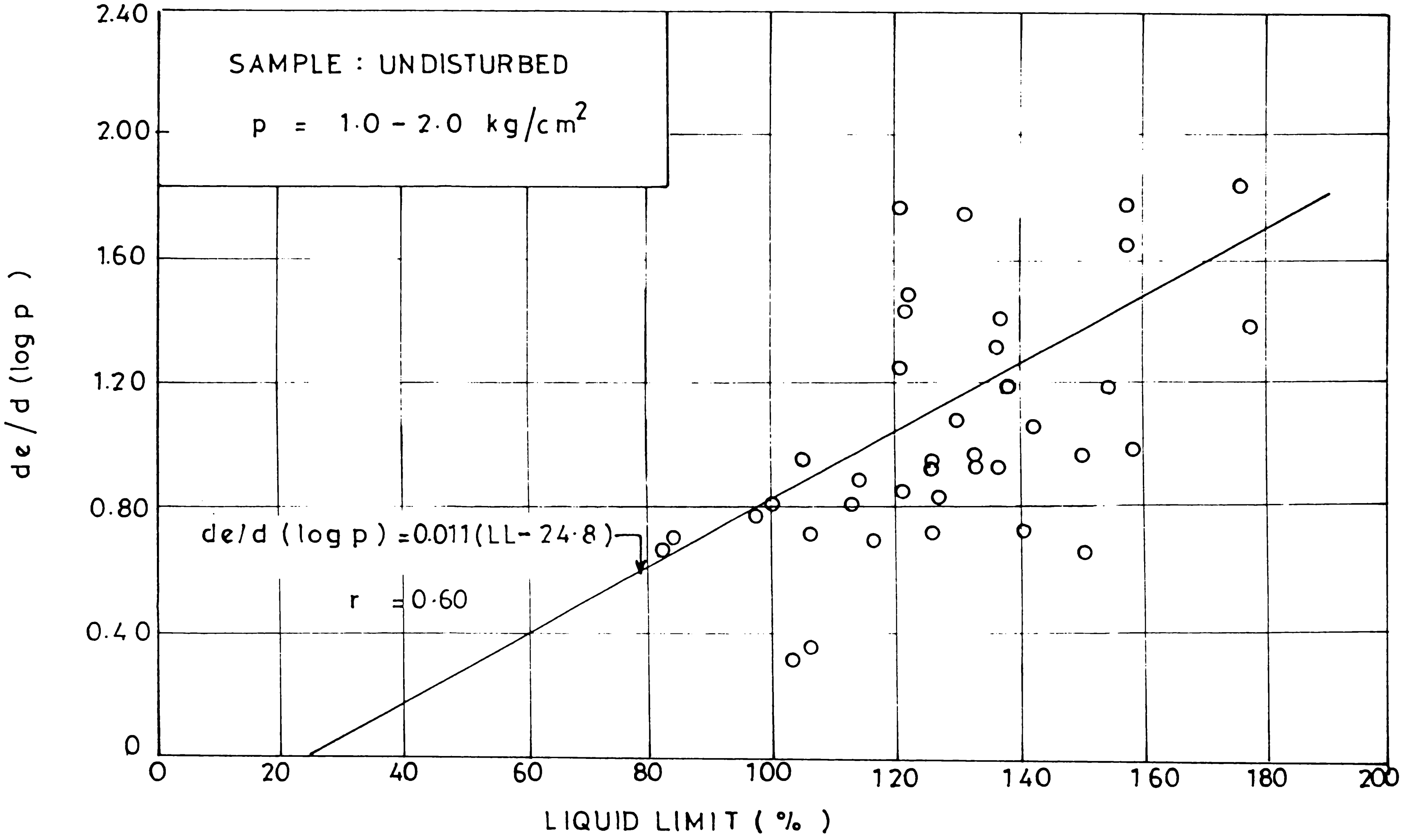

Fig. 5.3.2.1 $\frac{de}{d(\log p)}$ vs LIQUID LIMIT.

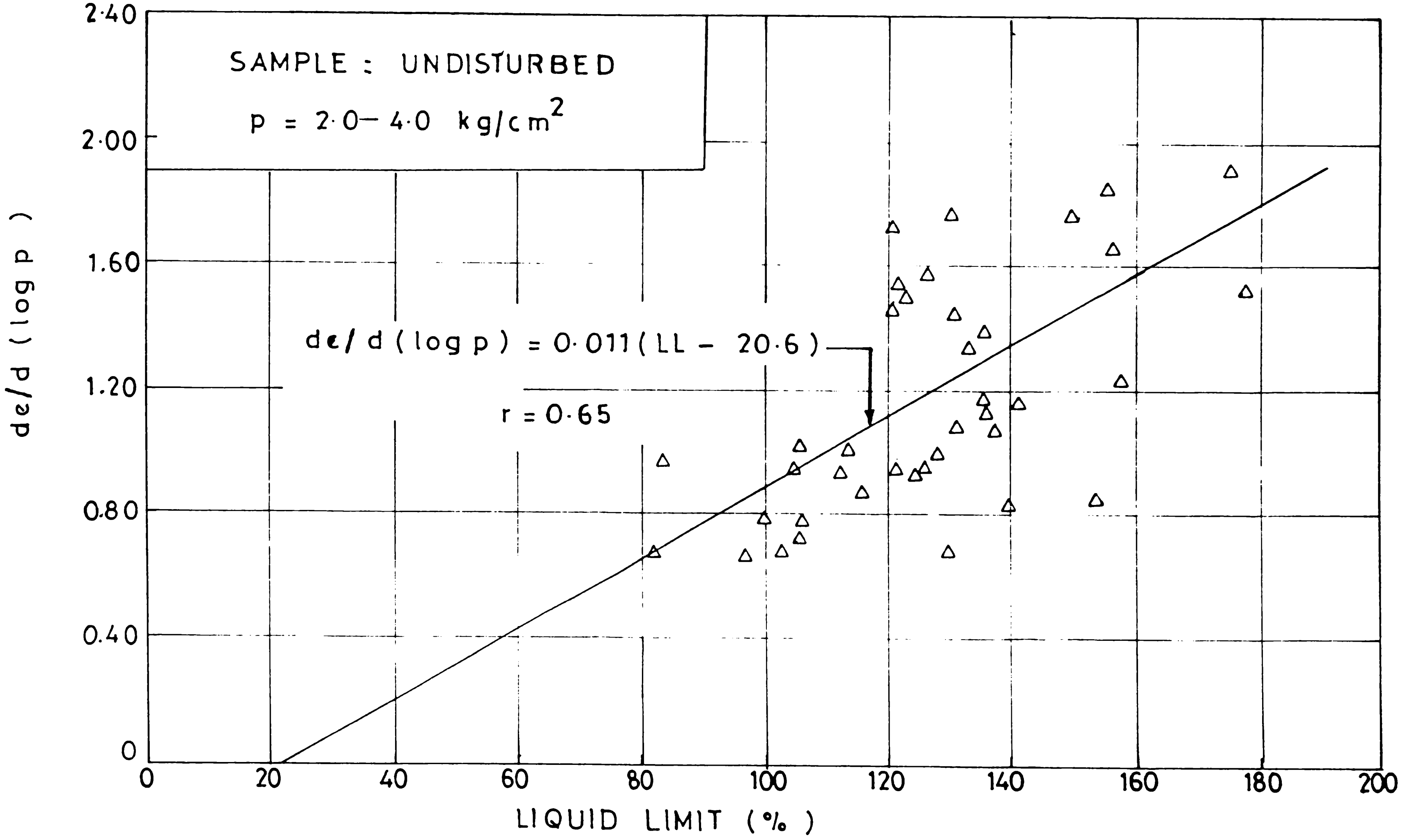

Fig. 5·3·2·2 de/d (log p) vs LIQUID LIMIT.

data have been plotted in Fig.5.3.2.3 for a pressure increment of 1 to 2 kg/cm^2. The statistical fit for the data is given below.

$$\frac{de/d(\log p)}{1 + e_o} = 0.0028(LL - 37.7) \qquad (5.3.2.4)$$

This has a correlation coefficient of 0.651 which is higher than the correlation coefficient for the relation between de/d(log p) and LL.

The relation between de/d(log p) and LL for remoulded clay is given in Fig.5.3.2.4. The equation for the line is

$$de/d(\log p) = 0.0087(LL - 12.6) \qquad (5.3.2.5)$$

where $r = 0.76$.

It can be seen that the undisturbed sample has given a comparatively lower correlation coefficient with a lot of scatter of points. This is possibly due to the non-uniform conditions of the undistirbed samples. On the other hand, because of the uniformity of soil samples, the coefficient of correlation is higher for the remoulded samples. It may be noted that the de/d(log p) - liquid limit relationship obtained for both undisturbed and remoulded samples is about 20-25% higher than those suggested by Terzaghi (1948)

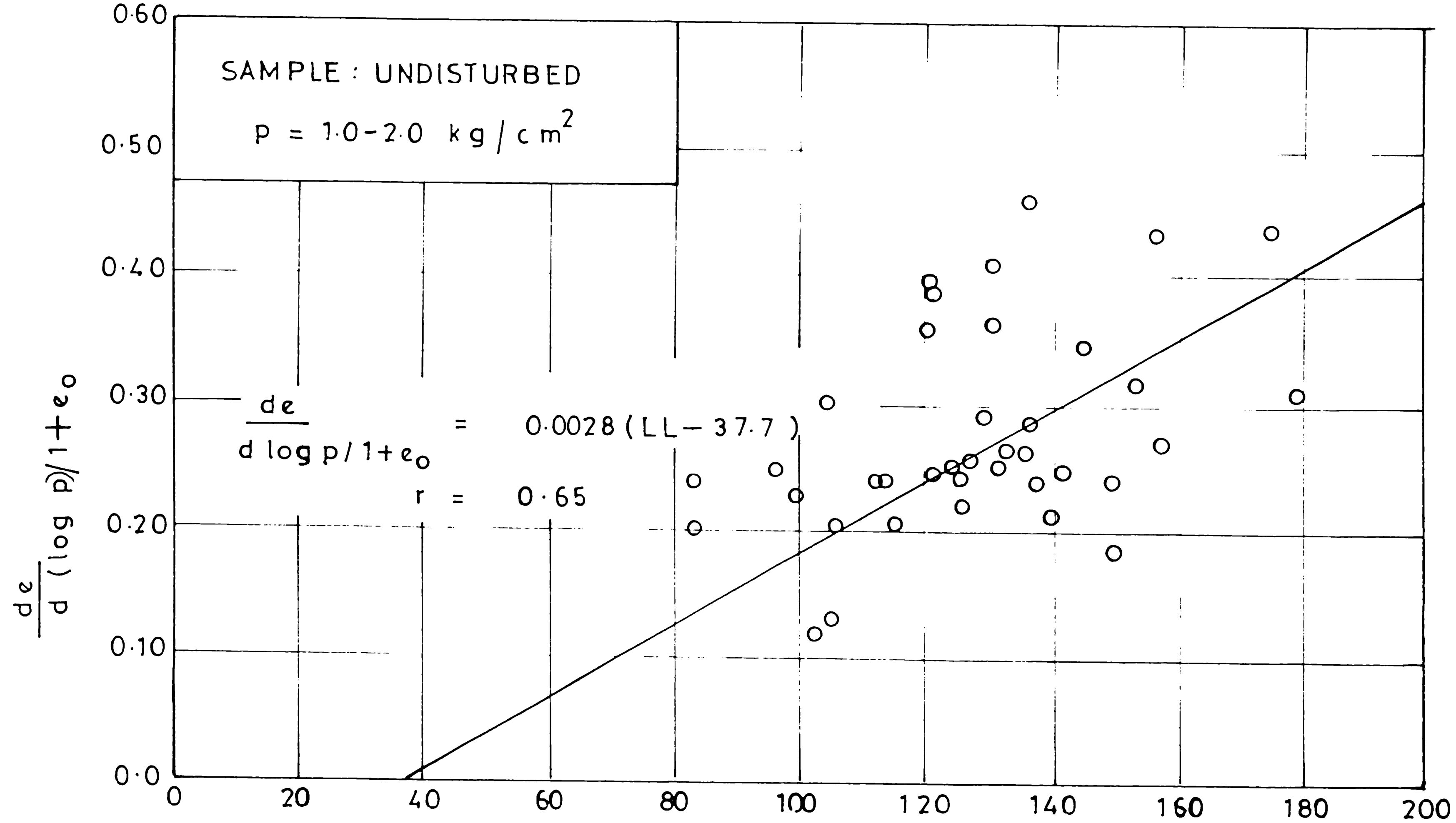

Fig.5.3.2.3 : $\frac{de/d(\log p)}{1+e_o}$ vs LIQUID LIMIT.

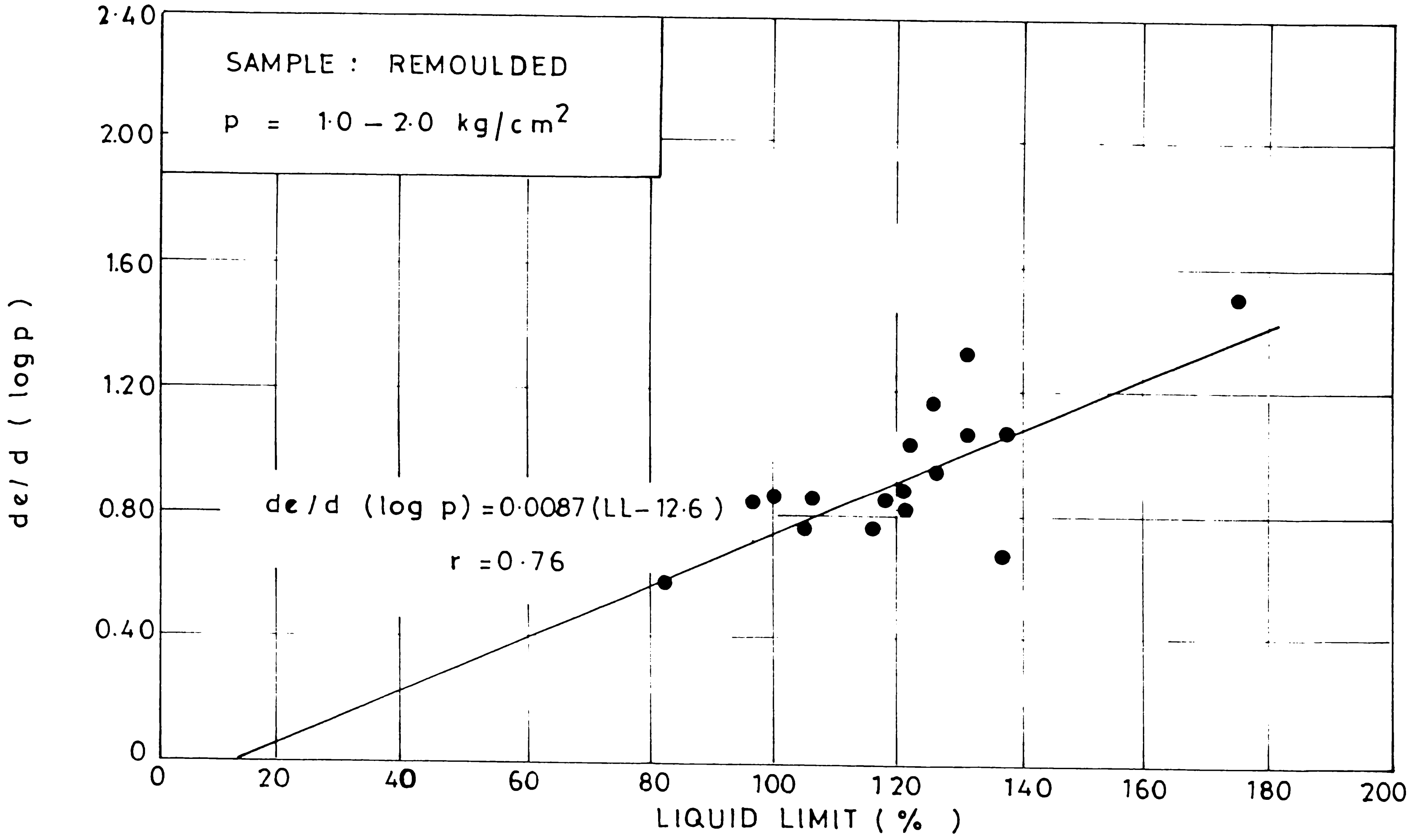

Fig. 5.3.2.4 de/d (log p) vs LIQUID LIMIT.

and Skempton (1944) for undisturbed and remoulded clays respectively, possibly due to the excessive compression of flocs present in Cochin marine clays. The fact that flocculation in Cochin marine clays is of a very high order has been brought out in the section of hydrometer analysis.

The de/d(log p) and liquid limit values for the air and oven dried samples are presented in Fig.5.3.2.5 with the linear relationship

$$de/d(\log p) = 0.0084(LL - 5) \qquad (5.3.2.6a)$$

$$r = 0.92$$

Fig.5.3.2.6 gives the de/d(log p) - liquid limit relation for all samples combined. The equation is

$$de/d(\log p) = 0.0075(LL - 17.8) \qquad (5.3.2.6b)$$

$$r = 0.70$$

The liquidity index for these clays varies over a range of 0.46 to 0.87 indicating that these clays could be taken as medium to very sensitive as per Bjerrum's classification (Bjerrum, 1954).

5.4 COMPRESSION INDEX AND ITS RELIABILITY

The compression index C_c is one of the best known parameters in Soil Mechanics. The relationship between void

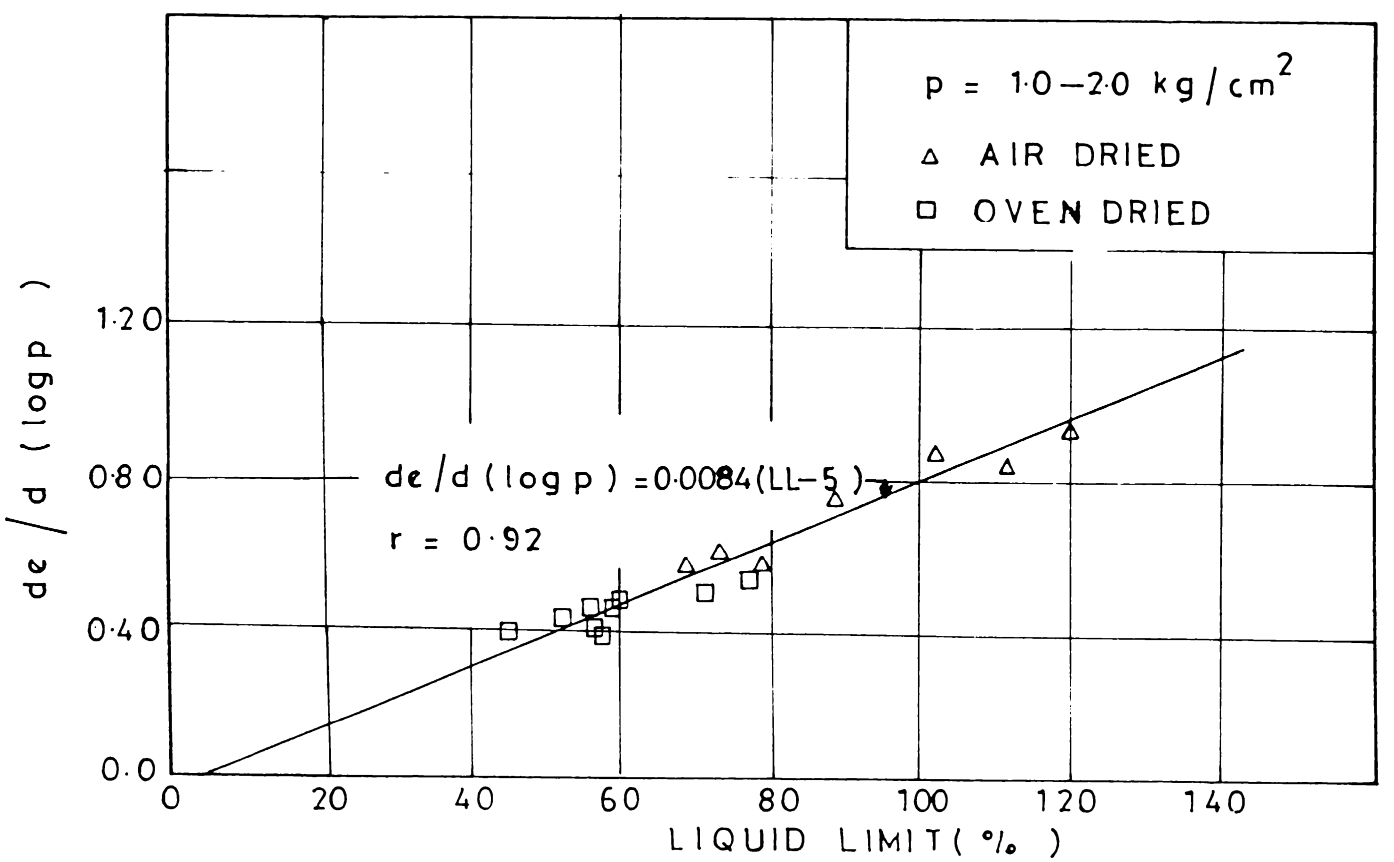

Fig. 5.3.2.5 de/d (log p) vs LIQUID LIMIT.

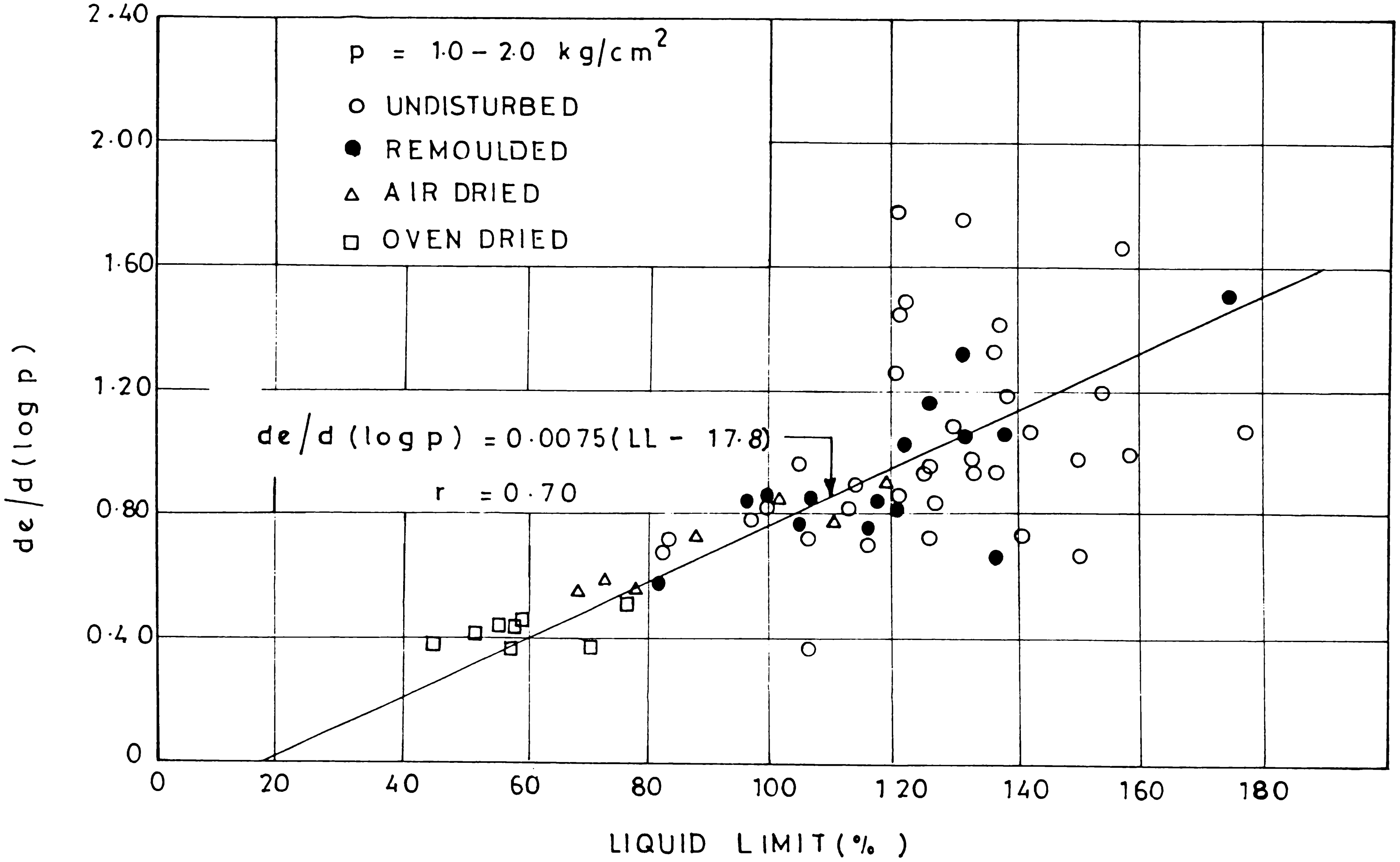

Fig. 5·3·2·6 de/d (log p) vs LIQUID LIMIT.

ratio e and logarithm of consolidation pressure p for ordinary clays can be represented by a straight line in the range of normal consolidation. This property of ordinary clays has become one of the most sacrosanct empirical relationship and perhaps it has been used so widely regardless of its reliability or applicability. The linearity of the e - log p relationship has been questionable especially in case of sensitive clays, organic soils and peats (Oikawa, 1987).

The relationship between C_c and the liquid limit (LL)

$$C_c = 0.007(LL - 10) \qquad (5.4.1)$$

for remoulded soils gained acceptability eventhough its suitability to all situations (especially in case of soils with high liquid limit) is not unquestionable (Wesley, 1988). The formula for the consolidation settlement as per IS: 8009 (Part I) 1976 is

$$S = \frac{C_c . H_i}{1+e_o} \log_{10} \frac{(p_o + \Delta p)}{p_o} \qquad (5.4.2)$$

It is evident from this expression that the soil parameter indicating compressibility is $C_c/(1+e_o)$ and not C_c alone. Comparison of compressibility would be better made on the basis of the compression ratio $C_c/(1+e_o)$ rather than compression index C_c (Saxena et al., 1978).

The variation of compression index de/d(log p) with respect to change in effective stress for undisturbed samples from Parur at a depth of 6.5 m is shown in Fig.5.4.1. The plots show a tendency for the value of de/d(log p) to increase upto pressures between 0.5 to 1.0 kg/cm^2. Relations between de/d(log p) and pressure are given for different durations such as 1 day, 2 days, 4 days, 8 days, 15 days and 30 days. After 0.5 to 1.0 kg/cm^2 the behaviour does not follow any regular pattern. The value of de/d(log p) tends to decrease at higher pressures for longer durations of stage loadings. The reason for this has been attributed to the development of bonds between the clay particles, discussed in section 5.5.

It has been already indicated that the compressibility behaviour is significantly influenced by the initial void ratio of each loading stage and compression ratio might represent the compressibility characteristics better than compression index C_c. In order to examine this, the compression ratio de/d(log p)/(1+e_i) has been plotted against log p for 1, 2 and 4 days duration in Fig.5.4.2 and for 8, 15 and 30 days in Fig.5.4.3. It can be seen that straight lines could be fitted in all the six cases in these two figures if the compression ratio value for the first loading stage viz.

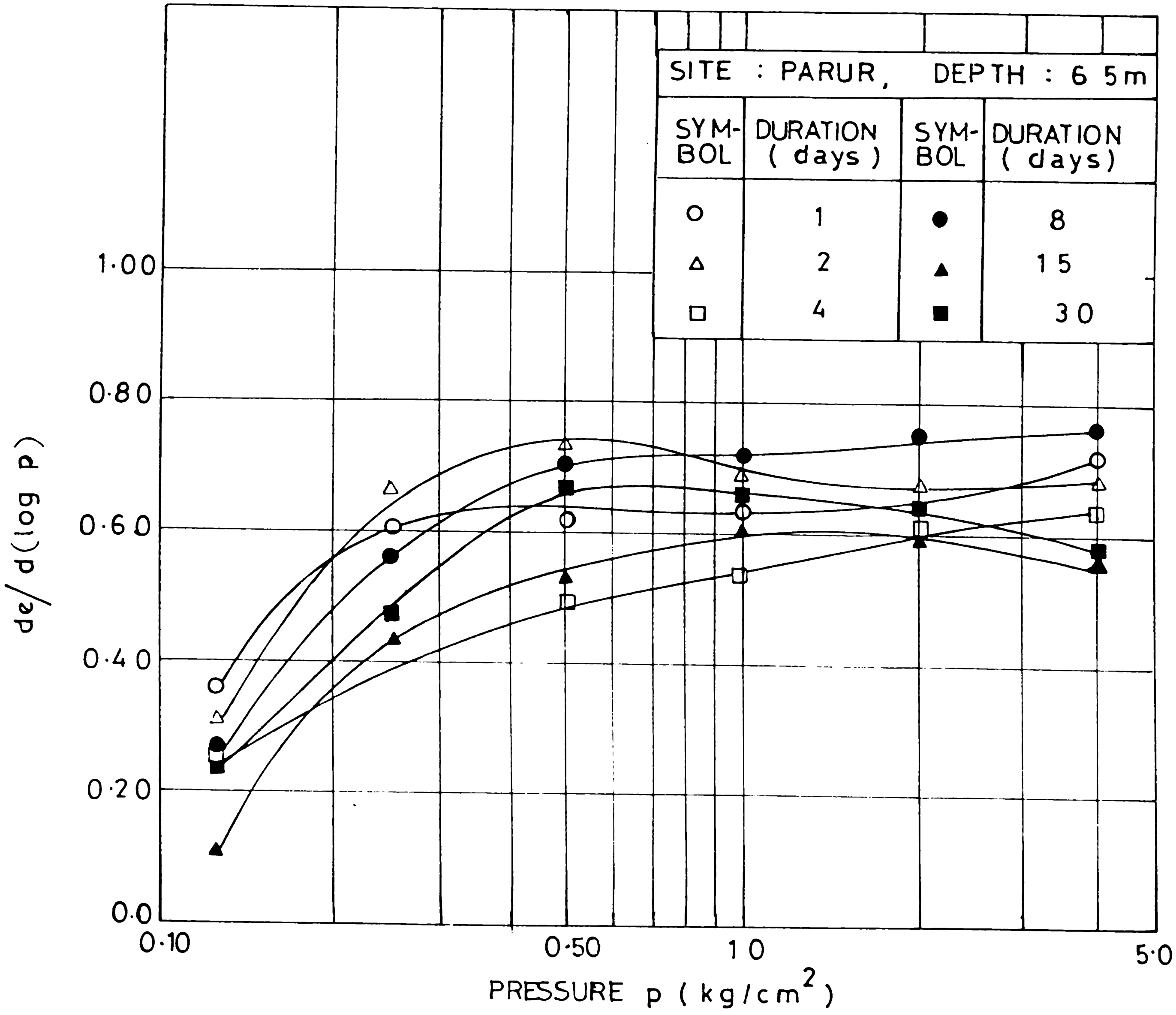

Fig. 5.4.1 de/d(log p) vs PRESSURE.

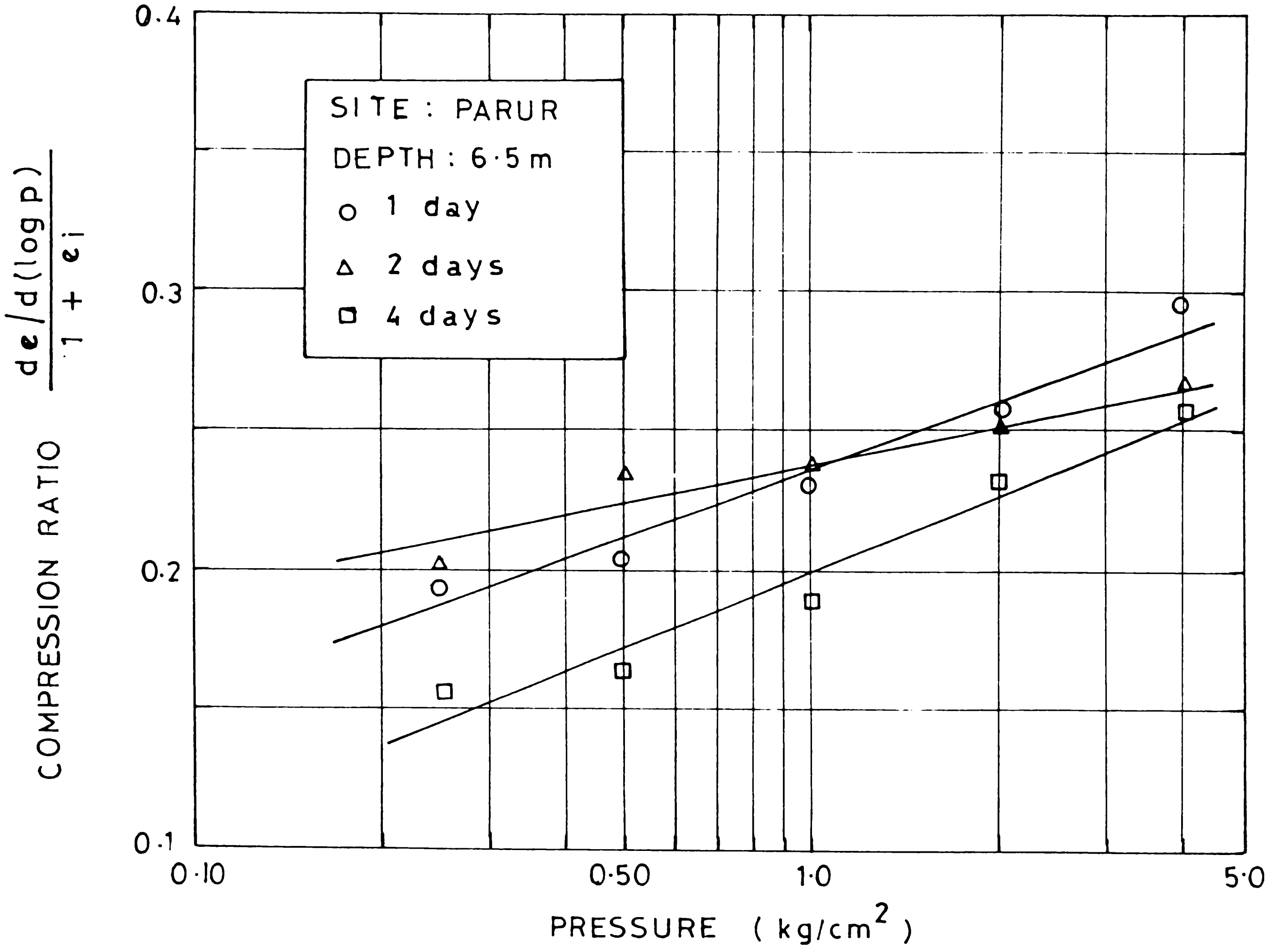

Fig. 5.4.2 : $\frac{de/d(\log p)}{1 + e_i}$ vs PRESSURE.

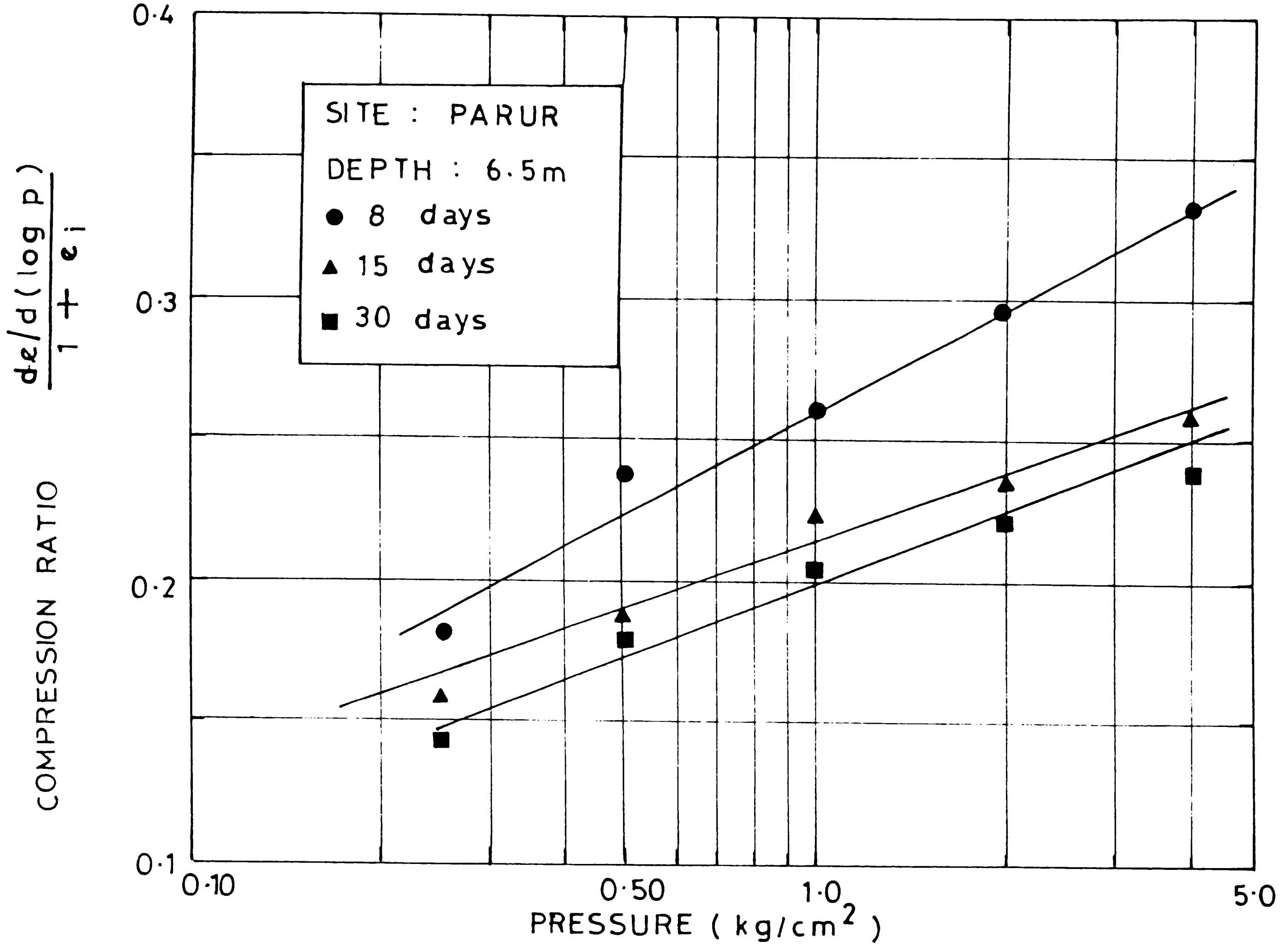

Fig. 5·4·3 : $\frac{de/d(\log p)}{1+e_i}$ vs PRESSURE.

0.0625 kg/cm^2 is left out. Since this point is influenced by the initial compression of the sample and mechanical adjustments of the loading system, its omission may not interfere with the general behaviour pattern. It can also be noted that the slope of the straight line decreases with increase in the duration of stage loading.

Fig.5.4.4 shows the variation of de/d(log p) with p for the same duration of stage loading for undisturbed samples collected from Parur at a depth of 13.5 m. It can be seen that the plots present a highly erratic behaviour. But the compression ratios for the same plotted in Figs. 5.4.5 and 5.4.6 show, uniform correlations. These figures clearly indicate that it is the compression ratio $de/d(\log p)/(1+e_i)$ which is more reliable in the study of compressibility characteristics than the compression index de/d(log p).

Fig.5.4.7 shows the de/d(log p) - log p plots for a series of consolidation tests carried out on four different specimens of soils which were originally consolidated at a sustained pressure of 1 kg/cm^2. The pressures were then released and full consolidation tests were carried out on the samples preconsolidated at 1 kg/cm^2. The failure of the

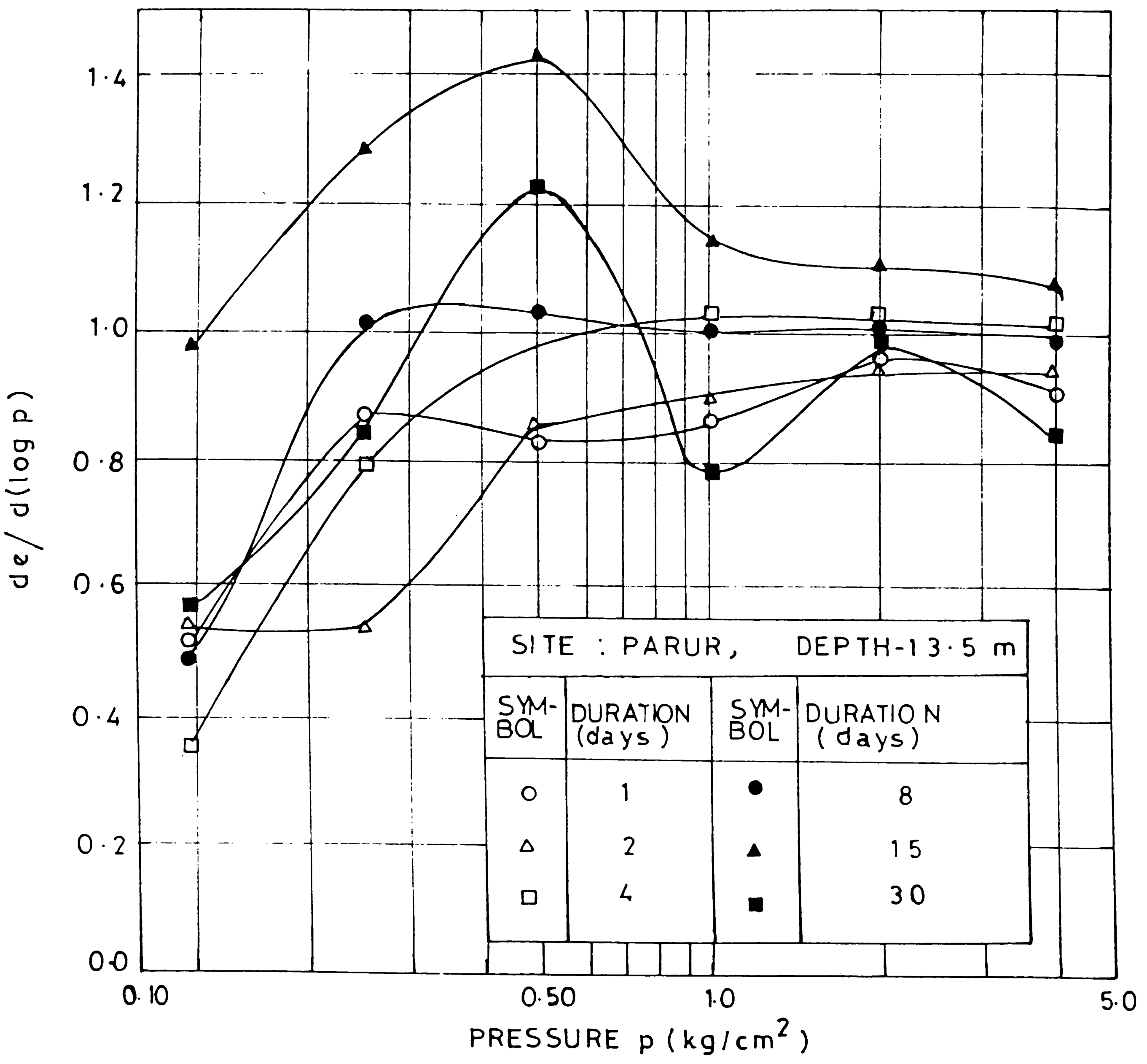

Fig. 5.4.4 de/d(log p) vs PRESSURE.

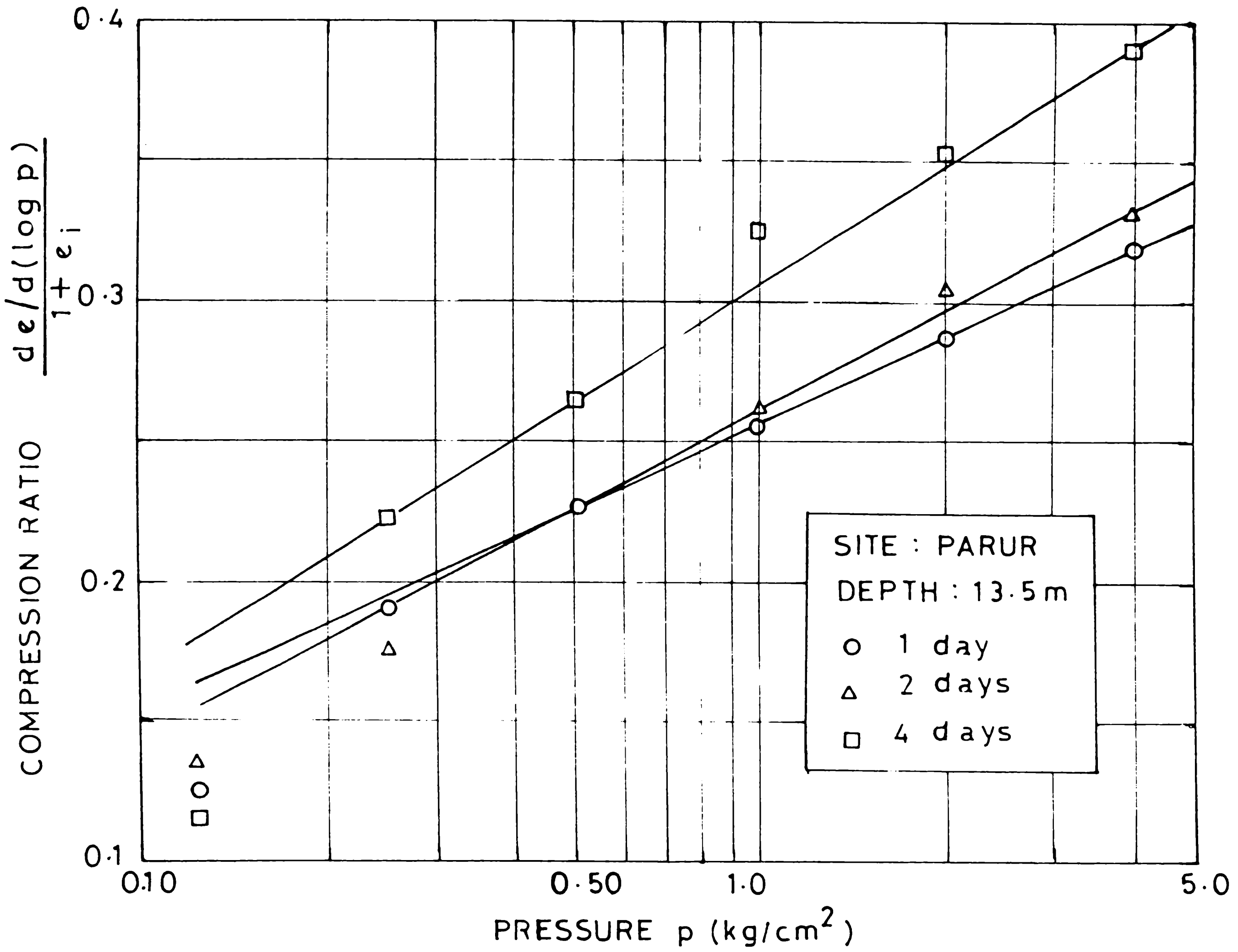

Fig. 5·4·5 $\frac{de/d(\log p)}{1+e_i}$ vs PRESSURE.

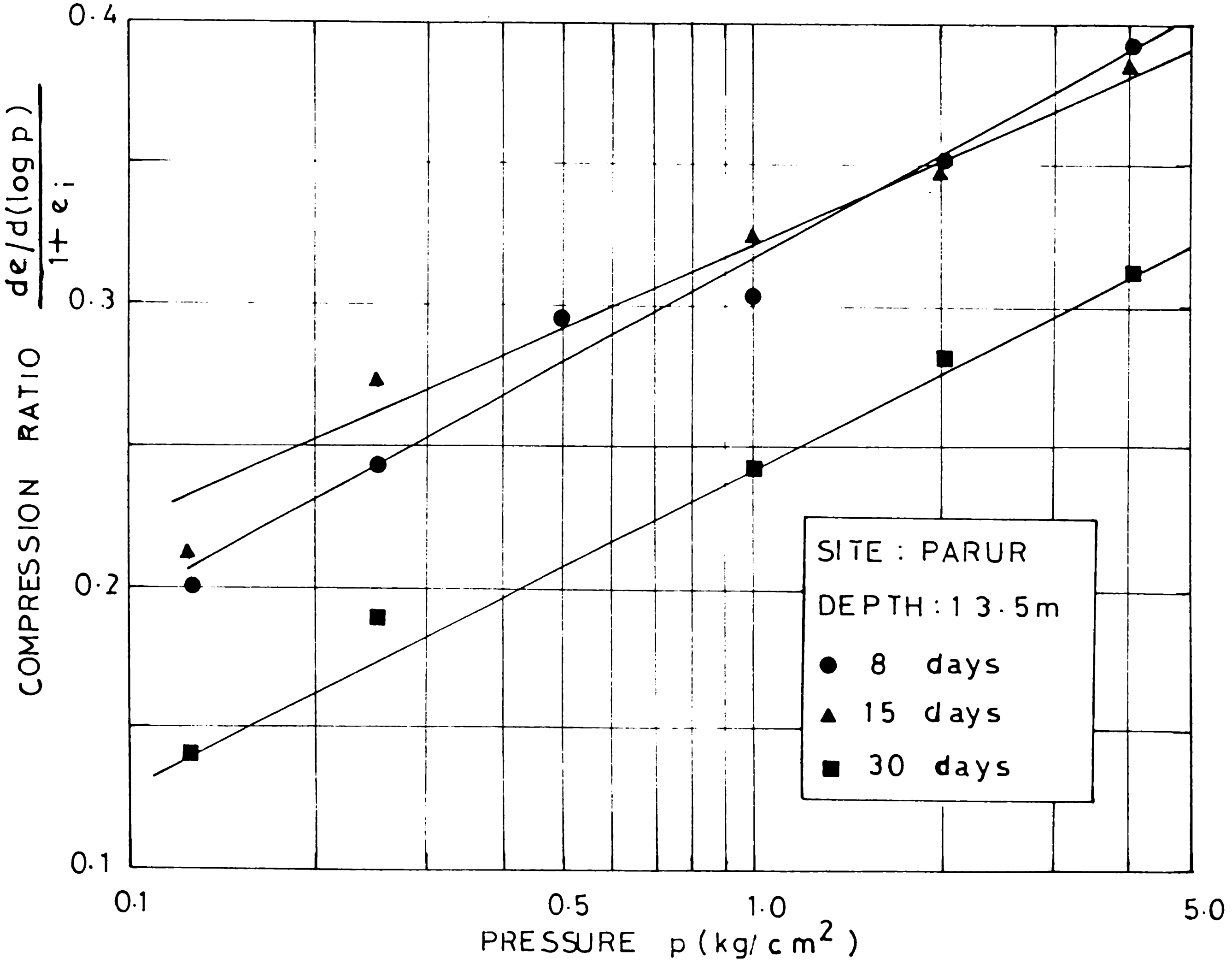

Fig. 5·4·6 $\frac{de/d(\log p)}{1+e_i}$ vs PRESSURE.

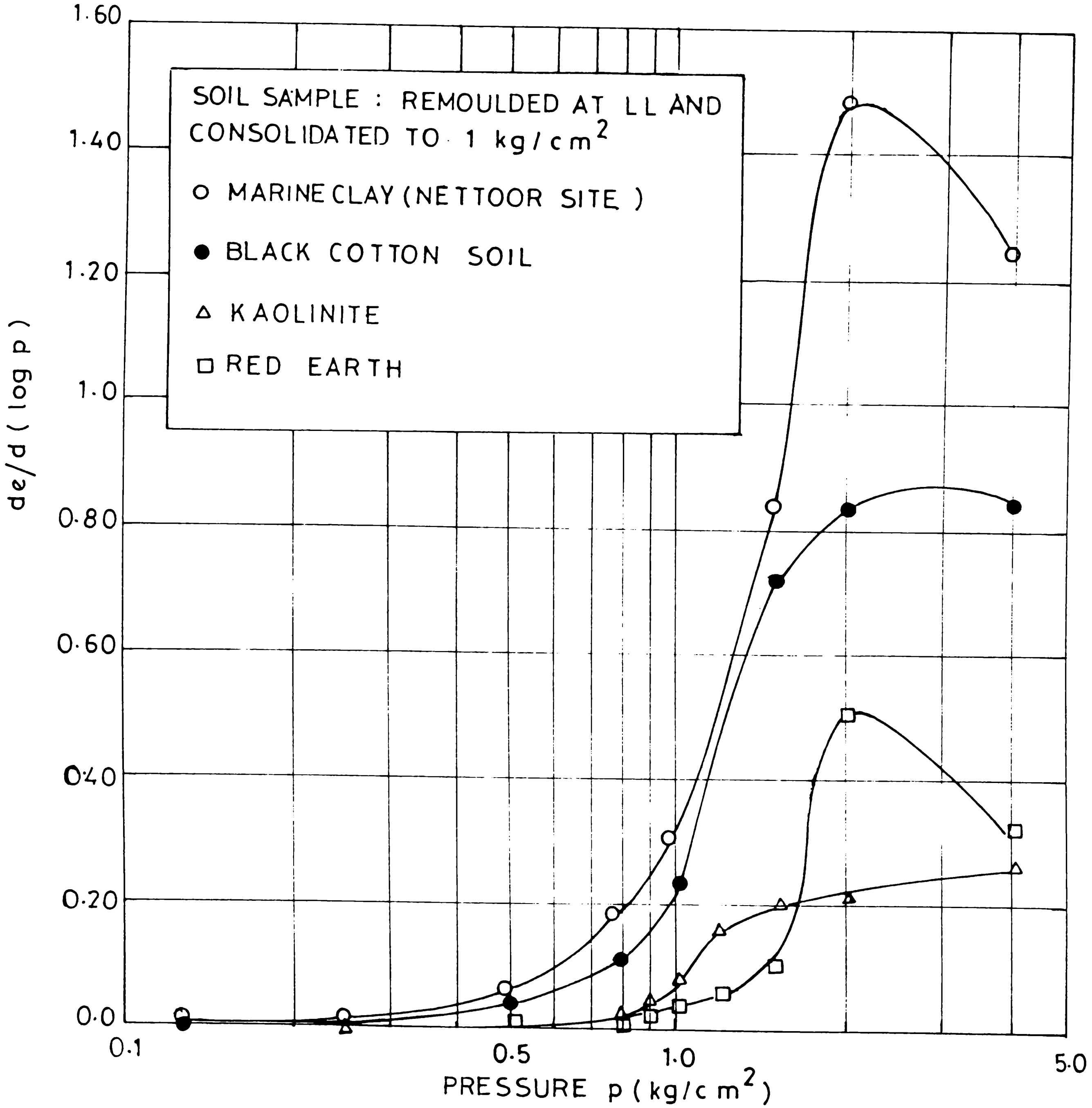

Fig. 5.4.7 de d(log p) vs PRESSURE.

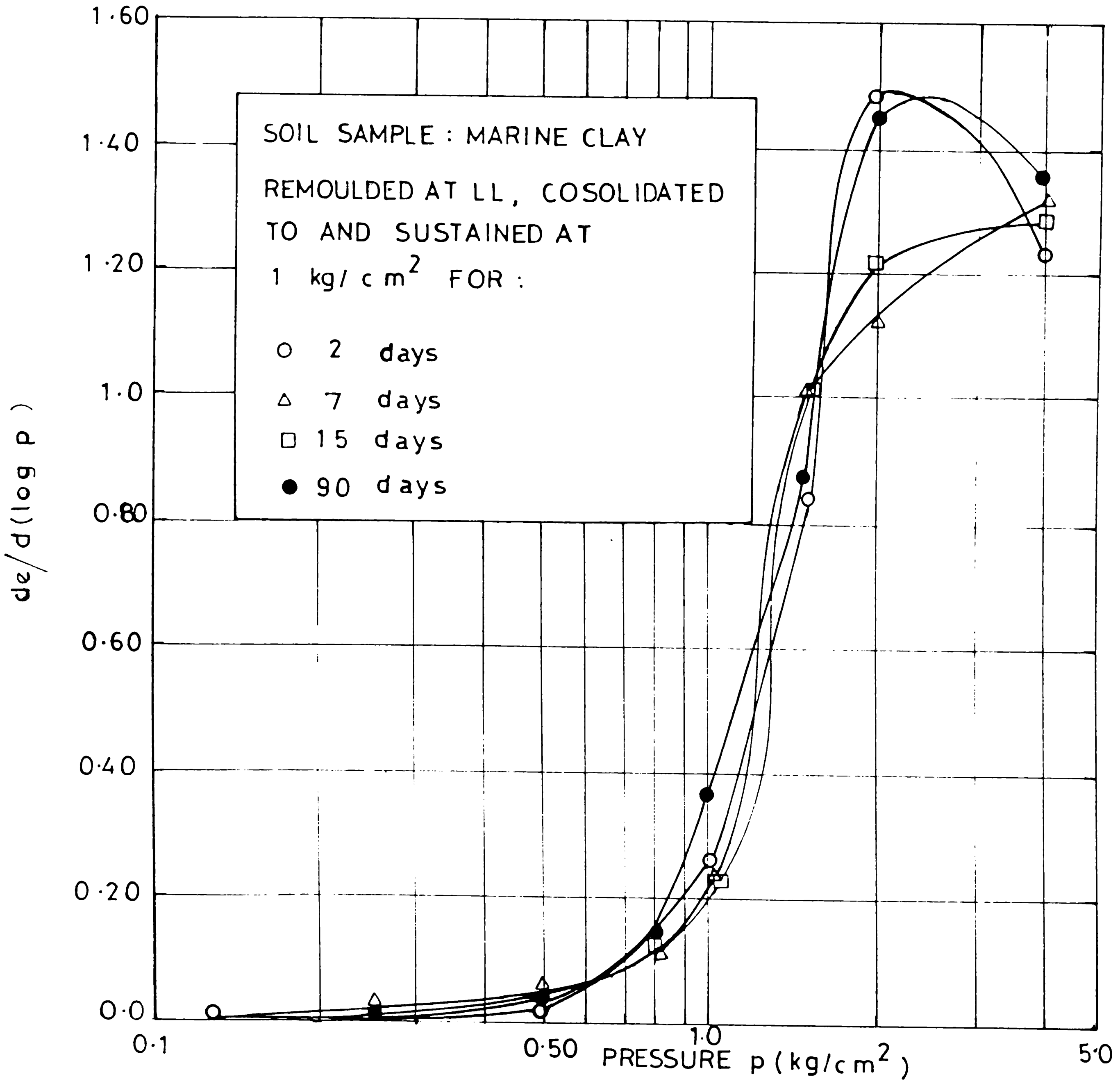

Fig. 5.4.8 : de/d (log p) vs PRESSURE.

fabric of the soils is clearly shown by the sudden increase in the value of compression index de/d(log p). Fig.5.4.8 presents another set of curves between de/d(log p) and log p for marine clay samples preconsolidated at 1 kg/cm^2 for different durations. The variation of de/d(log p) is similar to that in Fig.5.4.7 except that the final value of de/d(log p) tends to be almost same.

5.4.1 Prediction of total settlements from compression index

The determination of allowable soil pressure is one of the most important aspects in the practice of geotechnical engineering. The allowable soil pressure is directly related to the permissible settlements of the structure and its foundation. This necessitates an accurate assessment of the rate and magnitude of the consolidation settlement of a foundation due to the compressible layers beneath the foundation, under any load.

The process of determination of allowabsle bearing pressure involves the determination of the following parameters.

a) Compression Index, C_c

b) Preconsolidation Pressure, p_c.

While the I.S.Codes (IS: 1904, IS: 2720 (Part XV); IS: 8009 (Part I)) specify various methods and procedures to arrive at the above values and compute the quantum of settlement leading to the estimation of allowable soil pressure, quite often it is observed that the actual settlement due to consolidation is considerably less than the predicted settlement.

The city of Cochin and its suburbs consist of deep deposits of highly compressible marine clay with C_c values around 1.5 and natural moisture content very close to the liquid limit. It has been often observed that almost all the structures, founded on spread footings or raft foundations invariably show actual settlements far below the computed values. Since the actual settlements were significantly less than the computed values, designs based on geotechnical principles tended to be on the uneconomical side whereas foundations provided as per thumb rules proved more economical. These findings prompted the present study which shows that there exists a need for a relook into the code provisions to redefine certain terms and amend certain methods and procedures so that the settlement computations are more realistic and designs more economical.

The parameter, which controls the total settlement, is the compression index C_c. The total settlement is directly proportional to this value. C_c is defined (IS: 2720 (Part XV) as 'the slope of the straight line portion of the curve' (e - log p curve). In order to obtain a realistic picture of the problem, undisturbed samples were collected from Kumbalam, Cochin, the soil profile of which is shown in Fig.5.4.1.1.

Fig.5.4.1.2 shows a typical e - log p curve for an undisturbed sample of Cochin marine clay collected from a depth of 9.5 metres. As per the definitions (IS: 2720 (Part V)), the value of C_c is 1.69.

As per IS: 8009 (Part I), 'Code of Practice of Calculation of Settlements of Foundations - Shallow Foundations', for a clay which is not precompressed, the total settlement is calculated from

$$S_f = S_{oed} = \frac{H_t}{1+e_o} C_c \log(\frac{p_o+\Delta p)}{p_o})) \qquad (5.4.1.1)$$

where S_f = final settlement, m

S_{oed}= settlement computed from 1-D consolidation test

H_t = thickness of soil layers, m

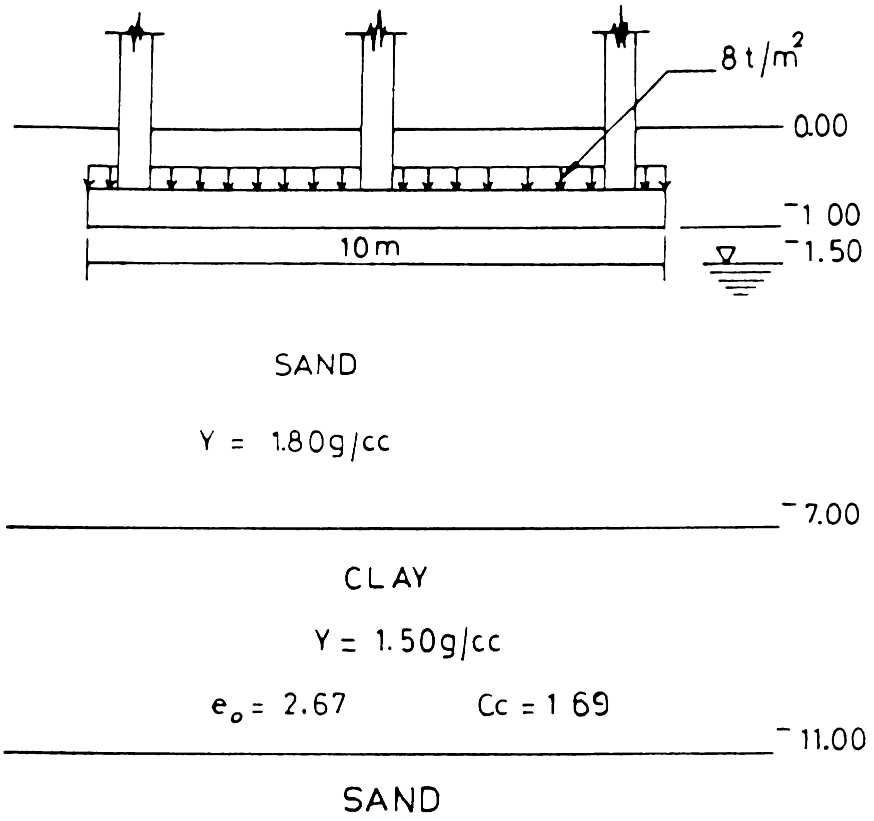

Fig. 5.4.1.1 FOUNDATION DETAILS WITH SOIL PROFILE.

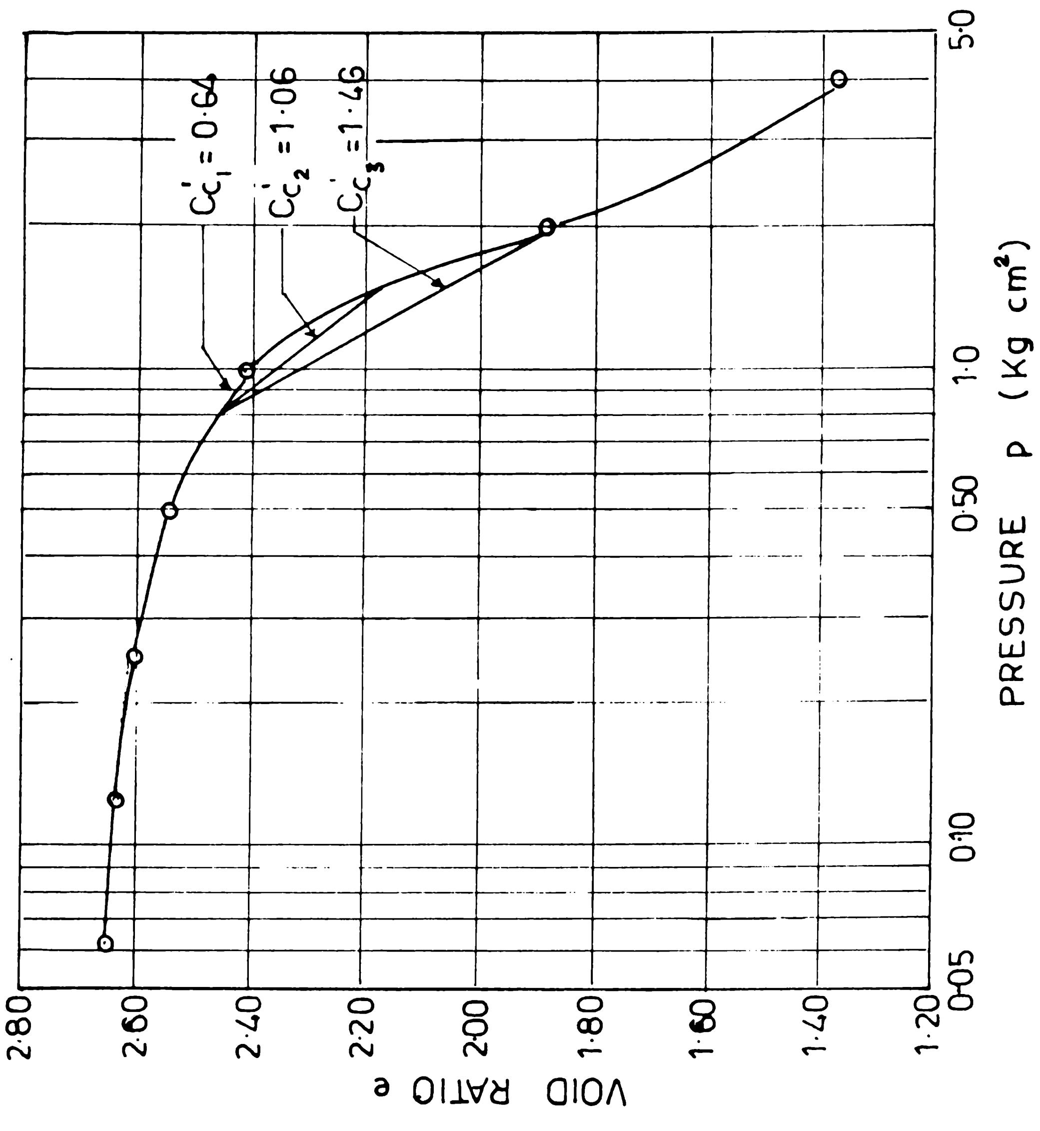

Fig. 5·4·1·2 SEGMENTAL COMPRESSION INDEX C'_c

e_o = initial void ratio

p_o = initial effective pressure at mid height of layers, kg/cm^2

Δp = pressure increment, kg/cm^2.

Field compression curve developed from the laboratory curve for the above soil sample, yields a C_c value of 1.99 for the clay of ordinary sensitivity, as per Appendix C of IS: 8009 (Part I).

For a 10 m square structure resting on a mat foundation the effective overburden pressure is 0.8 kg/cm^2 and increase in stress Δp = 0.246 kg/cm^2, at the middle of the clay layer for a foundation pressure of $8t/m^2$. The total settlement calculated from the above expression is 25.25 cm.

As per IS: 1904-1978 - 'Code of Practice for Structural Safety of Buildings: Shallow Foundations', the maximum permissible settlement for a reinforced concrete structure with raft foundations resting on plastic clay is 10 cm. Thus, as per code provisions, the allowable soil pressure is far less than $8t/m^2$. This, from experience, is far from reality. The allowable pressures obtained from codes are considerably lower than values that can be permitted judging from field experience. Thus strict adherence to the codes yields a highly uneconomical design in case of the highly compressible Cochin marine clays.

5.4.2 Segmental compression index

Since field observations in almost all cases show that the codal provisions yield unrealistically high values for C_c, it is felt that the value of C_c needs a redefinition. Further, C_c at present is taken as the slope of that portion of the e - log p curve where the soil is subjected to compressive stress of the order of 40 or 80t/m^2 which is rarely experienced in actual practice. The initial portion of the e - log p curve, especially that near the insitu stress, describes the compressibility characteristics of the clay in that pressure range better than the portions of the curve in the vicinity of stresses of 40 or 80t/m^2, which at present is taken as the measure of compressibility characteristics of the clay, irrespective of the stress range or stress increase. This, it is felt, is unrealistic and is the reason for the high values of total consolidation settlements computed from code provisions.

The true representation of the consolidation of the clay, when subjected to an increase in pressure Δp is given by the portion of the curve between the abscissas p_o and $p_o + \Delta p$ (Fig.5.4.1.2). The slope of the line joining points (e_o, p_o) and (e, $p_o + \Delta p$) can give a better representation of the field compression of the clay. The slope of this line, called the 'segmental compression index', C_c', is a variable quantity whose maximum value will always be less than C_c.

In the e - log p curve shown in Fig.5.4.1.2 for the stress range of 0.8 kg/cm^2 to 0.8 + 0.246 kg/cm^2, the segmental compression index works out to 0.64, which is considerably less than the original value of 1.99 from the field compression curve. The total settlement, with the new value for C_c' works out to 7.61 cm only. The estimation of the settlement could be further improved by dividing the thickness of the compressible medium into a number of layers and taking corresponding C_c'.

Thus a bearing capacity of $8t/m^2$ which was not permissible according to I.S.Code provisions is well within permissible limits as per the new method. As is clearly evident, segmental compression index method will give more realistic and economical designs.

Fine grained soils, under constant pressure, can develop bond strength. This will be reflected in the value of preconsolidation pressure as quasi preconsolidation pressure. For foundation loads, which induce a Δp within the quasi preconsolidation pressure, the consolidation settlement will be very marginal. This is truly represented by the segmental compression index approach where the C'_c will be almost equal to the slope of the tangent at p_o.

It is evident that C'_c is a quantity which varies between the slope of the tangent to the e - log p curve at p_o and the compression index C_c. Fig.5.4.2.1 gives the variation of C'_c with increase in the value of Δp for an overburden pressure of 0.8 kg/cm^2. It shows that C'_c reduces after a certain value of Δp(2 kg/cm^2). This is due to the reversal of curvature of the e - log p curve. For computation of settlement, where the increase in stress is more than the Δp corresponding to the peak value of C'_c, the peak value may be used in the settlement computations.

Fig.5.4.2.2 shows the percentage variation of the ratio of settlements calculated using C_c and C'_c respectively. It can be seen that for lower values of Δp, the

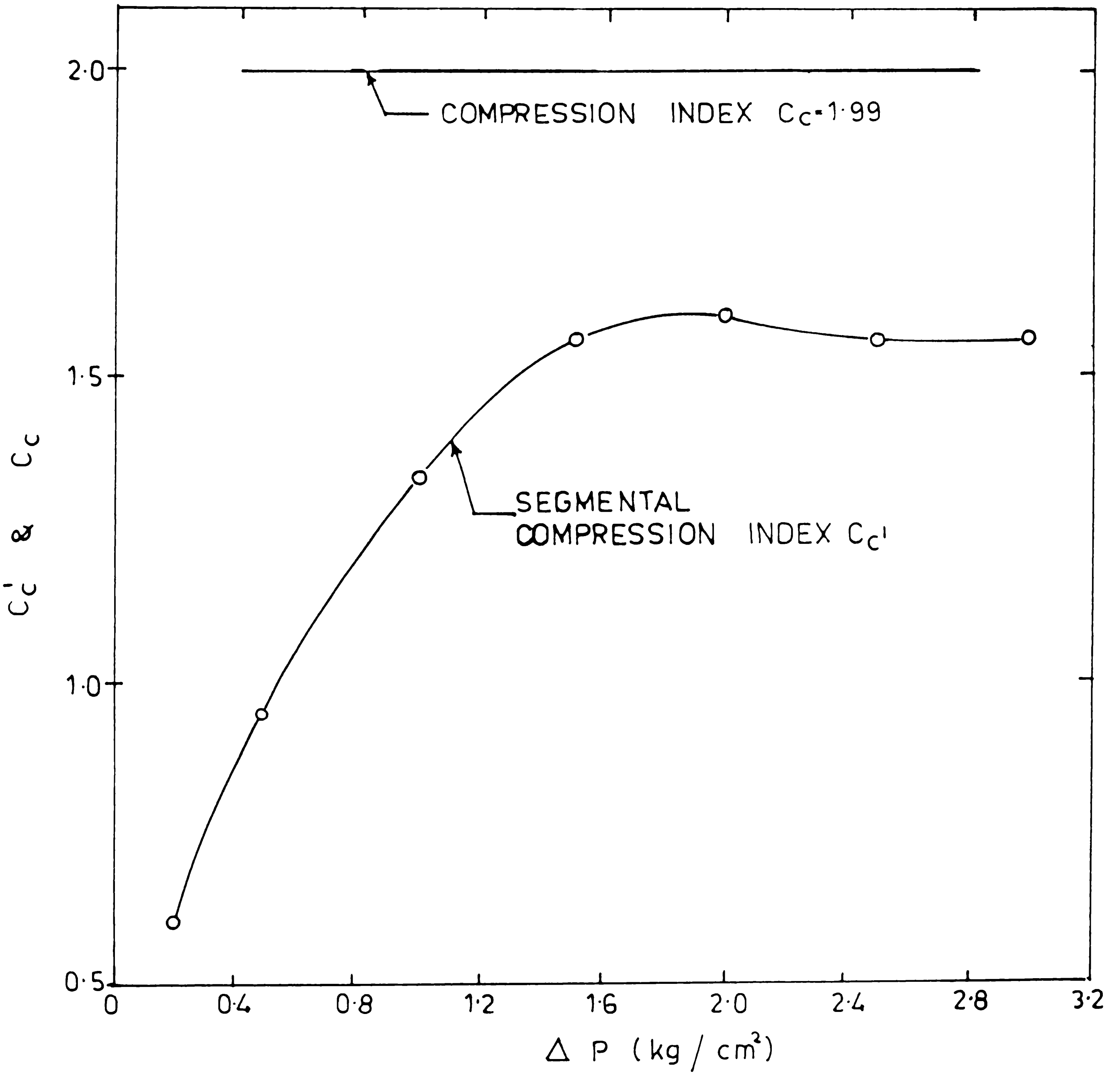

Fig. 5.4.2.1 VARIATION OF C_c' AND C_c WITH Δp

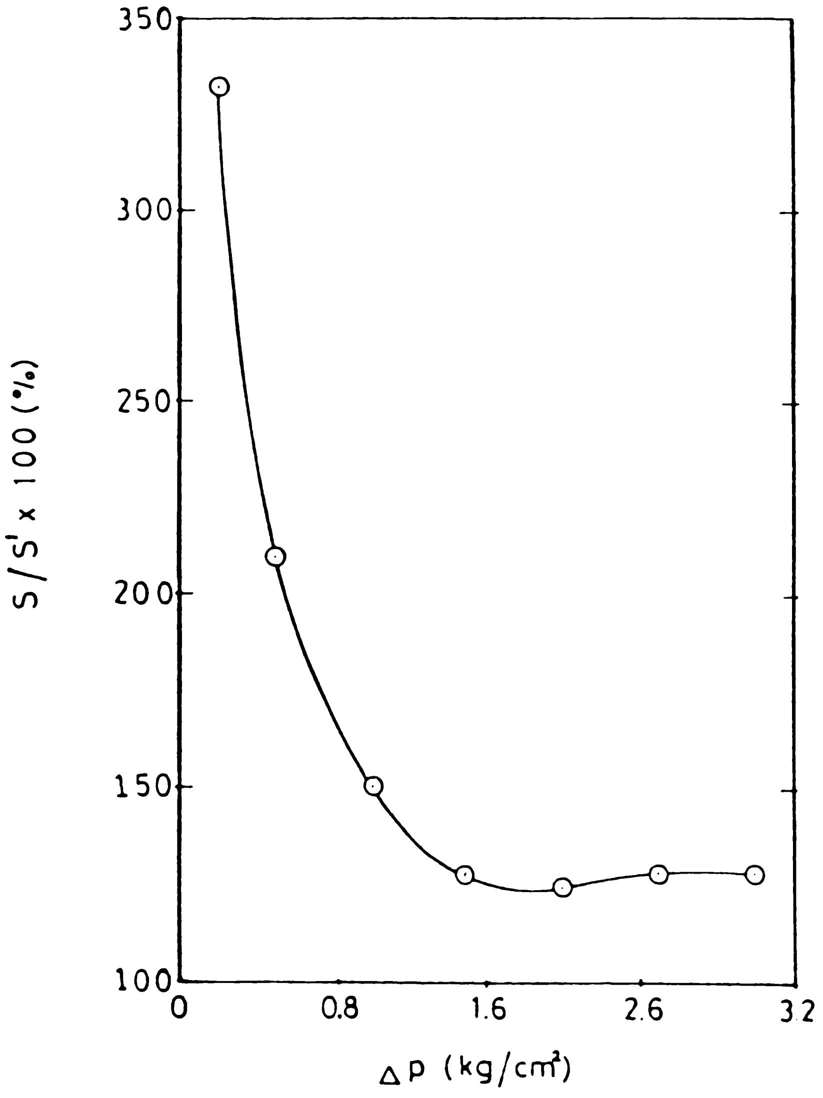

Fig. 5·4·2·2 : S/S' vs Δ p.

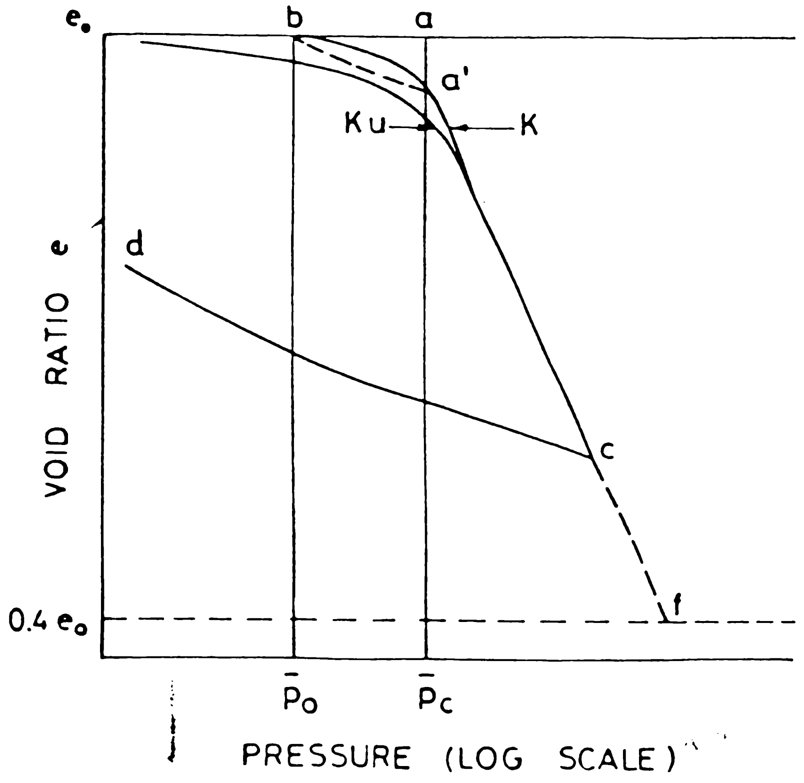

Fig: 5·4·3·1 C_c FOR OVERCONSOLIDATED CLAYS.

settlement as per I.S.Codes is about 332% of the value given by C'_c. The difference drops very fast and comes to a steady state for higher values of Δp.

5.4.3 Settlement of overconsolidated clays

Fig.5.4.3.1 shows the procedure for determining the compression indices for over consolidated clays as per Appendix C of IS: 8009 (Part I). K_u represents the laboratory curve. The portions between p_o (effective overburden) and p_c (preconsolidation pressure) is the recompression curve. The line ba' is drawn through b parallel to the laboratory rebound curve cd. ba'cf is treated as the field compression curve from which m_v is calculated.

$$\text{Settlement } s_f = \Delta p . m_v . H \qquad (5.4.3.1)$$

where m_v = coefficient of volume compressibility

H = thickness of compressible stratum, m.

The procedure does not elaborate on the computation of m_v from the field compression curve and lacks clarity.

Winterkorn and Fang (1975) give a better treatment of the problem wherein the total settlement is assumed to consist of two parts - S_e and S_c. S_e gives the recompression settlement upto the preconsolidation pressure p_c, computed from the settlement equation

$$S_e = \frac{H_t}{1+e_o} C_e \log_{10}\left(\frac{p_c}{p_o} \right) \qquad (5.4.3.2)$$

where C_e is the slope of the recompression curve. Similarly the consolidation settlement from the virgin portion of the e - log p curve,

$$S_c = \frac{H_t}{1+e_o} C_c \log_{10}\left(\frac{p_o + \Delta p}{p_c} \right) \qquad (5.4.3.3)$$

Total settlement $S_f = S_e + S_c$ (5.4.3.4)

In case of the proposed segmental compression index method, such a separation of the total settlement into two parts is not envisaged, as the segmental compression and virgin compression portions of the e - log p curve. Hence a differential treatment to overconsolidated and normally consolidated clays may not be necessary in case of the suggested method.

In conclusion, it can be said that consolidation settlements computed by I.S. procedures tend to overestimate the settlements of foundations resting on clayey soils.

This is due to an overestimation of the compression index, C_c irrespective of the stress range. The segmental compression index method where C'_c is selected based on the insitu stress and stress increase, will give more realistic values. Since the I.S.Codes tend to overestimate the consolidation settlements rendering foundation designs uneconomical, there is a need for a revision of the codes incorporating the above suggestions.

5.5 EFFECT OF DURATION OF LOADING ON COMPRESSIBILITY

The effect of duration of loading on the compressibility characteristics of cohesive soils has not received much attention from the research workers in geotechnical engineering, eventhough stress-strain behaviour of sensitive clays has been studied in detail by Bjerrum (1967) and Mitchell (1970) during their studies on Norwegian quick clays and cemented Leda clays found in eastern Canada. Until recently it was generally considered that the influence of time effects on the stress-strain behaviour of clays is relatively not very significant. Investigations on undisturbed and remoulded samples of Whangamarino clay by Newland and Alley (1957) tend to agree with the above hypothesis. But findings of Hamilton and Crawford (1959) during their investigations on Leda clay onfirm that factors such as duration, load increment ratio etc. influence the compressibility characteristics. Through a series of tests on remoulded samples, Richardson and Whitman (1963) demonstrated that the time dependency of both preconsolidation pressure and undrained shear strength was significant.

The volume change that takes place under constant effective stress is called secondary compression. During secondary compression, a cohesive clay develops increased strength and reserve resistance against further compression. The behaviour of the clays and the mechanism controlling the one-dimensional secondary compression have been explained taking into consideration the inter-particle attractive and repulsive forces, by Sridharan and Sreepada Rao (1982). The reserve resistance against compression developed during secondary compression increases with the time of sustained loading since the amount of secondary compression also increases. In this section it is proposed to investigate the time effects on the compressibility behaviour of Cochin marine clays.

5.5.1 Consolidation tests and results

A series of consolidation tests were carried out on marine clay samples from Parur at depths of 6.5 m, 10.5 m, 12.5 m and 13.5 m. Tests were carried out at durations of 1 day, 2 days, 4 days, 8 days, 15 days and 30 days. The load increment ratio was kept uniformly at 1.0. Each soil specimen was loaded upto 4.0 kg/cm^2 starting from a seating load of 0.0625 kg/cm^2.

Compression index, coefficient of consolidation, permeability and secondary compression were calculated at each loading stage. C_v was calculated by the rectangular hyperbola method in every case (Sridharan & Rao, 1981; Sridharan et al.,1987). The results are presented in the form of a series of curves.

Almost all the consolidation tests have been carried out on undisturbed samples to take into consideration the normal variations that can occur even in case of samples taken from the same depths from closely spaced bore holes. Consolidation tests on remoulded samples would have given more consistent results as the initial void ratio of all the specimen could be made uniform (close to liquid limit). The interference from the effects of preconsolidation pressure of the undisturbed samples could also have been avoided. But to study the field behaviour of the marine clays more closely, tests were carried out on undisturbed samples collected and preserved as explained in chapter III. The studies in this work are based mainly on natural deposits which are believed to form the most reliable basis for a geotechnical treatment, even if the results may be encumbered with some scattering due to lack of homogeneity of the natural clays (Bjerrum, 1954).

Fig.5.5.1.1 and 5.5.1.2 show the relationship between e/e_o and log p for two soil specimens from Parur collected from 13.5 m and 6.5 m respectively. The loading duration for each stage of loading was kept at 1, 2, 4, 8, 15 and 30 days. Since the initial void ratio of the specimens varied from 3.063 to 3.642, pressure was plotted against e/e_o so as to normalise the effects due to the variations in initial void ratio to some extent.

It can be seen from the curves that eventhough the pressure intensities are sustained for a longer duration, the corresponding deformations do not increase as one would have normally expected. In fact the equilibrium void ratio for the curve for durations of 15 days and 30 days are higher than the average values of void ratios. These plots clearly lend support to the hypothesis that the shear strength component of cohesion is increased when the pressures are allowed for longer durations.

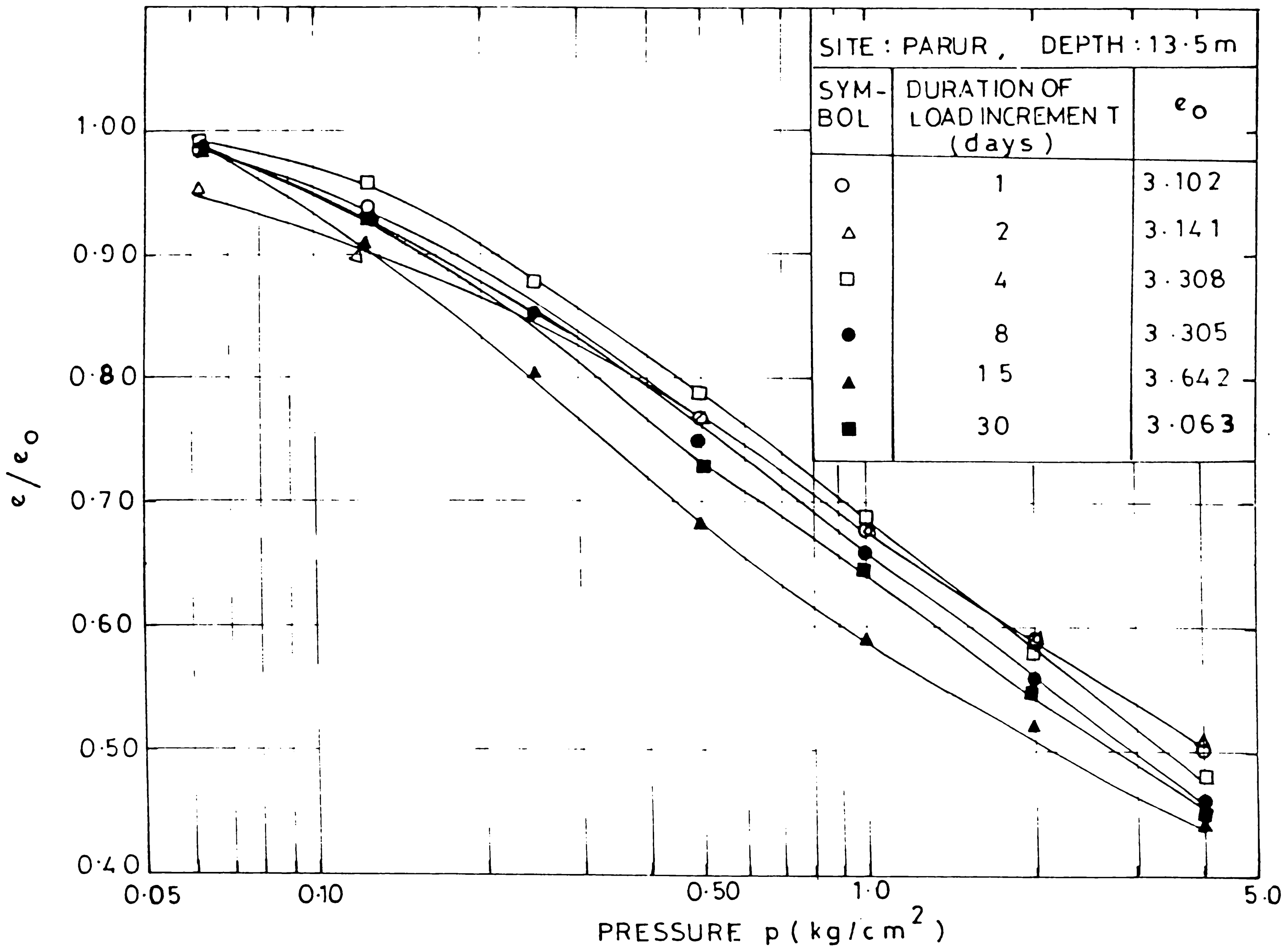

Fig. 5.5.1.1 e/e_o vs PRESSURE.

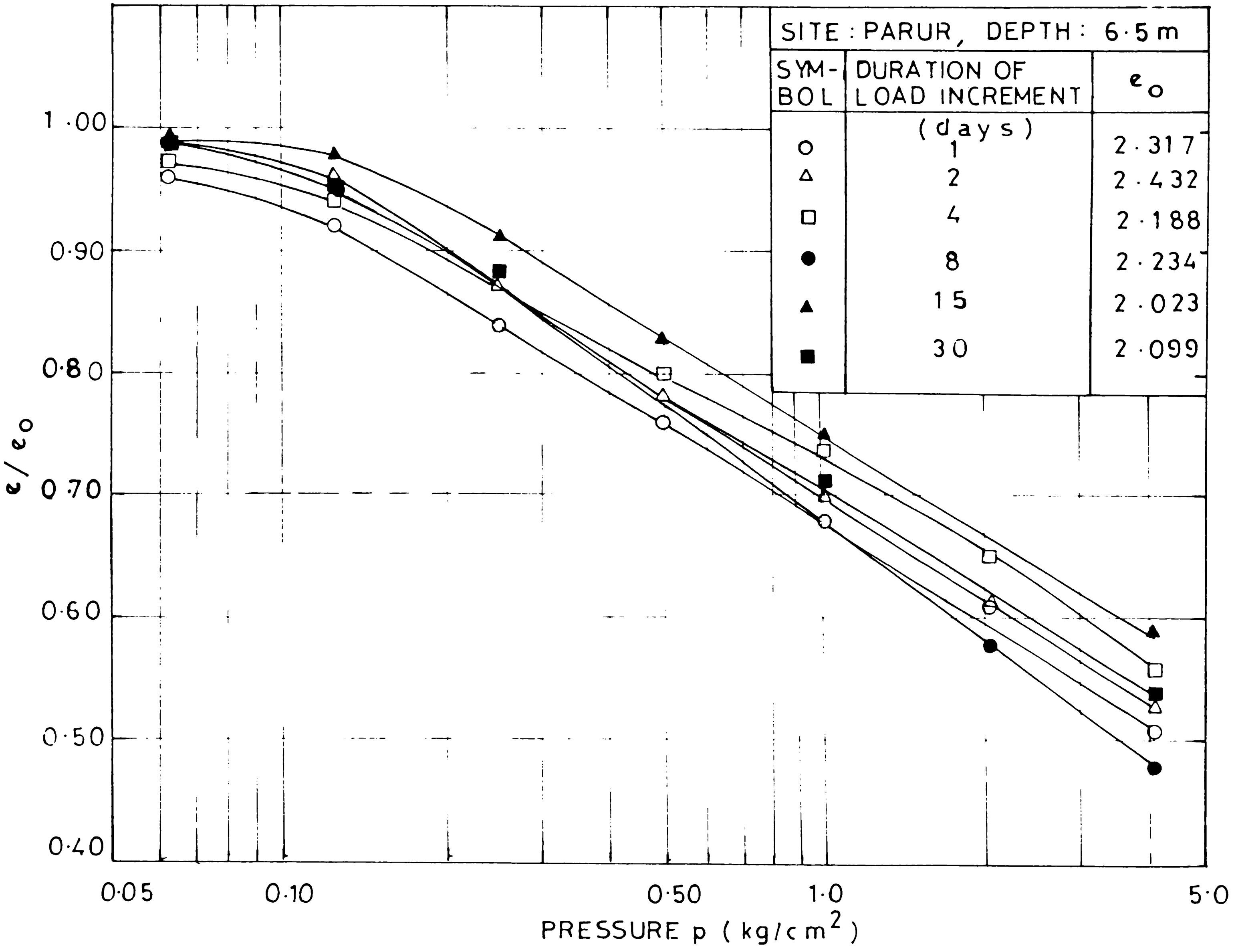

Fig. 5.5.1.2 e/e_o vs PRESSURE

In order to bring out this phenomenon with greater clarity, the deformation in number of divisions have been plotted against time on logarithmic scale in Fig.5.5.1.3 and 5.5.1.4. The former shows δ - log t relationship for two undisturbed samples from Parur collected from a depth of 13.5 m, for the pressure increase from 0.0625 to 0.125 kg/cm^2. In this pressure range, the deformation for a duration of two days is less than the deformations obtained for 30 days. Such behaviour has already been reported for lower ranges of pressure (N.S.Murthy, 1986). But a pressure increase from 2.0 to 4.0 kg/cm^2, the deformation for 30 days consolidation is considerably below that of 2 days consolidation (Fig.5.5.1.4).

To substantiate this behaviour further, the deformation of two samples from Parur, from a depth of 6.5 m, have been plotted in Fig.5.5.1.5 for the pressure range 0.25-0.50 kg/cm^2. The deformation for two days consolidation which starts with a lower value initially, overtakes the other. In Fig.5.5.1.6 the deformations for 30 days duration is consistently and considerably lower than the other.

Eventhough such a behaviour has been predicted by some of the earlier workers (Richardson & Whitman, 1963; Perloff and Ostenberg, 1963), some other investigators

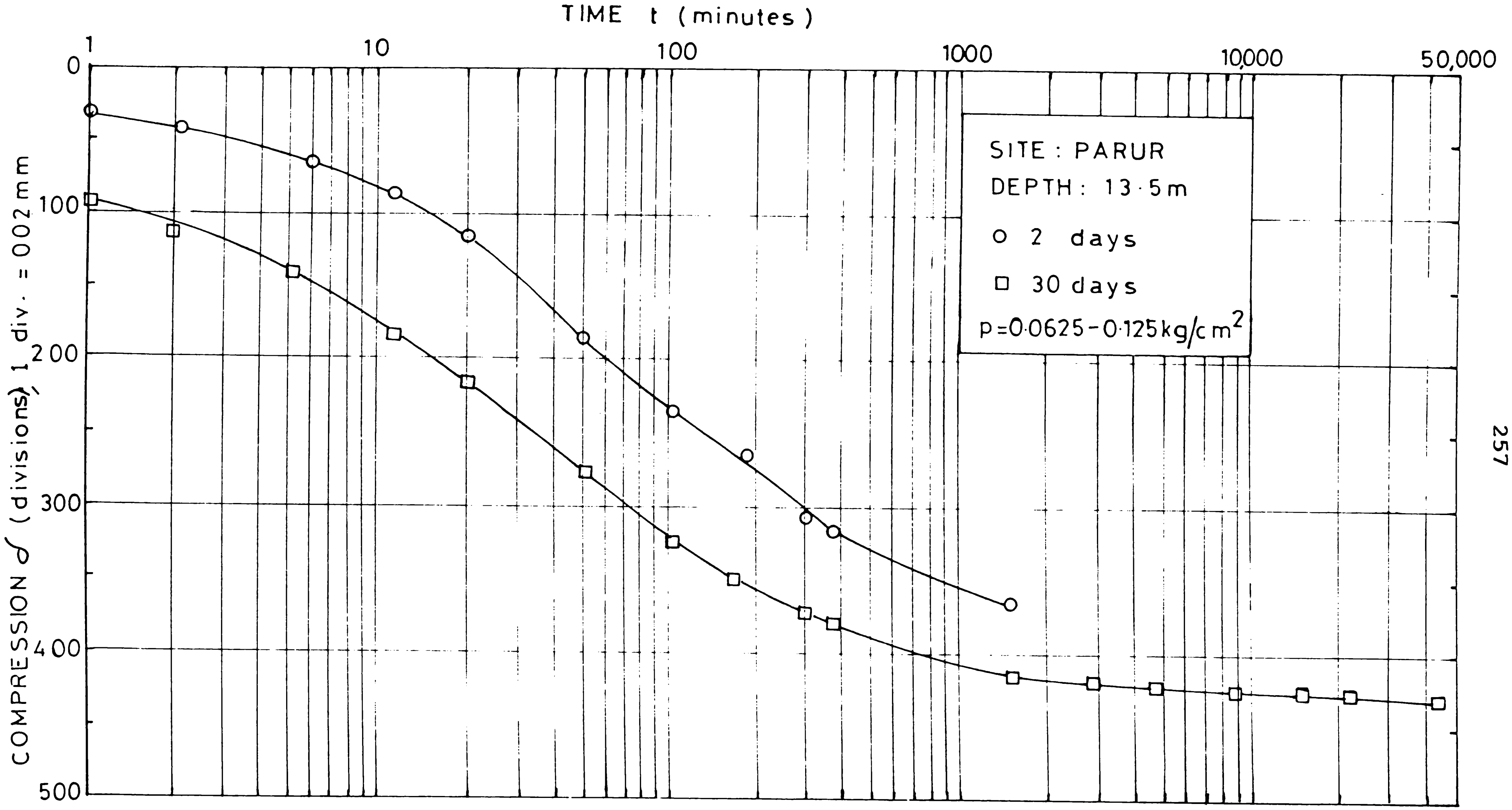

Fig. 5.5.1.3 δ - log t CURVES.

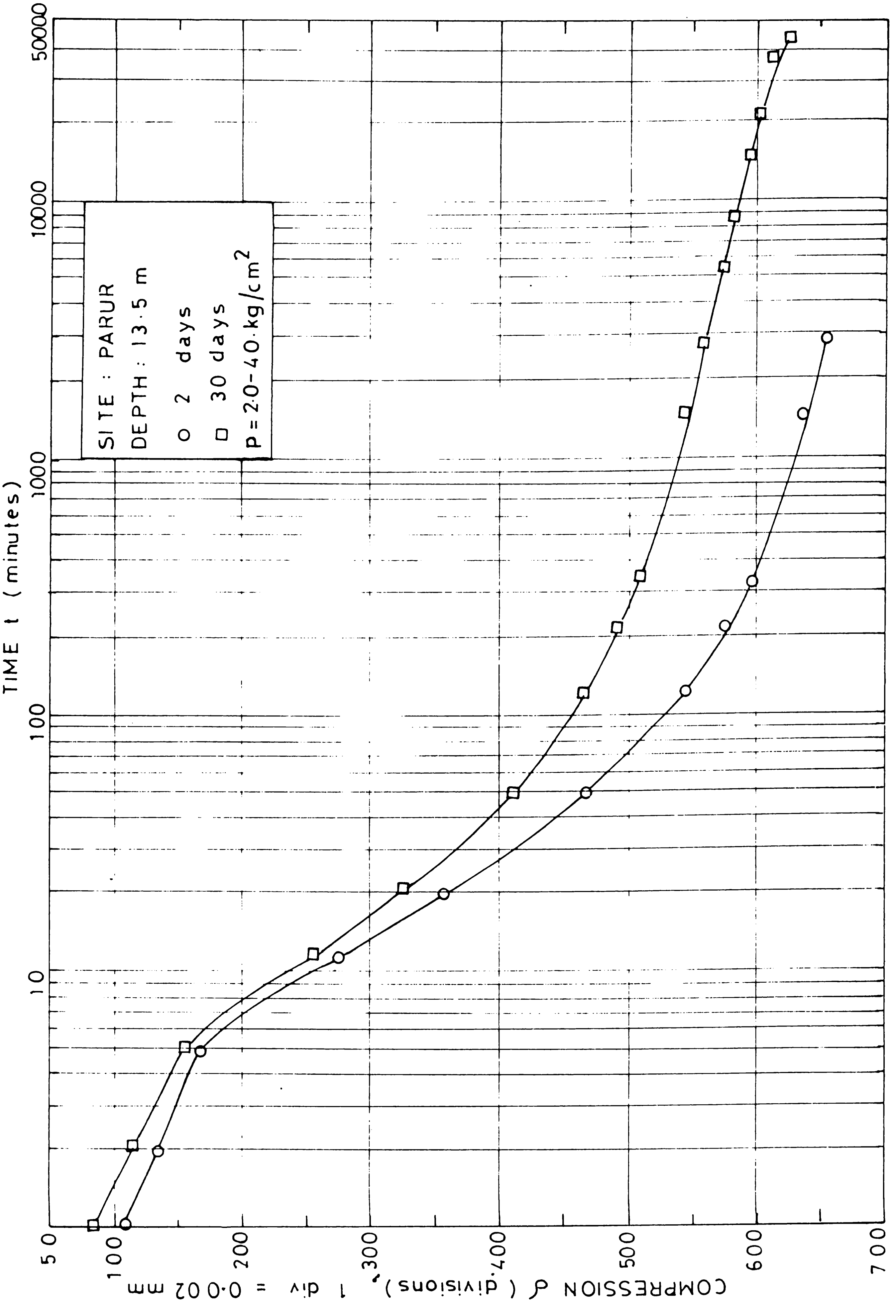

Fig. 5.5.1.4 δ - log t CURVES.

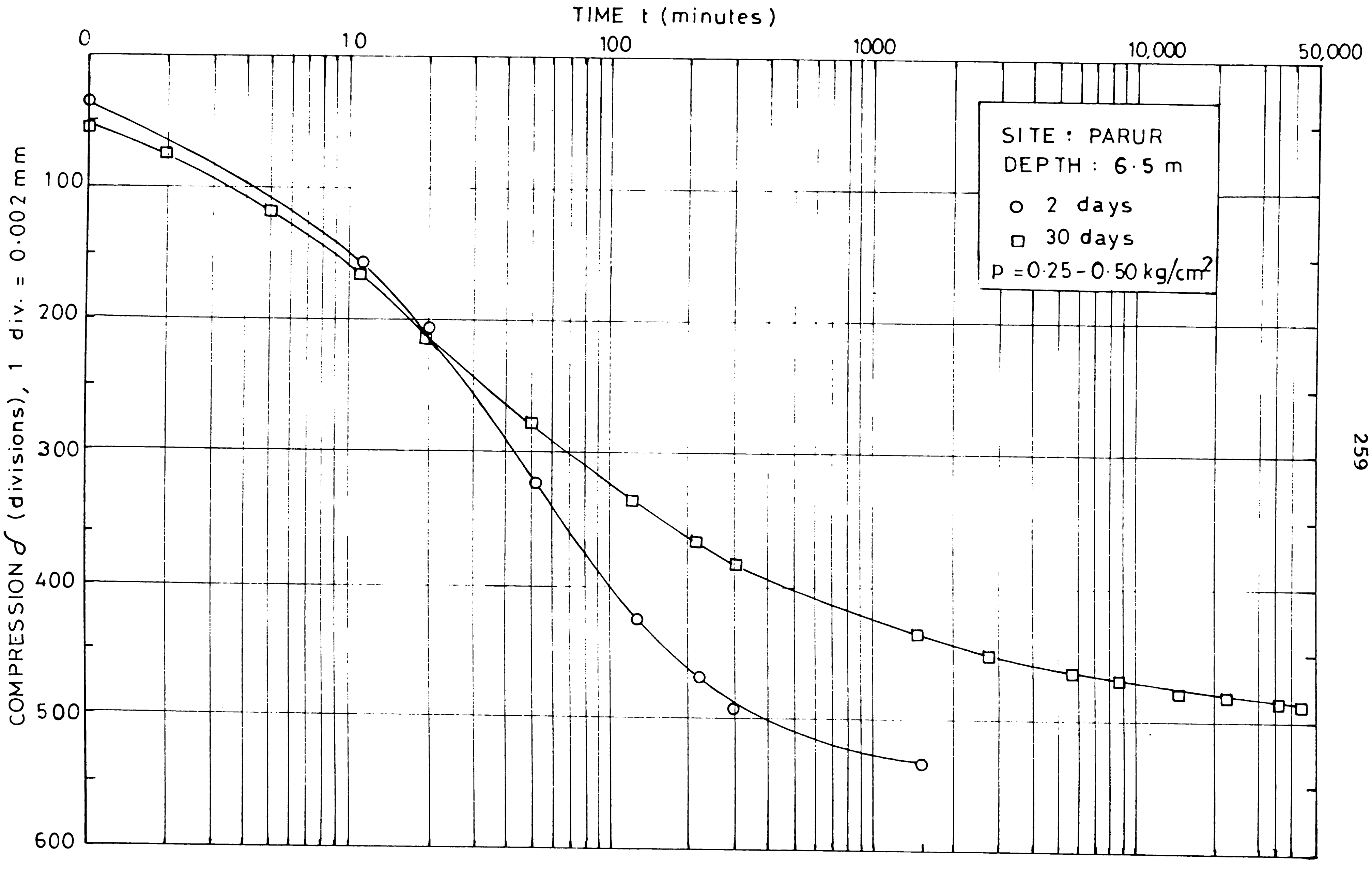

Fig. 5.5.1.5 δ-log t CURVES.

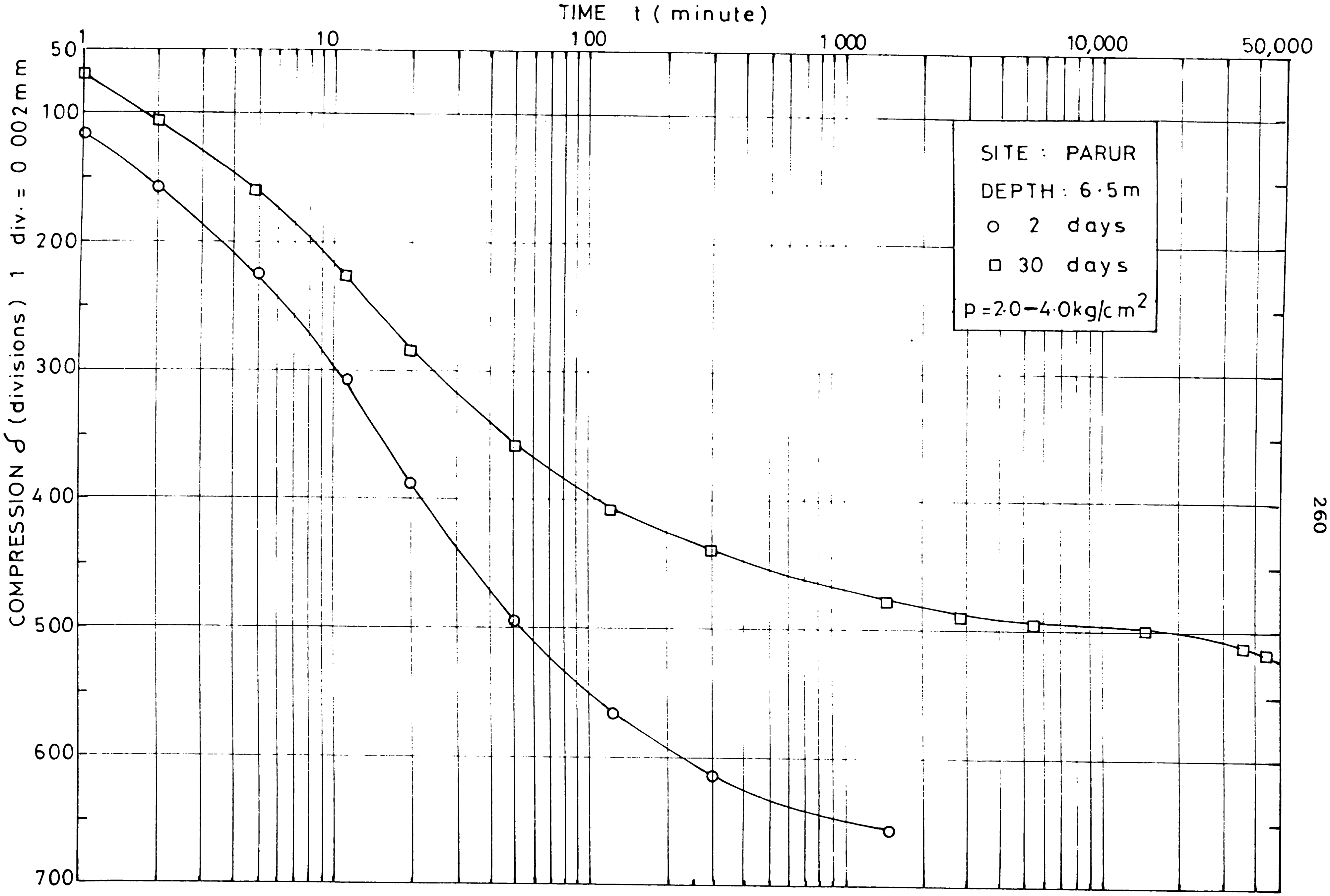

Fig. 5.5.1.6: δ - log t CURVES

reported results to the contrary (Grahmans, Grooks & Bell, 1983). But in the case of Cochin marine clays duration of each loading stage has remarkable influence on the volume change behaviour and the increase in strength is much more pronounced that those reported in the literature (Sangrey, 1972; Lo & Martin, 1972).

Another set of plots between compression δ and time in minutes are given in Figs.5.5.1.7a,b and c in support of the above. Fig.5.5.1.7a shows the δ - log t plot for a pressure increment of 0.5-1.0 kg/cm^2 for a remoulded sample from Parur for durations of 2 days and 15 days. Here the compression for 15 days is more. But for the next loading stage of 1.0-2.0 kg/cm^2 the compression for 15 days duration is lower than that for 2 days duration during the primary consolidation as shown in Fig.5.5.1.7b. The final compression is almost equal in both cases. But in Fig.5.5.1.7c, the total compression for 15 days duration is less than that for 2 days duration. These three plots clearly bring out the development of strength during secondary compression in the case of samples consolidated for long duration. In the case of pressure increment of 0.5-1.0 kg/cm^2, the total time available for consolidation for sample with 2 days duration, before current pressure increment was 6 days viz.

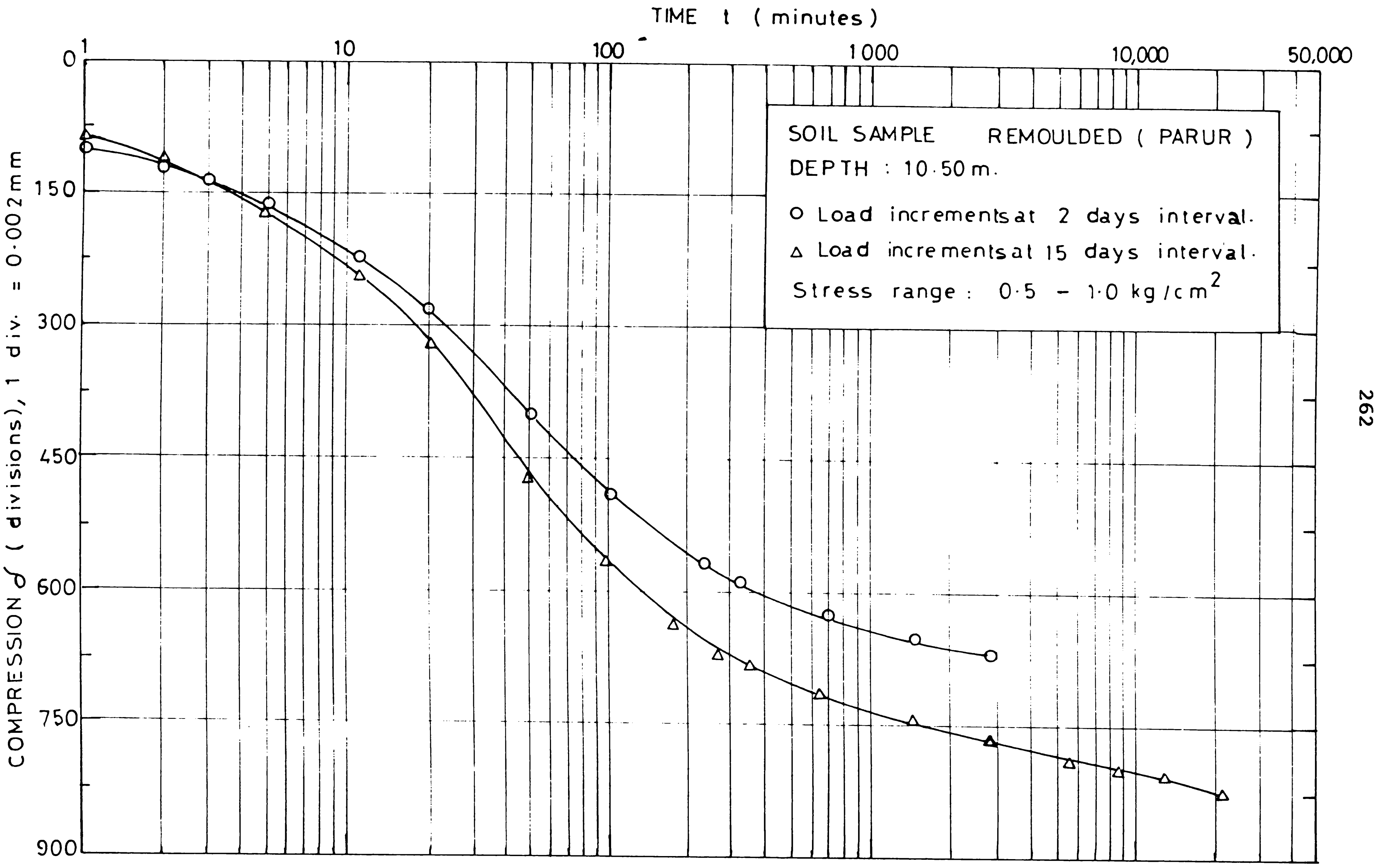

Fig. 5.5.1.7a: δ - log t CURVES.

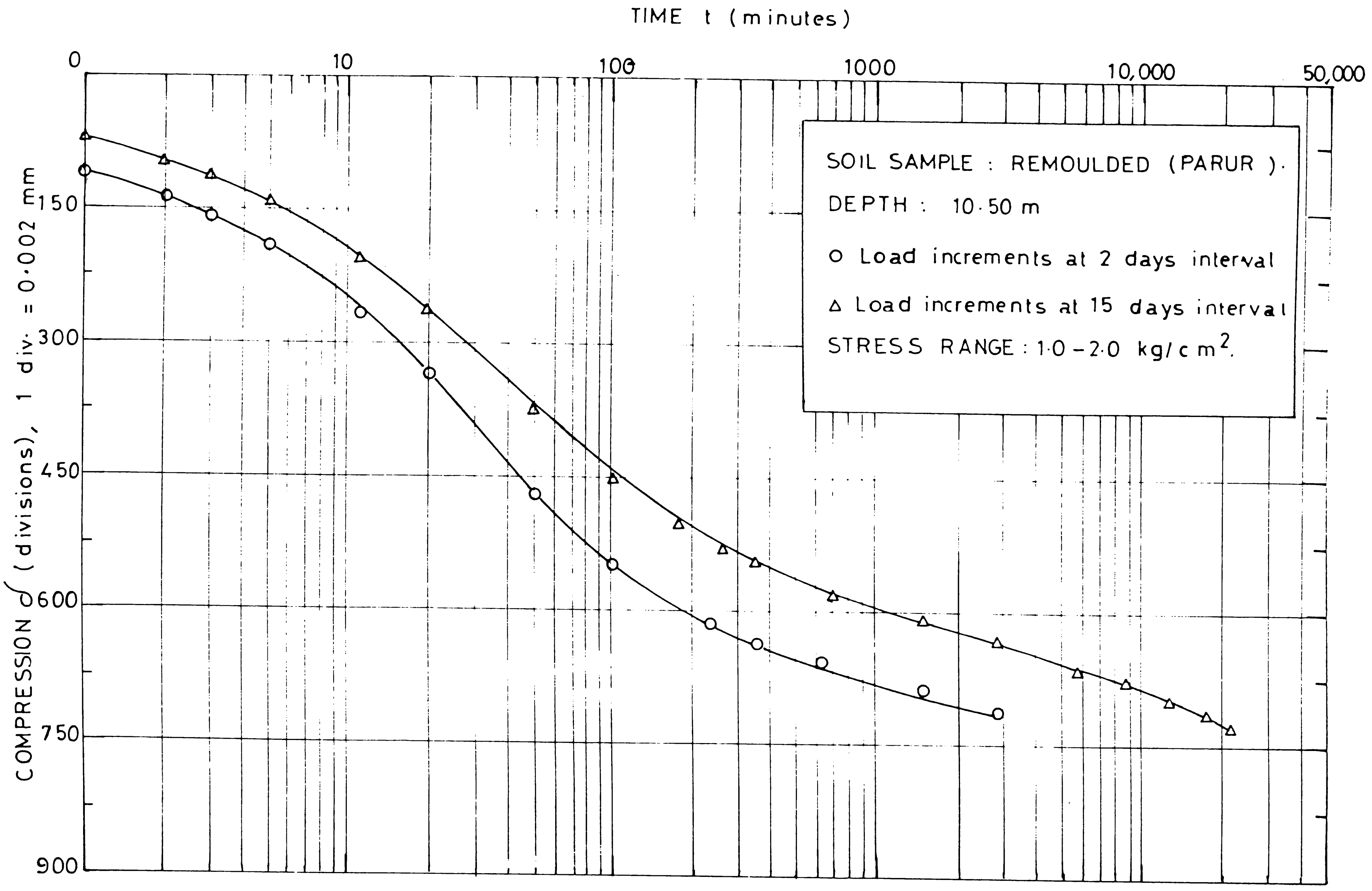

Fig. 5.5.1.7b δ - log t CURVES.

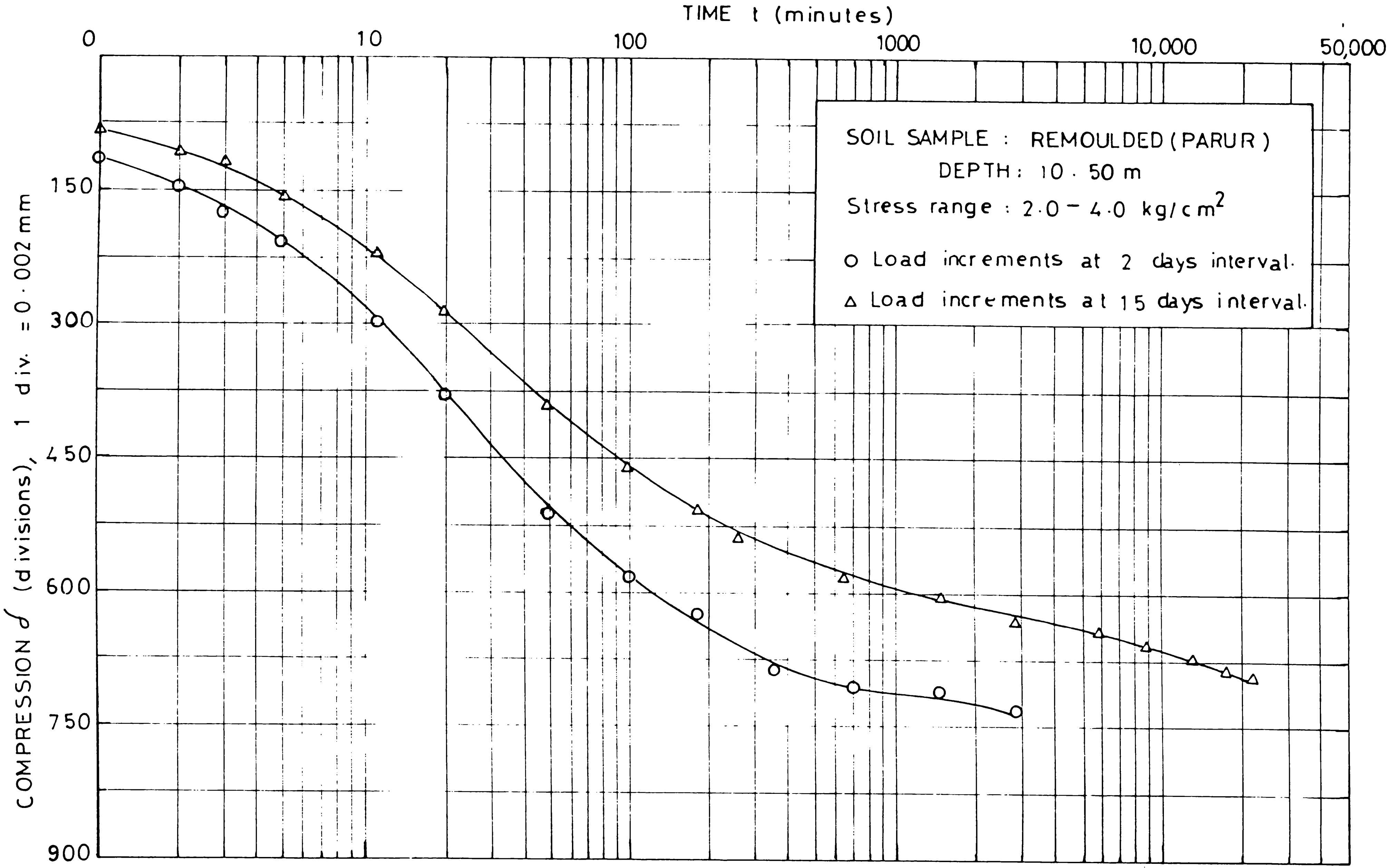

Fig. 5.5.1.7c δ log t CURVES.

2 days such for the load increments of 0.0625-0.125, 0.125-0.25 and 0.25-0.5 kg/cm^2. In case of a sample with 15 days duration the time available was 45 days. But the curves show that the second sample has not gained strength during these 45 days compared to the first sample. But in the next plot Fig.5.5.1.7b the deformations are almost equal for the load increment of 1.0-2.0 kg/cm^2. In the next figure (Fig.5.5.1.7c) the plot for 15 days duration consistently shows compression less than that for 15 days. This perhaps shows that the development of shear resistance during secondary compression takes place after a certain minimum pressure only.

It has already been shown in the earlier discussions on Figs.5.5.1.3 to 5.5.1.7c that soil specimens subjected to pressures for longer durations develop shear strength with respect to time. If it is so, the magnitude of primary consolidation for an increase in pressure should be less in case of specimens subjected to longer durations as the soil has become less compressible due to gain in shear strength. Therefore the primary consolidation expressed as a ratio of total consolidation should show a decline with respect to time.

In Fig.5.5.1.8 the ratio P_c/T_c, where P_c is the primary compression and T_c is the total compression for each stress increase, has been plotted against pressure intensities, for undisturbed marine clay samples from Parur collected from a depth of 6.5 metres, for durations of 1 day, 4 days and 30 days. It can be seen from the plot that the value of P_c/T_c decreases with duration. It also shows a similar behaviour with higher effective stresses.

When the primary compression shows a reduction with respect to total compression, there should be a complementary increase in the secondary compression. Fig.5.5.1.9 shows a plot between the ratio of secondary compression (S_c) and total compression (T_c) and the effective stress, for loading durations of 1 day, 4 days and 30 days from consolidation tests on undisturbed samples from Parur at 6.5 m depth. The plots show a regular increase in the value of S_c/T_c with duration. The increases in the value of S_c/T_c with respect to stress increase is marginal.

A close study of the figures discussed above shows that the amounts of initial and primary compressions obtained for the different durations are in opposite direction to that of secondary consolidation. There is a significant reduction in the values of initial and primary

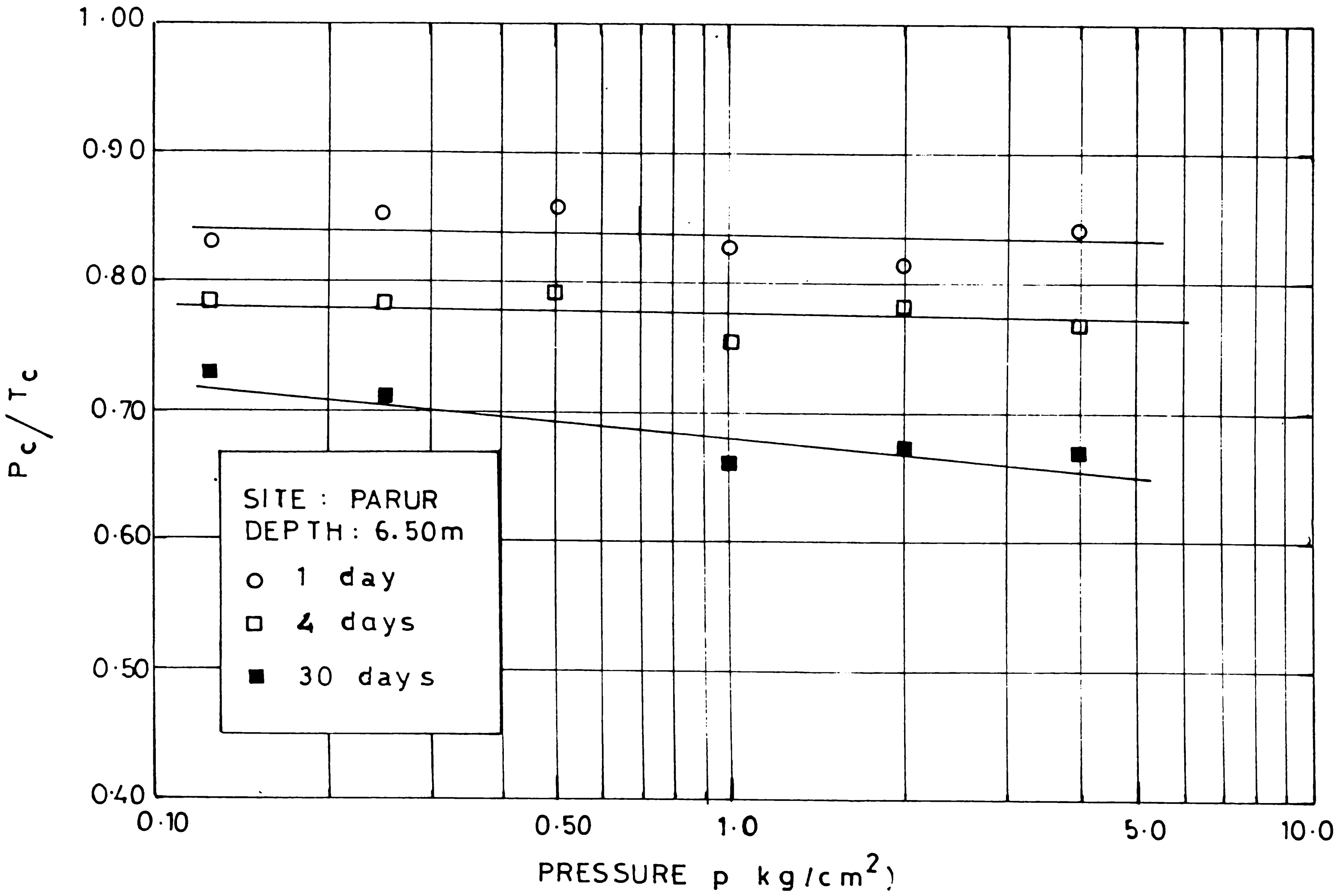

Fig. 5.5.1.8 : P_c/T_c vs PRESSURE.

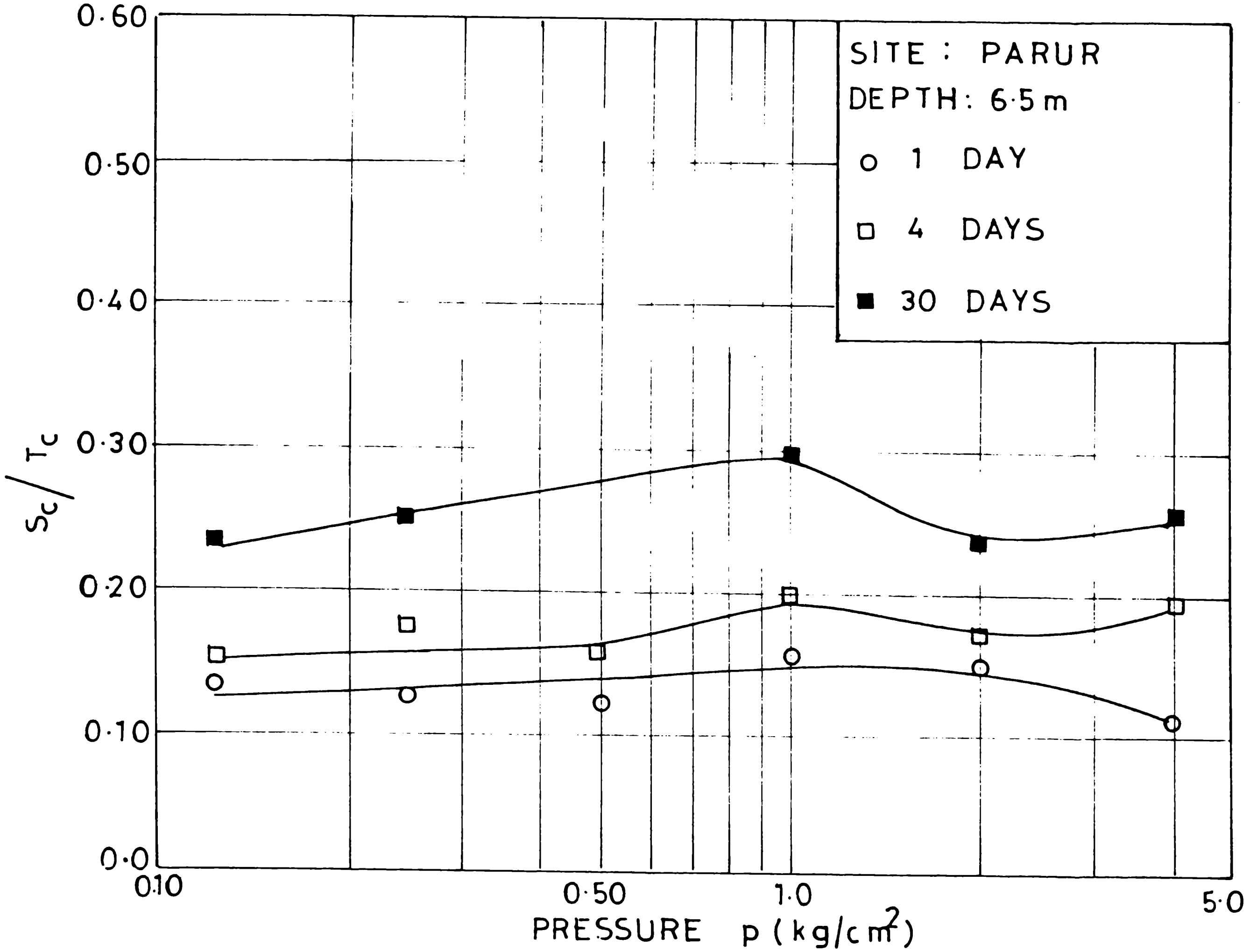

Fig. 5.5.1.9 S_c/T_c vs PRESSURE.

compression with increase in duration of load at a given pressure increment, while the reverse is true in case of secondary compression.

Eventhough there have been contradicting conclusions by research workers on the development of strength with duration of loading and consequent reduction in primary and total compression, this has been well established by the present series of tests on Cochin marine clays and by earlier workers (Leonards & Ramiah, 1959; Bjerrum, 1967; Allam & Sridharan, 1979). The cohesive component of shear strength is likely to be improved by growth of coulombic particle-cation-particle linkages and Van der waals bonds, while fabric changes occurring during secondary compression further improve the resistance to compression.

5.5.2 Rebound characteristics of consolidated marine clays

It has now become an established fact that marine clays develop strength and resistance to compressibility when they are subjected to effective stress for long durations. This has been attributed to the increase in the number of contact points between clay particles which increases the cohesion component of the shear strength is in plastic clays (Bjerrum, 1967). If this increase in strength is reflected in the soil fabric and if this strength gain due to development of bonds between clay particles, has some sort of permanance, it will try to retain its soil structure even when the load is removed. Of course there will be an inevitable elastic rebound; but the soil structure may manifest its strength gain through its rebound characteristics. In order to explore the possibility of such a volume-change behaviour upon removal of pressure, the rebound or swelling of the consolidated specimens of marine clay was studied in detail from the results of the consolidation tests.

Fig.5.5.2.1 shows the plot between the rebound of undisturbed marine clay specimen from Parur collected from a depth of 6.5 m. They have been consolidated upto 4.0 kg/cm^2 with a load increment ratio of 1. The duration of each consolidation pressure increment was chosen as 1 day, 4, 15, and 30 days. The rebound of these samples in number of divisions when the pressures were reduced from 4 to 0.0625 kg/cm^2 has been plotted with pressure for various durations. It can be seen that the rebound

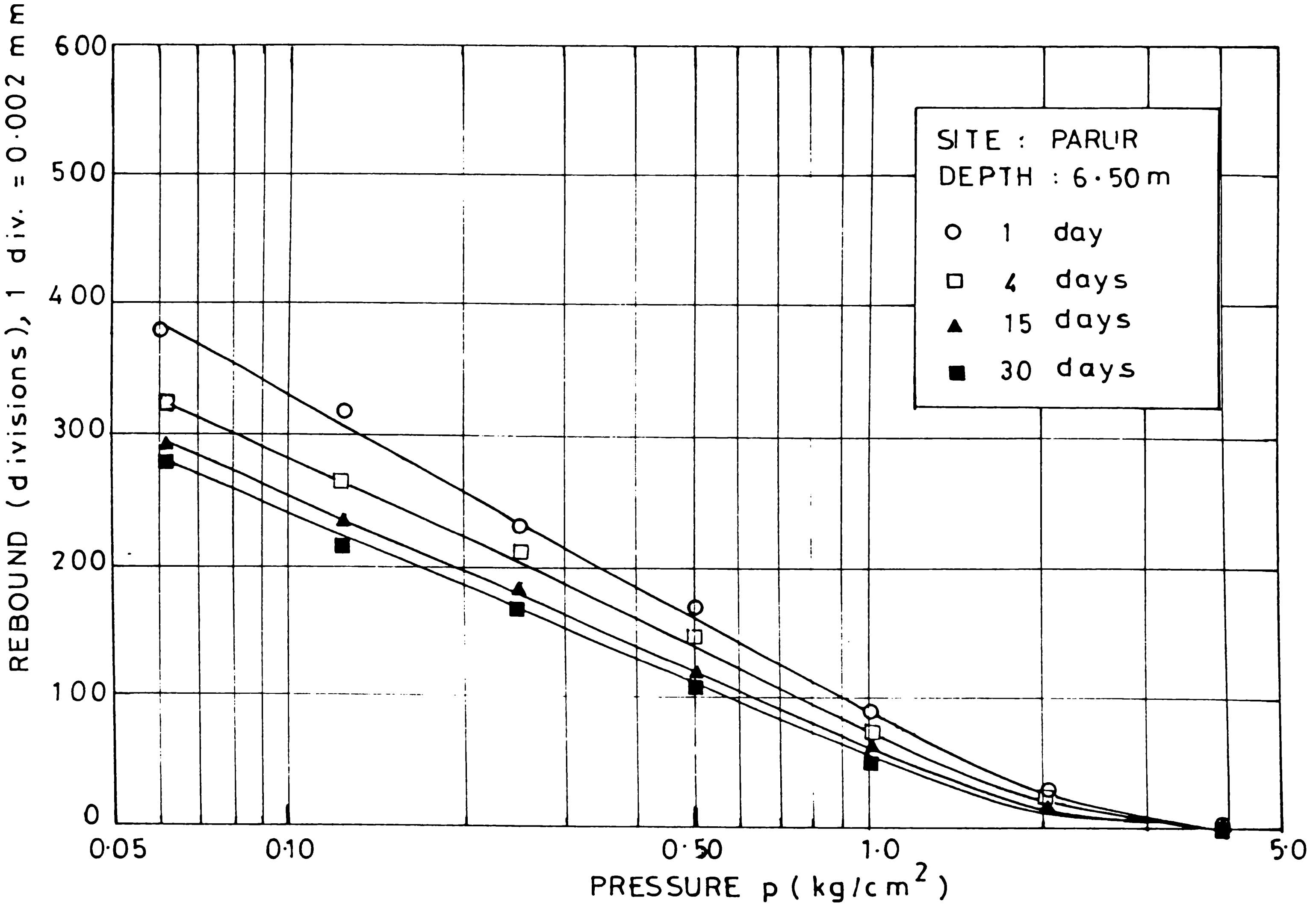

Fig. 5.5.2.1 : REBOUND vs PRESSURE

was maximum for samples consolidated with duration of 1 day and minimum for the duration of 30 days with the other two (4 days and 15 days) falling in between in that order.

5.6 PRECONSOLIDATION PRESSURE

The consolidation test provides basic information on the compressibility characteristics of soils for computations of the magnitude and time rate of settlement resulting from one-dimensional strain. The two most important consolidation characteristics viz. compression index/coefficient of volume decrease and coefficient of consolidation are significantly influenced by preconsolidation pressure. An important aspect of determination of the magnitude of settlement is the realistic estimation of the preconsolidation pressure. Casagrande (1936) defines preconsolidation pressure as the "largest overburden in which the soil had once been consolidated". But a soil may show a preconsolidation pressure higher than the maximum past pressure. This difference is attributed to the bonds formed due to long term secondary compression, aging and other physico-chemical factors (Leonards & Altshaeffl, 1964; Bjerrum, 1967). A soil may show a preconsolidation pressure caused by a variety of factors including overburden pressure that may or may not have reduced due to erosion, desiccation, temporary overloading, sustained seepage force etc. (Leonards, 1962).

It is generally accepted that an accurate determination of p_c is the most important step in predicting long term consolidation settlements and analysis of short term stability problems (Jamiolkowski et al., 1985). The major variables that can influence the measured p_c value are sample disturbances, test equipments and procedures to obtain the compression curve, environmental factors and the interpretative techniques used for the estimation of the preconsolidation pressure.

5.6.1 Methods available for determination of preconsolidation pressure

One of the earliest and most widely used procedure is the method proposed by Casagrande (1936). The other two methods are those of Burmister (1951) and Schmertmann (1955). Jamiolkowsi et al. (1985) discuss at length the mechanism of the preconsolidation pressure and also opine that the most popular method still is Casagrande's empirical construction technique. According to Leonards (1962), as there are no reliable methods for the determination of the in situ preconsolidation pressure, no suitable criteria exist for appraising the relative merits of the various methods that have been proposed for determining p_c. As a corollary to the detailed investigations carried out for the study of the consolidation characteristics of Cochin marine clays, a new method has been developed and its reliability tested in this work.

The Casagrande construction involves selecting the 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.

In Burmister's 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 procedure is obviously cumbersome and time consuming.

In the method suggested by Schmertmann, 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.

While in Casagrande's method, there could be personal errors involving the choice of the minimum radius of curvature, the other two methods are basically trial and error procedures with their inherent drawbacks. The main lacuna in assessing the validity of any of the above procedures is the non-availability of the real value of p_c to compare with. The series of tests on Cochin marine clays with long durations of loading helped to evolve a new method called log-log method for the determination of preconsolidation pressure.

5.6.2 Preconsolidation pressure by log-log method

In this method, p_c is obtained as the point of intersection of two straight lines extended from the linear portions on either end of the compression curve plotted to log scale on both axes (Fig.5.6.3.2). After running several series of consolidation tests with varied parameters such as soil type, sustained pressure, duration of sustained pressure, it has been confirmed that the log-log method is superior to all other existing methods.

The validity of the log-log method has been verified by conducting a series of consolidation tests wherein a pre-knowledge of the preconsolidation pressure was made available. This was achieved by consolidating the saturated soil samples from slurry level (initial water content nearly equal to their liquid limits) to a predetermined consolidation pressure for sufficient time. The specimens were then unloaded to a nominal initial load and then reloaded much beyond the predetermined sustained pressure. Thus the initial portion of the recompression curve upto predetermined sustained pressure is the overconsolidated portion and the portion beyond this is the virgin compression line. Use of this recompression line to check the validity of any method of determining p_c will have the distinct advantage of a prior knowledge of the preconsolidation pressure. The validity of the log-log method is examined in detail below.

5.6.3 Test results and analyses

Tests were conducted in standard oedometer set ups on eight different soils with widely varying compressibility characteristics with their liquid limits varying from 47 to 137%. The soils used were of different origin, five of them being Cochin marine clays one is black cotton soil from Benihalla in Karnataka State, a red earth from Kerala State and a typical kaolinite clay from Karnataka State. The physical properties of these soils are listed in Table 5.6.3.1. They are highly sensitive to air/oven drying. Hence they have been determined on natural moist samples.

Typical e - log p curves from consolidation tests are shown in Fig.5.6.3.1. The same results are plotted in Fig.5.6.3.2 taking logarithmic scale for both void ratio and pressure. Straight lines can be readily fitted to the initial and final portions of log e - log p curves, the point of intersection of which agrees well with the value of the previous sustained pressure which can be taken as the p_c

Table 5.6.3.1

PHYSICAL PROPERTIES

Sl. No.	Soil type	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)	Shrinkage limit (%)	Free swell index* (cc/)	Grain size distribution		
							Clay (%)	Silt (%)	Sand (%)
1.	Cochin marine clay (Site: Nettoor)	134.0	47.5	86.5	17.7	4.80	48	32	20
2.	Cochin marine clay (Site: Parur)	94.0	38.8	55.2	19.1	4.00	36	22	42
3.	Black cotton soil	93.0	39.2	53.9	9.0	2.90	62	30	8
4.	Red earth	52.5	35.3	17.2	27.1	2.50	44	24	32
5.	Kaolinite	47.0	34.8	12.2	30.2	1.15	9	62	29
6.	Cochin marine clay (Site: Kumbalam)	120.0	46.6	73.4	19.6	6.45	46	47	7
7.	Cochin marine clay (Site: Elamkulam)	105.0	40.5	64.5	19.4	6.45	46	47	7
8.	Cochin marine clay (Site: Maradu)	137.0	55.0	82.0	21.8	6.50	50	38	12

* It is the swollen sediment volume in a measuring jar expressed in cc/g (Sridharan et al., 1985 and 1986).

Table 5.6.3.2

COMPARISON OF P_c VALUES BY THE DIFFERENT METHODS

Sl. No.	Type of soil	Sustained pressure (kg/cm^2)	Sustained period (days)	Preconsolidation pressure (kg/cm^2) from			
				Casagrande method	loge-log p plot	Δe_1-log p plot	Δe_2-log p plot
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1.	Remoulded Cochin marine clay (Nettoor site)	1.00	2	1.30	1.08	1.05	1.05
2.	,,	1.00	7	1.30	1.15	1.10	1.10
3.	,,	1.00	15	1.35	1.19	1.15	1.10
4.	,,	1.00	90	1.30	1.22	1.25	1.15
5.	,,	2.00	15	2.45	2.20	2.25	2.10
6.	Remoulded Cochin marine clay (Nettoor site) -CRS test	1.00	2	1.15	1.08	1.05	1.10
7.	,,	1.00	15	1.10	1.18	1.10	1.08
8.	Cochin marine clay (Parur site)	1.00	2	1.25	1.05	1.10	1.05
9.	Red earth	1.00	2	1.50	1.20	--	1.40
10.	Black cotton soil	1.00	15	1.30	1.18	1.20	1.15
11,	,,	1.00	15	1.30	1.14	1.20	1.10
12.	,,	2.00	2	2.55	2.20	--	2.45
13.	,,	2.00	15	2.55	2.22	--	2.15

(Contd.)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
14.	Black cotton soil	2.00	22	2.40	2.20	--	2.00
15.	,,	2.00	96	2.65	2.25	--	2.35
16.	Kaolinite	1.00	2	1.25	1.02	1.05	1.00
17.	,,	1.00	15	1.30	1.05	1.10	1.05
18.	,,	2.00	2	1.90	2.00	--	1.20
19.	,,	2.00	22	2.20	2.30	--	1.90
20.	,,	2.00	96	2.80	2.50	--	2.10
21.	Marine clay treated with 3% lime	1.00	2	1.45	1.20	1.20	1.20
22.	,,	1.00	7	1.50	1.28	1.40	1.35
23.	,,	1.00	15	1.50	1.35	1.40	1.35
24.	,,	1.00	15	1.60	1.60	1.60	1.60
25.	Marine clay treated with 6% lime	2.00	15	2.40	2.22	--	2.10
26.	Undisturbed Cochin marine clay (Kumbalam site)	--	--	1.10	1.05	0.95	0.92
27.	Undisturbed Cochin marine clay (Elamkulam site)	--	--	0.66	0.63	0.70	0.66
28.	Undisturbed Cochin marine clay (Maradu site)	--	--	0.84	0.78	0.70	0.70

taking into consideration the quasi-preconsolidation pressure also. A series of much tests have been carried out and the results are presented in Table 5.6.3.2.

Leonards and Ramiah (1959) report that fine grained soils develop quasi preconsolidation pressure on sustained loading for long durations. Allam and Sridharan (1979) have brought out the significant increase in strength of soils consolidated to different aging periods. Keeping this in view, samples were loaded to predetermined pressures starting from slurry condition and kept for long durations, were unloaded and then reloaded. The duration of sustained pressure was varied from 2 to a maximum of 96 days.

Table 5.6.3.2 also contains results obtained with varied duration of sustained loading. Fig.5.6.3.3 shows typical test results of a marine clay specimen kept for a period of 90 days under a sustained pressure of 1.0 kg/cm^2. The existence of the quasi-preconsolidation pressure is evident from the illustration. Fig.5.6.3.4 presents the same data of the recompression curve in log-log scale. The intersection of straight lines drawn through the initial and final portions of the curve results in a pressure of 1.22 kg/cm^2 as against

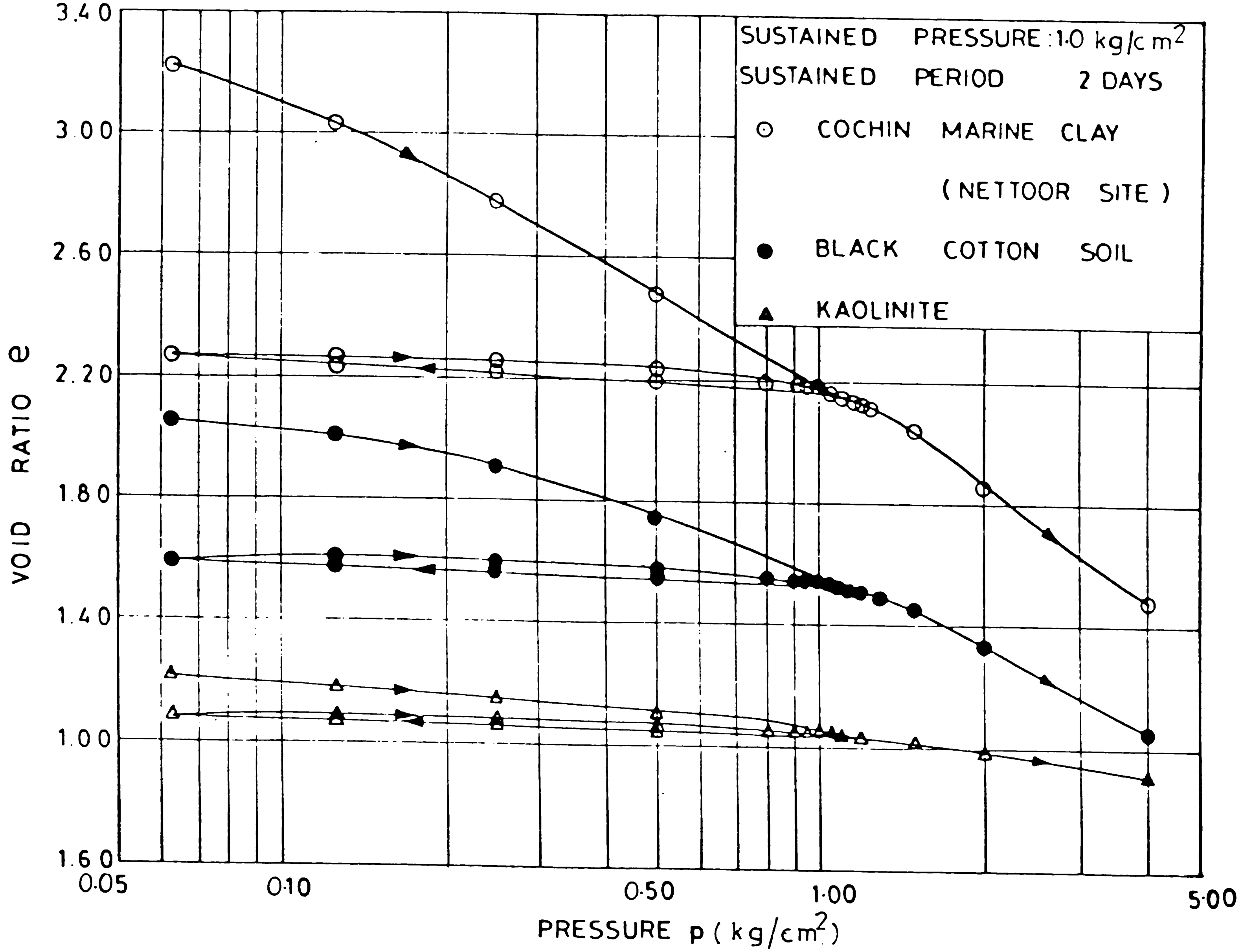

Fig. 5:6.3.1 TYPICAL e - log p CURVES.

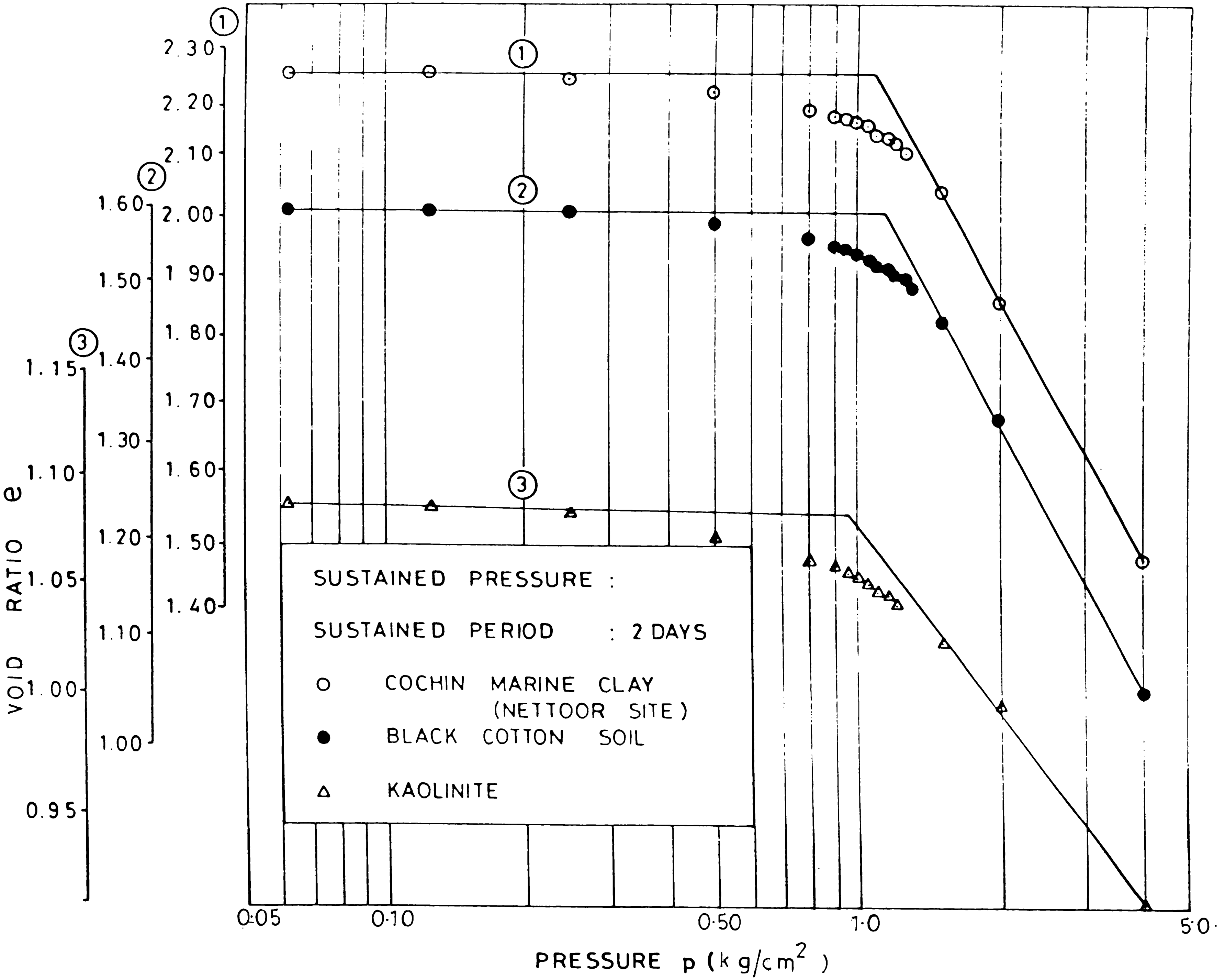

Fig. 5.6.3.2 : TYPICAL log e ~ log p CURVES.

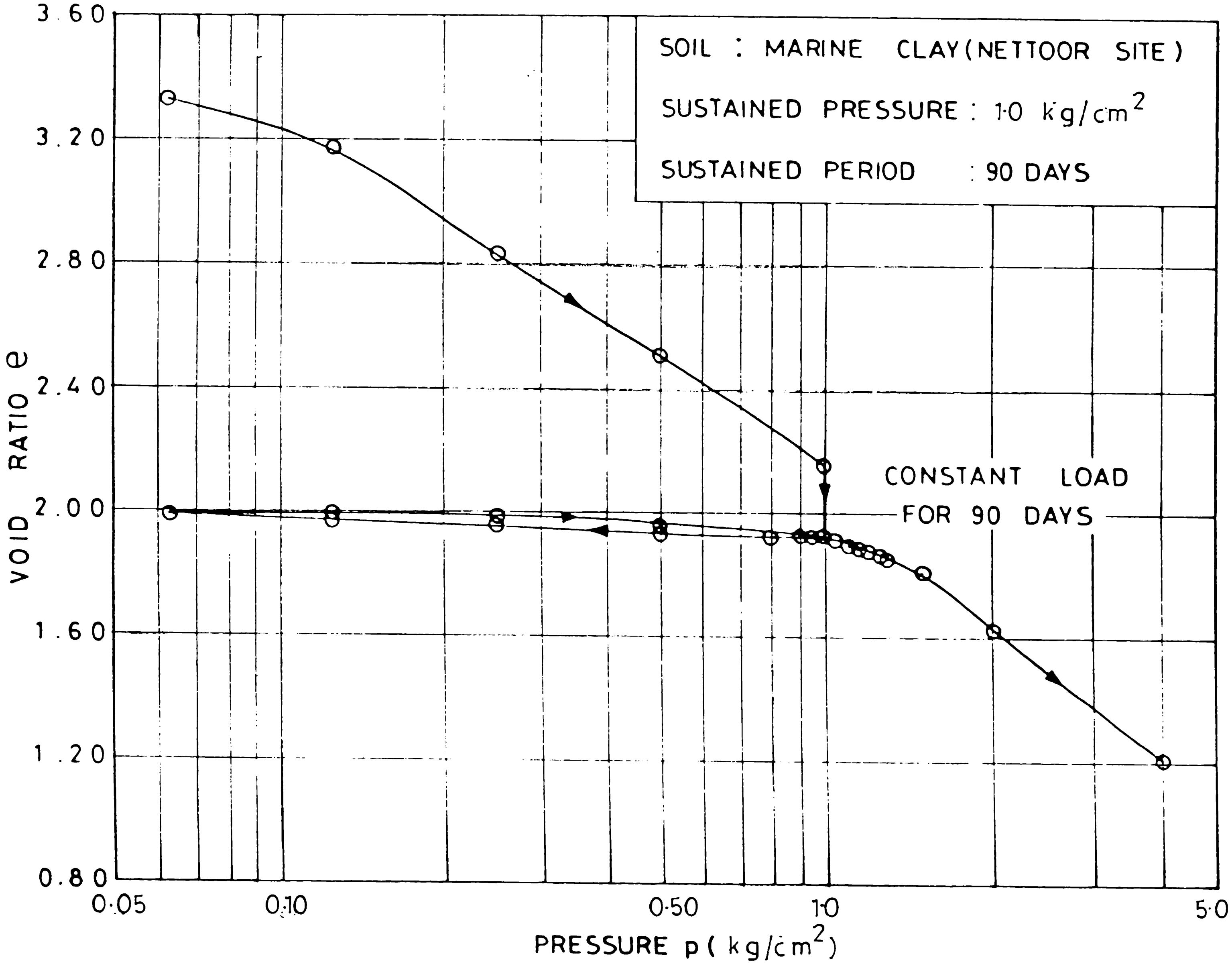

Fig. 5.6.3.3 e – log p CURVES.

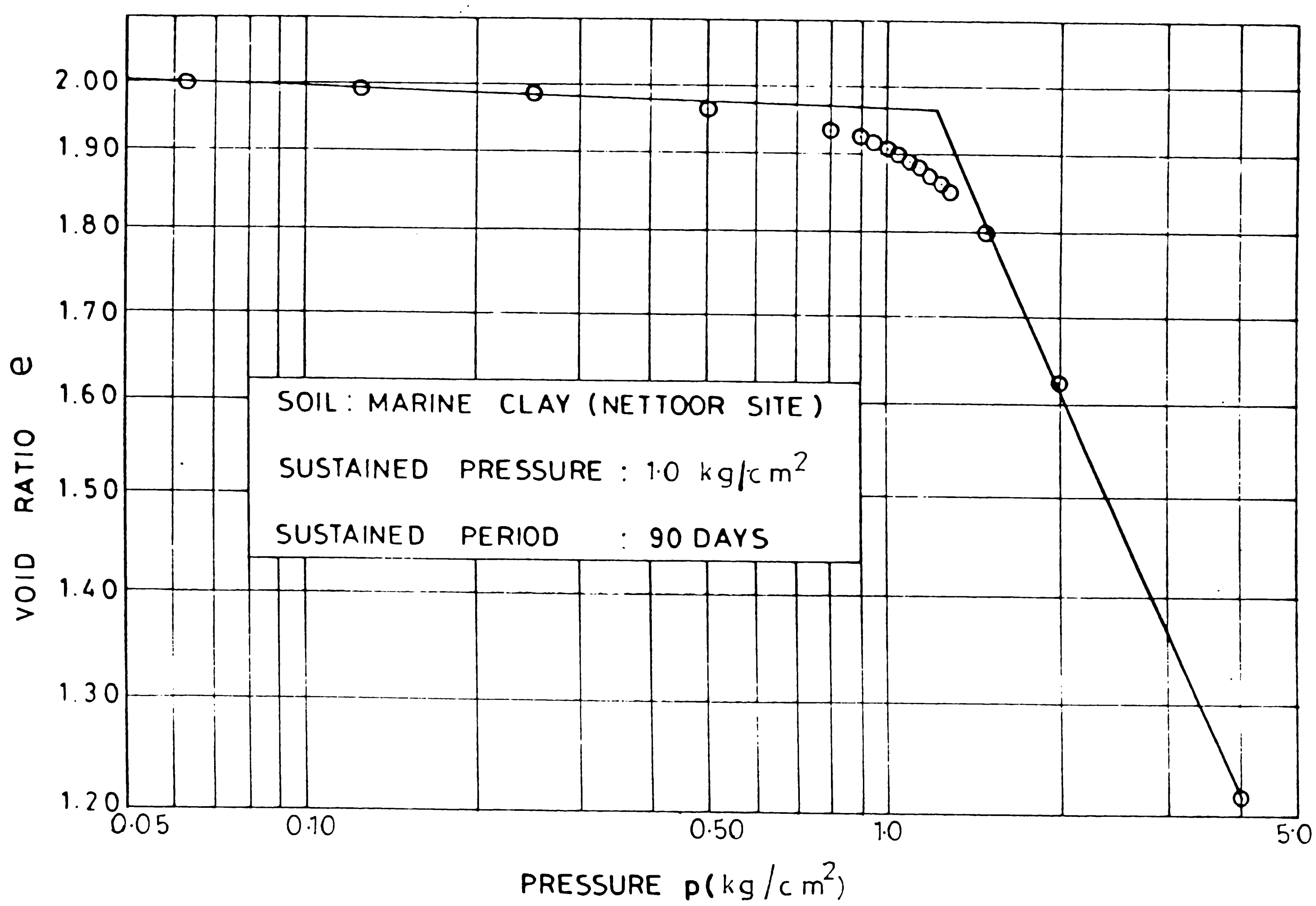

Fig. 5·6·3·4 log e – log p CURVES.

the sustained pressure of 1.0 kg/cm^2. The increase of 0.22 kg/cm^2 could only be attributed to the bonds developed due to aging under sustained load. More of such results can be seen in Table 5.6.3.2.

It is well known that addition of lime increases the strength of the soil fabric at the same water content. Further, due to pozzolanic action, the sample gains strength with time. Fig.5.6.3.5 shows typical results of Cochin marine clay treated with 3% lime and with four different sustained periods viz. 2, 7, 15 and 60 days. Because of pozzolanic action, the increase in preconsolidation pressure over and above the sustained pressure is evident from the curves. Fig.5.6.3.6 presents the same results in the log-log plot. It is clear that there is a substantial increase in p_c due to the duration of the sustained loading. Thus with the addition of lime, increase in bond strength with aging is quite significant. The results are presented in Table 5.6.3.2.

Fig. 5.6.3.1 presents typical e - log p curves obtained from consolidation tests on three undisturbed samples of Cochin marine clays. The corresponding log e - log p plots are shown in Fig. 5.6.3.7. It is clear

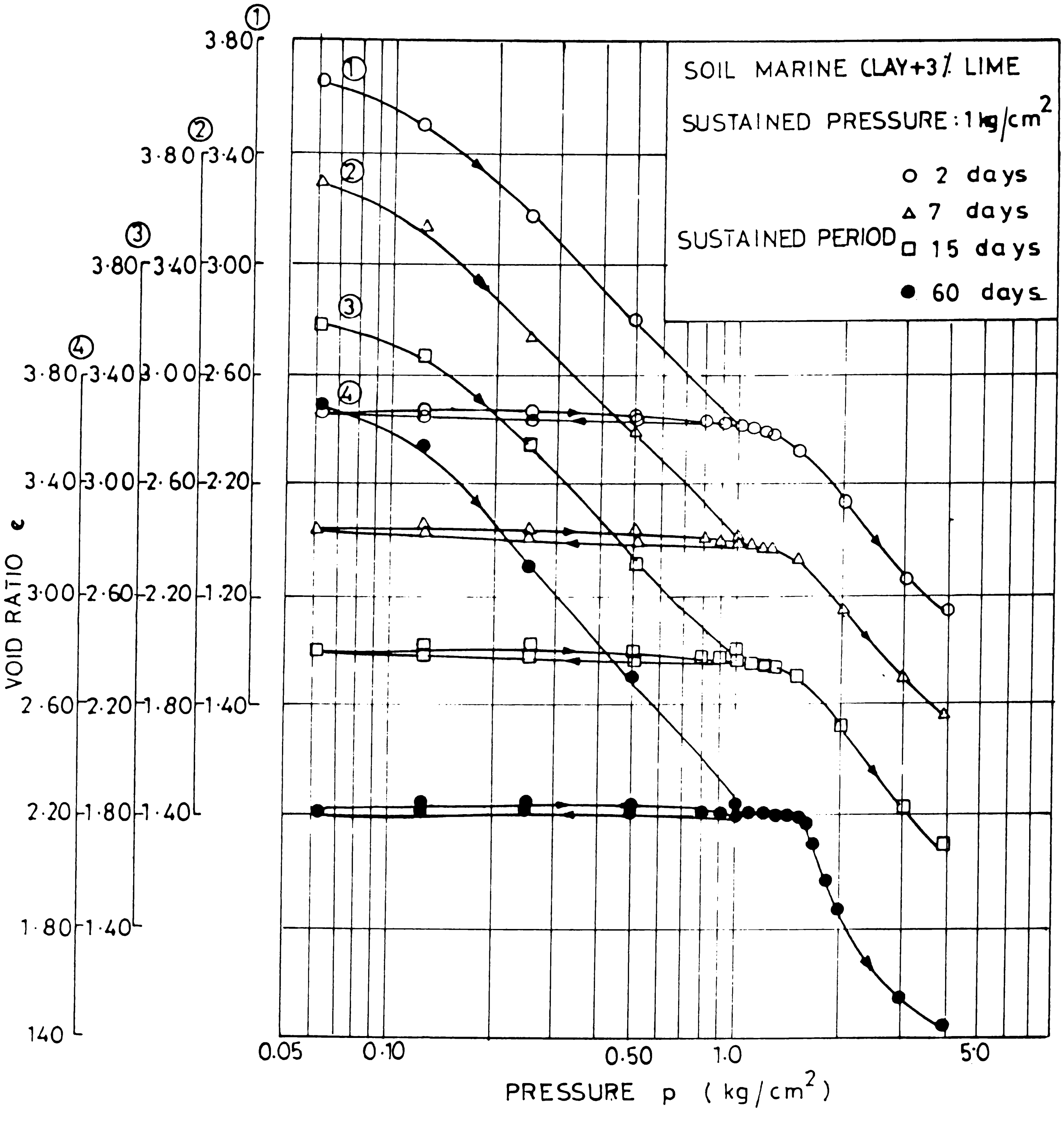

Fig. 5.6.3.5 e – log p CURVES.

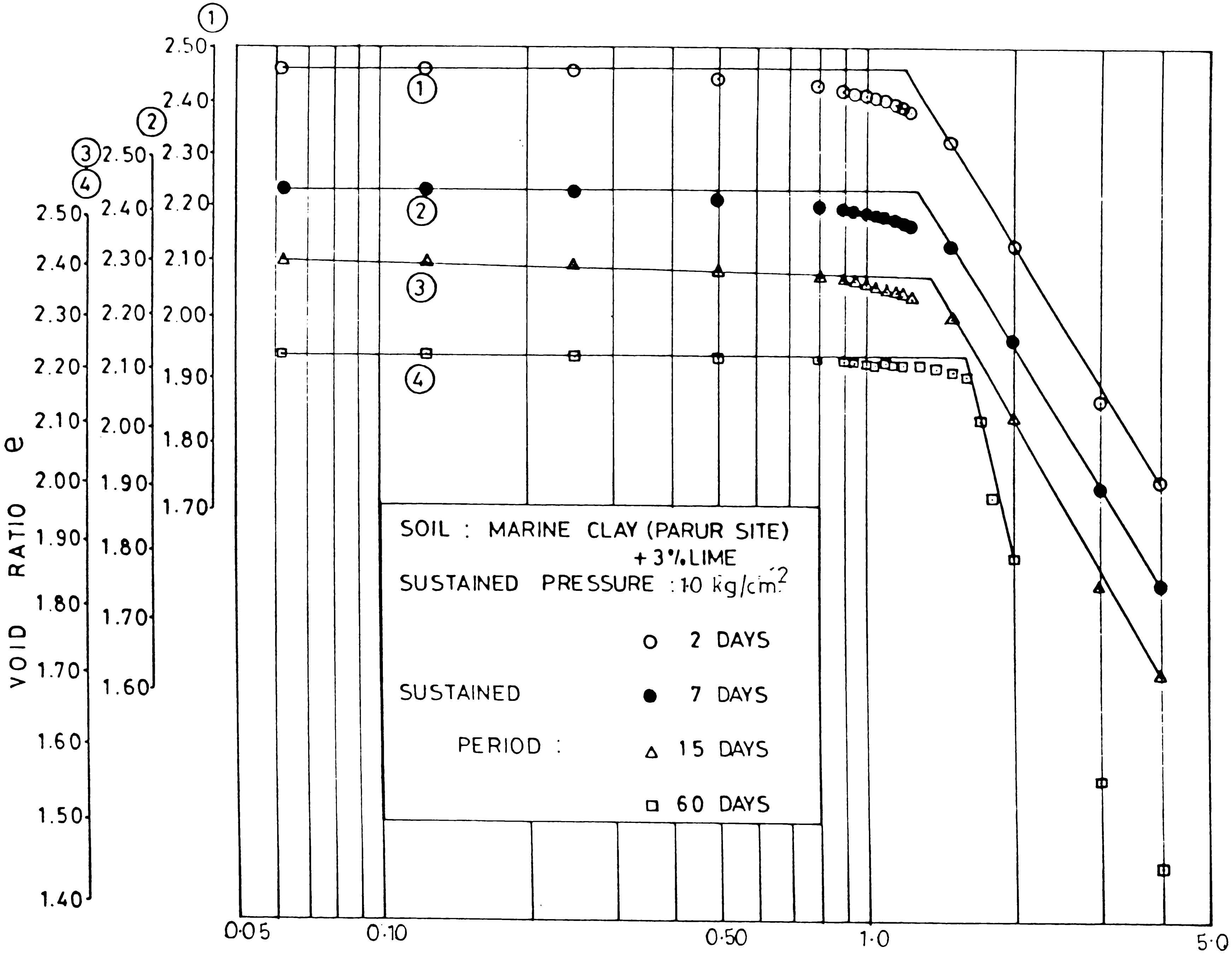

Fig. 5.6.3.6 log e – log p CURVES.

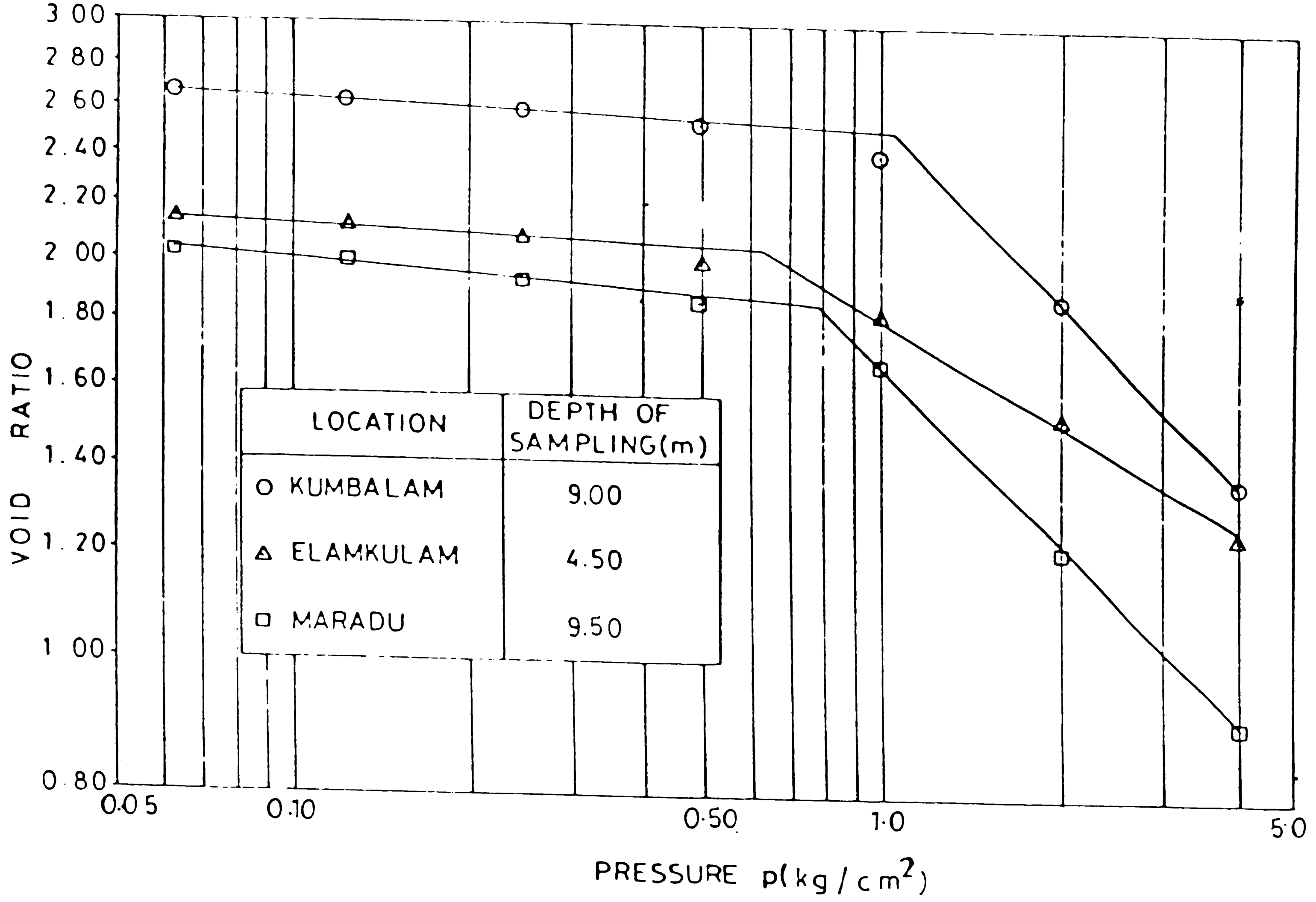

Fig. 5.6.3.7 : log e – log p PLOT

from the figure that the estimation of p_c is quite simple by the log - log method. These results are also included in Table 5.6.3.2.

A detailed comparative study between the p_c val ues given by the log-log method and the popular Casagrande method are also presented in Table 5.6.3.2. The superiority of the log-log method over the conventional Casagrande method is evident from the fact that results of log-log method are closer to the sustained pressure than those of the Casagrande method. It is further underscored by the consistent increase in the quasi-preconsolidation pressure with increase in duration of sustained period, while the increase in erratic in the case of results from Casagrande method.

5.6.4 Superiority of log-log method

The Casagrande method proposed in the thirties has been the only method in practice for the past half a century in spite of certain inherent drawbacks. Eventhough there have been 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 present series of tests clearly showed that the proposed 'log-log' method has considerable advantages over the existing methods for the determination of preconsolidation pressure. Since the tests were designed to give a pre-knowledge of the preconsolidation pressure, the results yielded by Casagrande method and log-log method could be compared with actual values of p_c. A comparative study indicates that the results by the Casagrande method were higher than the actual values of p_c. This has implications on the unsafe side.

As shown by Fig.5.6.4.1, the quasi-preconsolidation pressure increases with the period of sustained loading. While the log-log method clearly brought out this established fact, the results from the Casagrande method were erratic.

The greatest advantage of the log-log method is perhaps the elimination of errors due to personal judgement, as the preconsolidation pressure is obtained from the intersection of two straight lines while the selection of the point of maximum curvature plays a key role in the accuracy of the results given by the conventional Casagrande method. Thus the proposed log-log method is superior to the conventional methods for determination of preconsolidation pressure.

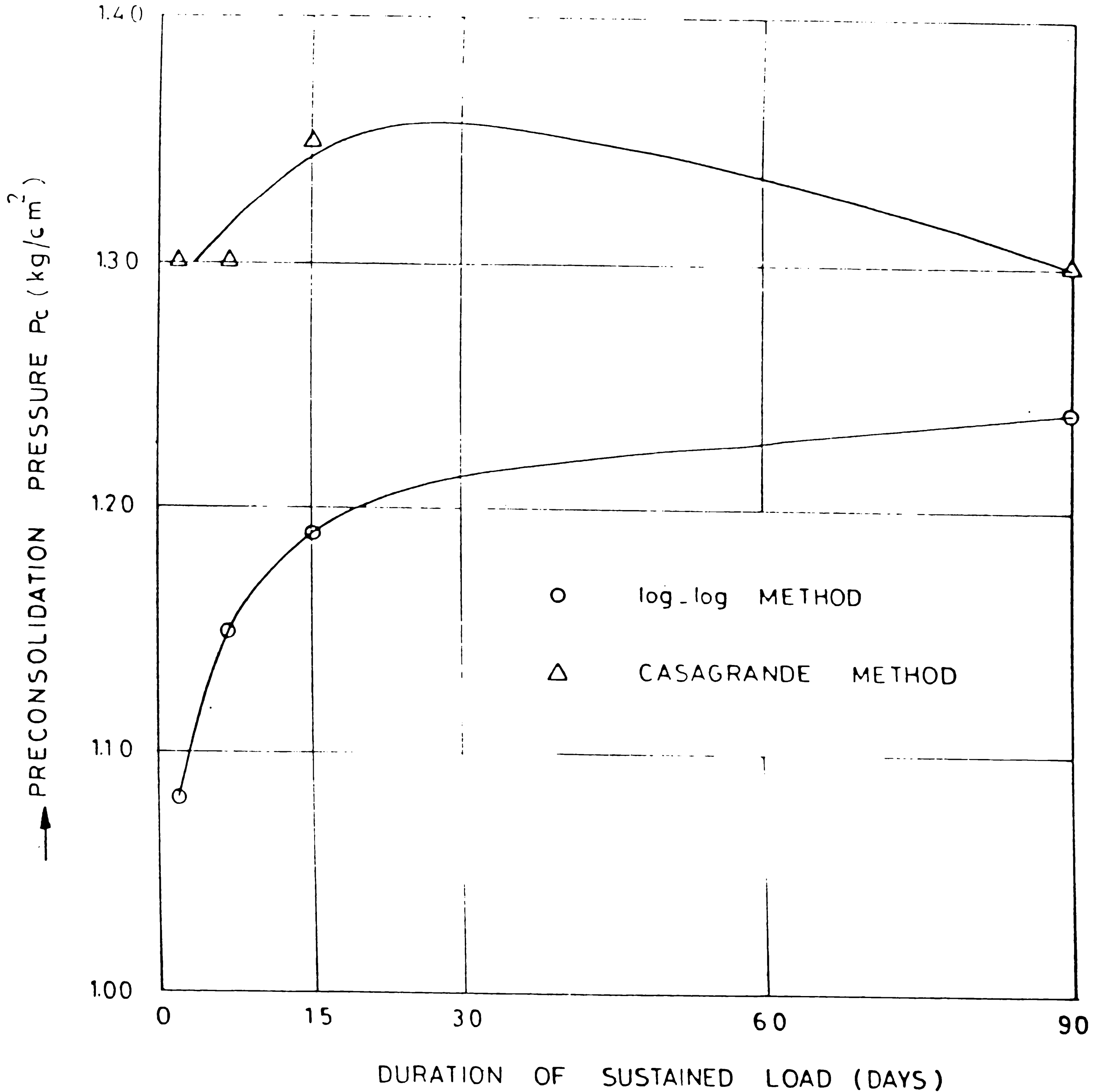

Fig. 5.6.4.1 P_c vs DURATION OF LOADING.

5.7 CONSOLIDATION CHARACTERISTICS OF COMPACTED MARINE CLAYS

Along the coastal belt of Indian Peninsula, long stretches such as Rann of Kutch in Gujarat, Calicut to Quilon in Kerala, and Visakhapatnam and its suburbs in Andhra, have deep deposits of marine clay which pose numerous problems to embankments constructed for highways and railways. Embankments built over soft marine clays often failed by rotational failure through base. Vast areas of shoulders or berms had to be constructed to prevent this failure which proved to be expensive. Hence improvement of engineering properties of marine clays as a base material and also as a construction material will necessarily form a part of a comprehensive study of the engineering behaviour of Cochin marine clays.

It has already been discussed in great detail, the enormous changes, that are brought in by air drying and oven drying of marine clays, in their index properties and consolidation characteristics. The aggregation of finer particles into coarser grains brought about during drying improves the compressibility characteristics of marine clays as shown by Fig. 5.7.2. The compressibility characteristics are considerably improved. The compression index of the air dried soil is 0.91 and oven dried soil is 0.42 as against 2.55 of the moist soil. Hence if the marine clay layers at the top is air dried by exposure to sunlight, its gains in engineering properties could be taken advantage of as a base course since the consolidation settlements are considerably reduced. Since air drying the top layers during summer by ploughing or similar means will be cheaper compared to the usually expensive stabilisation techniques, air drying can provide a cheap means of improving the geotechnical properties of marine clays.

Once the engineering properties of marine clays are improved by air drying, it can also be thought of as a possible construction material for the embankments. Hence compaction characteristics of the air dried marine clay samples were investigated and the results are presented below. Studies on oven dried samples were not taken up in detail due to their limitations regarding the feasibility in field applications.

Compaction studies were carried out on soil samples from Parur. The clay content of the original moist sample was 48%. On air drying, the fraction less than 2 microns dropped to 33% and oven drying brought down the percentage of clay size to 13%. The corresponding increase in coarser fractions can considerably alter the compaction and shear strength characteristics of Cochin marine clays.

Table 5.7.1 presents the physical properties of the marine clay samples used for the investigations.

Compaction was carried out as per procedures described in chapter III. Fig.5.7.1a shows the dry density--moisture content relationship for air dried and oven dried samples for both standard Proctor test and modified Proctor test. Fir air dried soil the maximum dry density achieved was 1.33 gm/cc at an optimum moisture content of 30.4% for standard test. For the modified compaction the corresponding values are 1.56 gm/cc and 23.2% respectively.

For the above tests, the samples were first air dried and compaction was done on air dried specimens by adding water to them. In order to find out whether the compaction characteristics will be similar in nature if the moist specimens are dried to the respective water contents instead of air drying first, another series of compaction studies were conducted with both the standard and modified procedures. The results of these tests are presented in Fig.5.7.1b. Eventhough the results are highly erratic without showing any peak or optimum value, this procedure of drying the samples was also adopted for a few investigations as the latter was more economical and less time consuming.

Table 5.7.1.

PHYSICAL PROPERTIES

Sl. No.	Physical Property	Moist soil	Air dried soil	Oven dried soil
1.	Liquid limit (%)	131.5	94.5	62.0
2.	Plastic limit (%)	53.9	45.2	40.5
3.	Plasticity index (%)	77.6	49.3	21.5
4.	Shrinkage limit (%)	18.1	18.8	19.2
5.	Grain size distribution			
	Clay (%)	48	33	13
	Silt (%)	34	47	55
	Sand (%)	18	20	32

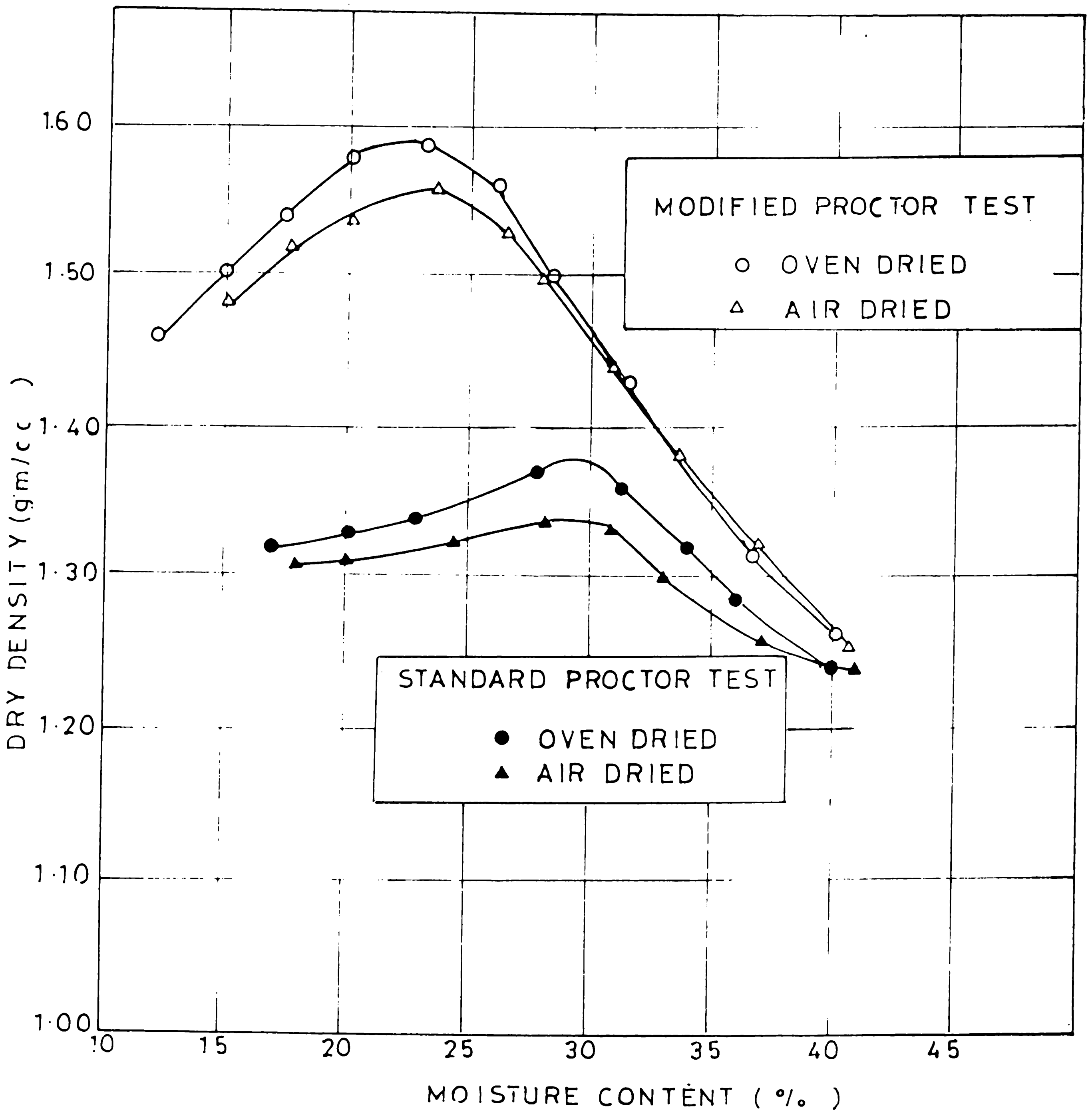

Fig. 5.7.1.a COMPACTION CURVES.

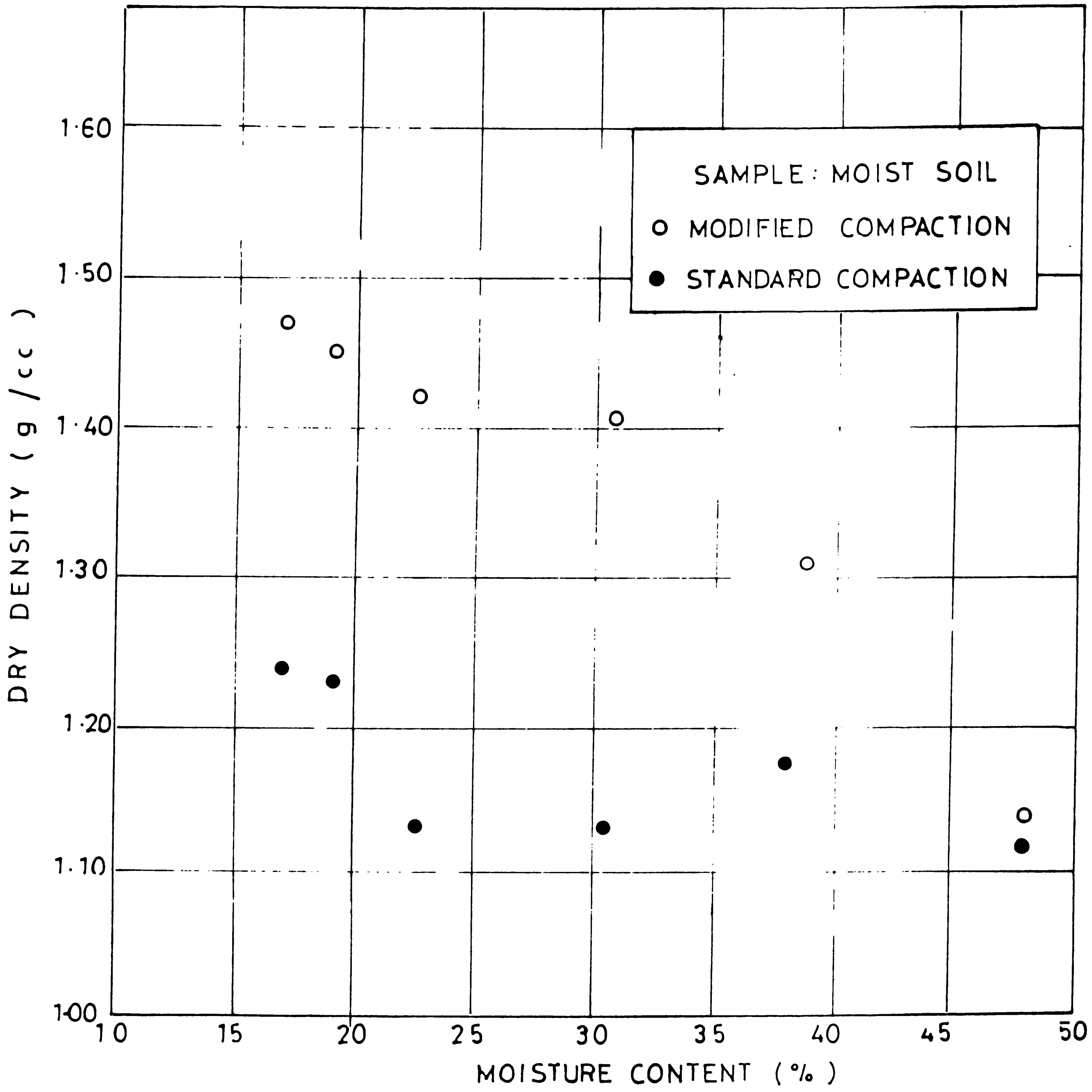

Fig. 5.7.1.b : MOISTURE CONTENT vs DRY DENSITY.

Consolidation tests were carried out on samples prepared at Proctor density (both standard and modified) and the results are presented in Fig.5.7.2. Since the samples prepared at optimum moisture content and maximum dry density were only partially saturated, the specimens would swell on inundation with water. To accommodate this possible swelling sample thickness was restricted to 1.5 cm instead of the usual 2 cm, for all the consolidation tests using dried samples. A comparative study of the e - log p curves for the moist soil at original void ratio and air dried samples at standard and modified Proctor density reveals the significant reduction in compressibility achieved through air drying. The figure also shows the test results on samples dried to optimum moisture content from moist state.

To bring out the remarkable reduction in compressibility obtained through air drying, deformation vs. time in minutes has been plotted in Fig.5.7.3. It gives the relation between and time for the stress increase of 0.5 to 1.0 kg/cm^2. The deformation for moist sample after 1440 minutes is 580 divisions. For air dried marine clay, given standard proctor compaction and modified proctor compaction, the corresponding values are 140 and 53 respectively. Thus the marine clays which pose numerous

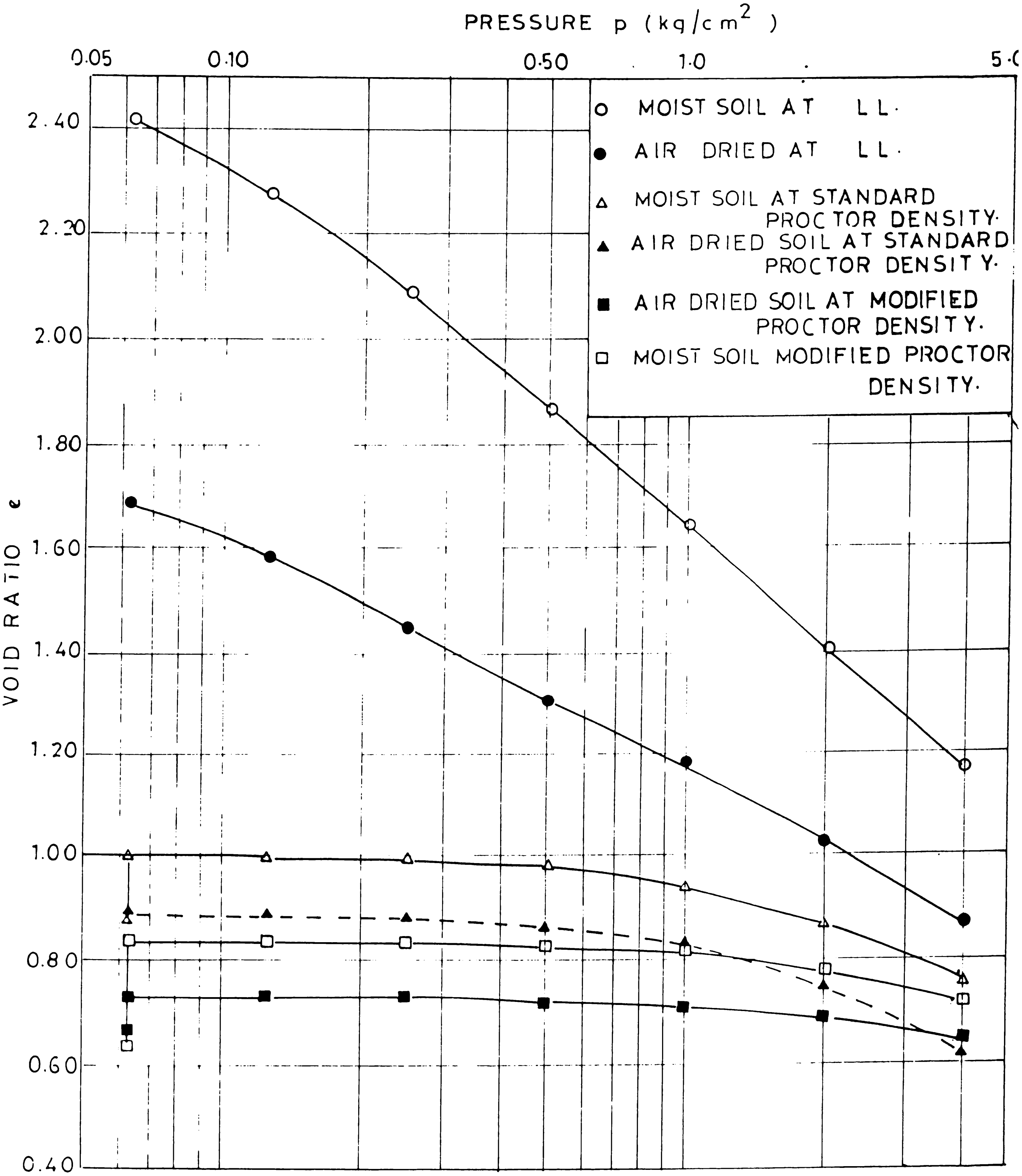

Fig. 5.7.2 e - log p CURVES.

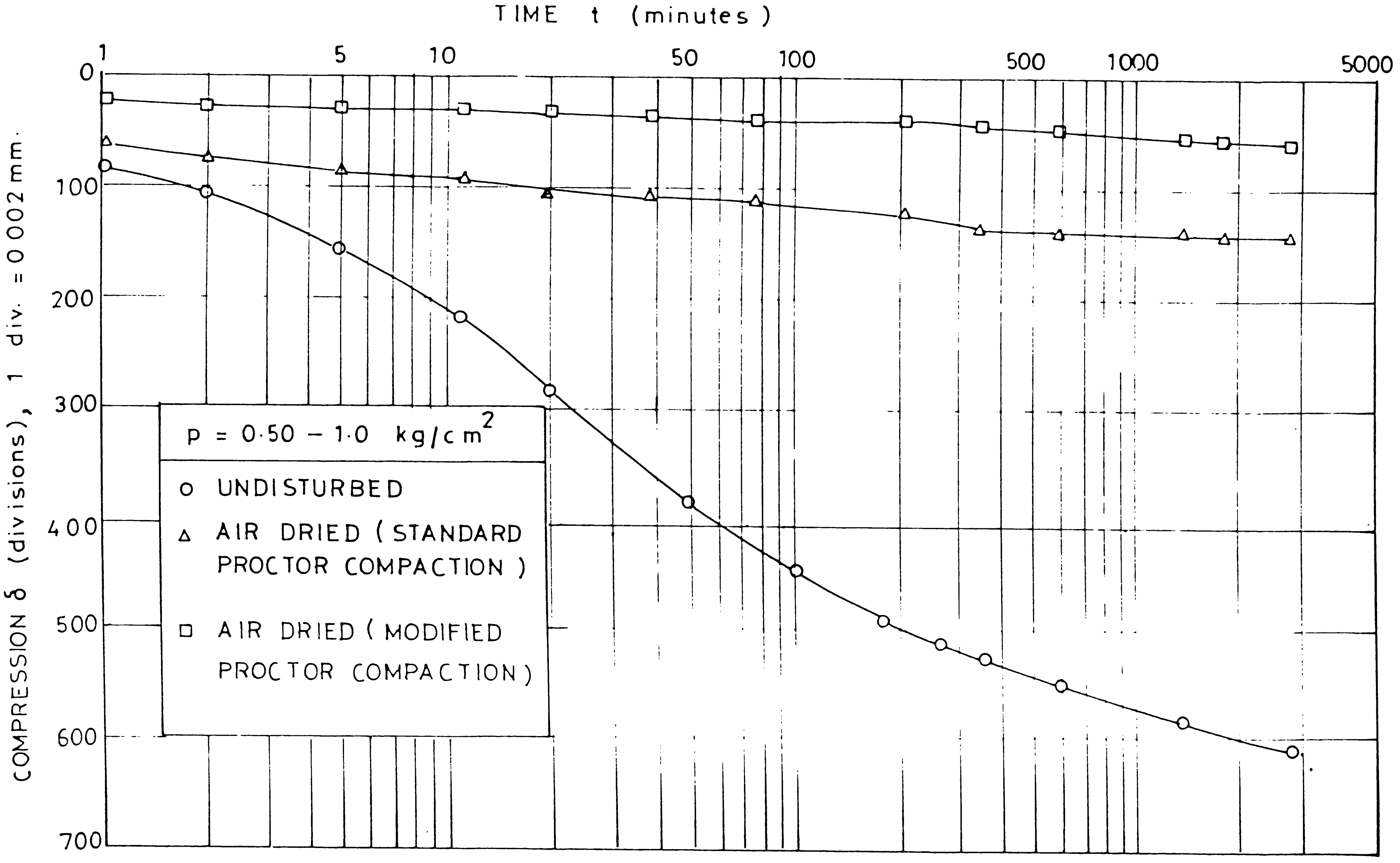

Fig. 5.7.3. : δ - log t CURVES.

problems to embankments by excessive settlement can be improved remarkably by an inexpensive treatment like air drying.

The values of de/d(log p), C_v and C are given in Table 5.7.2 for undisturbed, air dried and compacted at standard proctor density, air dried and compacted at modified proctor density, moist soil dried to standard optimum moisture content and moist soil dried to modified optimum moisture content.

5.7.1 Shear strength of compacted marine clays

Improvement in compressibility characteristics alone will not make the air dried marine clay a good construction material for embankments unless it is accompanied by considerable improvement in shear strength. Since the aggregation of finer particles into coarser fractions can improve the shear strength of air dried specimens of marine clays, a series of triaxial shear tests were conducted to measure the shear strength parameters viz. cohesion C and angle of internal friction ϕ, which are presented in Table 5.7.2.

Table 5.7.2

RESULTS OF COMPACTION TESTS AND TRIAXIAL SHEAR TESTS

Sl.No.	Description of sample	Compaction Tests		Triaxial shear tests			
		Optimum moisture content (%)	Maximum dry density (gm/cc)	Total Stress Parameters		Effective Stress	
				C (kg/cm^2)	ϕ (degrees)	C' (kg/cm^2)	ϕ' (degrees)
1.	Undisturbed sample	*97	* 0.73	0.07	6.5	0.08	8.2
2.	Air dried - Standard compaction	30.4	1.33	0.21	19.5	0.33	25.7
3.	Air dried - Modified compaction	23.2	1.56	0.94	25.7	1.10	25.7
4.	Moist soil dried to optimum moisture content and given standard compaction	30.4	1.33	0.16	23.8	0.33	24.5
5.	Moist soil dried to optimum moisture content and given modified proctor compaction	23.2	1.56	1.42	23.8	1.43	24.5

* These are the natural moisture content and dry density of moist samples from Parur.

5.8 INFLUENCE OF DIELECTRIC CONSTANT OF PORE MEDIUM ON ENGINEERING PROPERTIES

It is well recognised that the engineering properties of clays are dependent on the interaction of the clay particles with fluid contained in their pores (Sridharan, 1968; Sridharan & Rao, 1970, 1971, 1973; Kinsky, 1971). Most of the geotechnical properties can be linked with the electrical double layer of clays. The structure of the electrical double layer is determined to a great extent by the polarity of the pore fluid. This can be expressed in terms of the dielectric constant of the bulk material. In case of marine clays which will be fully saturated the bulk material is a two-phase medium. The dielectric constant of a dry silicate mineral is around 4 and that of water is about 80. The large difference in values clearly indicates that it is the dielectric constant of the pore medium which controls the behaviour of marine clays and their properties which are dependent on the value of dielectric constant. The equivalent thickness of the electrical double layer can be related to the square root of the dielectric constant.

To study the effects of the dielectric constant of different liquids on the engineering properties of clay, Kinsky et al. (1971) carried out a series of experiments on the hydraulic conductivity of cylindrical samples using organic fluids such as dioxene, ethanol, dimethyl sulphoxide, formamide and water. The value of dielectric constant ranged over 2.2 (dioxane) to 109 (formamide). The clay used was Wyoming bentonite in which montmorillonite represented 98% of clay minerals.

The hydraulic conductivity or coefficient of permeability of Wyoming bentonite samples 1 cm in height and 10 cm in diameter was 10^{-8} cm/sec for water. When dioxene whose dielectric constant was 2.2 compared to 80 for water, was used the permeability increased to 10^{-4}cm/sec. When water was allowed to flow again through the soil, instead of dioxene, the permeability was unchanged for one or two days. But after a few days the value of permeability slowly came down very close to the original value of 10^{-8} cm/sec.

The variation of permeability could be well explained by the dielectric property of the pore fluid. When a high dielectric fluid like water was used the thickness of double layer increased and the permeability was low. When dioxene was allowed to permeate through the sample, the thickness of double layer reduced considerably permitting free drainage. The permeability values for other organic fluids increased inversely proportional to the dielectric constant. Permeability of clayey soils can be increased by a number of order of magnitude as the dielectric constant is decreased.

When water was allowed to pass through a soil fully permeated with dioxene, the value of k did not increase appreciably for a number of days. But when formamide was allowed to flow through the sample, the permeability readily decreased to 10^{-7} cm/sec. This indicates that the reaction of different organic fluids with clay specimens are at different rates.

To study the effect of different pore fluids on the electrical double layer, a series of consolidation tests were carried out on oven dried samples of Cochin marine clays. The samples were initially loaded to 0.25 kg/cm^2 from a seating load of 0.0625 kg/cm^2. At this pressure, the following fluids were circulated for a number of days.

Pore fluid	Dielectric constant
1. Carbon tetrachloride	2.284
2. Acetone	20.70
3. Methanol	32.63
4. Water	80.4

After obtaining full saturation with the new pore fluid, the coefficient of permeability of the samples were measured. Fig.5.8.1 shows the plot between coefficient of permeability and head causing flow for the four fluids. It can be seen that carbon tetrachloride gives the highest values of k and water gives the lowest values for different heads. The permeability values given by acetone and methanol fall in between in the order of their dielectric constant. The slope of the straight lines giving k value for various heads, increases with decreasing dielectric constants.

Since water has the highest dielectric constant, it has maximum thickness for the electrical double layer. This restricts the cross sectional area available for flow. In case of carbon tetrachloride, since dielectric constant is only 2.284, the double layer thickness reduces and consequently increases the flow rate.

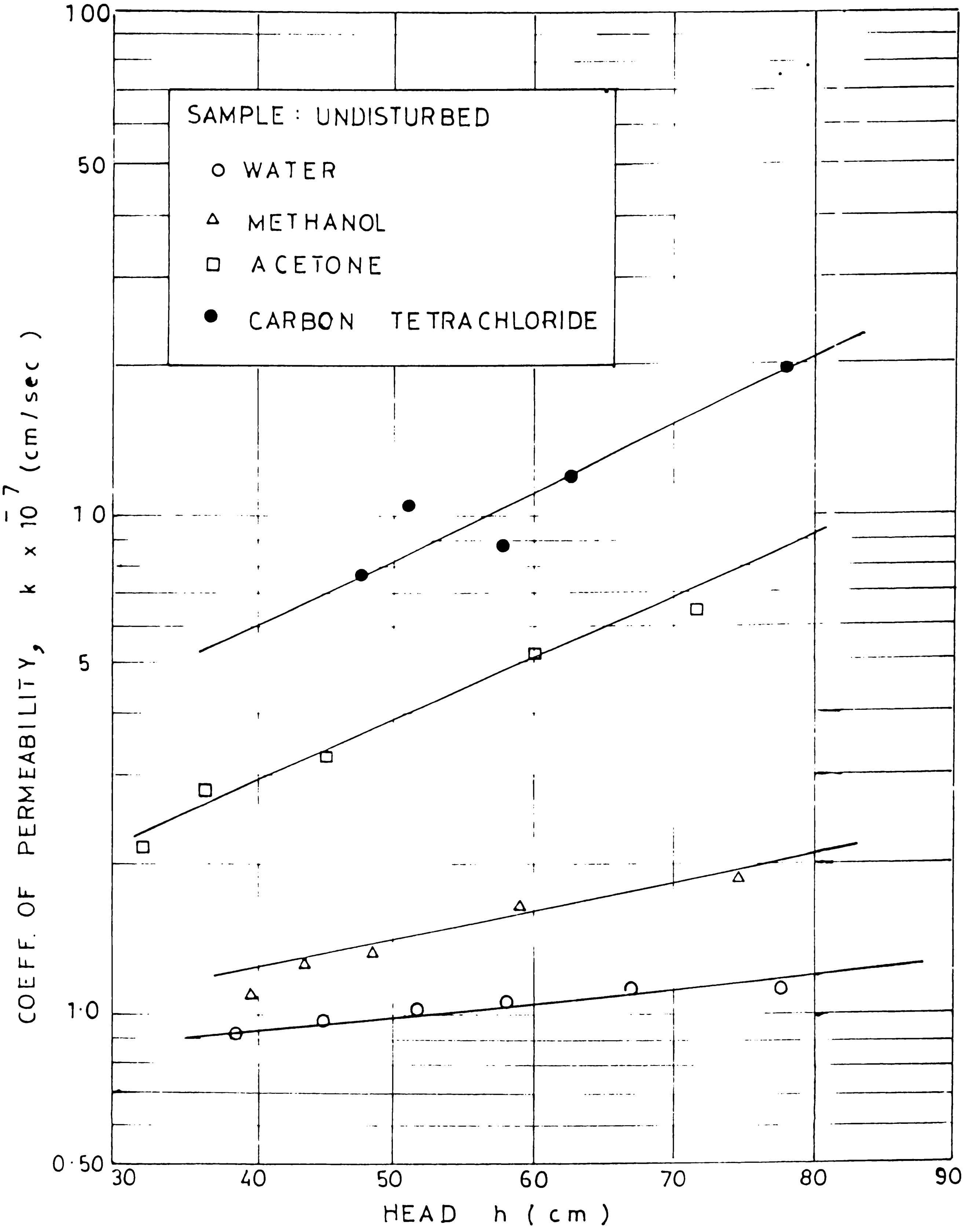

Fig. 5.8.1 HEAD vs PERMEABILITY.

5.8.1 Use of dielectric fluid to study compressibility behaviour

Sridharan and Venkatappa Rao (1970, 1972, 1973) have shown that there are two mechanisms which control the volume change behaviour of fine grained soils. In the first mechanism, where the compressibility of a clay is primarily controlled by the shear resistance at the near contact points, volume changes occur by shear displacements. In the second one the compressibility is governed by the long range electrical repulsive forces, primarily from the double layer. It is possible to find out which mechanism is applicable to a particular type of clay by conducting a series of consolidation tests wherein the shear resistance of the contact points or the electrical repulsive forces can be controlled. By reducing either of the above, if the compressibility could be increased the mechanism by which the compressibility of the clay is regulated could be identified.

The effective stress concept of Terzaghi is represented by

$$\bar{\sigma} = \sigma - u \qquad (5.8.1.1)$$

where $\bar{\sigma}$ = effective stress
σ = total applied pressure
u = pore water pressure

This could explain many of the phenomena associated with the strength and compressibility characteristics of soils. But it fails to explain some of the features in the strength characteristics of clays as the equation assumes that there are only two external forces acting on the soil grains viz. total stress σ and hydrostatic pressure u.

But it has been well established that both attractive and repulsive forces of an electrical nature exist between clay particles. Repulsive forces are primarily attributed to an interaction between double layers (Lambe, 1953, 1958; Seed et al., 1960). Repulsive forces are proportional to the dielectric constant of the pore medium (Bolt, 1956).

The attractive forces can be contributed by van der Waals' forces (Sridharan & Venkatappa Rao, 1973), London dispersion forces, coulombic attraction and hydrogen bonds (Lambe, 1953; Rosenquist, 1955). Some of these forces are inversely proportional to the dielectric constant of the medium and the distance between clay particles (Davies, 1965). Considering all these forces Lambe (1960) proposed a generalised equation

$$\sigma = \bar{\bar{\sigma}} a_m + \bar{\bar{u}} + R - A \tag{5.8.1.2}$$

where

σ = total stress

$\bar{\bar{\sigma}}$ = mineral to mineral contact stress

a_m = fraction of the total inter particle contact area

A = total interparticle attraction divided by total interparticle contact area

R = total interparticle repulsion divided by total interparticle contact area

$\bar{\bar{u}}$ = equivalent pore pressure.

This equation can be rewritten as effective stress

$$\sigma' = \sigma - \bar{\bar{u}} = \bar{\bar{\sigma}} a_m + R - A \qquad (5.8.1.3)$$

As per the equation effective stress increases with repulsive forces and decreases with attractive forces. This does not auger well with common concepts. Hence Sridharan (1968) rewrote this equation as

$$\bar{c} = \bar{\bar{\sigma}} a_m = \bar{\sigma} - \bar{u}_w - \bar{u}_a - R + A \qquad (5.8.1.4)$$

where

$\bar{C}$ = effective contact stress

$\bar{\sigma}$ = external applied pressure

$\bar{u}_w$ = effective pore water pressure

$\bar{u}_a$ = effective pore air pressure.

This equation shows that the effective contact stress is proportional to (A-R). It is difficult to assess quantitatively the exact values of the attractive or repulsive forces. But it is quite possible to regulate them by controlling the dielectric properties of the pore fluid. Since a study of this nature will help to identify the mechanism which governs the compressibility characteristics of Cochin marine clays, a well-designed series of consolidation tests involving change in pore fluid properties at various stages of the consolidation tests were undertaken.

5.8.2 Consolidation tests with different dielectric fluids

To find out the mechanism which controls the volume change behaviour of Cochin marine clays, series of consolidation tests were carried out on oven dried samples of Cochin marine clays from Parur. In all the tests the load increment ratio was kept at one. A duration of two days or the time required for the completion of primary consolidation was permitted for each load increment. Extreme precaution was taken to avoid errors due to side friction and air entrapment. Care was taken to minimise evaporation loss of pore fluid from the top of the cell and from the burette or reservoir, containing the fluid.

The following organic fluids were used for the consolidation tests on the oven dried marine clay specimens.

Pore fluid	**Dielectric constant**
1. Carbon tetrachloride	2.3
2. Ethanol	28.0
3. Methanol	32.6
4. Methanol + water	56.5
5. Water	80.4

Since there was considerable difference between the values of dielectric constants of water and the organic fluid close to it (methanol) another dielectric fluid was prepared by mixing equal volume of methanol and water and its dielectric constant was assumed as average of the values (Kinsky et al., 1971).

The specimen for consolidation tests using different organic fluids consisted of oven dried samples of Cochin marine clays packed dry into an oedometer ring. The thickness of the specimen was kept as 1.5 cm leaving a gap of 0.5 cm for possible swelling of the dry marine clay when it is inundated with the pore fluids of varying dielectric constant. After the samples have undergone the full swelling under a seating load of 0.0625 kg/cm^2, consolidation tests were conducted to find out equilibrium void ratio for each pressure intensity.

Fig.5.8.2.1 shows the plots of the consolidation test results. The change in thickness of the specimen expressed as a percentage of the original thickness$(\frac{\Delta H \times 100}{H})$ has been plotted against consolidation pressure for the five dielectric fluids. The curves exhibit a regular pattern in that the compression curve for the liquid with the highest dielectric constant viz. water, is the lowest and the curves for the fluid with the lowest dielectric constant, viz. carbon tetrachloride is the highest. The three other curves fall in between with the increasing values of dielectric constant from top to bottom. When the maximum compression of carbon tetrachloride is 7.5% only, the compression for the marine clay specimen mixed with water is as high as 24%. Thus the curves clearly show that the compression of the oven dried marine clay increases with the value of dielectric constant.

The relation between the equilibrium void ratio and the consolidation pressure is given in Fig.5.8.2.2. At the seating load, the sample washed with water has shown

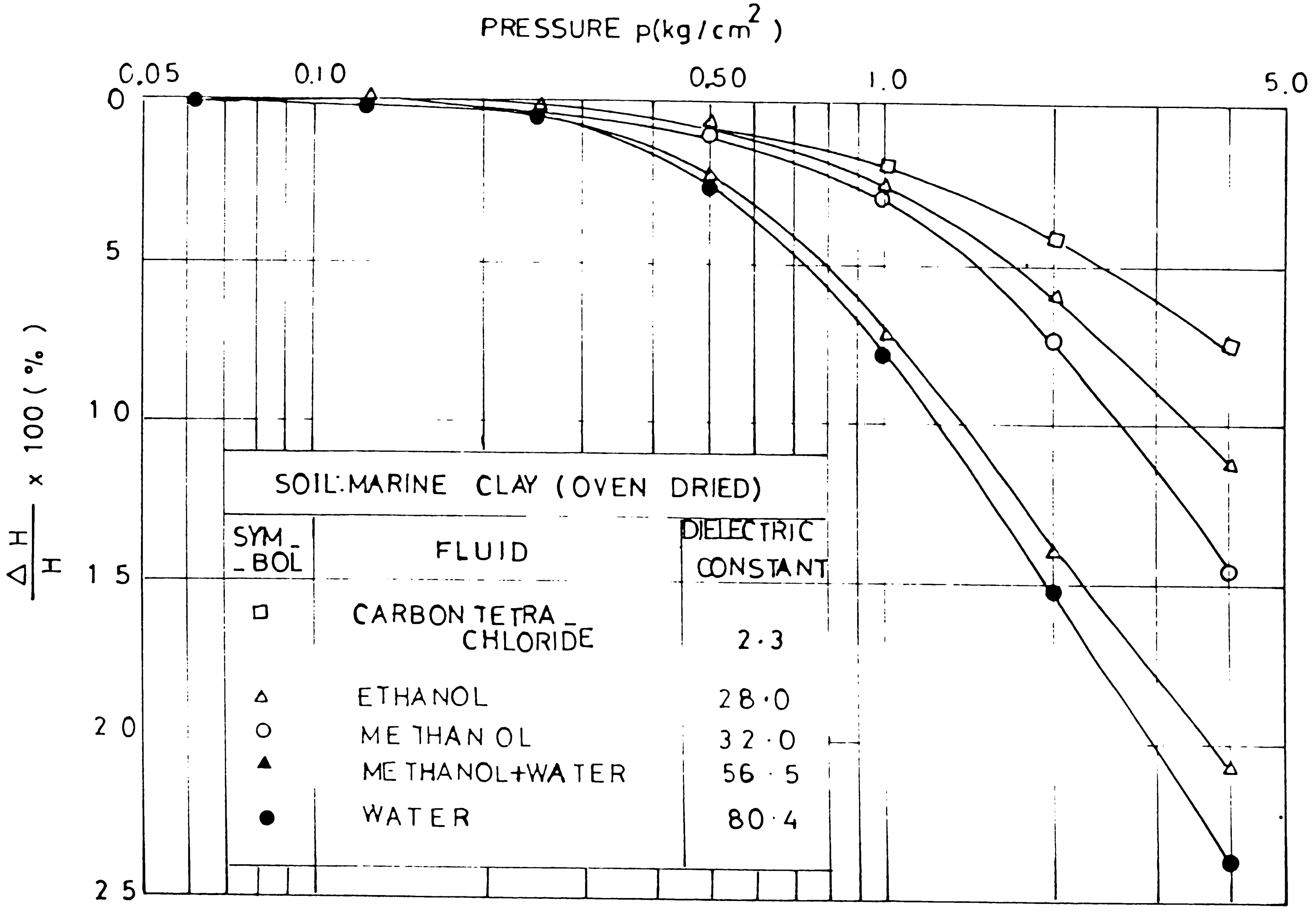

Fig. 5.8.2.1 : EFFECT OF DI ELECTRIC CONSTANT ON COMPRESSIBILITY.

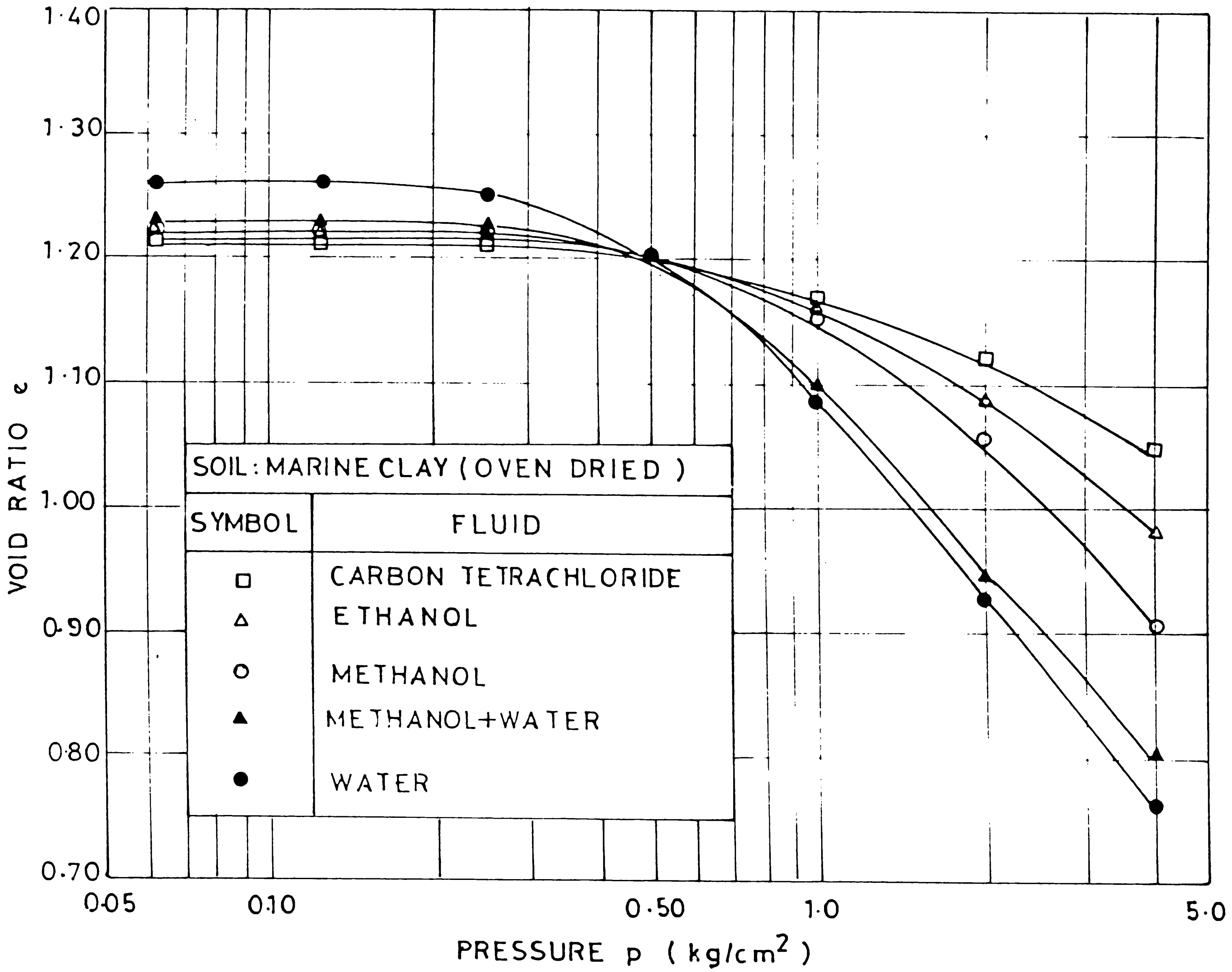

Fig. 5.8.2.2 e- log p CURVES.

maximum swelling and therefore has the highest void ratio. The void ratio of this specimen is higher than all other samples till a pressure of 0.5 kg/cm^2 is reached. The sample washed with carbon tetrachloride has the lowest values of equilibrium void ratio for the pressure intensities upto 0.5 kg/cm^2. As dielectric constant of pore fluid increases, the thickness of electric double layer also increases. The e - log p curves show that it is the repulsive forces that govern the compressibility of Cochin marine clays upto 0.5 kg/cm^2. The behaviour is similar to that of montmorillonite clays.

After a pressure of 0.5 kg/cm^2, the compressibility increases with dielectric constant of the pore fluid. This indicates that the mechanism of development of shear strength changes to the one in which attractive forces dominate. The shear strength now is improved by the reduction in distance between the clay platelets (Sridharan & Rao, 1973; Sridharan & Sudhakar Rao, 1985). This behaviour is similar to the pattern followed by kaolinite clays where the shear resistance and volume change behaviour are governed by factors like

i) frictional resistance
ii) clay fabric
iii) attractive forces arising from physiochemical mechanism (Sridharan & Sudhakar Rao, 1985).

Thus the pressure-equilibrium void ratio curves of oven dried samples of Cochin marine clays exhibit a peculiar behaviour wherein at lower pressures, the volume change behaviour is governed by repulsive forces and at higher intensities of pressure, the attractive forces dominate the shear resistance.

This change in the pattern of volume change behaviour might be due to the aggregation caused by drying. It may be noted that the oven dried marine clay was closer to the kaolinite clay in the plot between dielectric constant and liquid limit (Fig.4.12.1) and dielectric constant and free swell index (Fig.4.12.2).

Incidentally it may be of interest to note that while Terzaghi's effective stress concept fails to explain the contradictory behaviour of two basic clay minerals under identifical physico-chemical environment, the hypotheses put forth by Sridharan and Rao (1978) is able to fully explain this curious behaviour. This highlights the need for a wider acceptance for the new relation for effective stress.

The variation of de/d(log p) with pressure for different dielectric fluids is shown in Fig.5.8.2.3. The

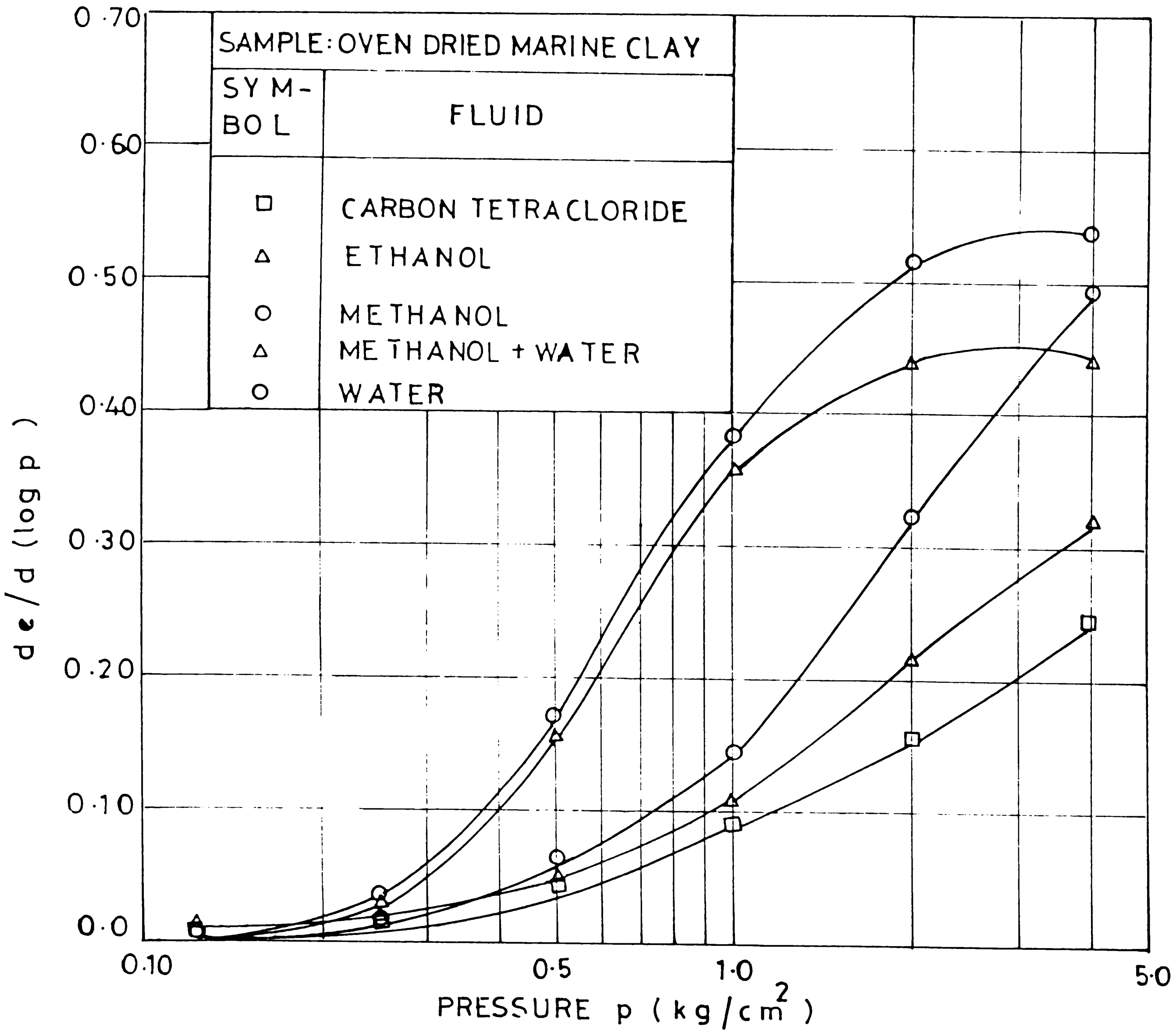

Fig. 5.8.2.3 de/d(log p) vs PRESSURE.

value of de/d(log p) is minimum for carbon tetrachloride and maximum for water. It steadily increases both with consolidation pressure and with dielectric constant of pore fluid. Dielectric constant ϵ vs. $\frac{\Delta H}{H}$ x 100 is plotted in Fig.5.8.2.4. As indicated by the previous figures, compressibility increases with respect to pressure as well as with dielectric constant of pore fluid.

The swelling characteristics of the oven dried marine clay is brought out by the rebound characteristics of the consolidated samples shown in Fig.5.8.2.5. Figure shows the rebounded of the consolidated specimens when the pressure was reduced in steps from 4 kg/cm^2 to the seating load of 0.0625 kg/cm^2. The highest rebound is shown by water and lowest is by carbon tetrachloride. The volume increase is proportional to the value of dielectric constant of pore fluid indicating that the increase in volume is due to the higher thickness of the electrical double layer.

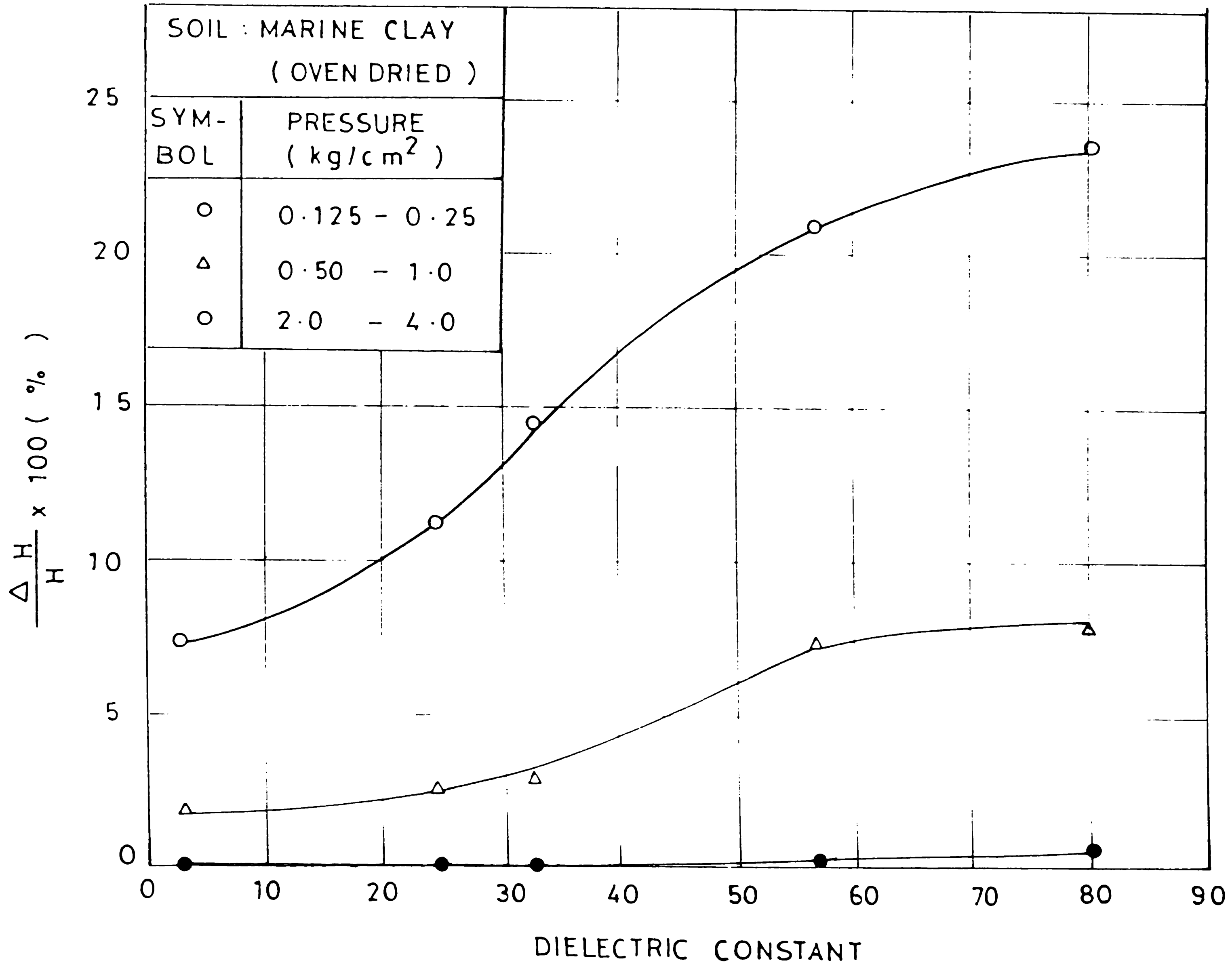

Fig. 5.8.2.4 : $\frac{\Delta H}{H}$ vs DI ELECTRIC CONSTANT.

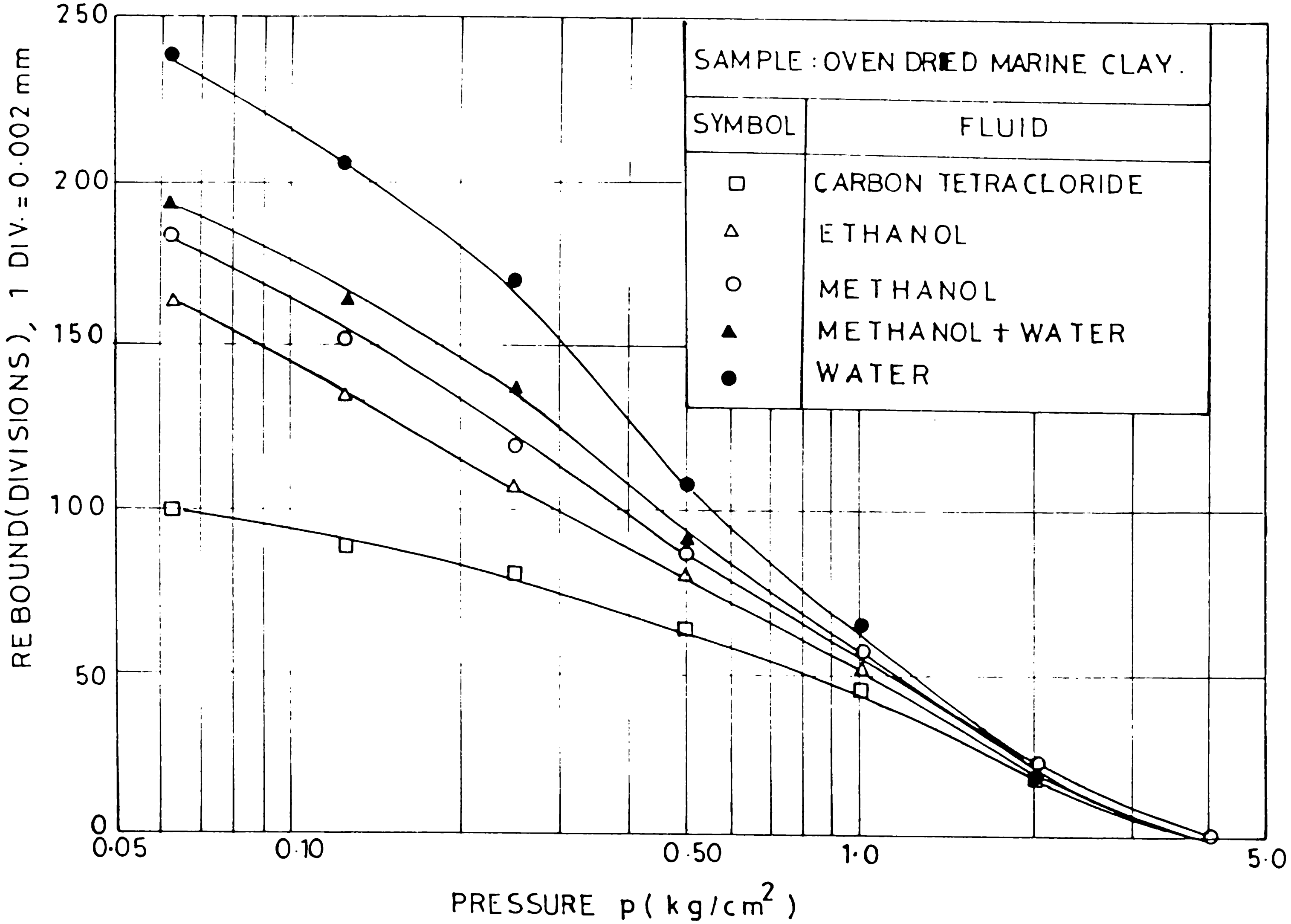

Fig. 5.8.2.5 REBOUND vs PRESSURE.

Chapter VI

ENGINEERING BEHAVIOUR OF LIME TREATED MARINE CLAYS

6.1 INTRODUCTION

The use of lime as a stabilising agent dates back to the early Romans who used it in the construction of their roads, especially the famous 'Via Appian'. This road has given outstanding service since its construction and is still maintained as a traffic artery (Leonards, 1962).

Clay minerals carry regular negative charges on their surface and to a large extent are responsible for imparting plasticity to the soil sample. Such soils have been successfully stabilised with lime which decreases the plasticity of the soil. Lime stabilised soils find wide application in subbase courses in highway and airport pavements and as foundation bed for buildings.

Although the details of the chemistry of lime stabilisation is not yet fully understood (Thompson, 1966) the improvements in engineering characteristics of the lime stabilised soils are attributed to

i) cation exchange
ii) flocculation
iii) carbonation
iv) pozzolanic reaction

The general order of replaceability of the common cations associated with soils is given by the lytropic series (Grim, 1953) $Na^+ < K^+ < Ca^{++}$ Mg^{++}. Any cation will tend to replace the cation to the left of it and monovalent cations are usually replaced by multivalent cations. The addition of lime supplies an excess of Ca^{++} and cation exchange will occur.

The addition of lime to a fine grained soil causes flocculation and agglomeration of the clay fraction. Electrolyte content of pore fluid influences the activity (Herzog & Mitchell, 1963).

Lime, especially when the soil is exposed to atmosphere, reacts with carbon dioxide and forms relatively weak cementing agent of calcium carbonate.

The pozzolanic reaction is the reaction between soil silica and/or alumina and lime to form various types of cementing agents. These reactions are considered as the major contributing factor to strength development in lime soil mixtures by quite a few workers (Thompson, 1964; Eades, 1962; Narain & Singh, 1966). The complexity of the whole process is brought out by the hypotheses proposed by the following workers for the nature of soil-lime reaction (Humad, 1987).

1. Miller and McNicol (1958) suggested formation of $CaCO_3$ from $Ca(OH)_2$.

2. Galloway and Bucharan (1951) advocated cation exchange.

3. Arba (1948) proposed formation of calcium alumina silicate complexes from added lime and the free silica and alumina.

4. Laguros et al. (1956) proposed flocculation as a result of decreased potential.

5. Clare and Cruchley (1957) suggested flocculation due to proton transfer and subsequent binding by Ca^{++}.

As the main objective of the present investigations on lime treated marine clay specimens are improvements in shear strength and compressibility characteristics of these soft clays, prime importance was given to measurement of shear strength and compressibility characteristics rather than going into the details of the reactions caused by various possible stabilising agents. About 18 different additives were tried and the improvement in shear strength was studied by laboratory vane shear tests. This series of tests clearly indicated that lime was the most suitable stabilising agent as far as marine clays are concerned. Hence detailed investigations were carried out on the potential of lime as stabilising agent for Cochin marine clays.

6.2 SELECTION OF A SUITABLE STABILISING AGENT

In an attempt to identify the most suitable stabilising agent for marine clays, the following additives were tried on 'moist' marine clay samples. Two industrial wastes tried did not show promise.

1. Sodium hydroxide
2. Sodium chloride
3. Sodium silicate
4. Potassium hydroxide
5. Potassium chloride
6. Potassium dichromate
7. Ferric chloride
8. Calcium hydroxide
9. Calcium chloride
10. Calcium carbonate

11. Calcium sulphate (gypsum)
12. Sodium carbonate
13. Sodium hexametaphosphate
14. Potassium permanganate
15. Magnesium chloride
16. Aluminium chloride
17. Lime
18. Cement

Each of the above additives was mixed with 'moist' soil at a rate of six percent by weight. The weight of additives to be added was determined by comparing with equivalent dry weight of the 'moist' sample. The sample was mixed thoroughly and transferred to vane shear moulds. The specimens were then subjected to laboratory vane shear test after curing for one week and one month. Three specimens were tested each time and average value was taken for the shear strength. The shear strength values after one week and one month gave a comparative evaluation of the potential of the additive as a stabilising agent. The results are presented in Table 6.2.1.

The table also gives the Atterberg limits and shrinkage limit of treated soils found out immediately after mixing. Out of the 18 additives tried, lime, calcium hydroxide and cement showed very impressive gains in strength. Sodium silicate, potassium dichromate and aluminium chloride gave

Table 6.2.1

VANE SHEAR TEST RESULTS

		Cochin marine clay					Red Earth				
Sl. No.	Salt	Liquid limit (%)	Plastic limit (%)	Shrink-kage limit (%)	Shear strength (kg/cm^2)		Liquid limit (%)	Plastic limit (%)	Shrink-age limit (%)	Shear strength (kg/cm^2)	
					cured for 1 week	cured for 1 month				cured for 1 week	cured for 1 month
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
	Untreated	136.5	55.6	20.10	0.026	0.026	52.5	35.3	27.1	0.028	--
1.	Lime	147	83	34.30	0.276	0.497	49.5	--	38.5	0.051	0.064
2.	Sodium hydroxide	120.5	52	23.15	0.043	--	64.5	32.5	14.80	Very high	0.270
3.	Sodium chloride	115	46.1	18.40	0.036	--	38.0	25.7	19.10	0.031	--
4.	Sodium silicate	101.5	49.8	25.35	0.084	0.113	39.5	25.6	21.20	0.101	0.095
5.	Potassium hydroxide	133	60.2	26.48	0.065	--	60.0	32.8	18.6	0.452	0.326

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
6.	Potassium chloride	126	55.9	22.93	0.058	--	50	26.9	15.50	0.031	--
7.	Potassium dichromate	106	50.8	23.92	0.096	0.127	47	26.7	21.0	0.039	0.040
8.	Ferric chloride	129	58.3	--	0.051	0.072	46.5	--	14.4	--	--
9.	Calcium hydroxide	--	--	--	0.158	0.329	--	--	--	--	--
10.	Calcium chloride	--	--	--	0.044	--	--	--	--	--	--
11.	Calcium carbonate	--	--	--	0.033	--	--	--	--	--	--
12.	Calcium sulphate (gypsum)	--	--	--	0.042	0.052	--	--	--	--	--

(Contd.)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
13.	Sodium carbonate	--	--	--	0.035	0.042	--	--	--	--	--
14.	Sodium hexameta phosphate + sodium carbonate	--	--	--	0.048	0.053	--	--	--	--	--
15.	Potassium permanganate	--	--	--	0.052	0.054	--	--	--	--	--
16.	Magnesium chloride	--	--	--	0.031	0.040	--	--	--	--	--
17.	Aluminium chloride	--	--	--	0.081	0.071	--	--	--	--	--
18.	Cement	--	--	--	0.187	0.320	--	--	--	--	--

marginal increase in shear strength. Others did not have much effect on the strength of marine clays.

Lime showed the highest promise as a stabilising agent. Within one week, the gain in strength was more than 10 times. After one month curing, the shear strength was about 18 times the original strength.

For a comparative study of the action of the additive on a non-marine soil, the same additives were added to samples of red earth with a consistency close to that of Cochin marine clays. The gain in strength due to addition of lime was marginal in case of red earth. But sodium hydroxide and potassium hydroxide registered good gain in strength during the first week. But the strength was found to lose with time. Since other additives did not show promise with Cochin marine clays, they were not tried on red earth.

Cement also gave very encouraging results as a stabilising agent for marine clays. However further studies in the direction of improving the shear strength and compressibility characteristics of Cochin marine clays were restricted to lime only.

6.3 PHYSICAL PROPERTIES OF LIME TREATED SOILS

In most cases, the effect of lime on plasticity of clay is more or less instantaneous (Bell, 1988). Calcium ions from lime cause a reduction in plasticity of cohesive soils and make them more friable and more easily workable (Dandson & Handy, 1960). Only very small quantities of lime are required to bring about these changes in plasticity (Bell, 1988).

The addition of lime increases the plastic limit. Eventhough the variation of liquid limit with increase in lime percent is not very regular that of plastic limit is a steady increase. This brings down the plasticity index accompanied obviously by a reduction in plasticity.

Studies on the effect of lime on liquid limit of different soils give conflicting results. Investigations in India on black cotton soils showed a rise in liquid limit (Uppal & Bhatia, 1958). Similar observations have been made by Spangler and Patel (1949). But Clare and Cruchley (1957) feel that the influence on liquid limit is more complex.

In case of Cochin marine clays the effect of lime has been erratic. The physical properties of lime treated

Table 6.3.1

PHYSICAL PROPERTIES OF LIME TREATED MARINE CLAYS

Site: Parur

Sl.No.	Percentage of lime	Curing period	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)	Shrinkage limit (%)	Free Swell index (cc/gm)
1.	0	--	129	56.5	72.5	30.30	6.40
2.	3	1 week	144	50.8	93.2	22.3	6.2
		1 month	134	54.2	79.8	20.0	7.5
3.	6	0 day	138.5	71.5	67.0	38.8	8.3
		3 days	142	66.7	75.3	30.6	--
		1 week	145	83.0	62	31.5	7.6
		2 weeks	144	77.0	67.0	33.7	7.5
		1 month	147	73.1	73.9	32.7	7.7
		2 months	146	76.0	70.0	32.1	7.1
		3 months	144	76.0	68.0	32.5	6.6
		6 months	137.5	69.6	67.9	28.3	--

Table 6.3.2

PHYSICAL PROPERTIES OF LIME TREATED SOILS

Sl.No.	Percentage of lime	Curing period	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)	Shrinkage limit (%)	Free Swell index (cc/gm)
1.	0	--	129	56.5	72.5	20.30	6.40
2.	9	1 week	127.5	55.8	71.7	33.7	7.7
		1 month	136	76.7	59.3	43.3	6.0
		2 months	149	80.7	69.3	46.9	7.1
		3 months	144	71.4	72.6	35.2	6.1
		6 months	141	76.5	64.5	52.3	6.3
3.	12	1 week	142	69.8	72.2	37.2	8.1
		1 month	143	80.5	62.5	39.7	7.1
		2 months	145.5	94.5	52.0	57.0	6.4
		3 months	142.5	85.0	57.5	50.0	6.5
		6 months	152	96.0	56.0	60.5	6.1

soil for various lime contents and different periods of curing are presented in Tables 6.3.1 and 6.3.2. It can be seen from Tables 6.3.1 and 6.3.2 that liquid limit initially increases with time and then decreases. With time there is a general decrease in plasticity index which is seen for all percentages of lime. For both 9% and 12% of lime the shrinkage limit significantly increases with time whereas for 6%, shrinkage limit increases significantly with time initially and then marginally decreases.

The variation of liquid limit with increase in lime content for curing periods of 1 week, 1, 2, 3 and 6 months can be seen from the tables. It can also be seen from the tables that only very low percentages of lime are required for the changes in Atterberg limits which is in agreement with the findings of Sherwood (1967). The plasticity index tends to decrease with increase in lime content.

Compared to the Atterberg limits, the variation in shrinkage limit is more steady with increase in lime content as shown by the tables. The shrinkage limits tend to increase with lime content. This could perhaps be attributed to flocculation and agglomeration (Thompson, 1966). The free swell index increases initially but on curing the value slowly decreases.

6.4 COMPRESSIBILITY CHARACTERISTICS OF TREATED MARINE CLAYS

The selection of the most suitable stabilising agent for Cochin marine clays was mainly done based on the development of shear strength of treated samples measured by the laboratory vane shear tests. There has been a phenomenal increase in shear strength of marine clays with lime treatment. But it had to be verified whether these gains in strength are accompanied by an improvement in compressibility characteristics since excessive consolidation settlement was the major concern of the foundation engineers dealing with Cochin marine clays. In order to verify this and allied properties, a series of consolidation tests were carried out on marine clay samples treated with different percentages of lime and cured for various periods.

Fig.6.4.1 shows the compression curves for undisturbed, remoulded and lime treated marine clay samples from Parur site collected from a depth of 7.5 m. The compression curves of treated soils are above the curves for untreated samples eventhough the initial moisture content of treated soil was 106% compared to 87% of undisturbed sample. The compressibility characteristics improve with time as shown by the upper curve when the sample was cured for one month before the test, compared to the lower curve for which no time was allowed for curing.

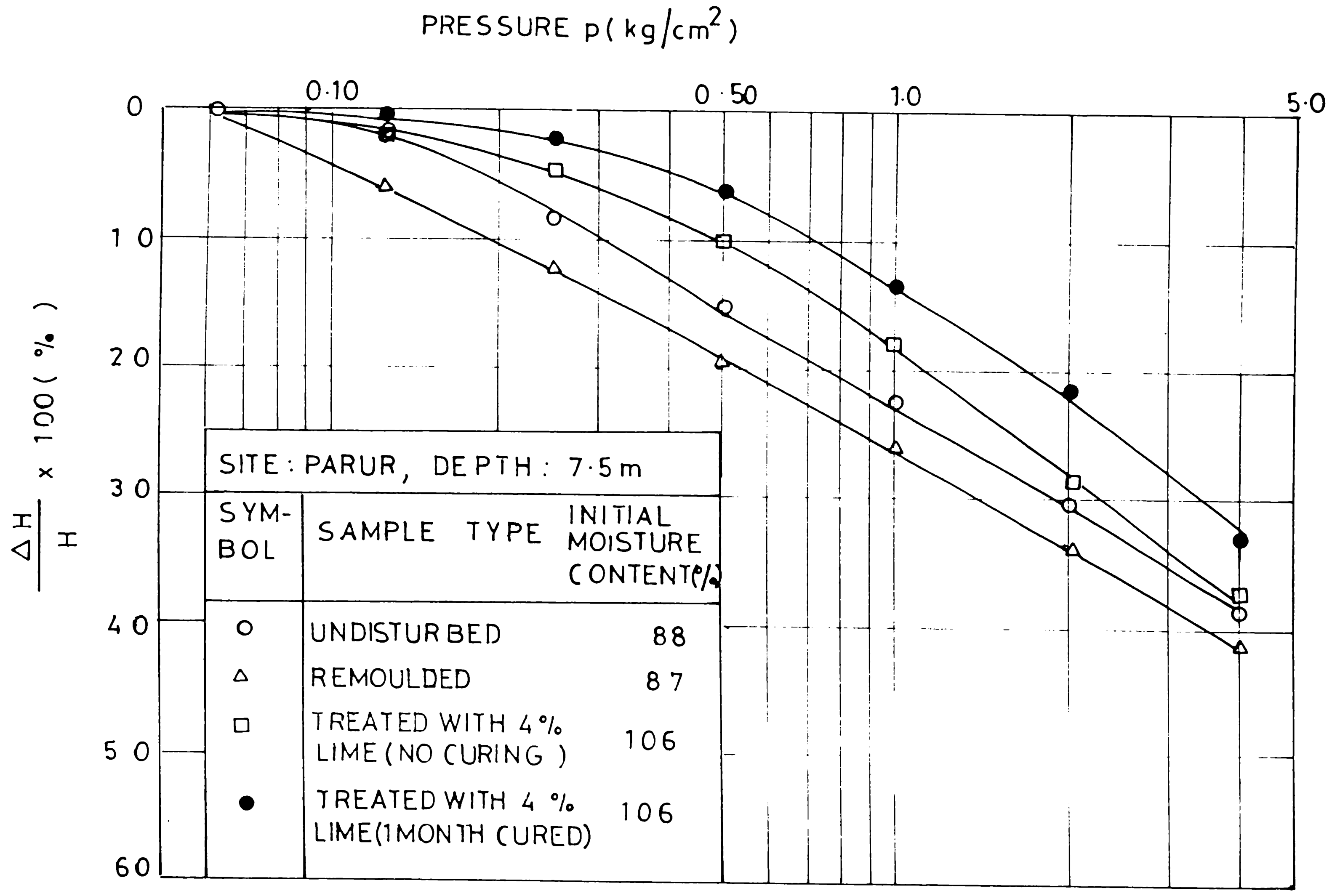

Fig. 6·4·1 COMPRESSION CURVES.

The improvement in compressibility characteristics are more conspicuous at lower pressure levels than at higher pressures. For the consolidation pressure of 0.25 kg/cm^2, the $\frac{\Delta H}{H}$ value is only 4% compared to 12.5% for the remoulded soil. The comparison has been drawn with remoulded marine clay as the treated soil gets thoroughly remoulded during mixing with lime. But at a pressure of 4 kg/cm^2 $\frac{\Delta H}{H}$ values are 33% for treated soil cured for one month and 42% for remoulded soil.

Fig.6.4.2a shows the compression curves for lime treated soils cured for various durations of 3 days, one week and two weeks along with the curve for undisturbed soil. It can be seen from the plots that the compressibility characteristics steadily improve with curing period. The ΔH values at 0.5 kg/cm^2 for the specimen cured for 14 days is only 3% compared to 14.5% for undisturbed sample. This indicates that the consolidation settlements will be only 20% of the settlement that may be realised in case of untreated soils. But at higher pressures, for example at 4.0 kg/cm^2, the corresponding values are 38% and 28% which represents a settlement ratio far below the earlier pair of values.

A study of the four curves show that, the stabilised soil develops a preconsolidation pressure (bond strength) slightly above 0.5 kg/cm^2. Till this point the magnitude of compression is insignificant. Once the consolidation pressure crosses this p_c value, the settlements for treated soils tend to increase. It can also be noted that the portions of the compression curves after this p_c value are more or less parallel. These results perhaps indicate that the consolidation settlements from a treated soil will be considerably less than that of untreated soil if the total pressure $p_o + \Delta p$ are kept well below the p_c values developed through stabilisation.

It can also be seen from the curves that while the behaviour of untreated soil present a gradual failure, more elastic in nature, the compression curves of treated soils give a picture of brittle behaviour.

Fig.6.4.2b, c and d present the compression curves for lime percents of 6, 9 and 12%. The plots confirm the earlier findings.

Fig.6.4.2d gives the compression curves for 12% lime, for curing periods of 0 day, 8 days, 1 month, 3 months, 6 months and 18 months. It can be seen that there is a

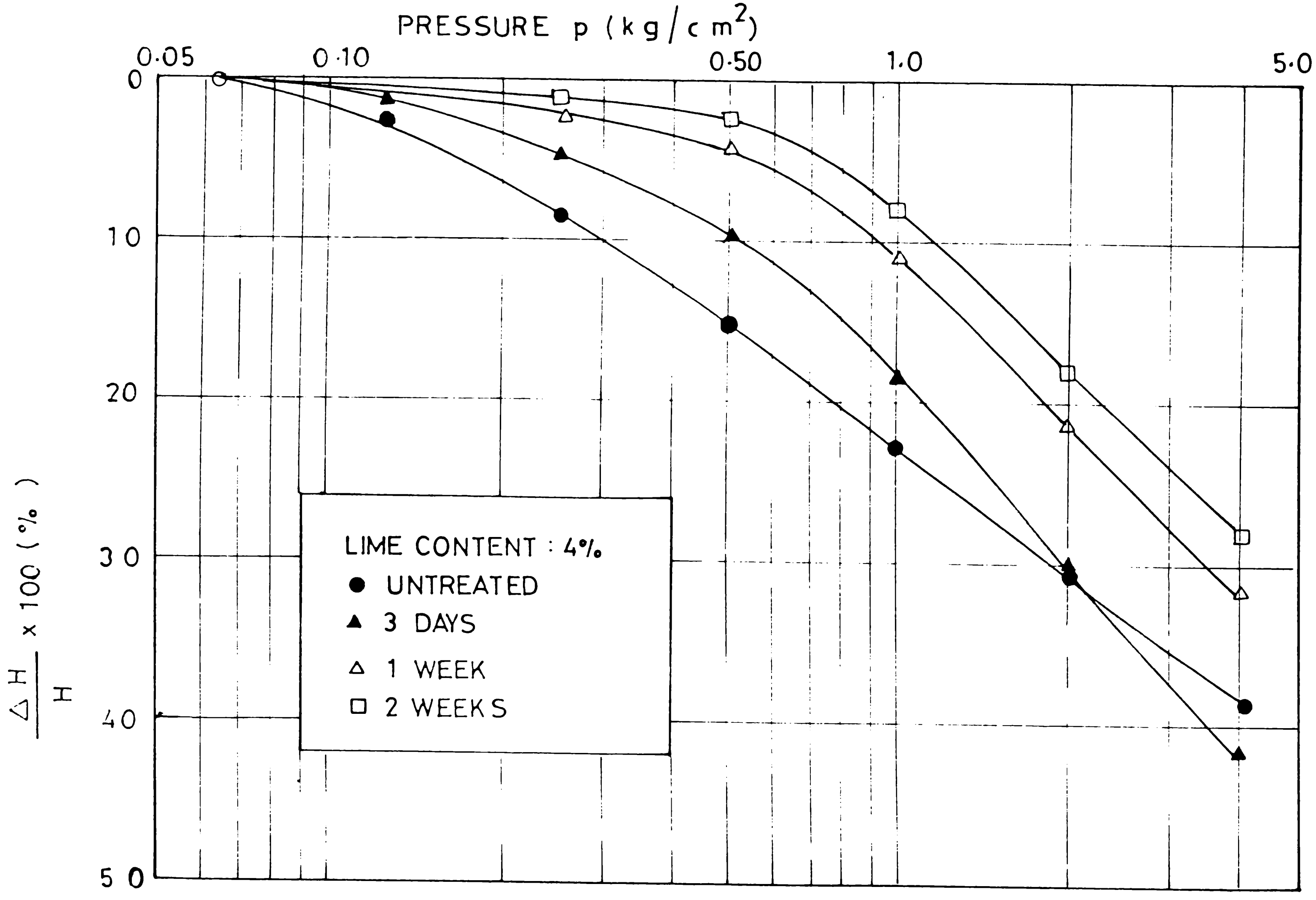

Fig. 6.4.2a COMPRESSION CURVES.

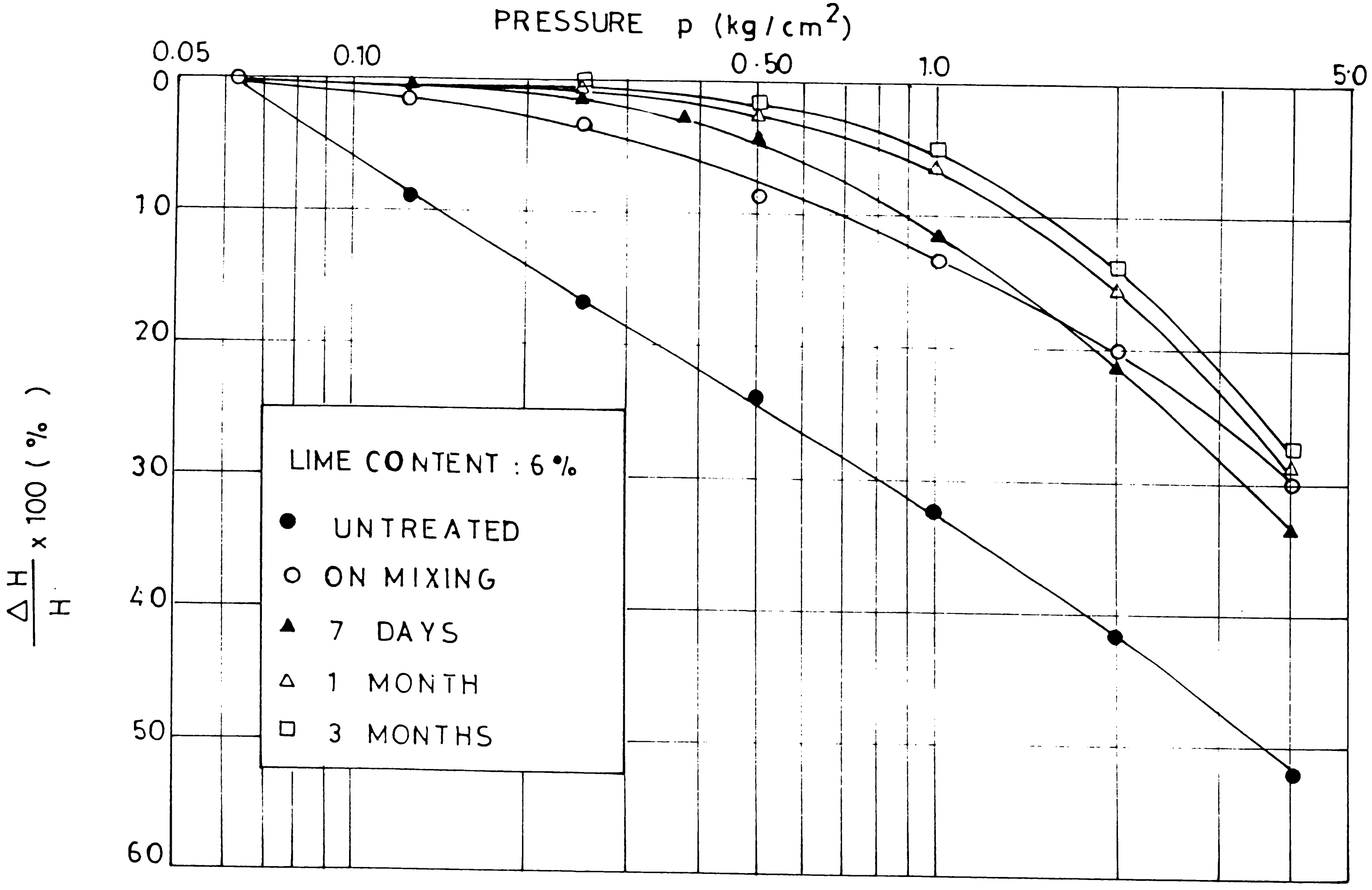

Fig. 6.4.2b : COMPRESSION CURVES.

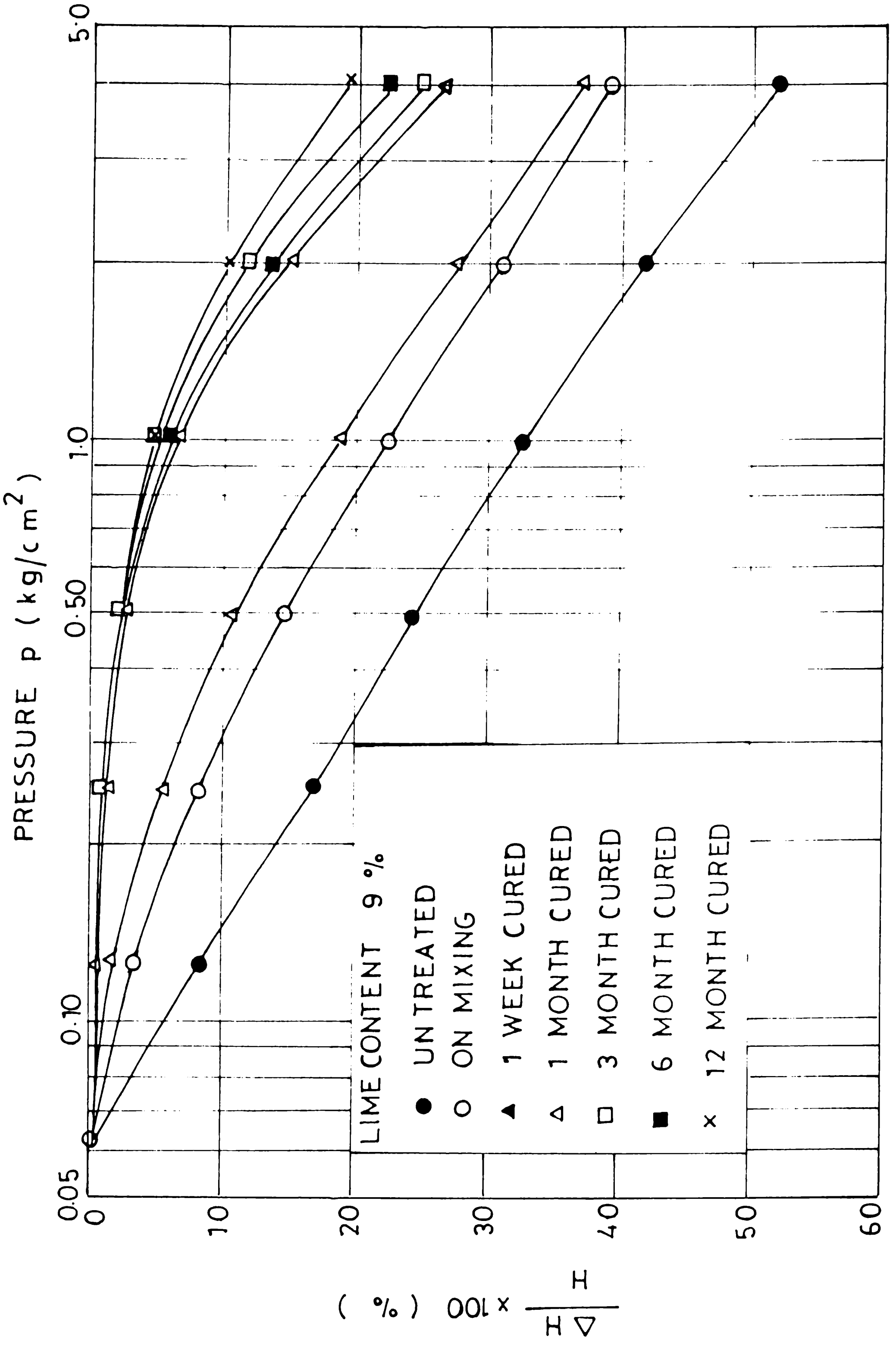

Fig. 6.4.2c : COMPRESSION CURVES.

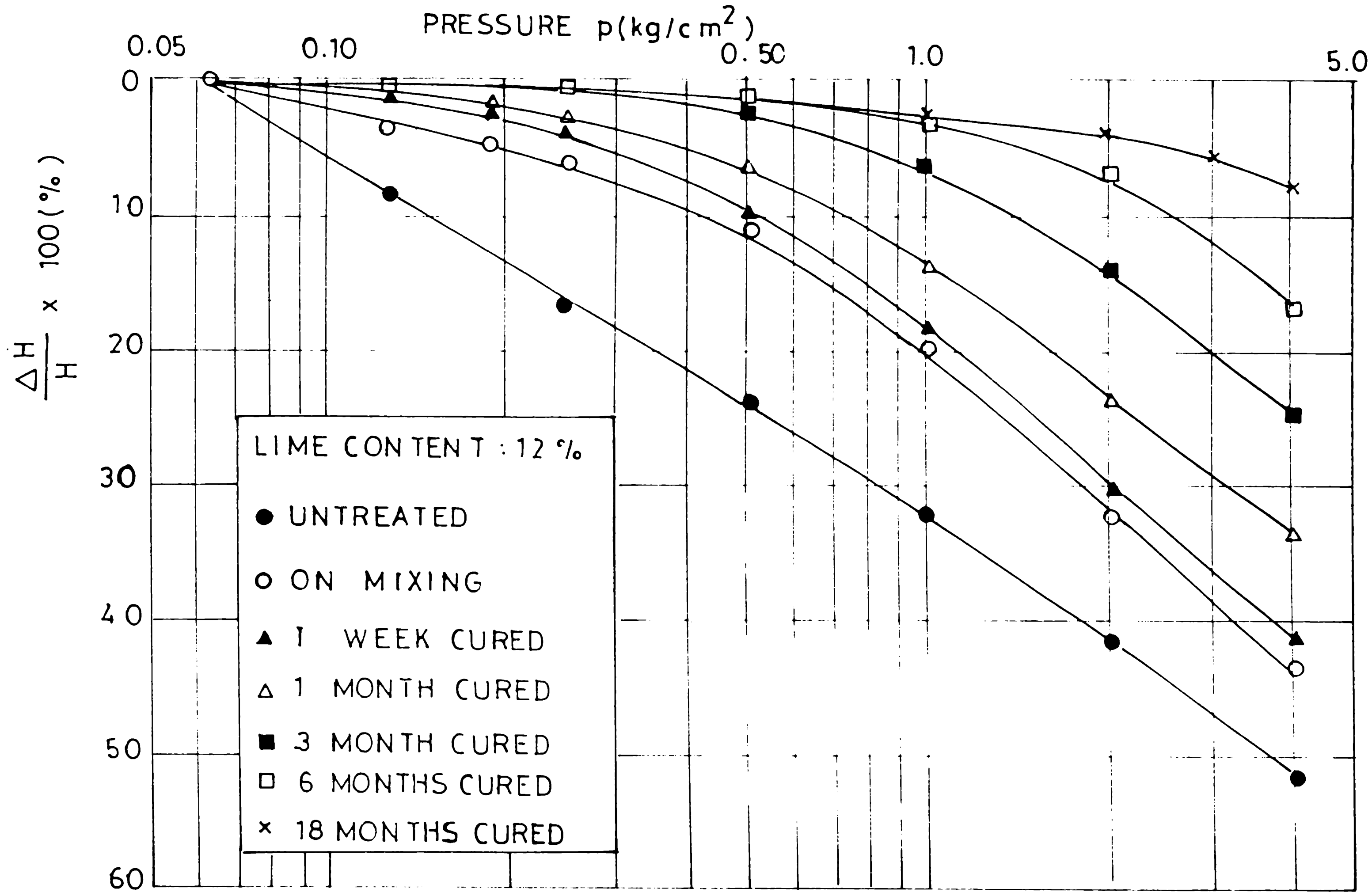

Fig. 6.4.2d : COMPRESSION CURVES.

steady increase in preconsolidation pressure developed with time allowed for curing, as indicated by the shift of the point of maximum curvature to the right.

Figures 6.4.3a, b and c present an interesting behaviour exhibited by treated soils which was noticeable while comparing vane shear strength values also. Fig.6.4.3a shows the compression curves for lime percents of 6, 9 and 12 for a curing period of 1 week. Highest resistance to consolidation is exhibited by 6% curve with 12% curve keeping a low profile. Fig.6.4.3b gives the compression curves for specimens cured for three months where all the three curves show more or less similar consolidation settlements.

Fig.6.4.3c shows the $\frac{\Delta H}{H}$ vs. log p plots for specimens treated with 6, 9 and 12% lime and cured for six months. Here the specimens treated with 12% lime occupies the highest position with 9% and 6% curves falling below in that order. This indicates that the process of stabilisation is faster when the lime content is less. As the lime percent is increased the rate of chemical reaction is much lower and it takes much longer to gain strength. This phenomenon has been observed in comparative studies

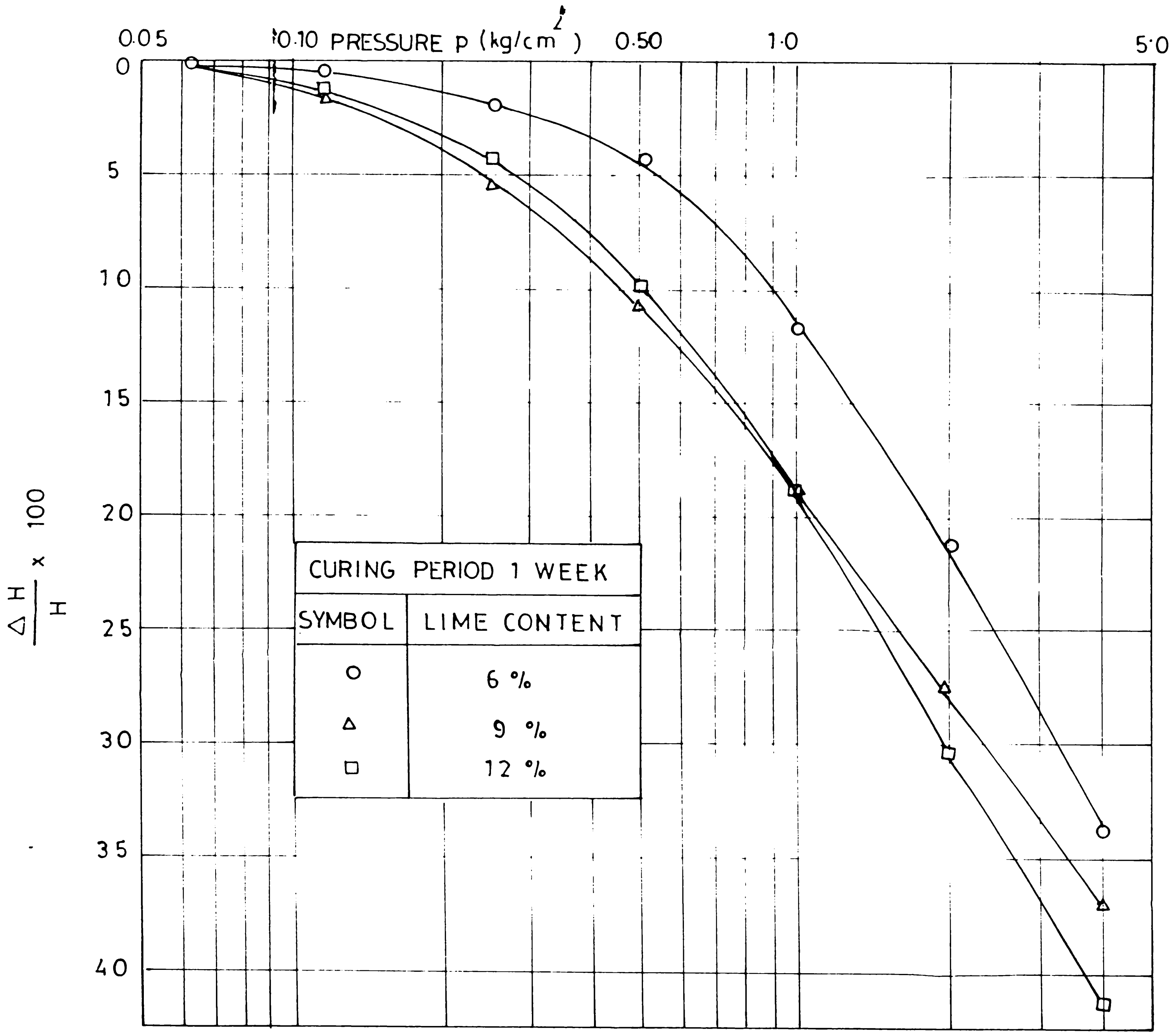

Fig. 6.4.3a : COMPRESSION CURVES.

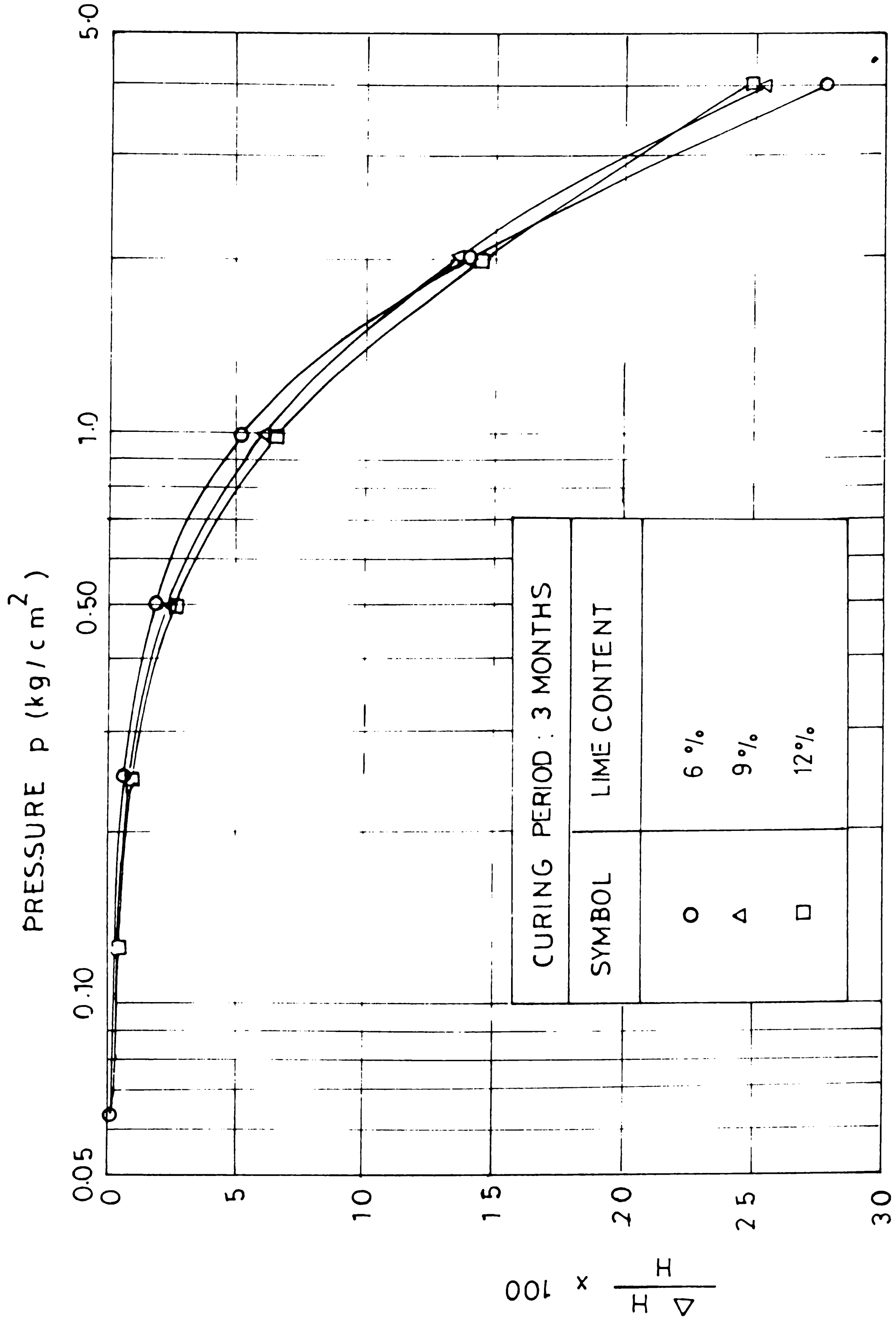

Fig. 6·4·3b : COMPRESSION CURVES.

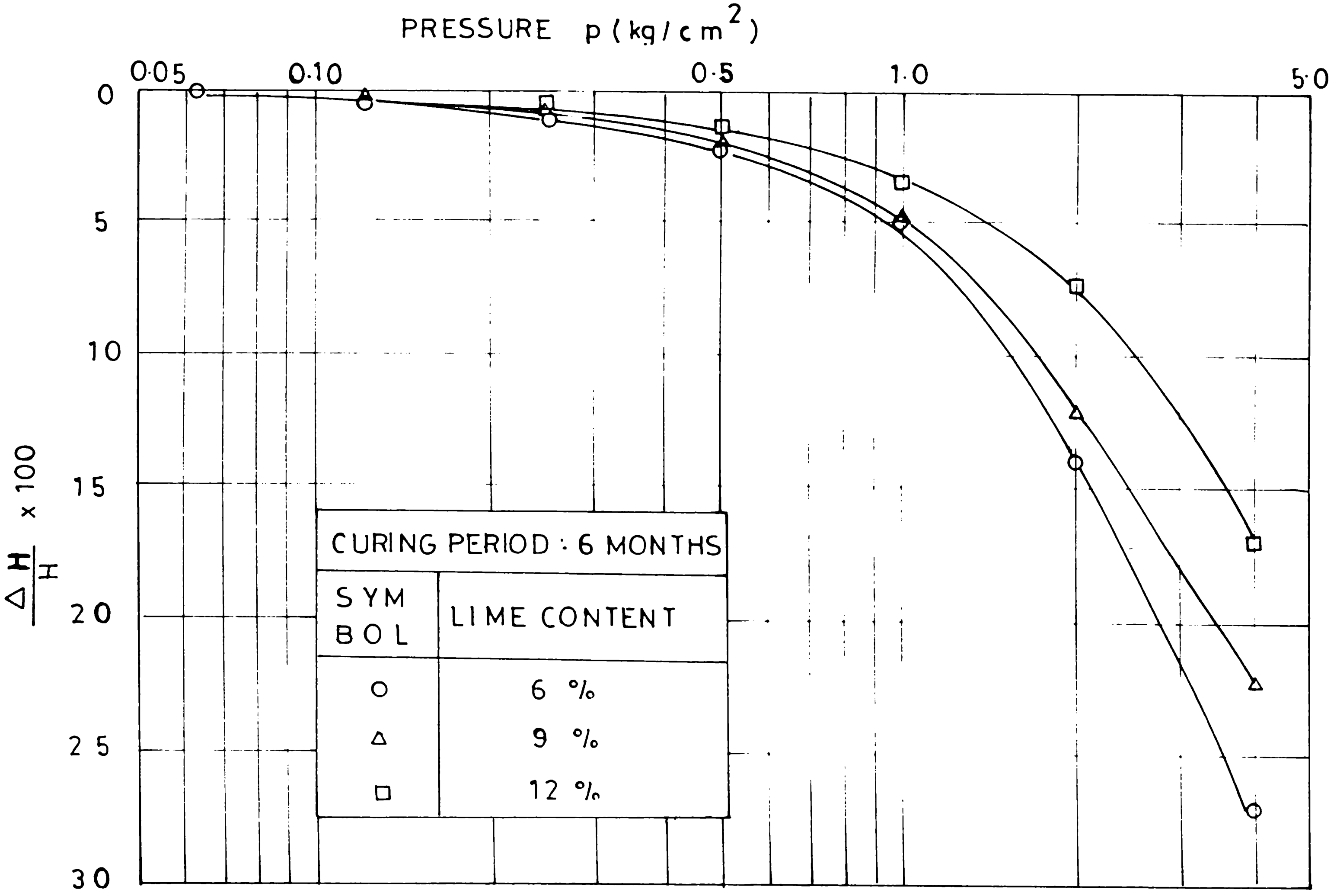

Fig. 6.4.3c COMPRESSION CURVES.

involving varying percentages of lime content and same curing period.

Fig.6.4.4 shows the ratio of deformation of treated samples and deformation of untreated remoulded samples for a lime content of 9%, for samples cured for 1 week, 1, 2 and 3 months. It can be seen that the ratio of ΔH treated/ ΔH untreated is very low for lower consolidation pressures and the ratio increases with intensity of pressure. The change in rate of increase is around 0.5 kg/cm^2 as pointed out earlier. This may correspond to the preconsolidation pressures developed during curing.

6.4.1 Variation of de/d (log p)

Fig.6.4.1.1a shows the values of de/d (log p) for various curing periods for samples treated with 6, 9 and 12% lime for a pressure of 0.0625-0.125 kg/cm^2. The conclusion that lower percentages of lime treatment show a faster rate in strength gain is confirmed by the plots in this figure. The value of de/d (log p) is as low as 0.1 for a specimen with 6% lime without any curing. In the case of the specimen treated with 12% lime the value for de/d (log p) is as high as 0.64. The value for 9%

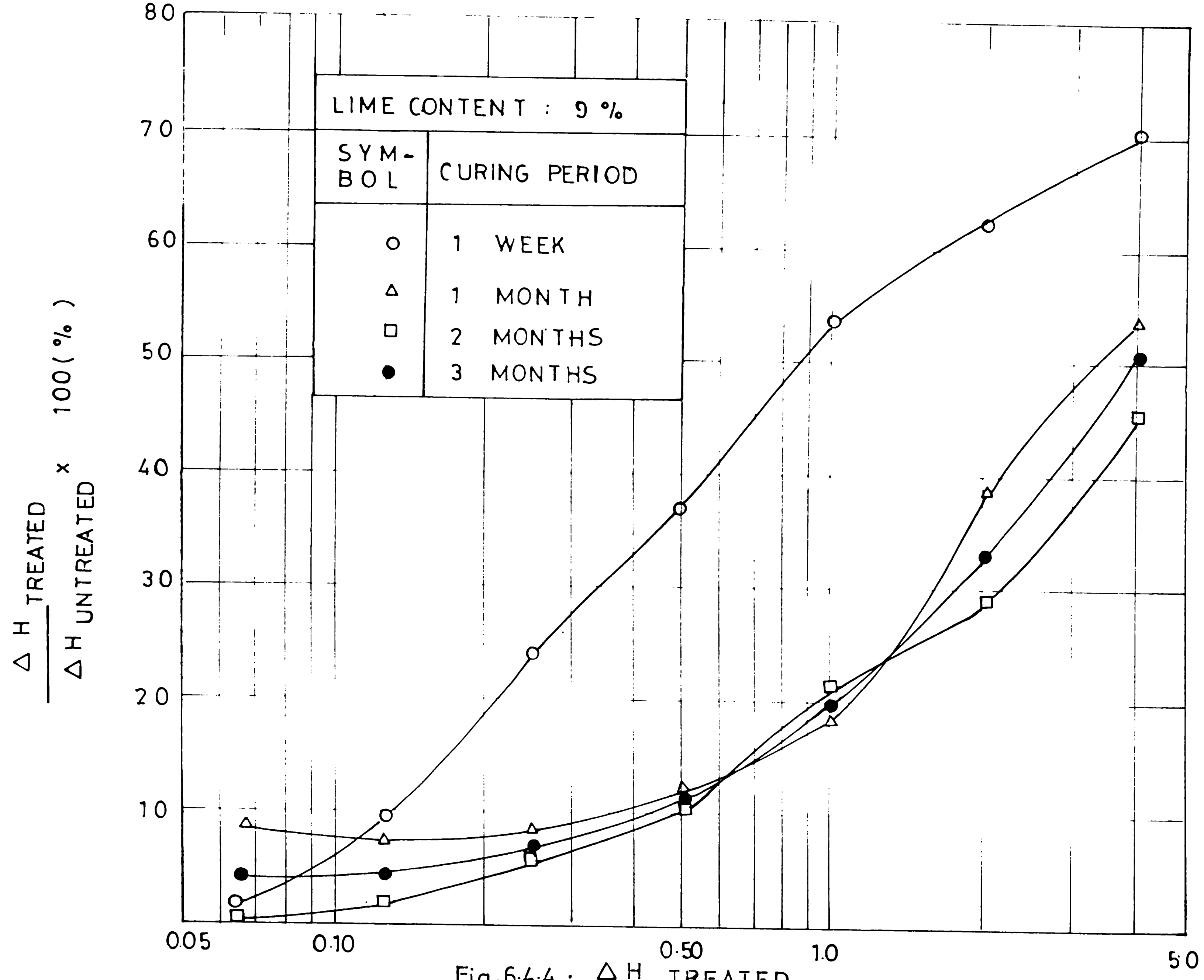

Fig.6.4.4 : $\frac{\Delta H \text{ TREATED}}{\Delta H \text{ UNTREATED}}$ vs PRESSURE .

lime falls in between. The de/d (log p) value for the sample treated with 12% lime slowly reduces for higher curing periods ultimately reaching a steady value below the curve for 9% lime content.

Fig.6.4.1.1b shows the same plots for the pressure increment from 0.25-0.375 kg/cm^2. It can be seen that the curves are closer to each other for curing periods longer than 90 to 100 days. But in Fig.6.4.1.1c the values of de/d (log p) for 6% lime are the highest for the pressure increment of 2.0-4.0 kg/cm^2. The de/d (log p) values for 9% and 12% lime fall below the plot for 6%. The variation in de/d (log p) values with curing period is very marginal compared to the previous two sets of curves.

It has already been pointed out that the compression curves of treated soils indicate the development of a preconsolidation pressure (bond strength) around 0.5 kg/cm^2. The consolidation settlements are very low when the pressure intensities are lower than the preconsolidation pressure developed. The de/d (log p) values are very low for the pressure increments of 0.0625 to 0.125 kg/cm^2 and 0.25 to 0.375 kg/cm^2, as shown in Figs. 6.4.1.1a and b. The same figures show very high values

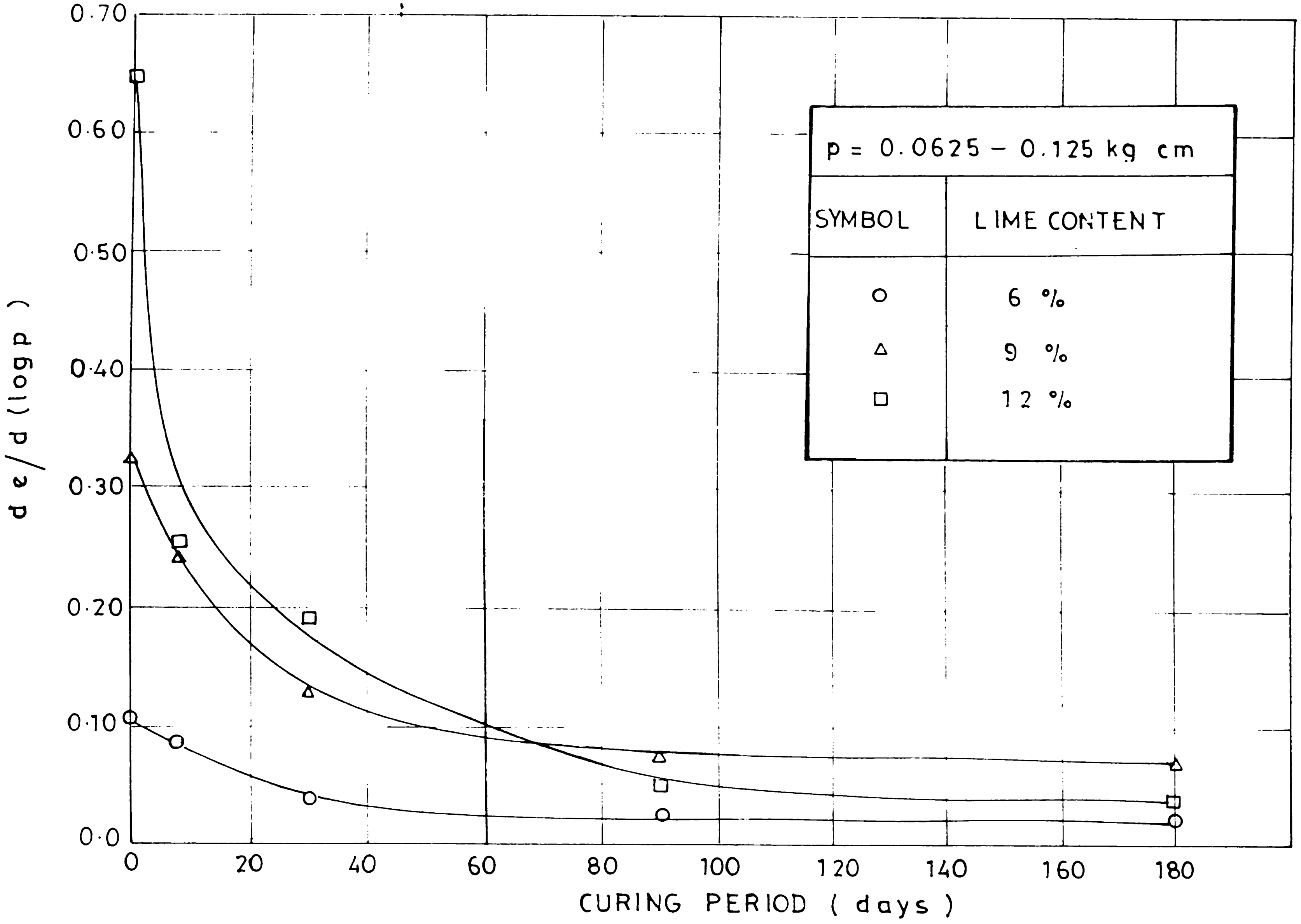

Fig. 6·4·1·1a de/d(log p) vs CURING PERIOD

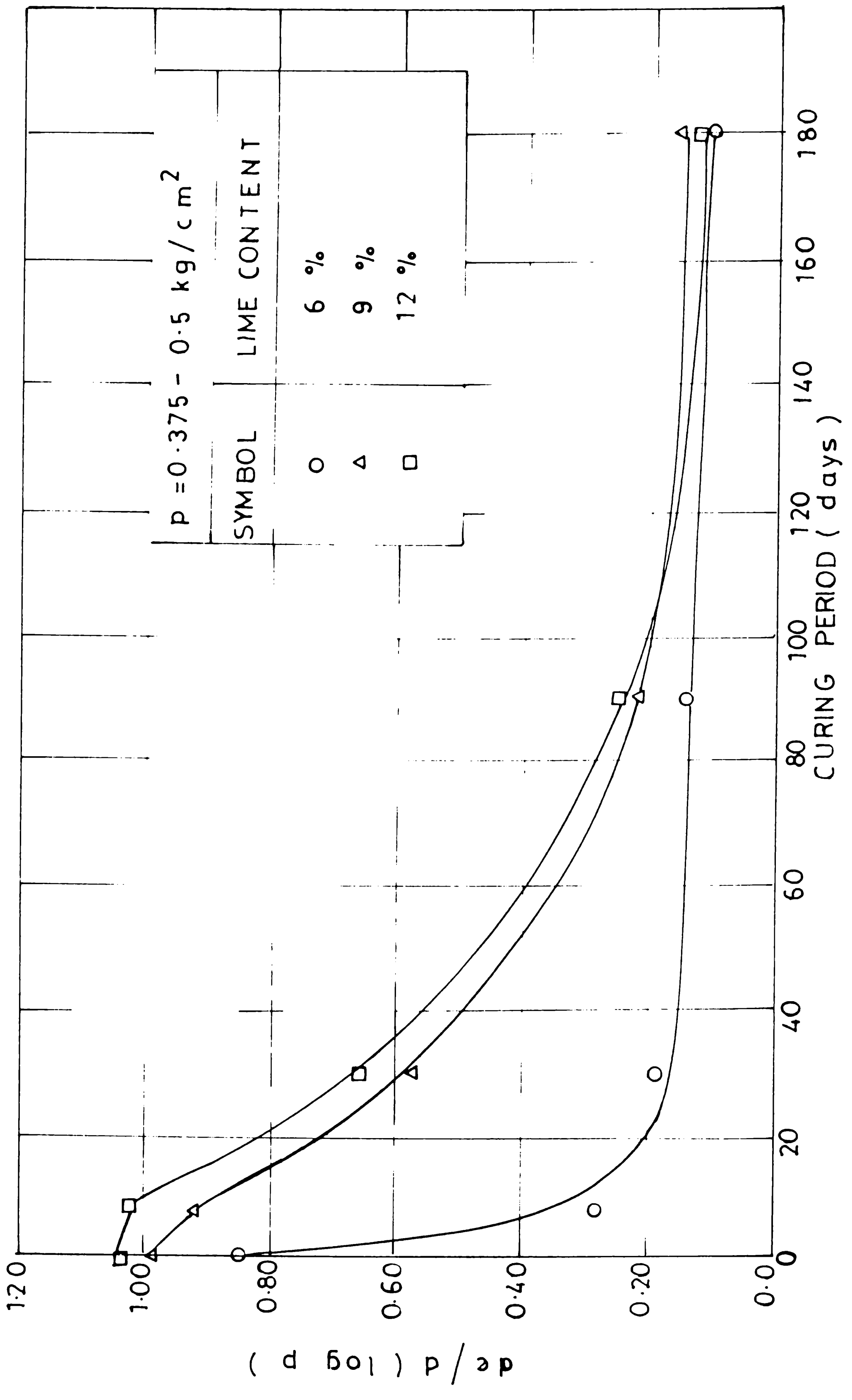

Fig. 6.4.1.1b dε/d(log p) vs CURING PERIOD.

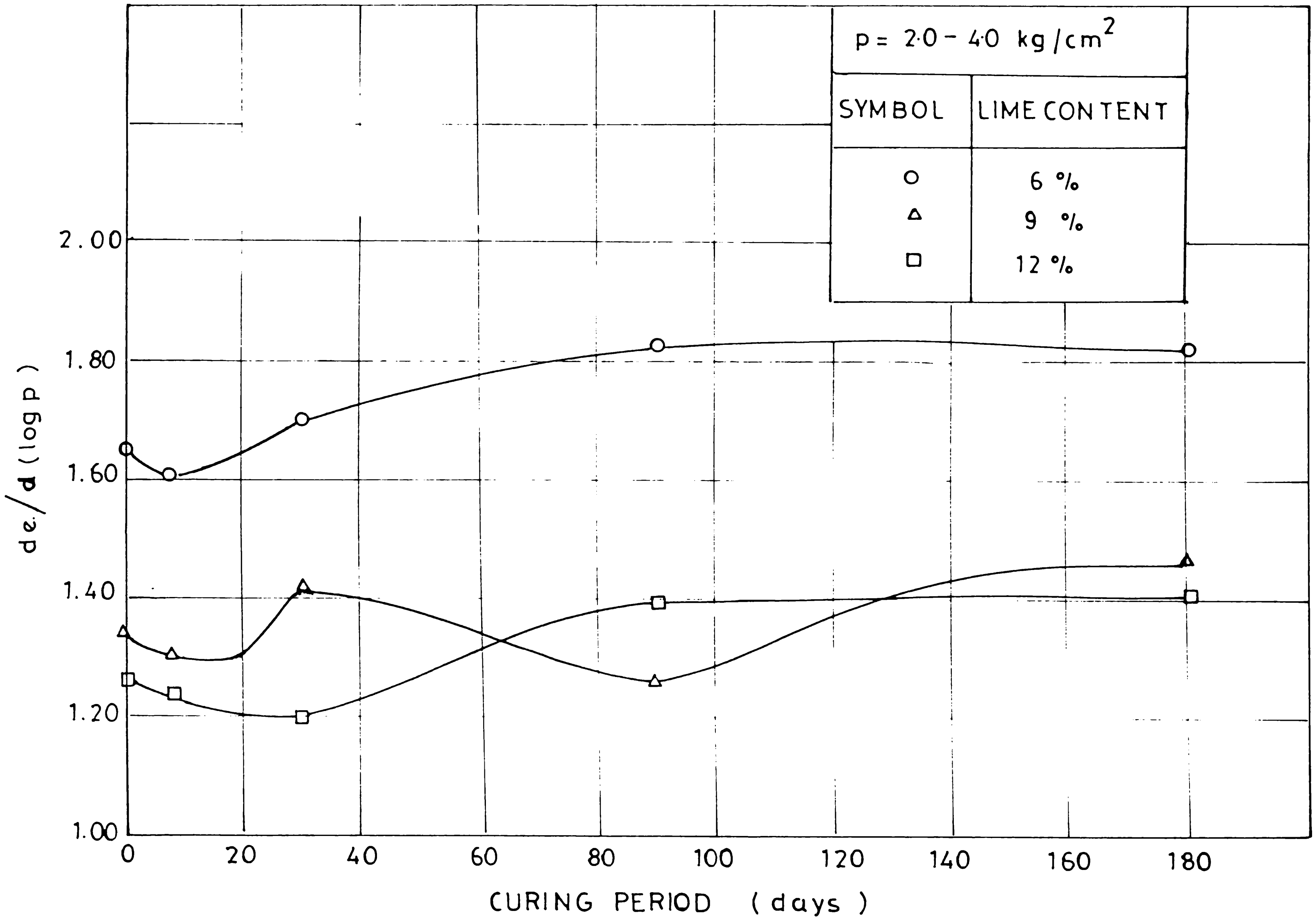

Fig. 6·4·1·1c de/d (log p) vs CURING PERIOD.

for de/d (log p) when no curing is allowed for the samples. The de/d (log p) values are high in such cases as there was no time for the bond strength to build up. Once p_c values are built up with curing period, the values of de/d (log p) drop down to steady low values. But once the pressure increment is higher than the maximum p_c that can be developed for the given lime content and period of curing allowed, de/d (log p) shows a regular steady value irrespective of the curing period as shown by Fig. 6.4.1.1c. Thus an accurate assessment of preconsolidation pressure that can develop for a given lime content and curing period is essential before designs involving stabilised soils, are taken up for field applications.

Fig.6.4.1.2a shows the plot between de/d (log p) and curing period for marine clays stabilised with 6% lime for the pressure range of 0.0625-0.125 kg/cm^2, 0.25-0.375 kg/cm^2 and 2.0-4.0 kg/cm^2. Fig.6.4.1.2b gives the same relations for 12% lime. Both these figures are in agreement with the findings from Figs.6.4.1.1a, b and c.

The variation of de/d (log p) with the percentage of lime added to the soil is given in Fig.6.4.1.3 for three pressure increments. The curing period allowed

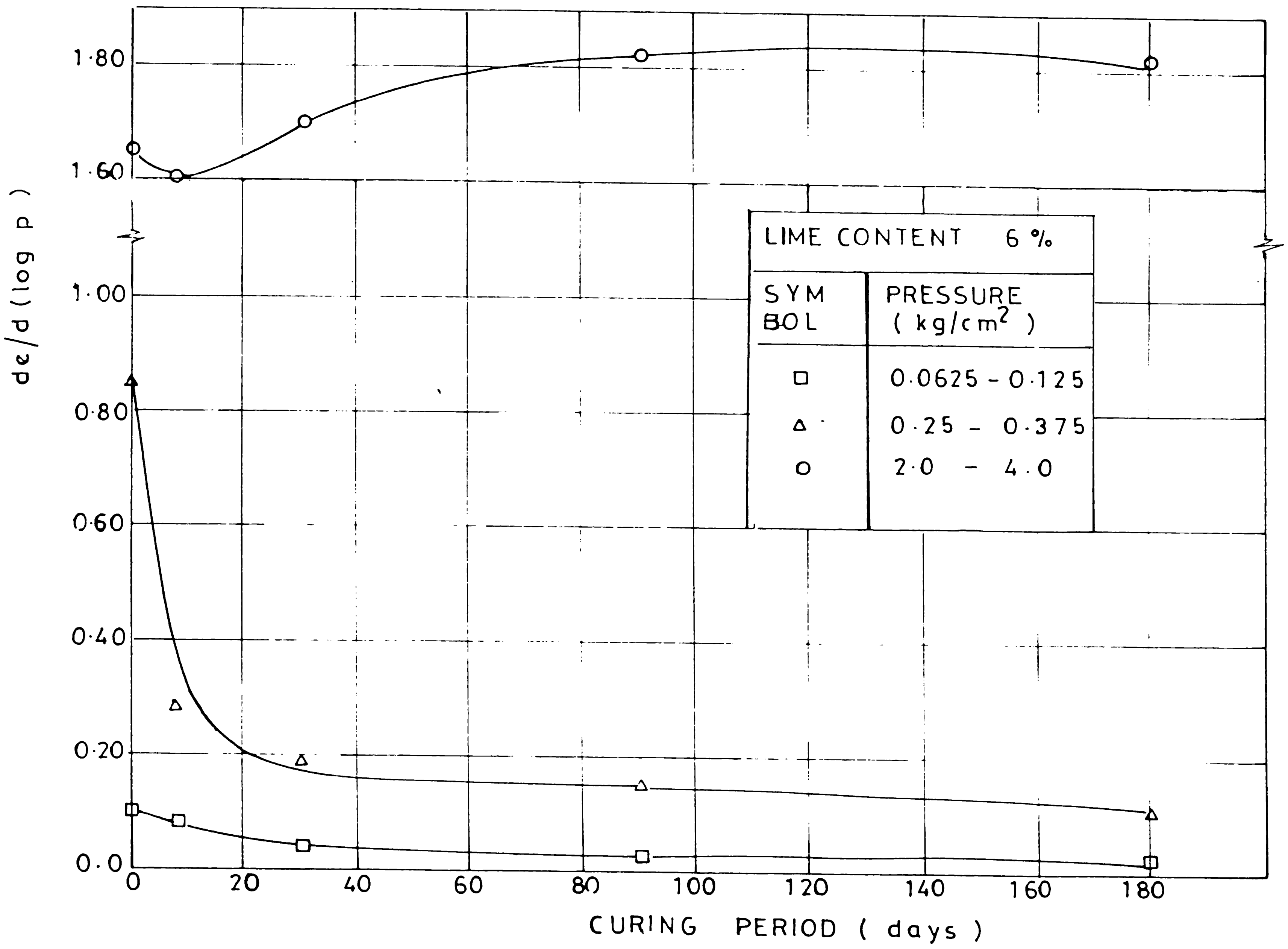

Fig. 6·4·1·2a d e/d(log p) vs CURING PERIOD .

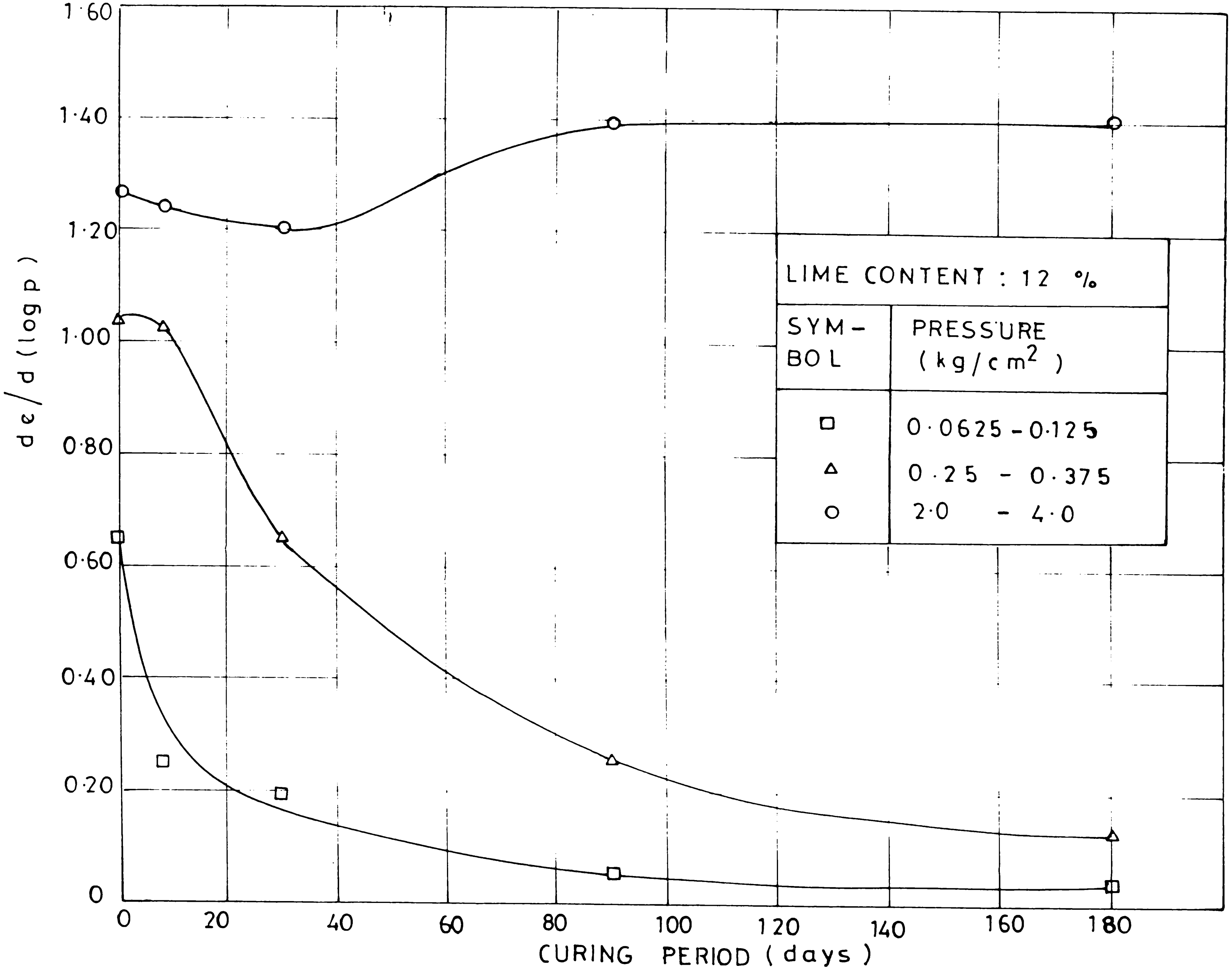

Fig. 6·4·1·2b d e/d (log p) vs CURING PERIOD·

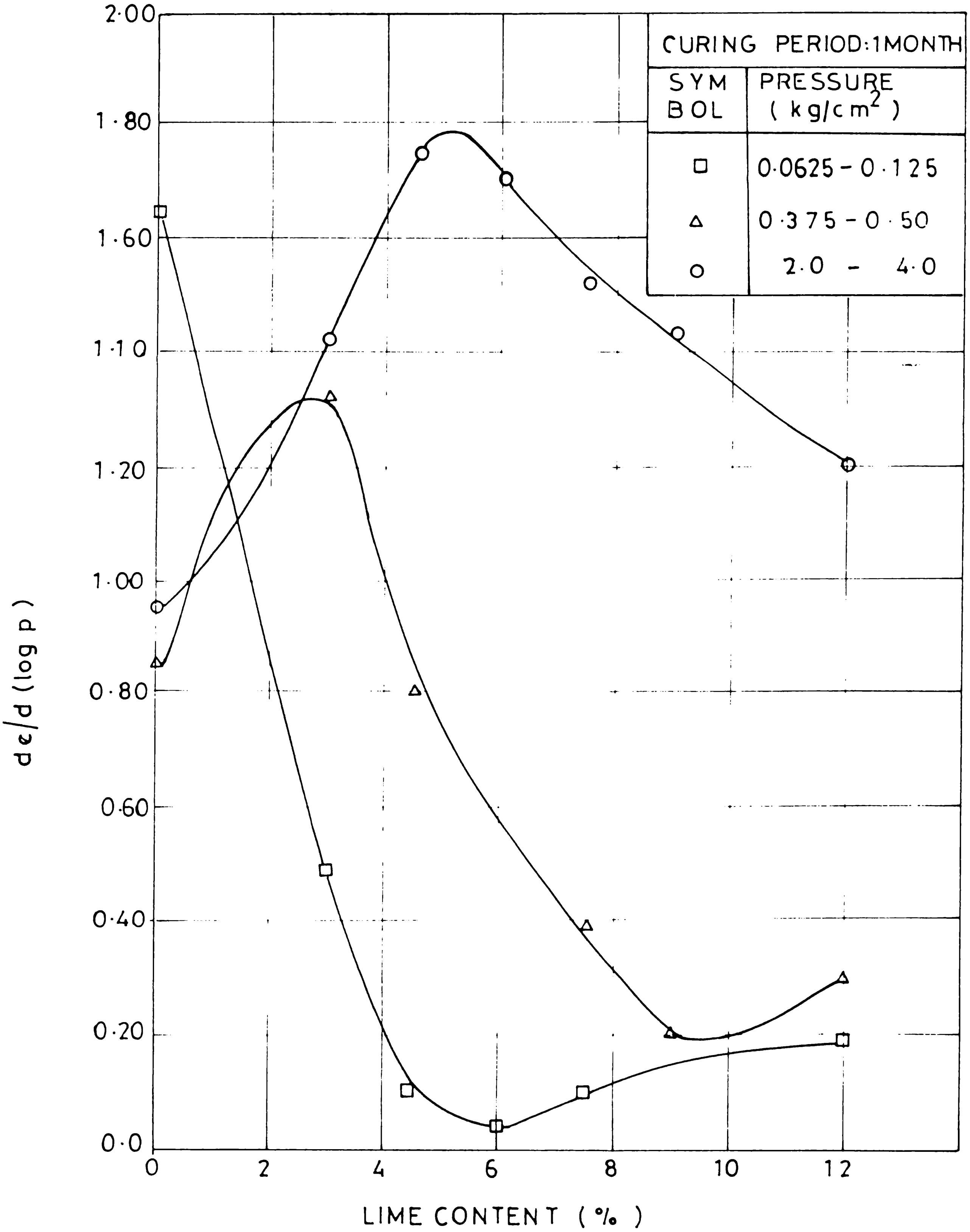

Fig. 6.4.1.3 : de/d (log p) vs LIME CONTENT.

was one month. These curves give some useful information regarding the most ideal lime content. For a pressure increment of 0.0625-0.125 kg/cm^2 for 3% lime the value of de/d (log p) is about 0.49. When the lime content is increased, the value of de/d (log p) decreases and reaches a minimum value at 6% lime content. For higher lime contents the value for one month curing increases. Since de/d (log p) is directly proportional to consolidation settlement, the consolidation settlement of treated clay layer which can be given one month for curing will be least for a lime content of around 6%. Lower lime content will obviously give a higher value of de/d (log p). Lime contents higher than 6% can give a lower value for de/d (log p) provided a longer duration can be permitted for curing.

Similarly for a pressure range of 0.375 to 0.5 kg/cm^2 the lowest value of de/d (log p) is given by a lime percent of around 9 for a curing period of one month. Since the lowest value of de/d (log p) has a bearing on the preconsolidation pressure developed through stabilisation, a higher lime content is required to develop the higher p_c within a month to meet the higher pressure.

This shows that studies of this nature will help to optimise the lime content for parameters such as pressure increment and period of curing.

The relation between de/d(log p) and consolidation pressure are shown in Fig.6.4.1.4a for a curing period of one week. The values increase with pressure and then start dropping down. The higher lime content gives higher values of de/d(log p). The reasons for this have already been discussed. Since the rate of development of shear strength is inversely proportional to the lime content, the sample with a lower lime content will show higher strength or lower de/d(log p) for short curing periods. Here the curing period was only one week.

How the de/d(log p) curves will change the trend is shown by Fig.6.4.1.4b where the samples have been allowed to cure for 6 months. Here the sample with 12% shows the highest strength or lowest values for de/d(log p). The figure also indicates that the difference in resistance to deformation between 6% and 9% is much higher than the difference between 9% and 12%.

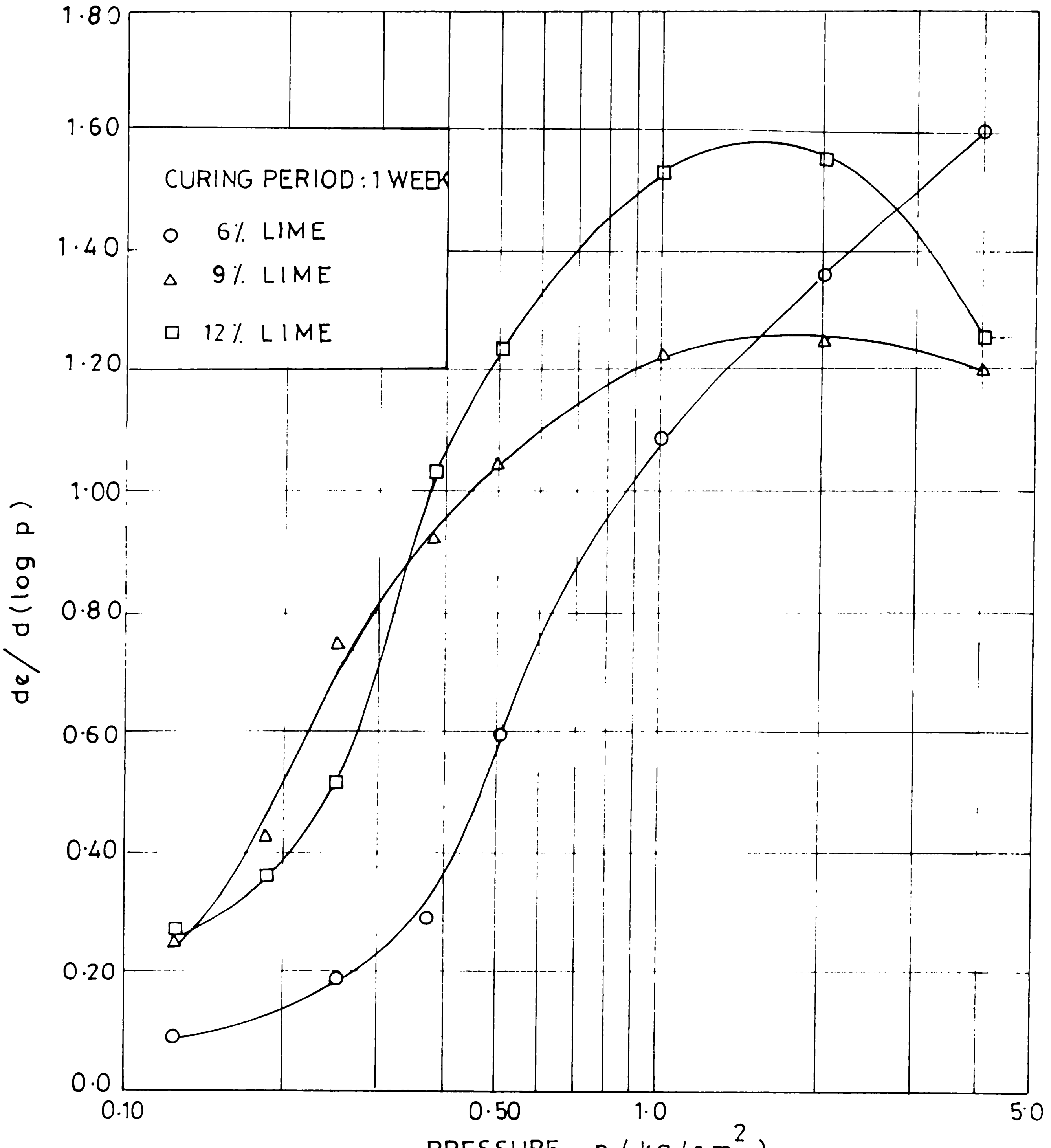

Fig. 6·4·1·4a de/d(log p) vs PRESSURE.

The variation of de/d(log p) with pressure is presented in Fig.6.4.1.5 for five samples which have been cured for 1 week, 1 month, 3 months and 6 months. The plot for a sample for which no curing was given is also given in the figure. A comparative study of the curves in the higher pressure region, show that the value of de/d(log p) is least for the sample for which no time was given for curing. The values of the sample cured for 1 week is the next lowest. The curves for the samples cured for 1, 2 and 3 months are very close to each other. Here a lower value for de/d(log p) does not show higher strength. For a sample which has not been permitted to cure, no preconsolidation pressure has developed. Hence the e - log p curve will not show any sharp variation in slope.

In Fig.6.4.2c the curve for remoulded sample shows a very uniform slope throughout. Hence its de/d(log p) is the lowest. The sample treated with lime and for which no curing was allowed, has the next least compression index as well as de/d(log p). The next sample has a slightly higher value for de/d(log p). Hence for a treated sample which has developed some bond strength will show

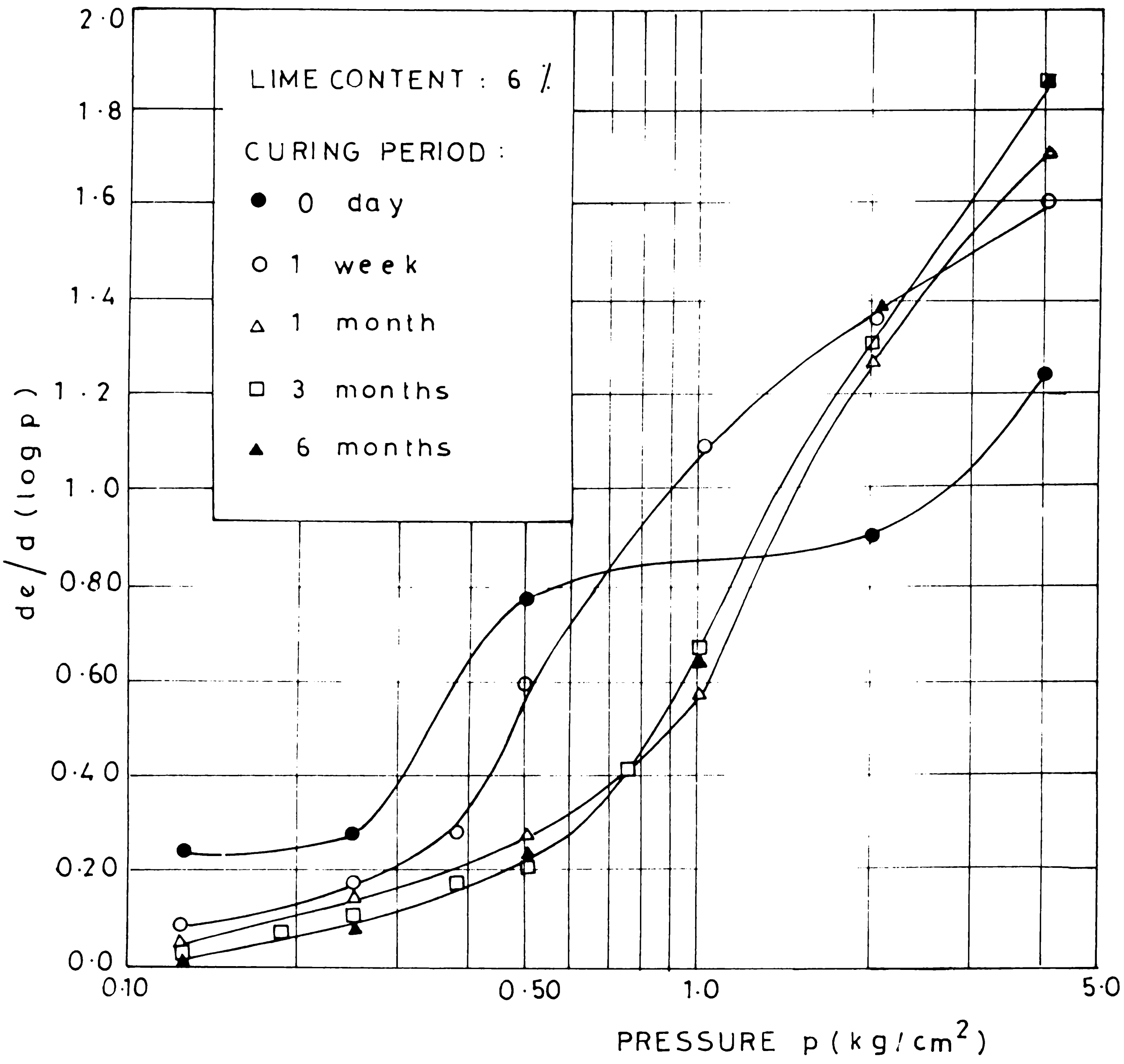

Fig. 6·4·1·5 de/d(log p) vs PRESSURE·

low values of de/d(log p) at lower pressures and high values at higher pressures.

6.4.2 C_v and $C_{\alpha\epsilon}$ for lime treated clays

As lime stabilisation causes flocculation or aggregation of finer grains, the permeability increases with curing. This has been shown by the curves in Fig. 6.4.2.1 where C_v has been plotted against curing period for 6, 9 and 12% of lime. As expected C_v has been found to increase with time.

Fig.6.4.2.2 shows the variation in C_v with increase in lime content. The value increases upto about 8% and decreases. A possible explanation for this can be the clogging of voids by the products of stabilisation (Brandl, 1981). By lime stabilisation the mineral particles are cemented within the soil-lime mixture. The fine skeleton is embedded partly within the gelatinous intermediate mass. Hardening products of the binder grow into the voids of the soil aggregates, changing the void structure. This can bring down the value of coefficient of consolidation or permeability.

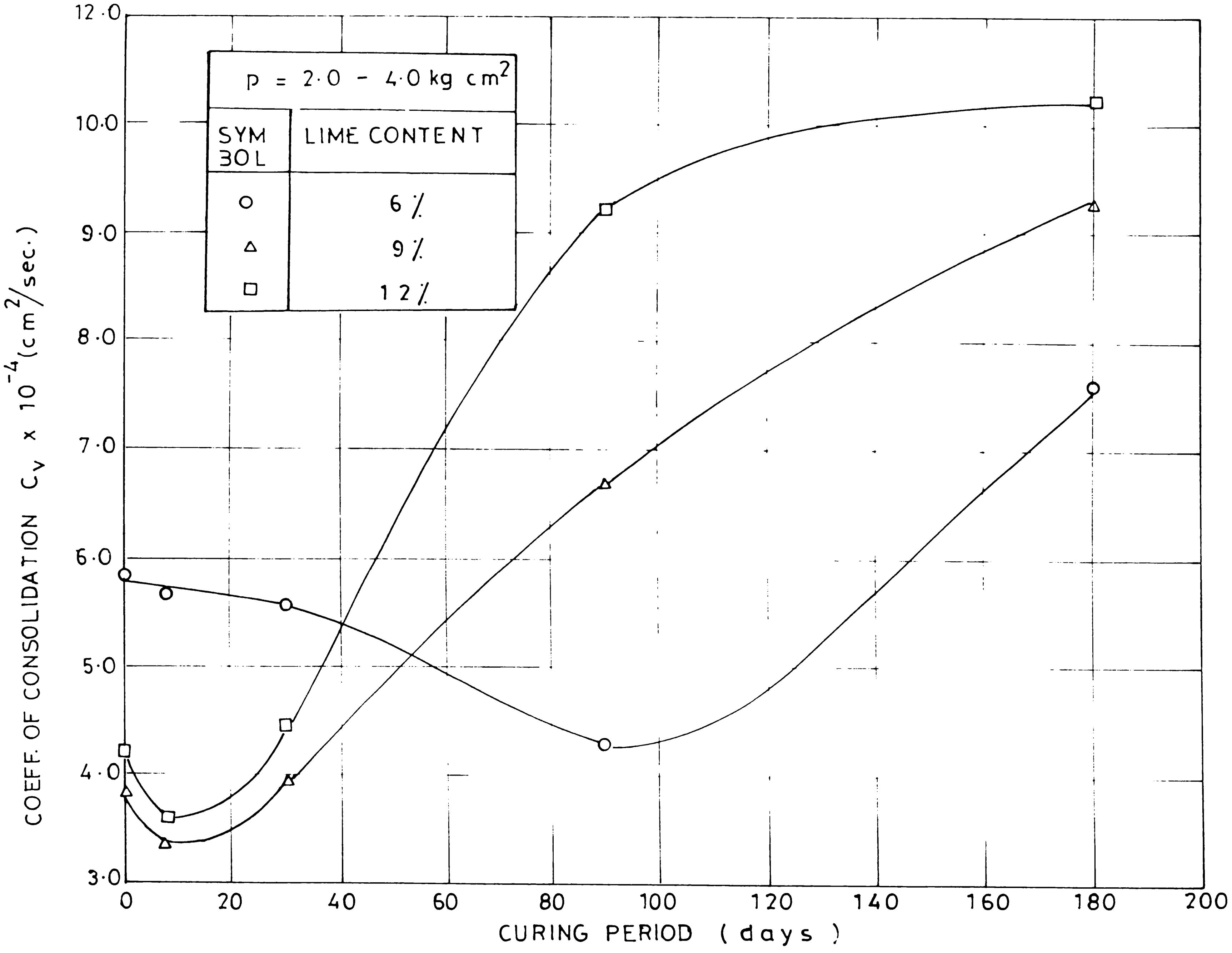

Fig. 6·4·2·1 C_v vs CURING PERIOD.

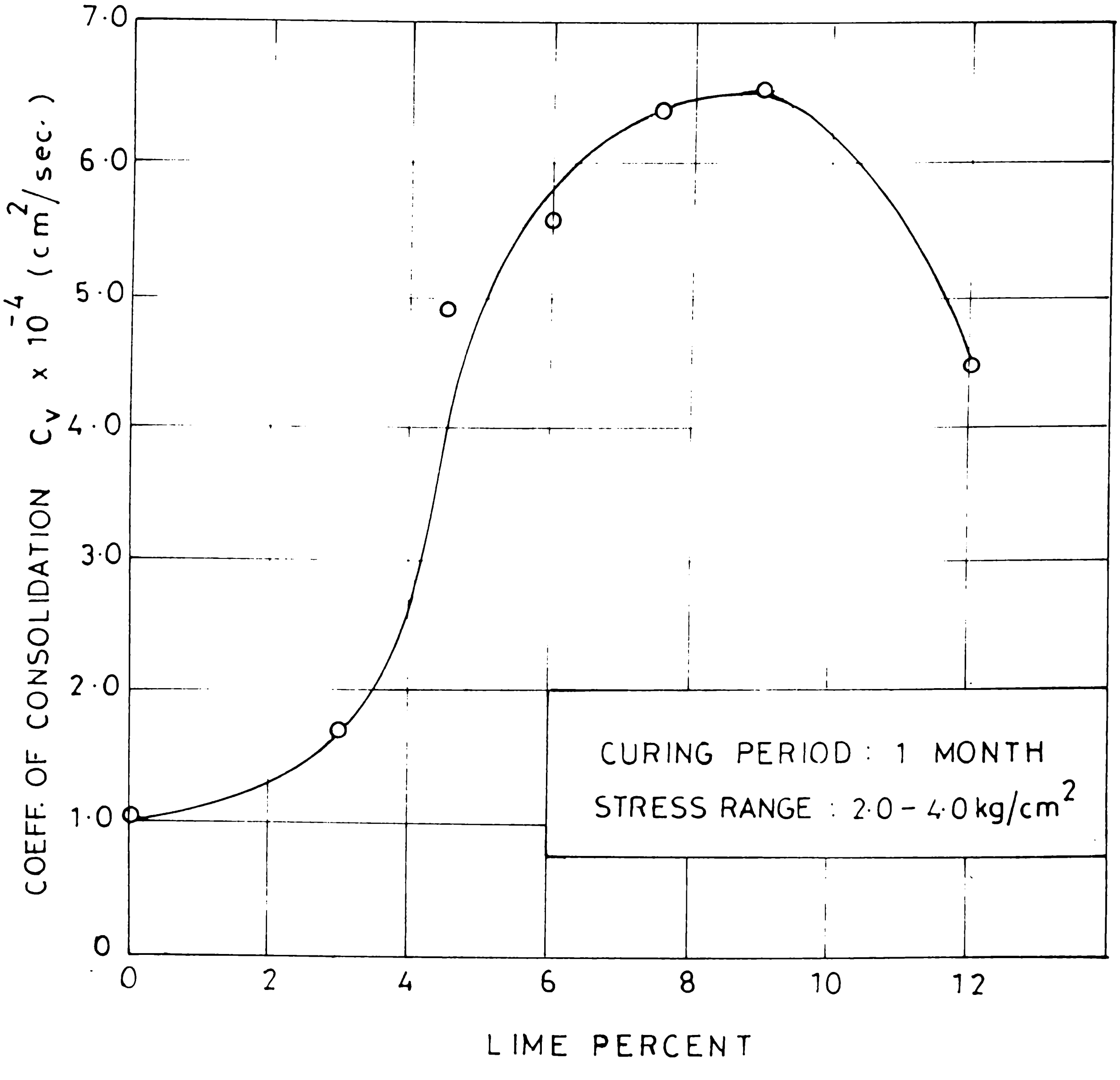

Fig. 6·4·2·2 C_V vs LIME PERCENT.

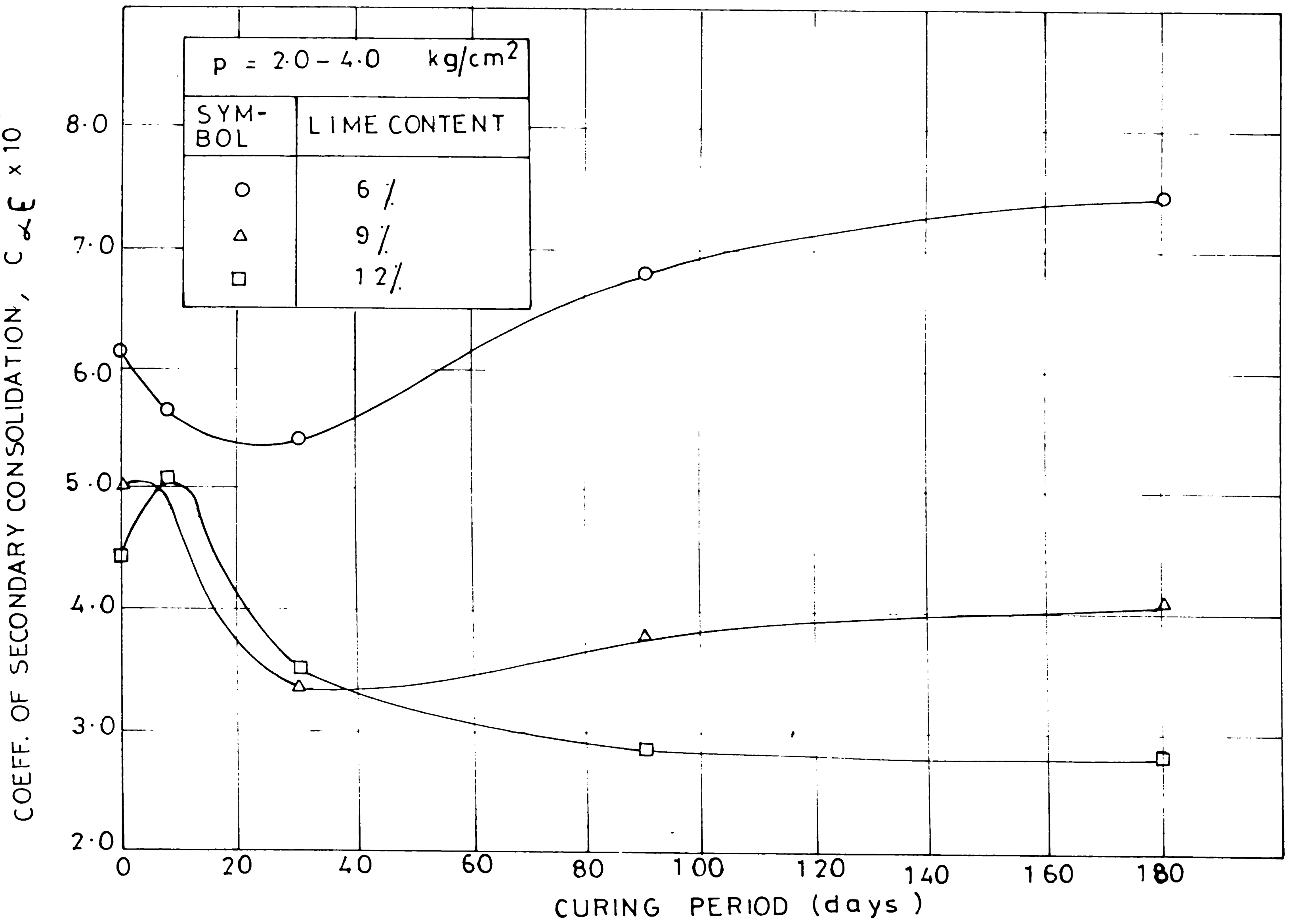

Fig. 6·4·2·3 $C_{\alpha\epsilon}$ vs CURING PERIOD.

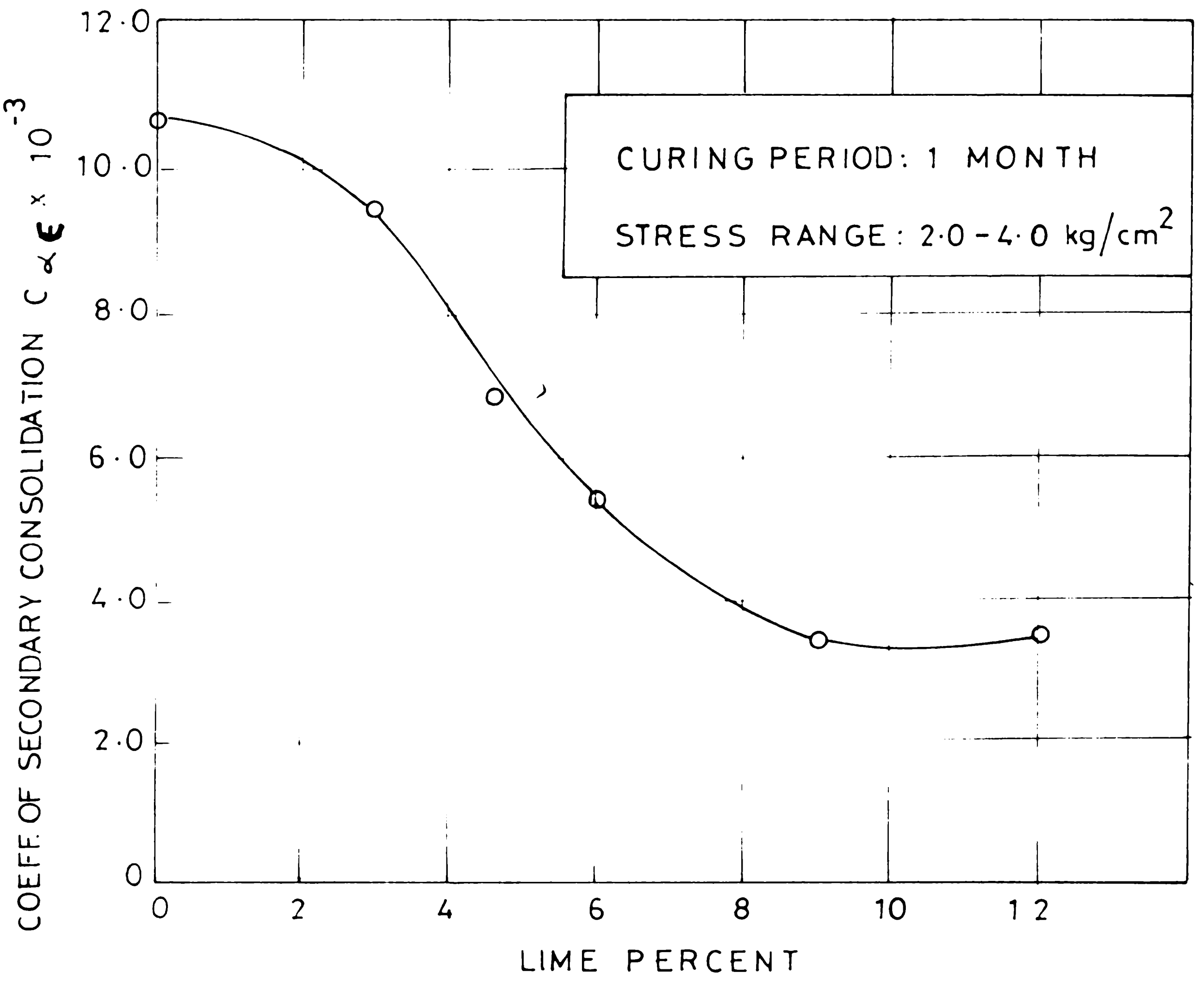

Fig. 6.4.2.4 $C_{\alpha\epsilon}$ vs LIME PERCENT

As the newly formed chemical bonds between the grains become harder and harder, the coefficient of secondary consolidation should increase. Hence $C_{\alpha\epsilon}$ should increase with curing periods and the consolidation test results also show the same trend as in Fig.6.4.2.3. Fig. 6.4.2.4 shows the variation of $C_{\alpha\epsilon}$ with lime percent for a curing period of one month. $C_{\alpha\epsilon}$ reduces with lime content.

6.4.3 Development of bond strength

Fig.6.4.3.1 presents an interesting feature of stabilising process when lime is added to Cochin marine clays. It shows the plot between preconsolidation pressure or the bond strength developed and lime content for four curing periods, viz. 1 week, 1, 3 and 6 months. The curve for 1 week duration shows that in case of samples cured for 1 week, the highest strength, 0.76 kg/cm^2, is given by 6% lime. A lime content above this shows a reduced strength. A higher lime content will certainly give a strength higher than 0.76 kg/cm^2. But it has to be given a longer duration for curing. It shows that if the curing period that can be allowed before a stabilised soil is used for an engineering application, is only one week, the optimum lime content which gives maximum strength is 6%. Similarly the optimum lime contents for 1 and

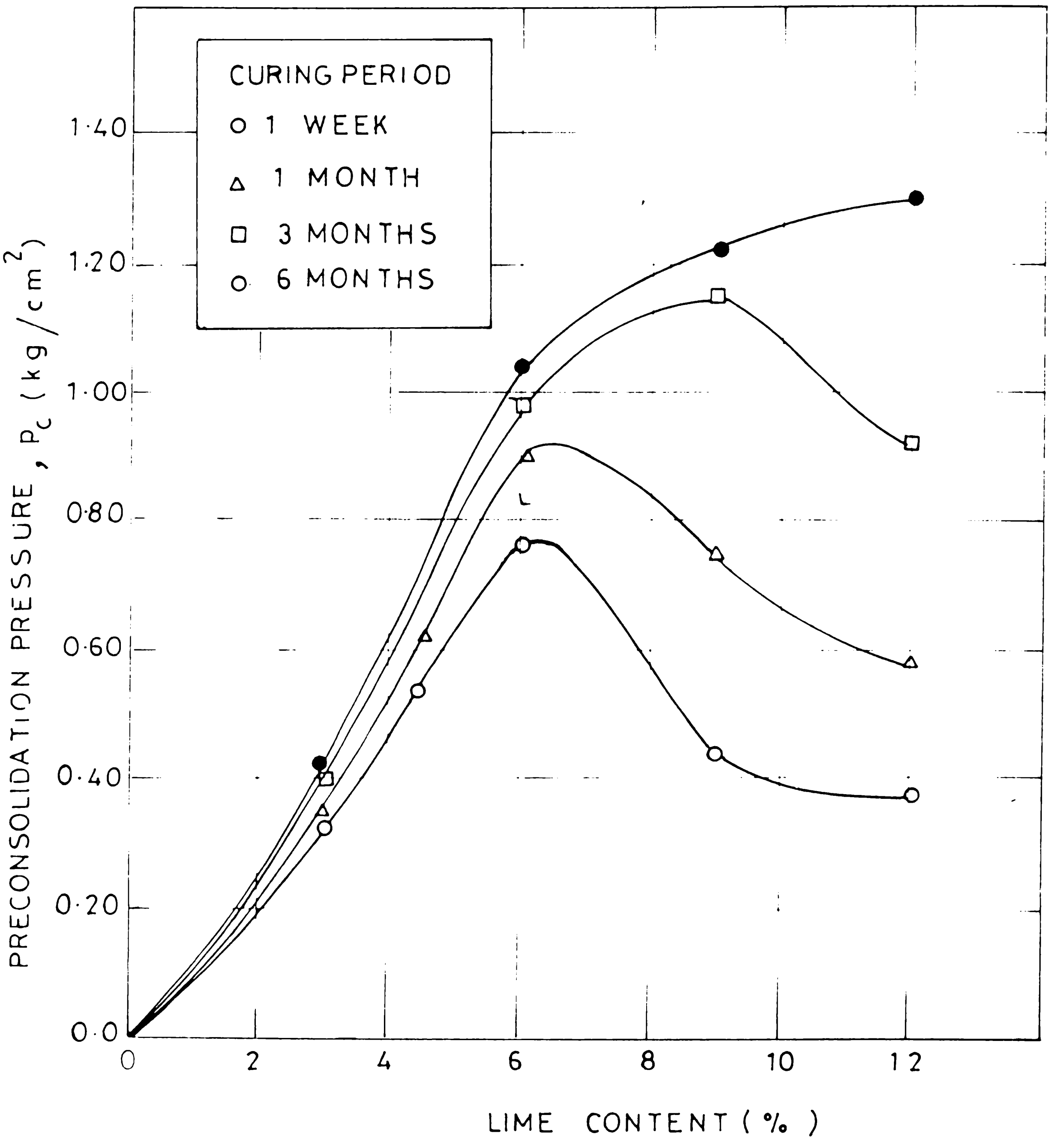

Fig. 6·4·3·1 P_c vs LIME CONTENT.

3 months of curing are 7% and 9% respectively. In case of samples cured for six months the optimum lime content is more than 12%. This shows that depending upon the time available, one can design a stabilised soil which will give the highest shear strength with least lime content for a given curing period. These curves are analogous to compaction curves where optimum lime content, preconsolidation pressure and curing period have respective analogies in optimum moisture content, maximum dry density and compactive effort.

The development of preconsolidation pressure with respect to curing period for lime contents of 6%, 9% and 12% are shown in Fig.6.4.3.2. It can be seen from the graph that 6% picks up a strength of 0.76 kg/cm^2 on the 7th day whereas the strengths developed by 9% and 12% of lime are 0.44 kg/cm^2, 0.36 kg/cm^2 respectively. After six months of curing 12% occupies the highest position and the other lime contents follow in that order. This again proves that lower lime contents gain strength faster than higher percentages of lime.

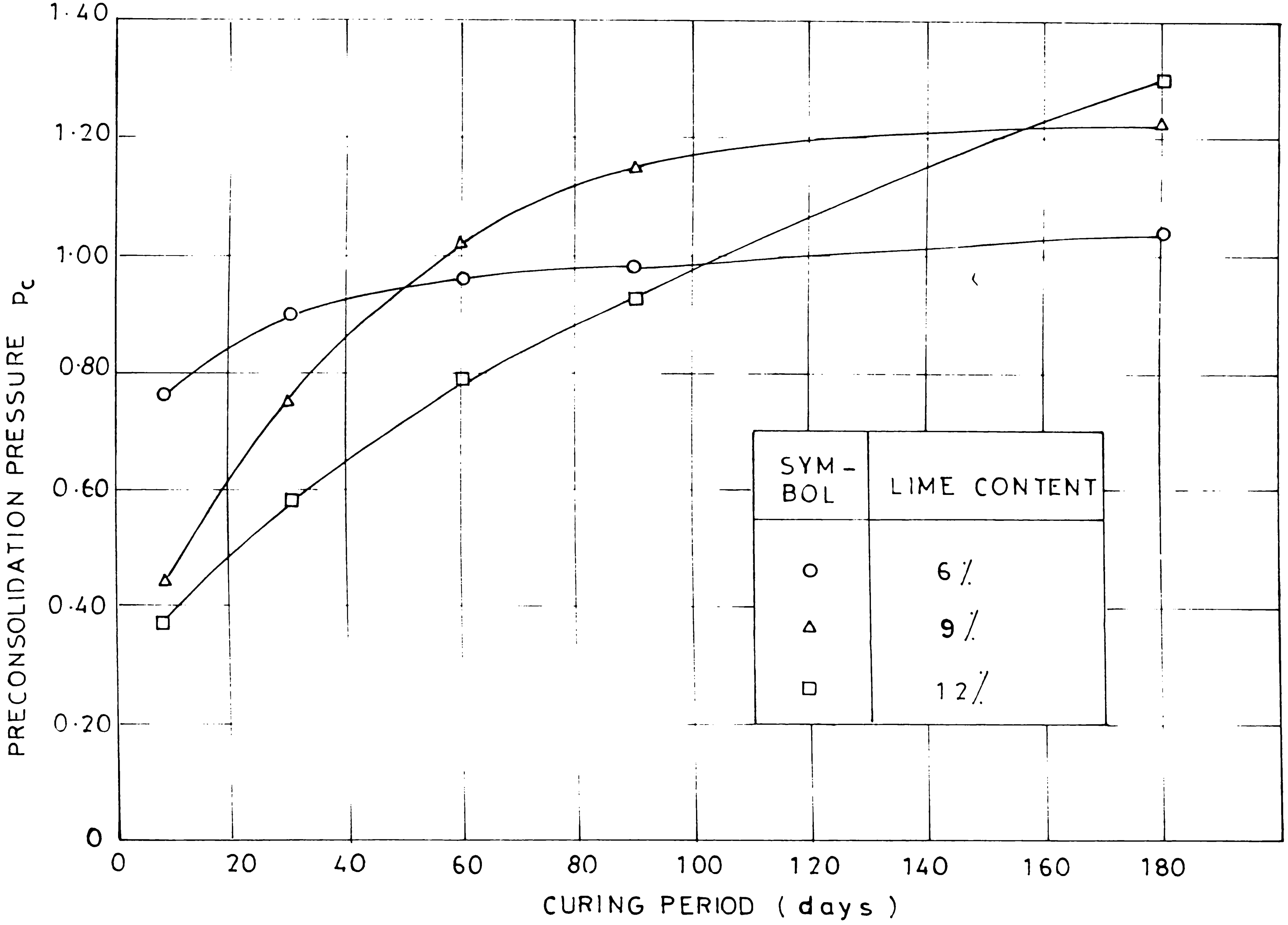

Fig. 6·4·3·2 P_c vs CURING PERIOD.

6.4.4 Duration of load increment

Fig.6.4.4.1a, b and c show the compression vs. log p curves for 2%, 4% and 6% lime. The first pair of curves show $\frac{\Delta H}{H}$ - log p relation for two lime-treated samples with no curing for two loading durations of 2 days and 8 days. The second figure shows similar curves for 4% lime. Both show that the curve for 8 days duration has undergone greater compression than that for 2 days duration. This is contrary to the results of consolidation tests on untreated samples of Cochin marine clays where the strength improved with duration especially at higher stress levels. In case of lime treated marine clays such development of strength is not available. The strength gain is due to bonds developed between grains due to physico-chemical action of lime, which is unaffected by loading stage or loading durations. Fig.6.4.4.1c shows the $\frac{\Delta H}{H}$ vs. log p curves for two samples with 6% lime content cured for 1 month. Eventhough the loading durations were different, the curves are very close to each other indicating that duration of loading has little effect on strength gain.

6.4.5 Comparison with other soils

For a comparative study of the improvement in compressibility characteristics between Cochin marine

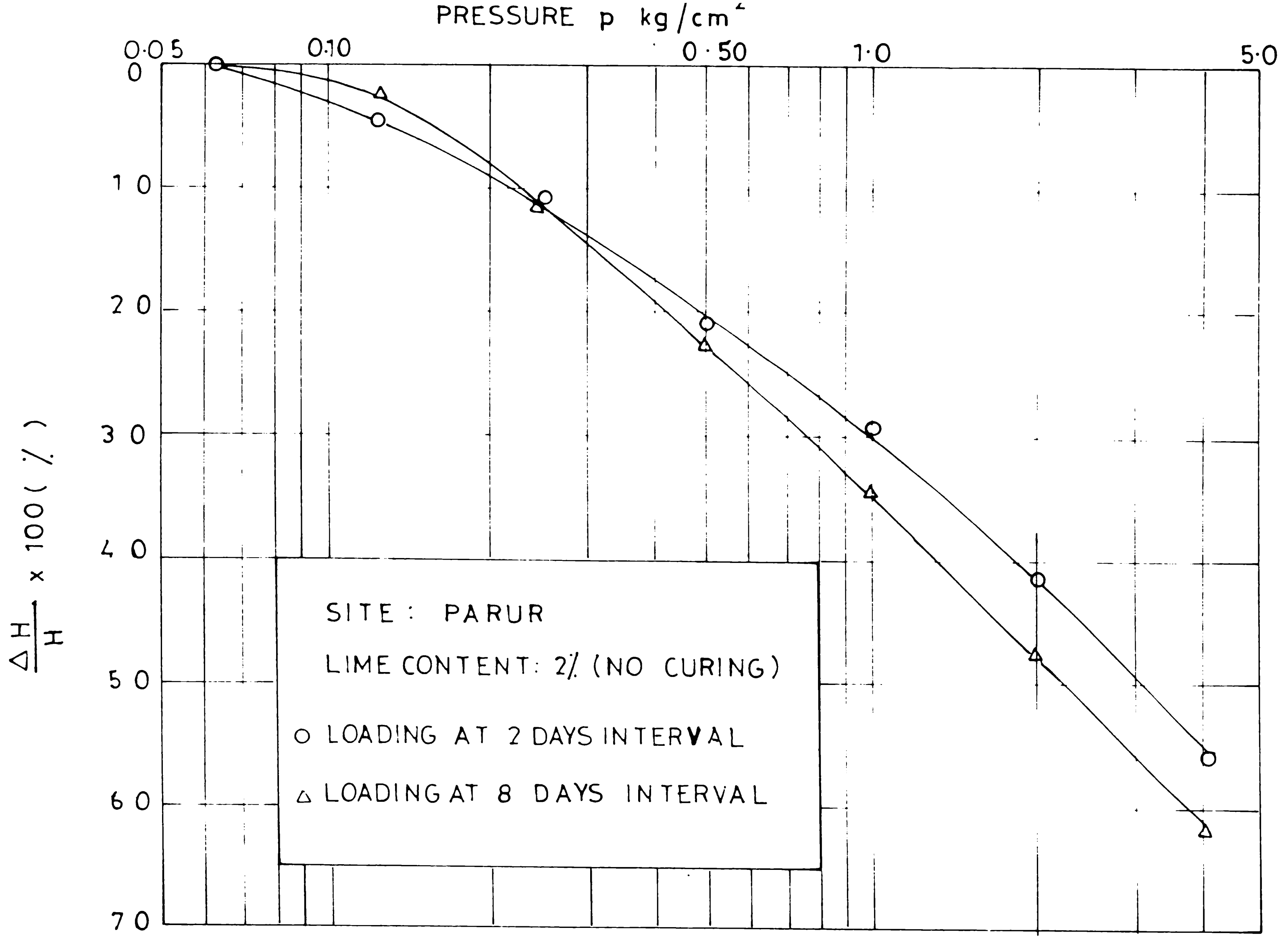

Fig. 6·4·4·1a $\frac{\Delta H}{H}$ vs PRESSURE·

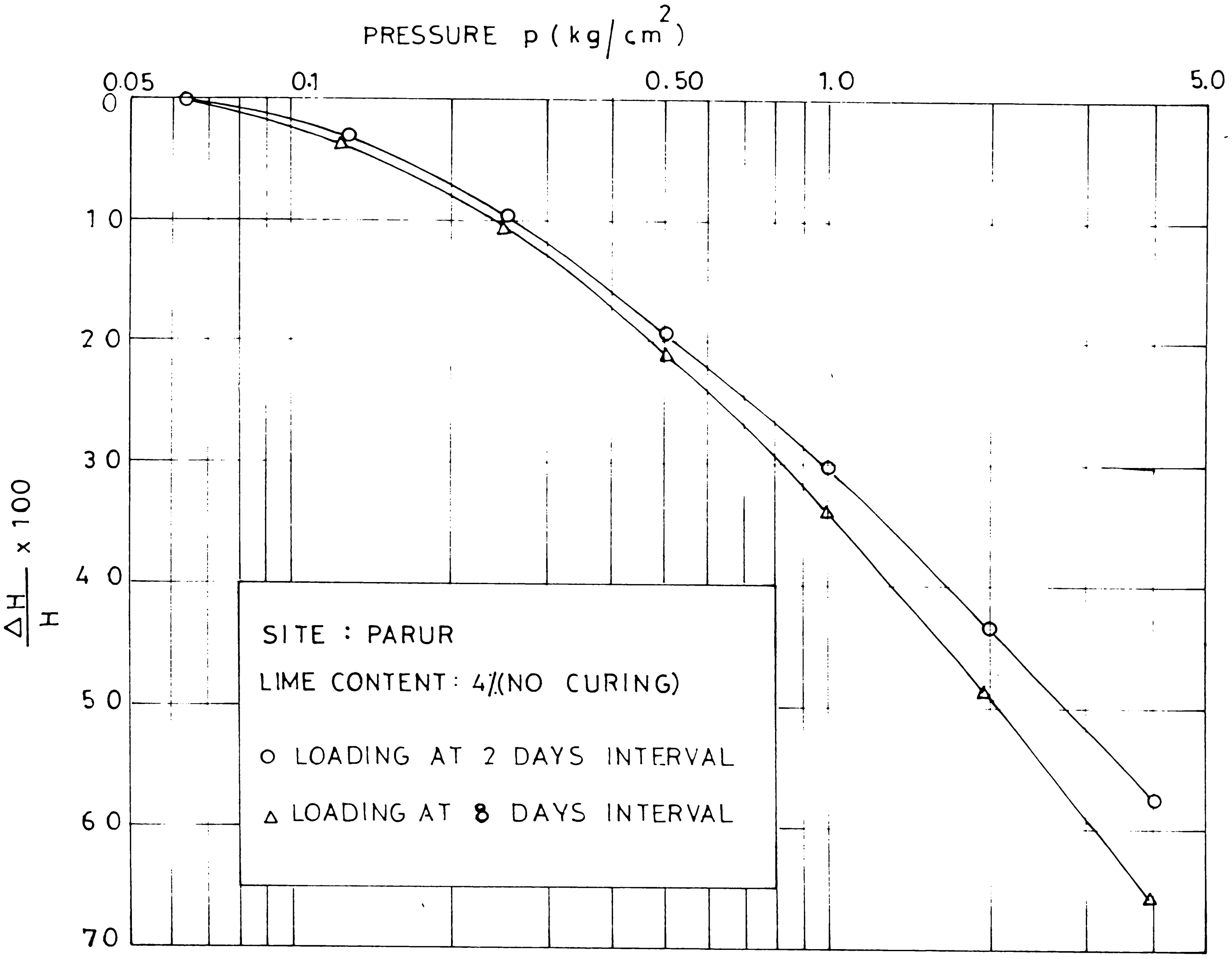

Fig.6.4.4.1b: $\frac{\Delta H}{H}$ vs PRESSURE.

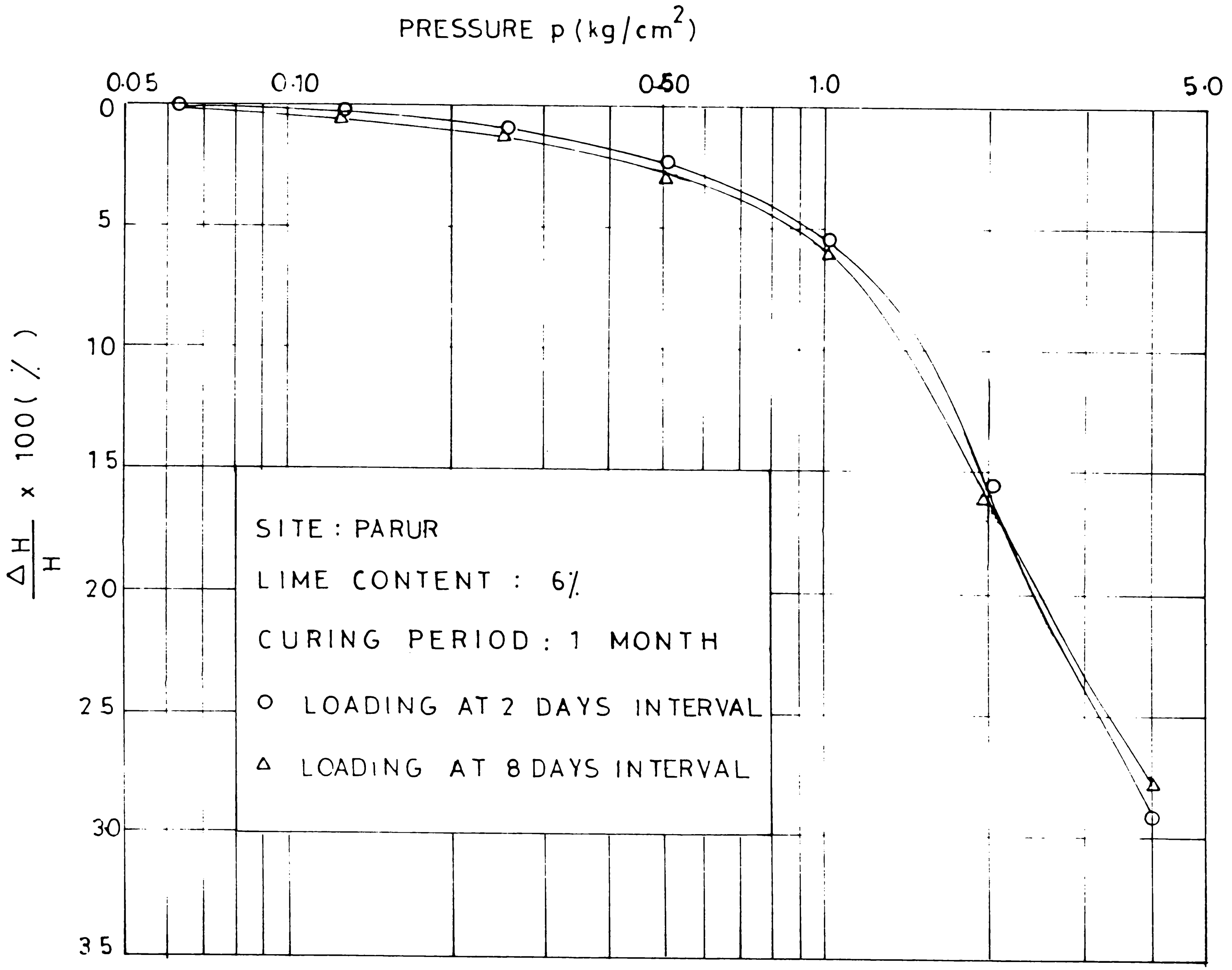

Fig. 6·4·4·1c : $\frac{\Delta H}{H}$ vs PRESSURE clays and black cotton soil, the compression curves are shown in Fig.6.4.5.1a and b. Fig.6.4.5.2 shows the relation between de/d(log p) and log p for the two soils. The figures indicate that the effect of lime on black cotton soil is much more pronounced than on Cochin marine clays.

For all the discussions so far, the results from tests carried out on lime-treated samples from Parur only were considered. To verify whether the findings are equally applicable to samples from other locations in Greater Cochin area, typical compression curves for lime-treated samples from Kumbalam and Maradu are presented in Figs.6.4.5.3b and c for untreated samples, samples immediately after mixing and samples cured for 1 week and 1 month. The curves are in good agreement with each other.

To study the development of bond strength under continuous strain, consolidation tests were conducted on two samples, keeping the total time spent the same. The first one was not given any time for curing, but more duration was allowed for each load increment. In the second, sample was allowed to cure, but the duration for each load increment was reduced so that both tests took

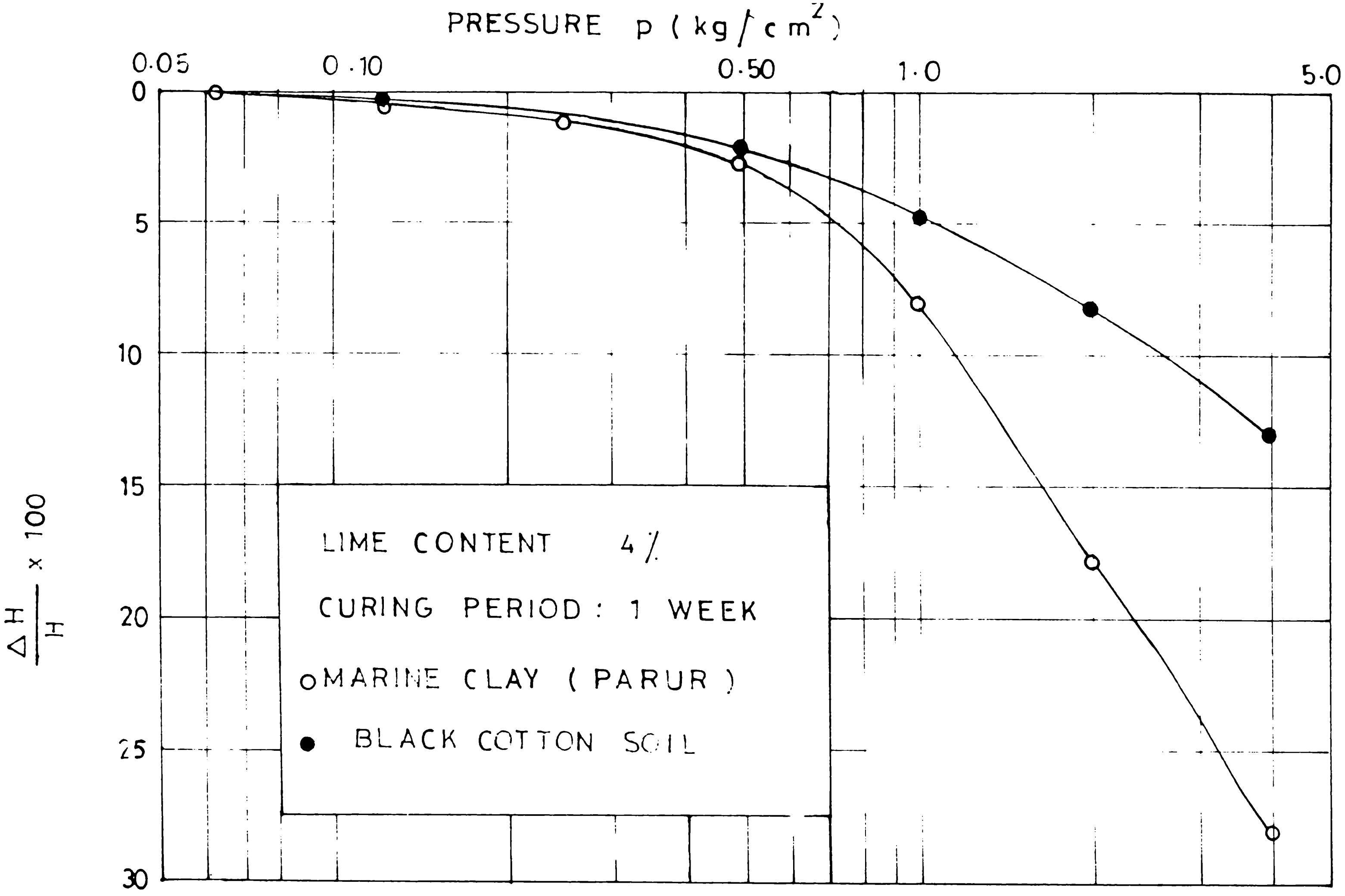

Fig. 6·4·5·1a: $\frac{\Delta H}{H}$ vs PRESSURE.

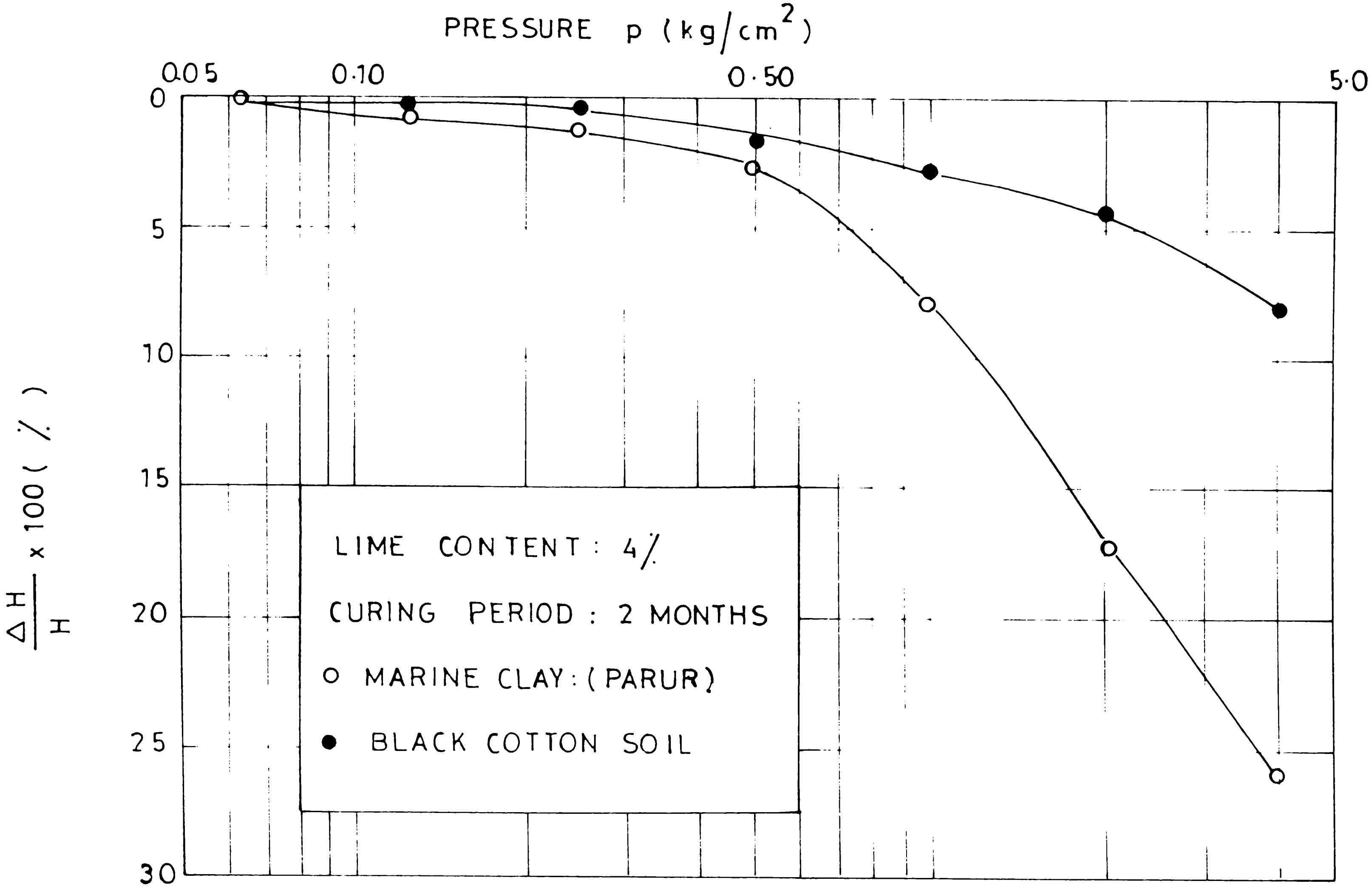

Fig. 6.4.5.1b: $\frac{\Delta H}{H}$ vs PRESSURE.

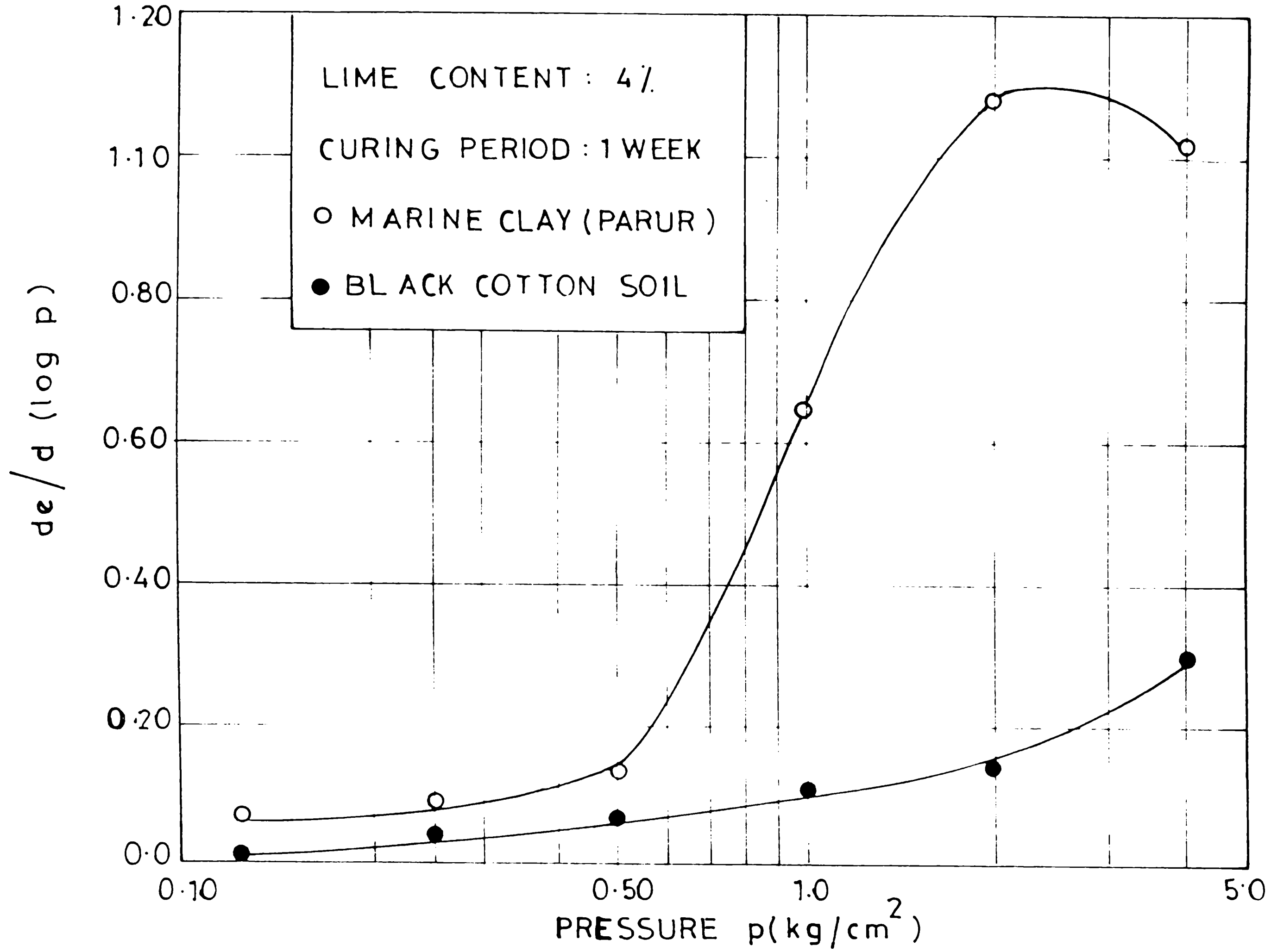

Fig. 6.4.5.2 de/d (log p) vs PRESSURE.

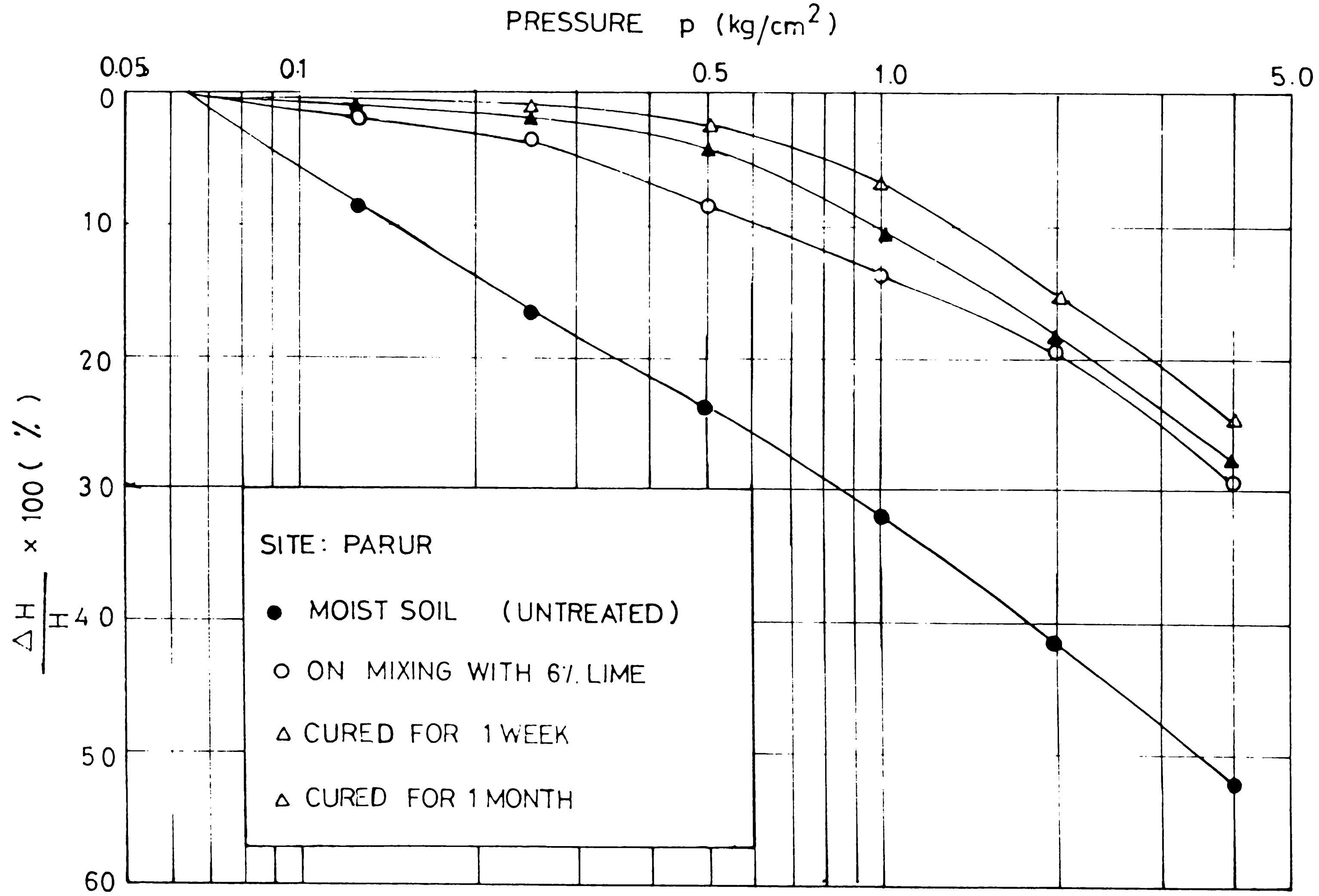

Fig. 6.4.5.3a : $\frac{\Delta H}{H}$ vs PRESSURE.

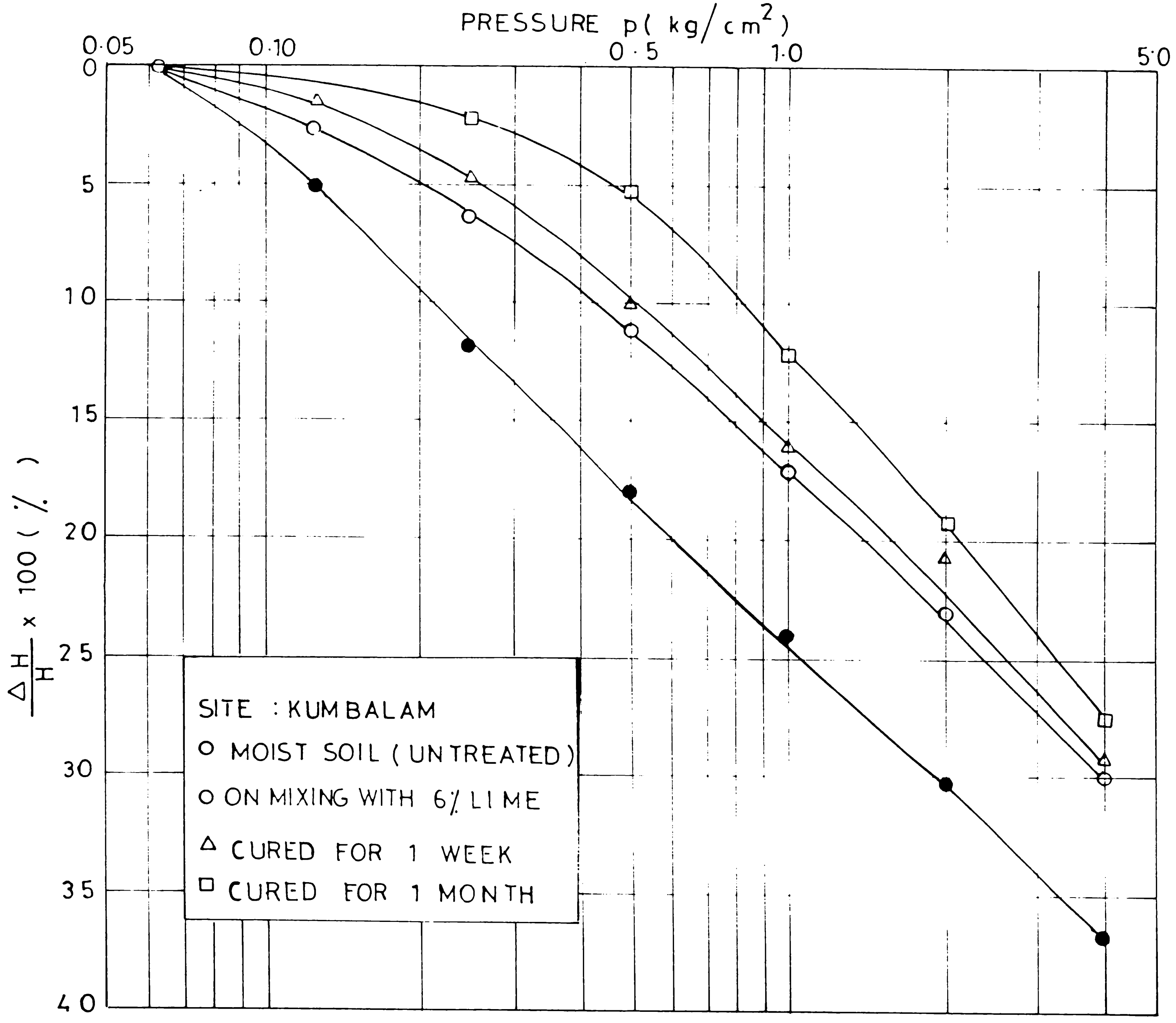

Fig. 6·4·5·3b : $\frac{\Delta H}{H}$ vs PRESSURE.

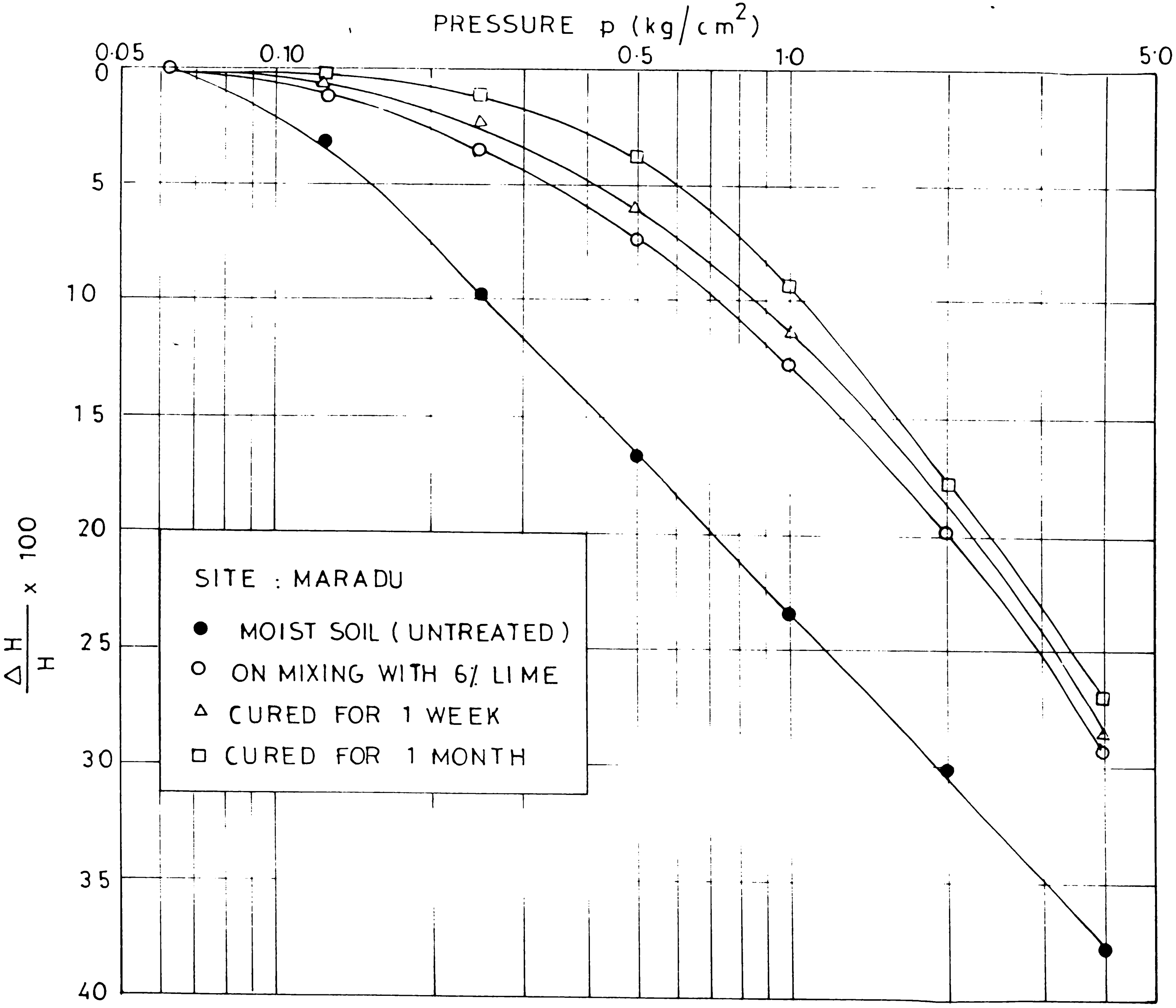

Fig. 6·4·5·3c $\frac{\Delta H}{H}$ vs PRESSURE.

the same time (45 days). The compression curve for both the samples are given in Fig.6.4.5.4. The sample which was not allowed to cure showed greater compressibility indicating that the development of bonds when the soil is subjected to pressure is less than that when the soil is allowed to cure free of pressure.

6.4.6 Effect of sustained loading

One of the aims of this investigation was to study the engineering behaviour of Cochin marine clays when they are subjected to constant pressure such as from a building. It has already been shown that untreated clay develops strength when they are subjected to sustained loading and the resistance to compression considerably improves with the duration of loading in consolidation tests. In case of treated marine soils, the duration of load increment does not have any influence on the development of bond strength. In order to study the behaviour of stabilised soil beds under sustained loading, a series of consolidation tests were carried out to assess the development of preconsolidation pressure. Fig.6.4.6.1 shows the e - log p curves for two samples remoulded at their respective liquid limits mixing with 3% and 6% lime.

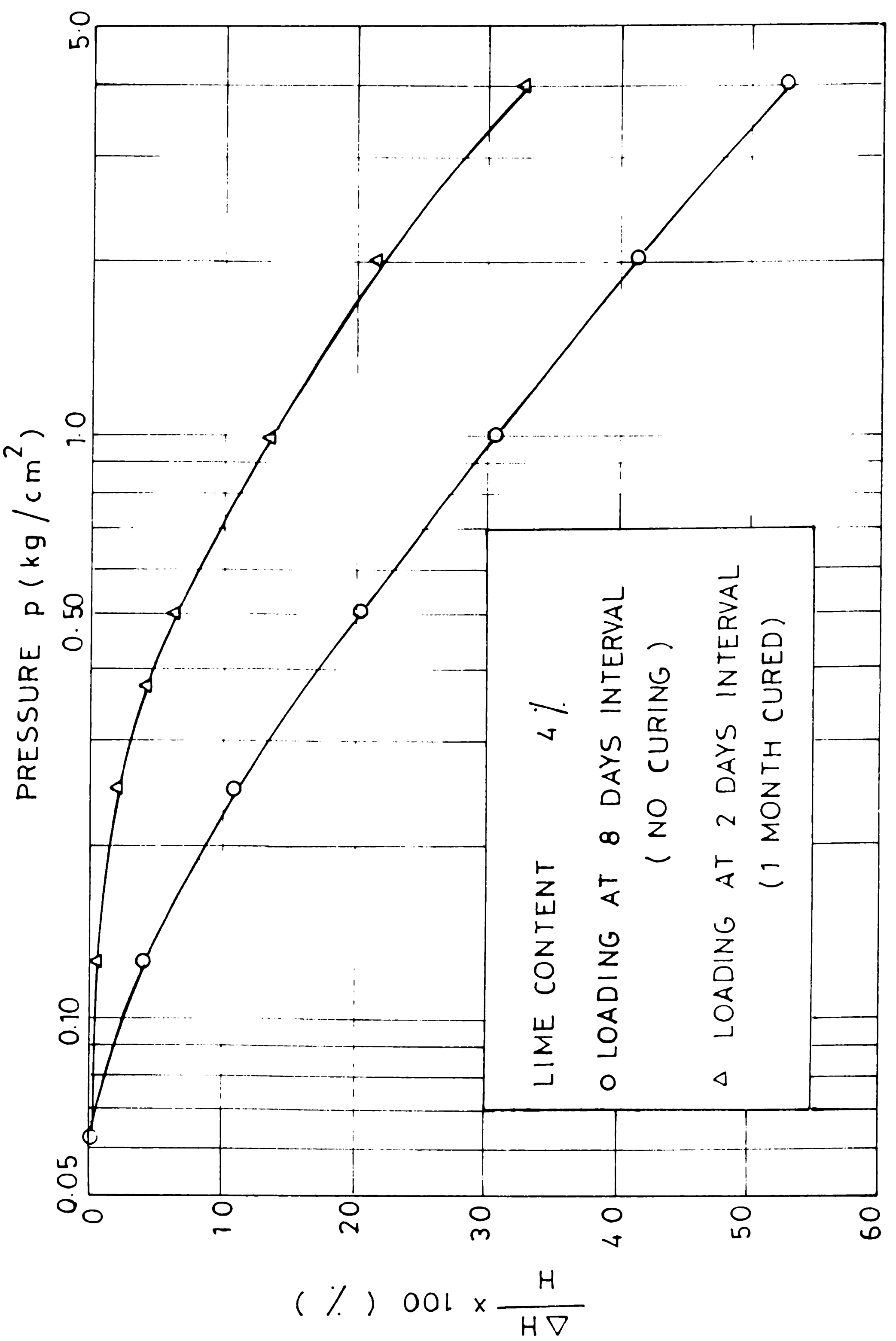

Fig. 6·4·5·4 $\frac{\Delta H}{H}$ vs PRESSURE.

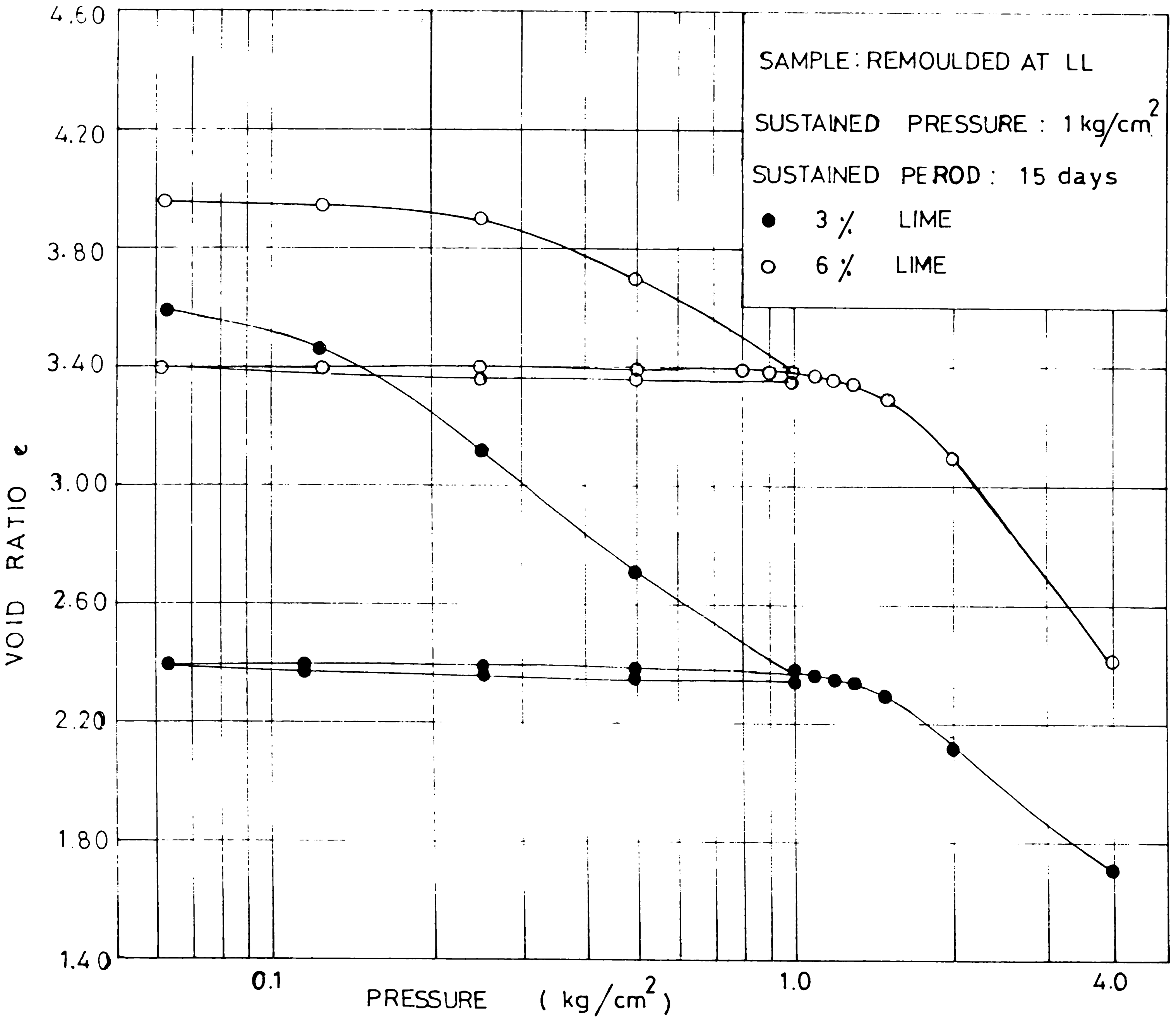

Fig. 6·4·6·1 e-log p CURVES.

In the oedometer ring the samples were kept at a pressure of 1 kg/cm^2 for 15 days. The pressure was then released to seating load of 0.0625 kg/cm^2 and a full consolidation test was run to determine p_c. Both the curves show that there is increase in the preconsolidation pressure by 0.25-0.3 kg/cm^2 in each case. It may be noted that higher lime content does not show higher strength as one would normally expect. This may be perhaps due to the short duration where samples with low lime content will be more active.

Fig.5.6.3.5 shows the e - log p curves for marine clay samples mixed with 3% lime. They were consolidated under a sustained pressure of 1 kg/cm^2 for duration of 2 days, 7 days, 15 days and 60 days. The four plots show that there is a regular increase in preconsolidation pressure and it is proportional to the period of sustained load. When the increase in p_c for 2 days loading is hardly 0.2 kg/cm^2, the same for 60 days duration is 0.6 kg/cm^2. The increase in p_c from sustained pressure consistently increases with duration of sustained loading.

Fig.6.4.6.2a and b shows two e - log p curves of samples with lime contents of 6% and 9% cured for

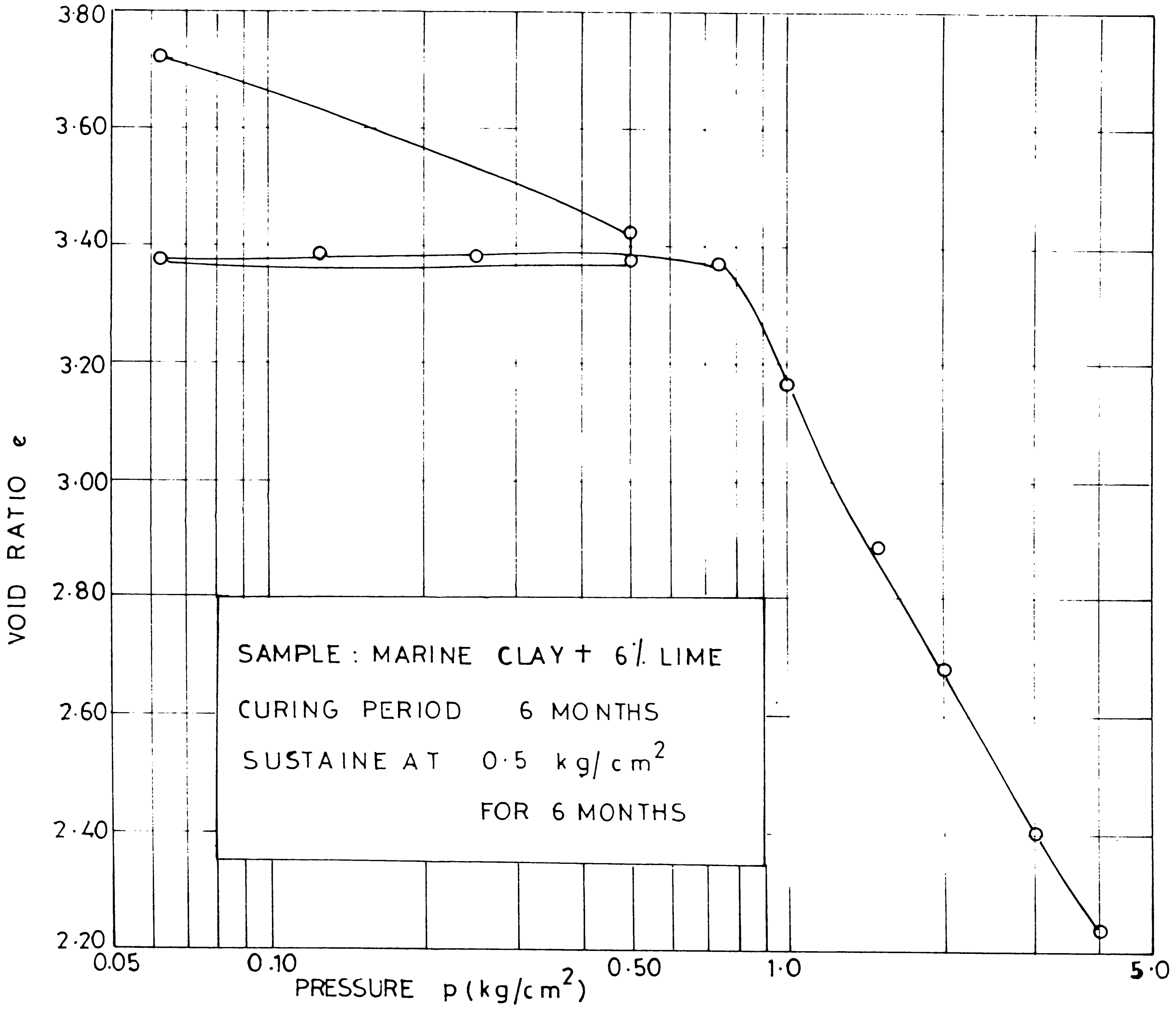

Fig. 6·4·6·2a e - log p CURVE

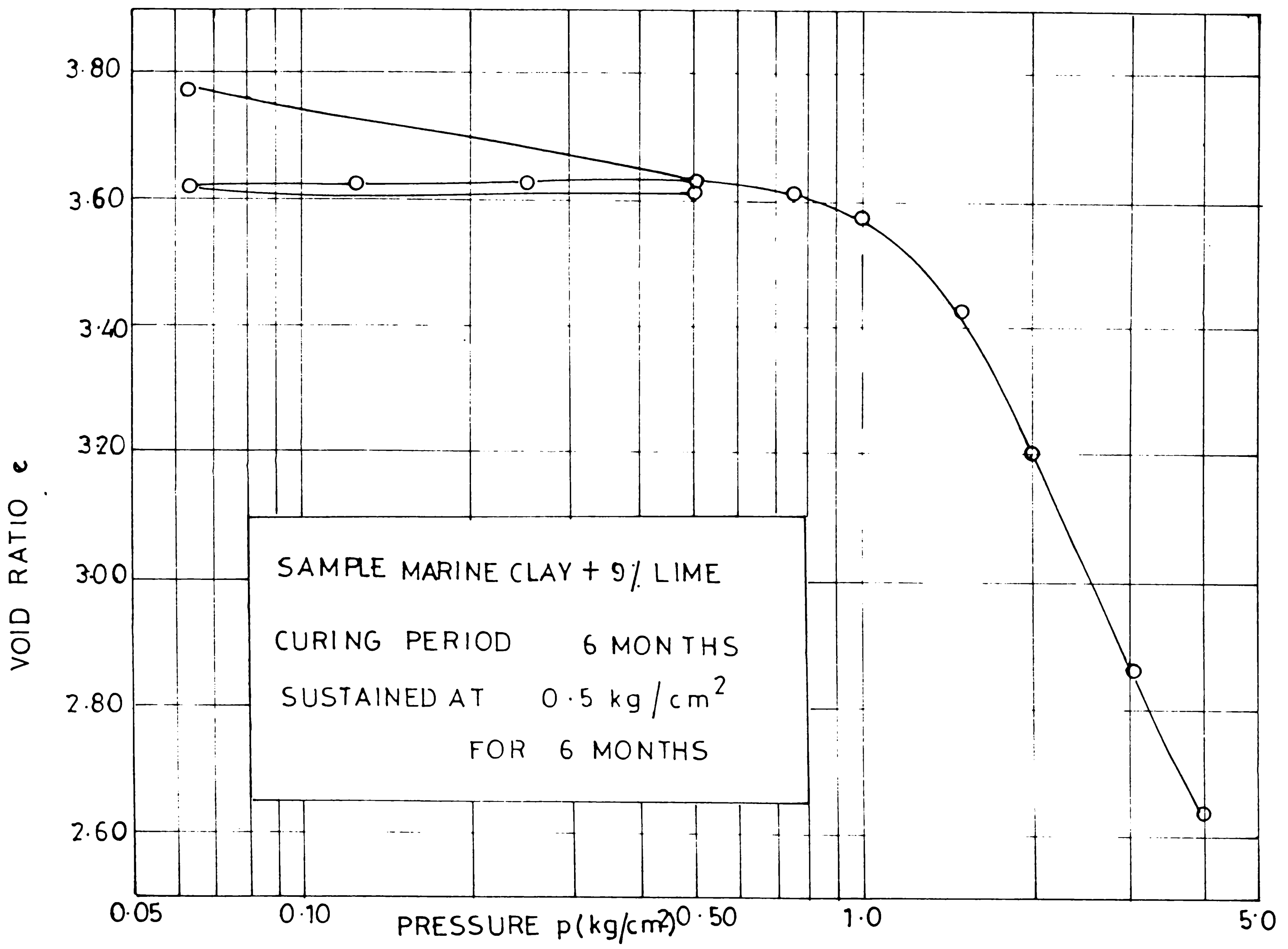

Fig. 6·4·6·2b e-log p CURVE

six months. Then they were consolidated for six months in an oedometer ring under a stained pressure of 0.5 kg/cm^2. The load was then released to seating load and full consolidation tests were conducted. The two e - log p curves show that there is considerable increase in p_c. The total compression for 6% lime is much more than the total compression for 9% lime.

Chapter VII

ELECTRICAL CHARACTERISTICS OF MARINE CLAYS

7.1 INTRODUCTION

In recent years, geophysical methods of exploring the subsurface have gained momentum in geotechnical feasibility studies encountered in civil engineering (Mitra, 1984; Murthy & Chandra, 1981). Among various geotechnical methods available, electrical resistivity survey has been very popular as a rapid and relatively inexpensive method. Eventhough its earlier applications were more in search of metals and minerals, of late, geotechnical engineers have been taking advantage of the versatality of the technique (Malhotra, 1981; Raman, 1984; Rao et al., 1987; Ravindranath et al., 1983). Geophysical measurements essentially involve search for the subsurface by physical measurements from the earth's surface. The two geophysical methods which are of maximum use in engineering studies are electrical and seismic prospecting techniques.

In resistivity prospecting, direct current is impressed into the ground through a pair of metallic electrodes while the potential difference between two other intermediate points is measured. From a knowledge of the electrode separation, potential difference and current, the apparent resistivity can be calculated. If the ground is homogeneous, this is the true ground resistivity. Since a homogeneous subsurface is very uncommon, it is the weighted average of the resistivities of the formations through which current is passed, that one gets from an electrical resistivity survey. It is from an analysis of the variation of this quantity with change in electrode spacing and position that deduction about subsurface can be made. Hence interpretation techniques are equally important in a successful subsurface exploration by any geophysicals method (Kate & Khichchumal, 1983).

The popular methods that are used for interpretation of the resistivity data for arriving at the soil profiles are

i) Moore's Cumulative Plot with Hummel's Extension
ii) Direct slope
iii) Inverse slope
iv) Barnes layer
v) Master curves

The values of the apparent resistivity obtained by any one of the above methods are then compared with standard absolute resistivity values to recognise the type and depth of the various layers.

While significant work has been carried out in connection with field oriented studies, no information is available with regard to its usefulness in the laboratory. In this chapter an attempt has been made to investigate whether this technique could be used advantageously to relate it with engineering properties.

7.2 RESISTIVITY MEASUREMENTS ON COCHIN MARINE CLAYS

The resistivity survey provides the values of apparent resistivities. Resistivity soundings provide the values of apparent resistivities ρ_a in a layered system at known depths. However the values of absolute resistivities are required to identify the various layers in subsurface profiles. The apparent resistivity at any depth is a function of absolute resistivities and thicknesses of various strata in the subsoil through which current passes. Hence the value of absolute resistivity is essential for purposes of correlation between field resistivity measurements and geotechnical properties of Cochin marine clays.

The absolute resistivity of soil is the resistance offered to the flow of current by a soil cylinder of unit cross sectional area and unit length (Chauhan & Kate, 1983).

$$\rho = \frac{R.A.}{L}$$

where

ρ = Resistivity of the soil (ohm-cm)

R = Resistance offered by a cylindrical soil specimen (ohms)

A = Cross sectional area of specimen (cm^2)

L = Length of the specimen (cm)

Laboratory results on electrical resistivity studies have been scanty. Preliminary results on the effect of density and moisture content on resistivity of soils have been presented by Chauhan and Kate (1983). A critical examination of the limited available data showed that it was desirable to investigate the electrical resistivity of Cochin marine clays. Since these tests are very simple and least time consuming, any correlations between the electrical resistivity and other engineering properties will be of immense help. With this in view, studies were initiated with regard to the electrical resistivity characteristics of Cochin marine clays.

Laboratory tests

The experimental set up to measure the absolute resistivity of clays consists of a simple perspex mould of size 4 cm x 4 cm x 8 cm. Well polished copper plates of size 3.9 cm x 3.9 cm and 1 mm thick are used as end plates. Oven dried marine clay sample was remoulded into the perspex mould at the desired water content and density. The required density was achieved by static compaction. One end of the connecting wires was welded to the face of the two copper end plates, and the other end of wire to a highly sensitive multimeter. The sample is kept in the upright position and a nominal weight of 0.2 kg was kept on the top of the top copper plate in order to ensure perfect contact between the soil and the copper plate. The self weight of the sample and the nominal weight ensure good contact at the bottom.

In the case of undisturbed samples no mould was used. The resistivity of the undisturbed sample was measured by keeping the undisturbed sample directly on the bottom copper plate and the top copper plate along with the nominal weight was placed on the top of the sample. For undisturbed specimen, the size used was 3.8 cm diameter and 7.5 cm height.

Fig.7.2.1 presents the plot between dry density and resistivity of oven dried Cochin marine clays for various moisture contents. Since the value of resistivity was found to vary from 800 ohm-cm to 500×10^3 ohm-cm, the resistivity was plotted to log scale. The curves show sharp variations in absolute resistivity for variations in dry density as well as moisture content. Fig.7.2.2 shows the relations between resistivity and moisture content for different dry densities. Since both curves have regular patterns, they can be utilised where density measurements have to be taken at short durations. It may be noted from both the curves that the changes in resistivity for normal changes in density and moisture content is of a very high order. Since density also represents the porosity and hence the volume of medium of separation, the resistivity is strongly influenced. Hence such sharp variations can be expected.

For purposes of comparison, results have also been obtained on partially saturated kaolinite clay samples at different moisture contents and dry densities. The results are shown in Figs.7.2.3 and 7.2.4.

7.2.1 Effect of sample dimensions on electrical resistivity

Resistivity measurements reported so far by various workers are normally on small cylindrical specimens. For

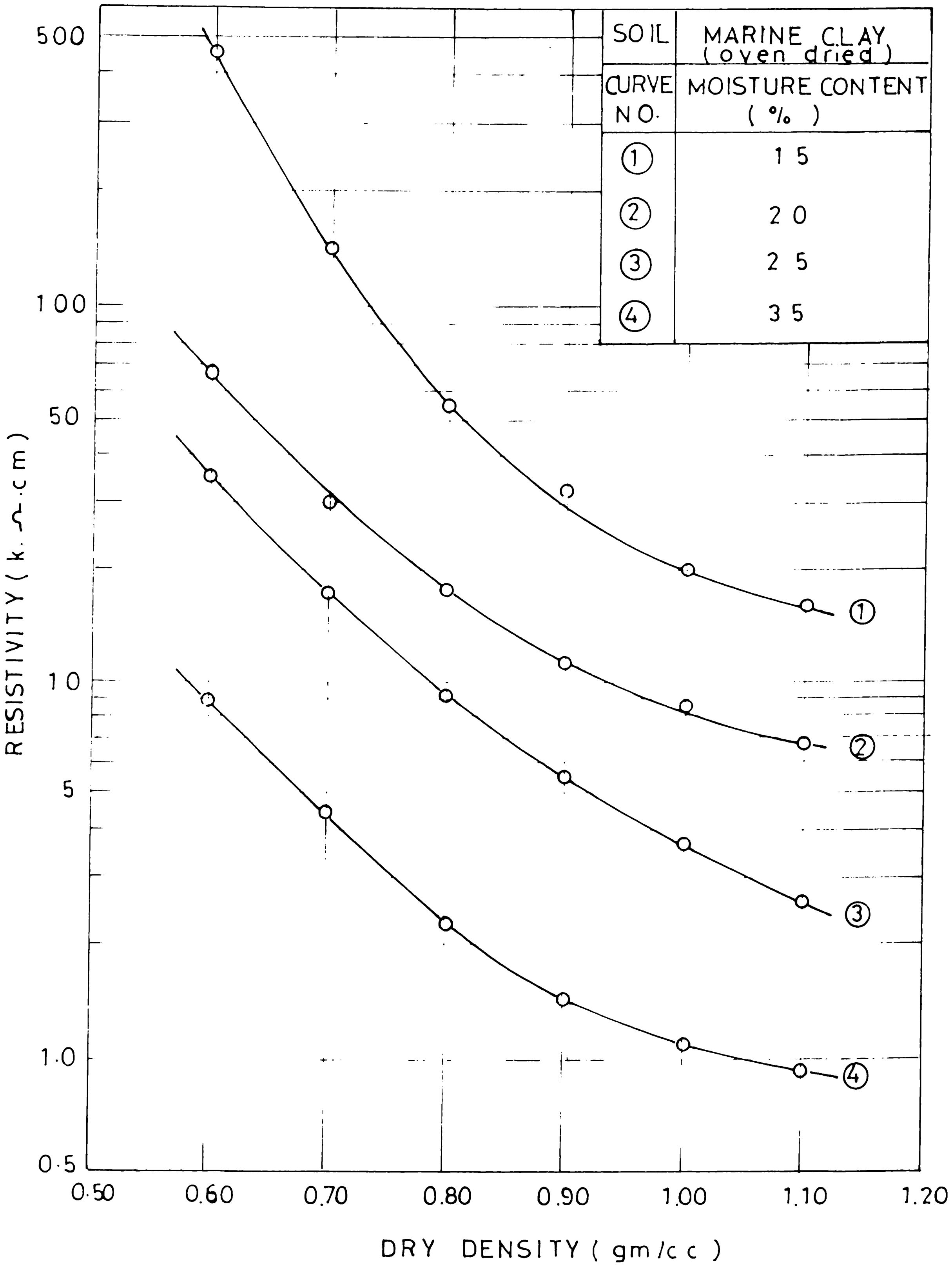

Fig. 7.2.1 RESISTIVITY vs DRY DENSITY.

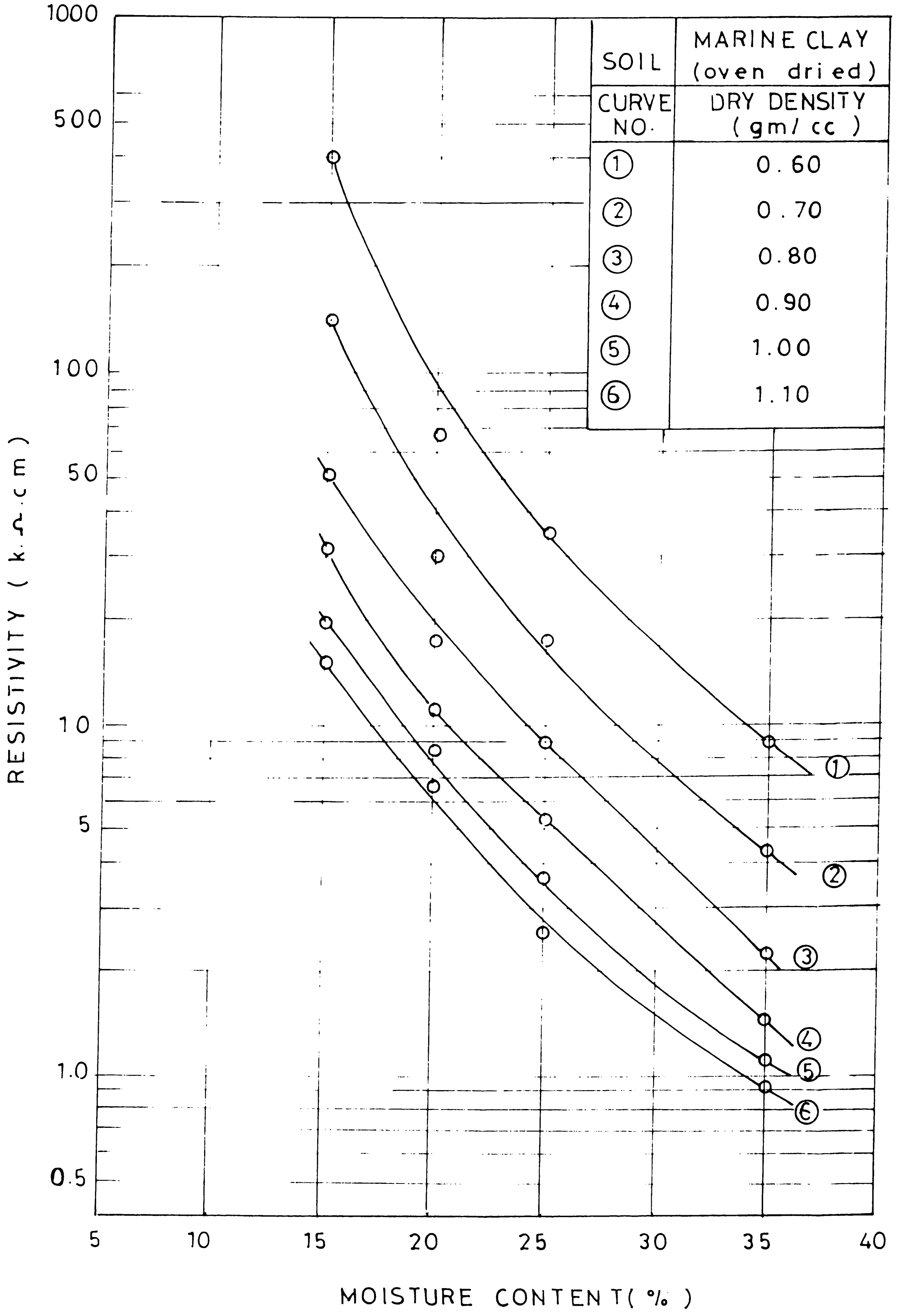

Fig. 7. 2. 2 RESISTIVITY vs MOISTURE CONTENT.

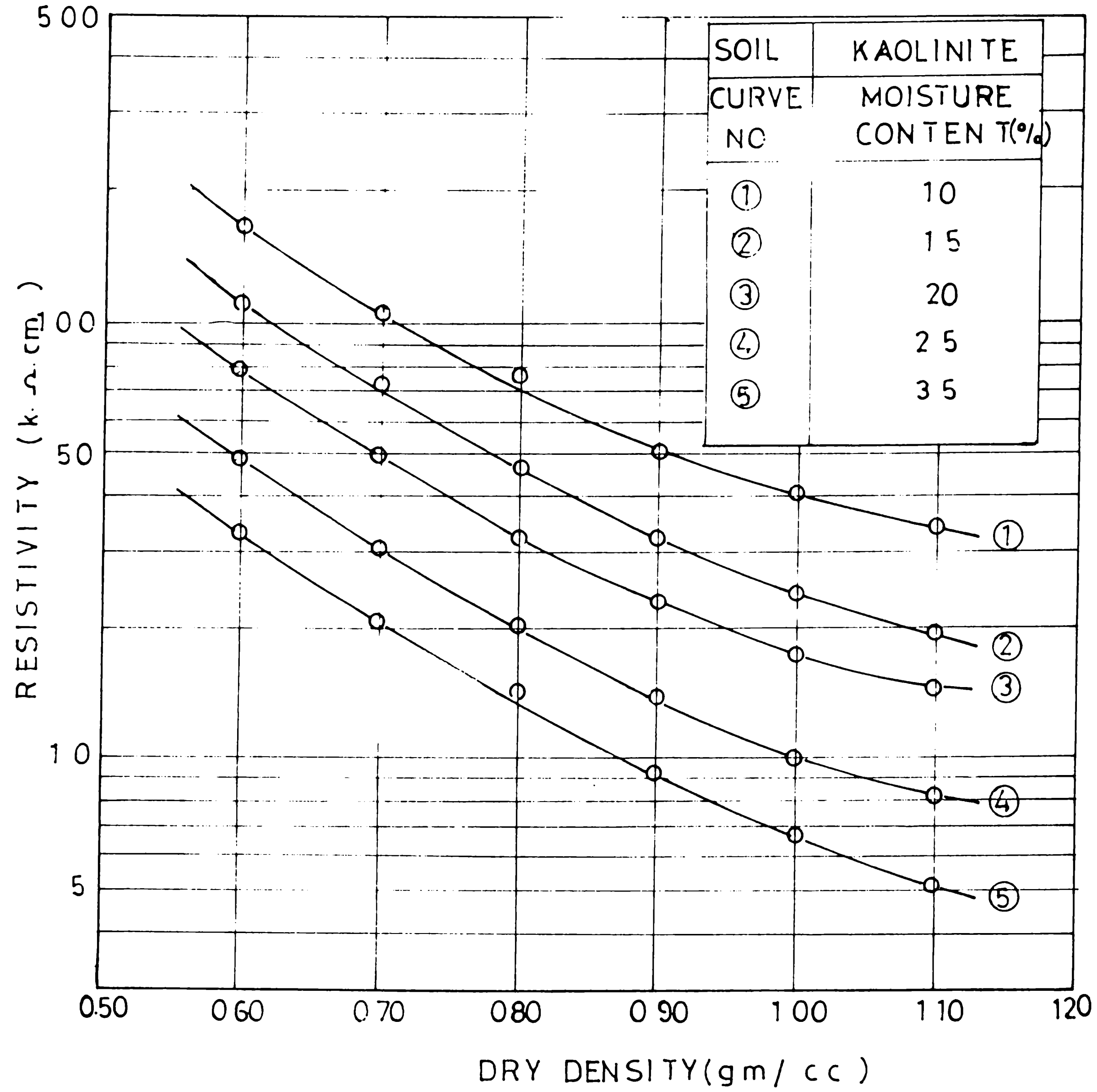

Fig. 7. 2. 3. RESISTIVITY vs DRY DENSITY.

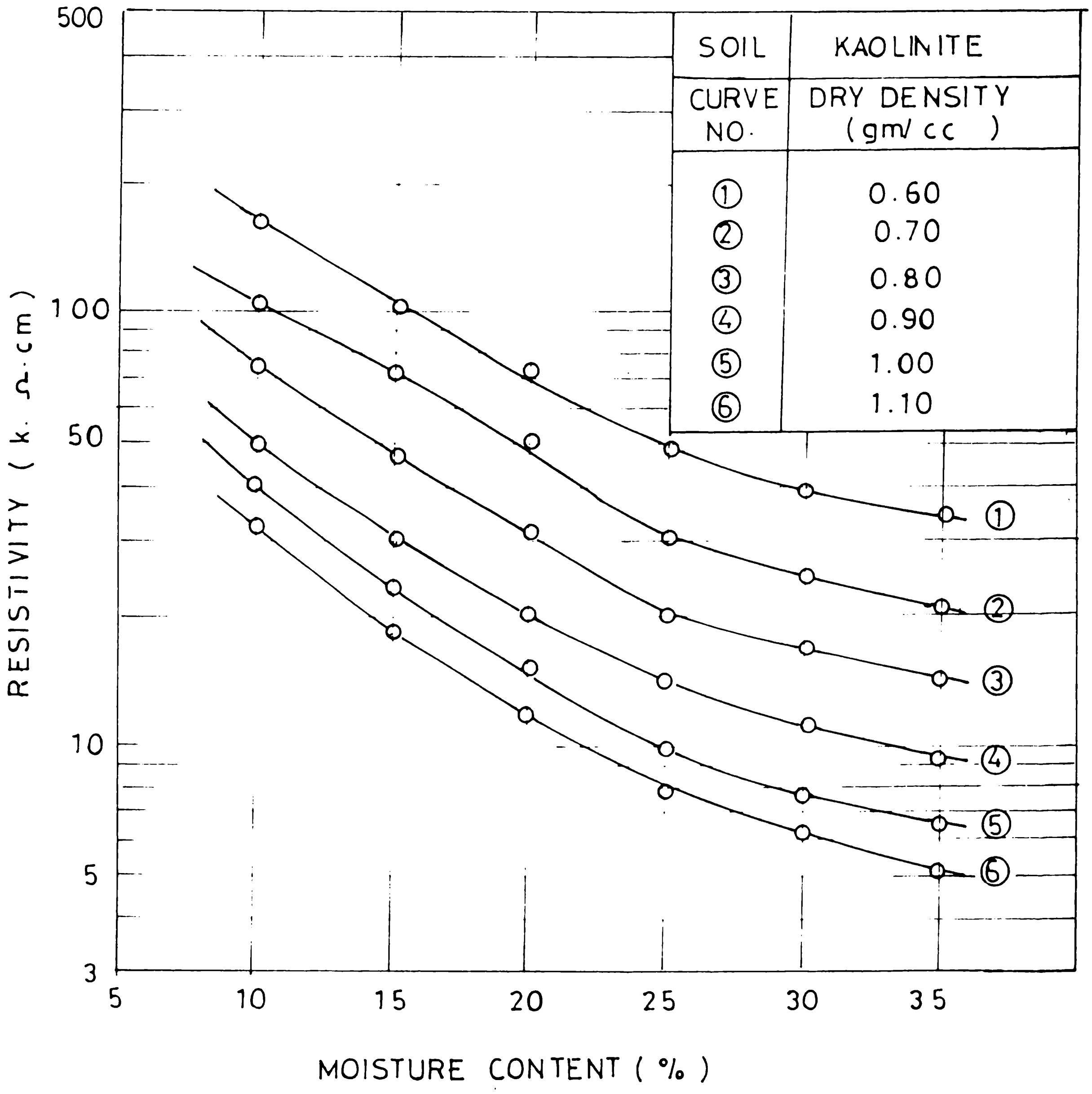

Fig. 7. 2. 4. RESISTIVITY vs MOISTURE CONTENT.

fine grained soils, measurements on such specimens can give representative values. But in geophysical exploration, where the resistivity methods have maximum applications, the absolute resistivity of soils with coarser grains also will have to be determined so that an accurate interpretation of the field test data is made possible.

In order to find out the influence of sample sizes on square prismatic samples, resistivity measurements were carried out on samples of different areas of cross section and lengths. The areas of cross section chosen were 16 cm^2, 36 cm^2, 64 cm^2 and 100 cm^2. The lengths of samples tested were 2.5, 5.0, 7.5, 10.0, 12.5, 15.0, 17.5 and 20.0 cm. The tests were carried out on oven dried marine clay samples with different water contents. The results are presented in Figs.7.2.1.1 to 7.2.1.4.

Fig.7.2.1.1 shows the plot between lengths of sample and resistivity of oven dried specimens for four areas of cross section for a water content of 40% and dry density of 1.2 gm/cc. It can be seen that the value of absolute resistivity tends to decrease with length of the sample. The rate of change is faster for shorter specimens than for longer specimens.

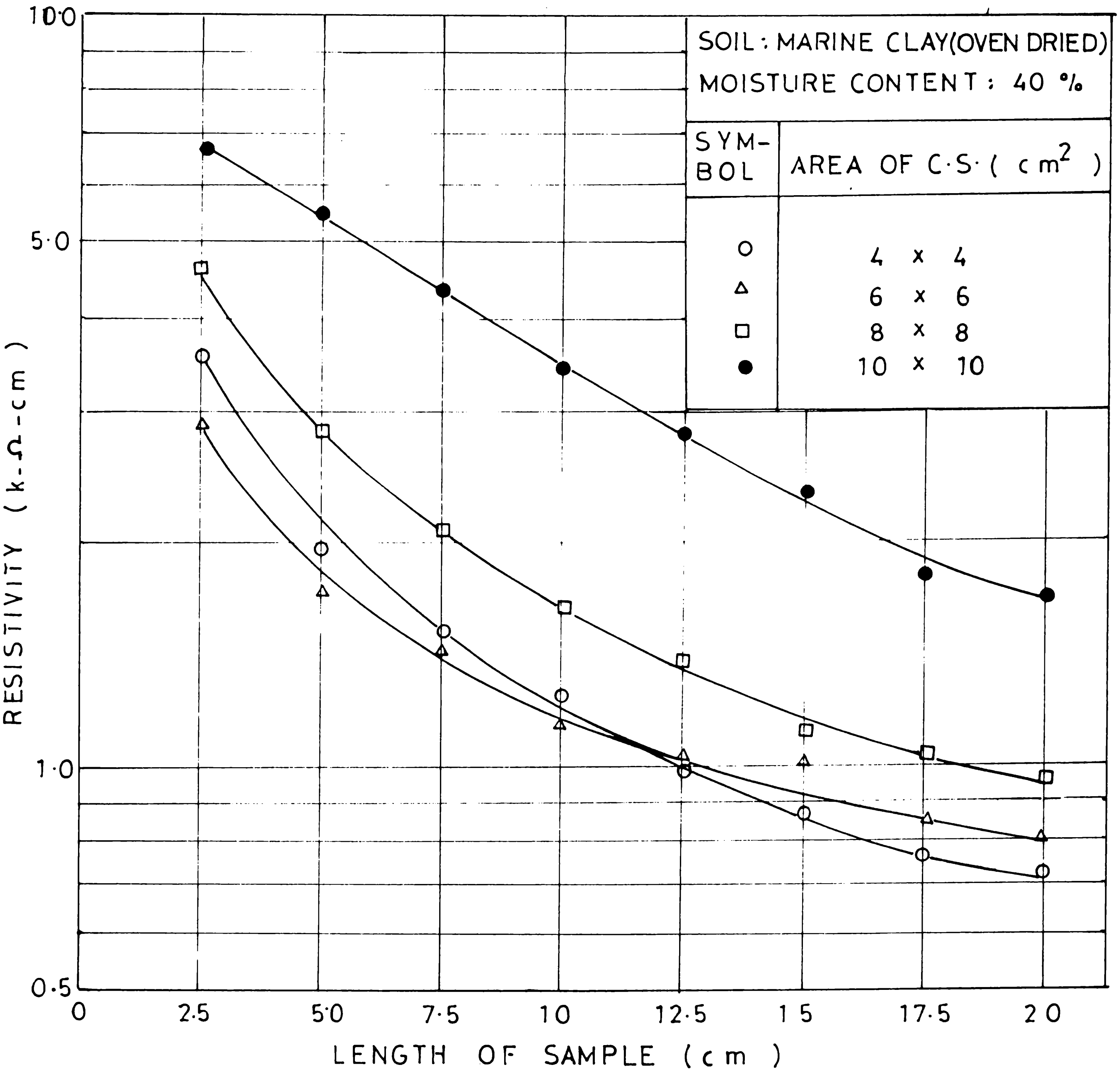

Fig. 7.2.1.1 RESISTIVITY vs LENGTH OF SAMPLE.

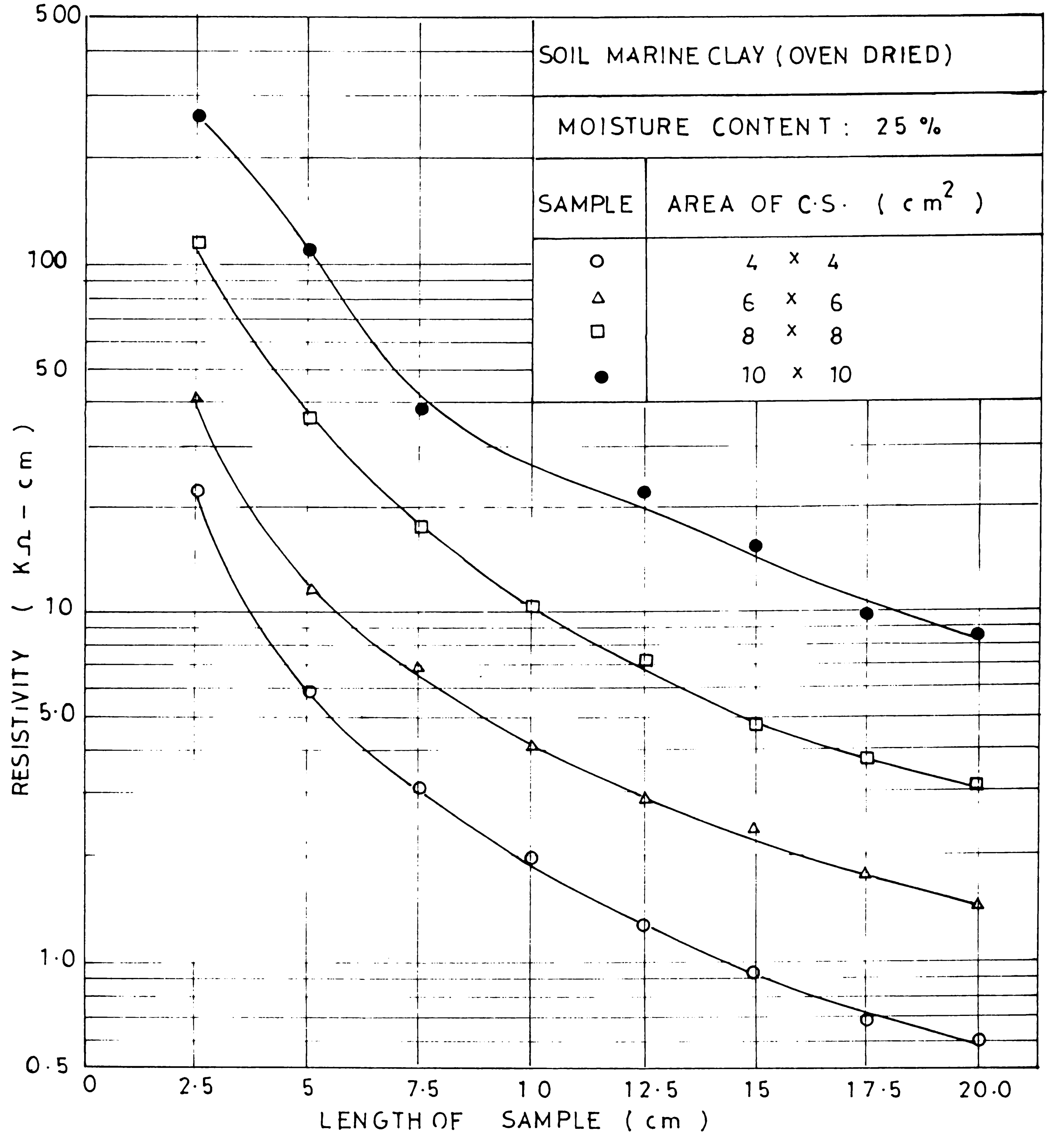

Fig. 7·2.1·2 RESISTIVITY vs LENGTH OF SAMPLE·

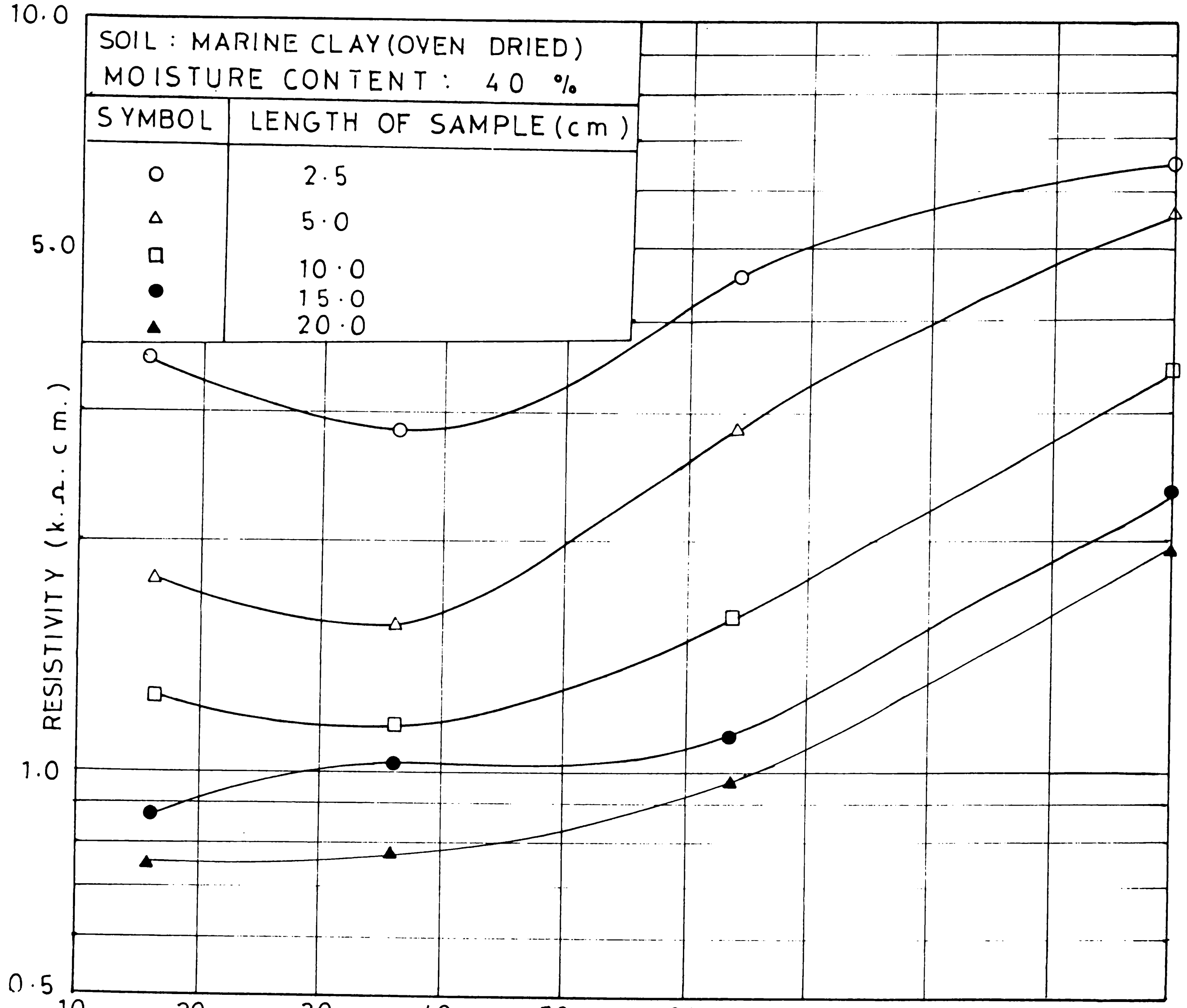

Fig. 7.2.1.3 RESISTIVITY vs CROSS SETIONAL AREA.

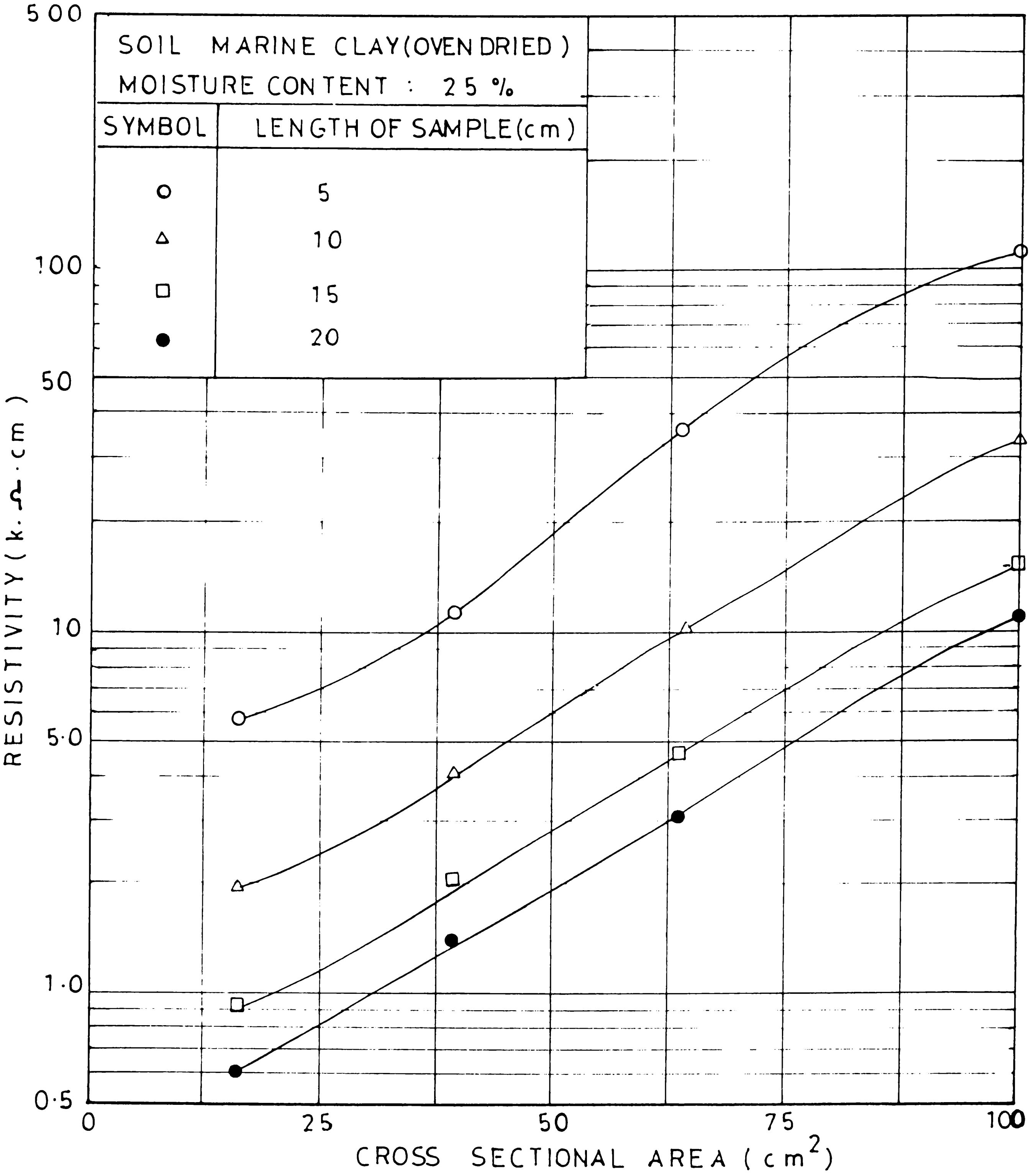

Fig. 7.2.1.4 RESISTIVITY vs AREA OF CROSS SECTION.

Fig.7.2.1.2 shows the variation of resistivity with length of sample for the different areas of cross section of oven dried marine clay samples for a water content of 25%. Here also the ρ values decrease with length of specimen.

Figs.7.2.1.3 and 7.2.1.4 show the relation between absolute resistivity and area of cross section for four typical lengths. The water content in Fig.7.2.1.3 is 40% and 7.2.1.4 is 25%. The curves show that the resistivity values increase with cross sectional area.

Since the values of absolute resistivity are dependent on the dimensions of the sample, in case of square prismatic specimens, there is a need for standardising the sample dimensions. One possible suggestion could be that the sample size of 37.5 mm x 75 mm, which is the standard size, may be adopted.

7.2.2 Resistivity of samples and dielectric constant of pore fluids

The resistivity of the soil is also a function of the dielectric constant of the pore fluids. Fig.7.2.2.1 shows the relation between the fluid content and the resistivity of oven dried marine clay collected from Parur site from a depth of 7.5 metres. The variation of resistivity has been plotted against the fluid content for a dry density of 1.2 gm/cc. For four pore fluids viz. carbon tetrachloride, acetone, methanol and water. The values indicate that the resistivity of a soil decreases as the dielectric constant of the pore fluid increases. In the case of acetone, methanol and water the electrical resistivity of the clay decreases with the fluid content as expected. But for carbon tetrachloride the resistance increased with higher fluid contents.

For purposes of comparison, resistivity studies were conducted on kaolinite and bentonite. The results are presented in Figs.7.2.2.2 and 7.2.2.3.

It may be mentioned here that among the four different dielectric fluids used the thickness of electrical double layer is maximum for water and least for carbon tetrachloride. This may be possibly the reason for their relative positions of the different curves.

7.3 RELIABILITY OF RESISTIVITY MEASUREMENTS

At present absolute resistivity ρ of a soil sample is determined by finding out the resistance R of specimen

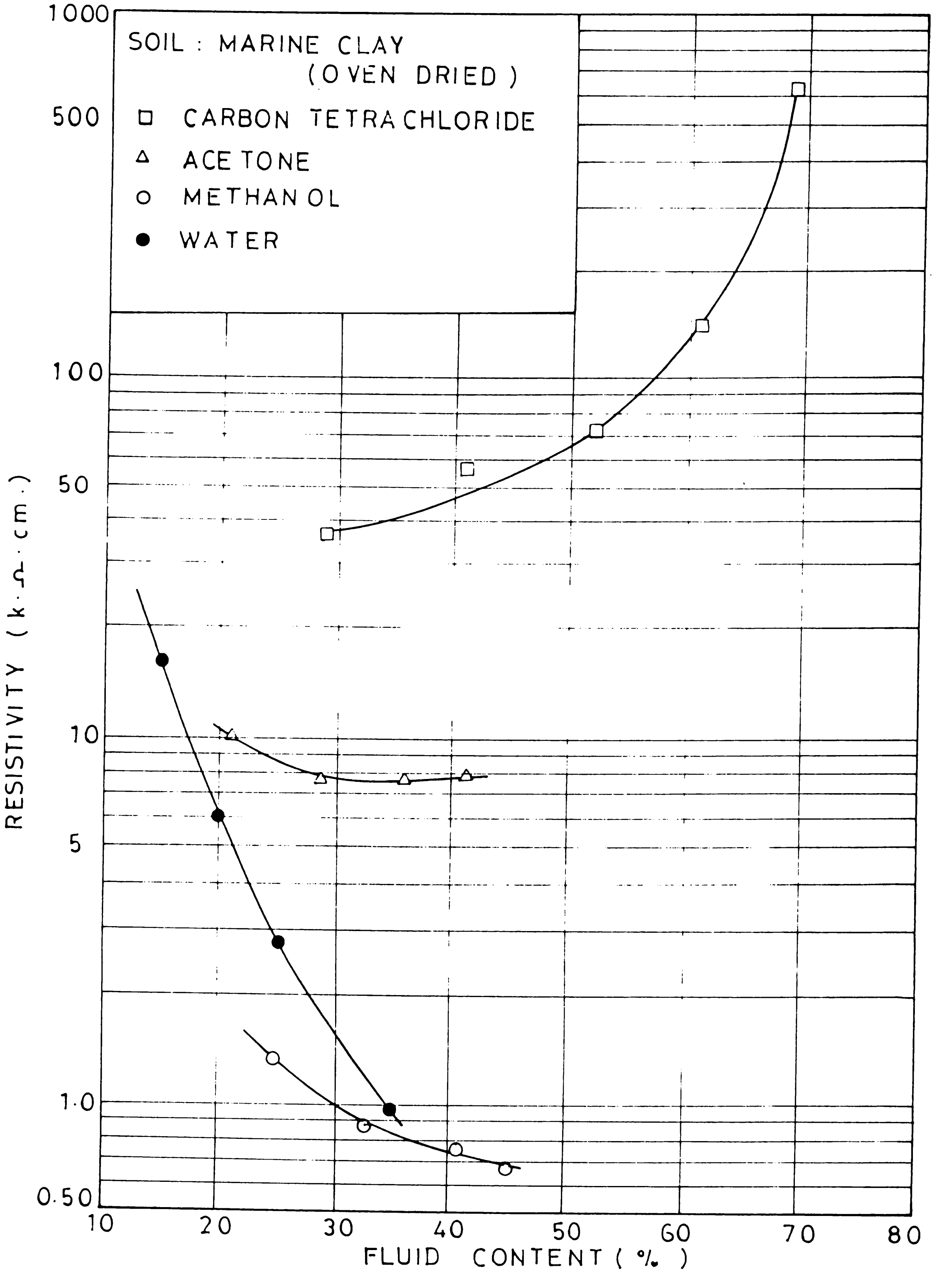

Fig. 7.2.2.1 RESISTIVITY vs FLUID CONTENT.

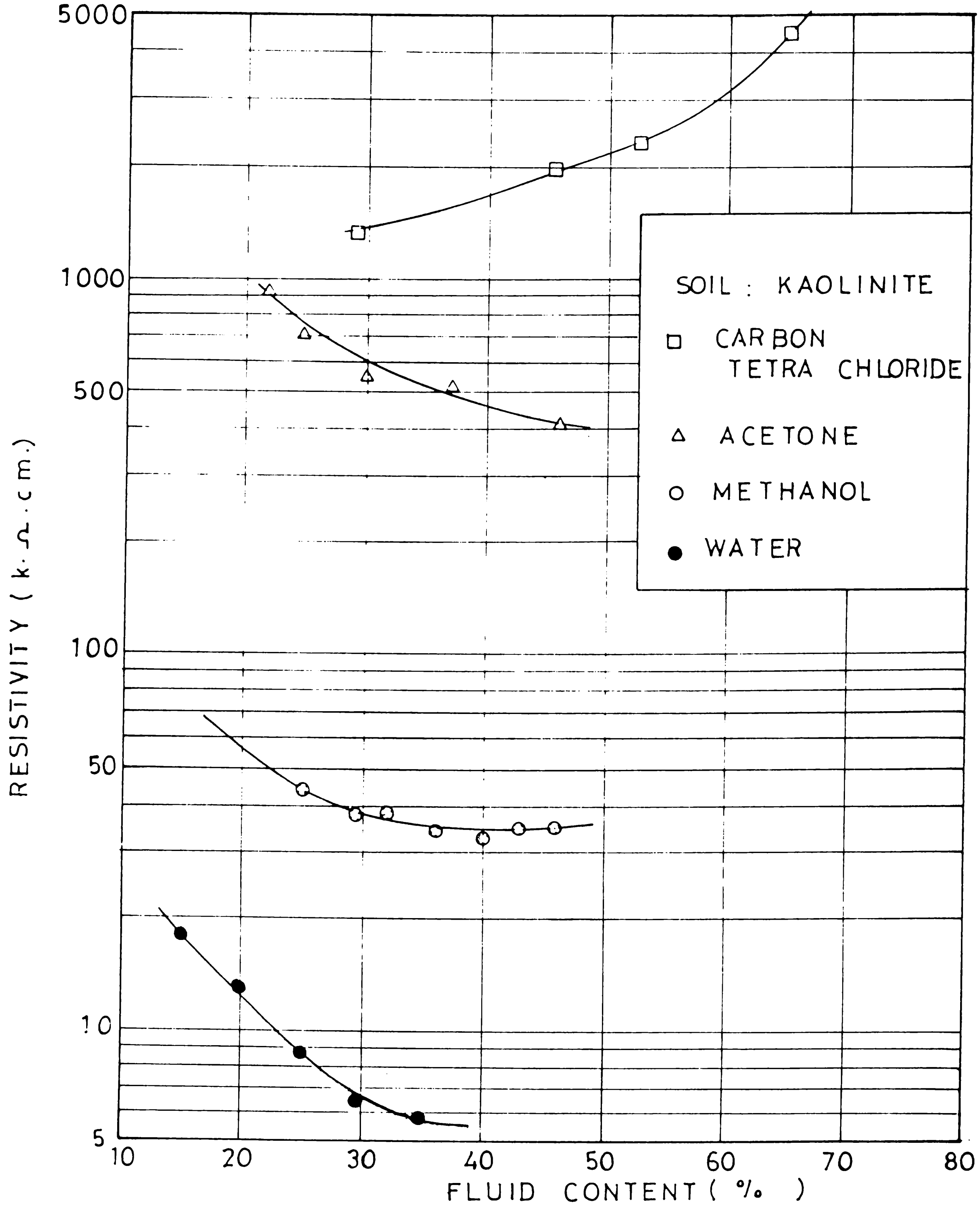

Fig. 7.2.2.2 RESISTIVITY vs FLUID CONTENT.

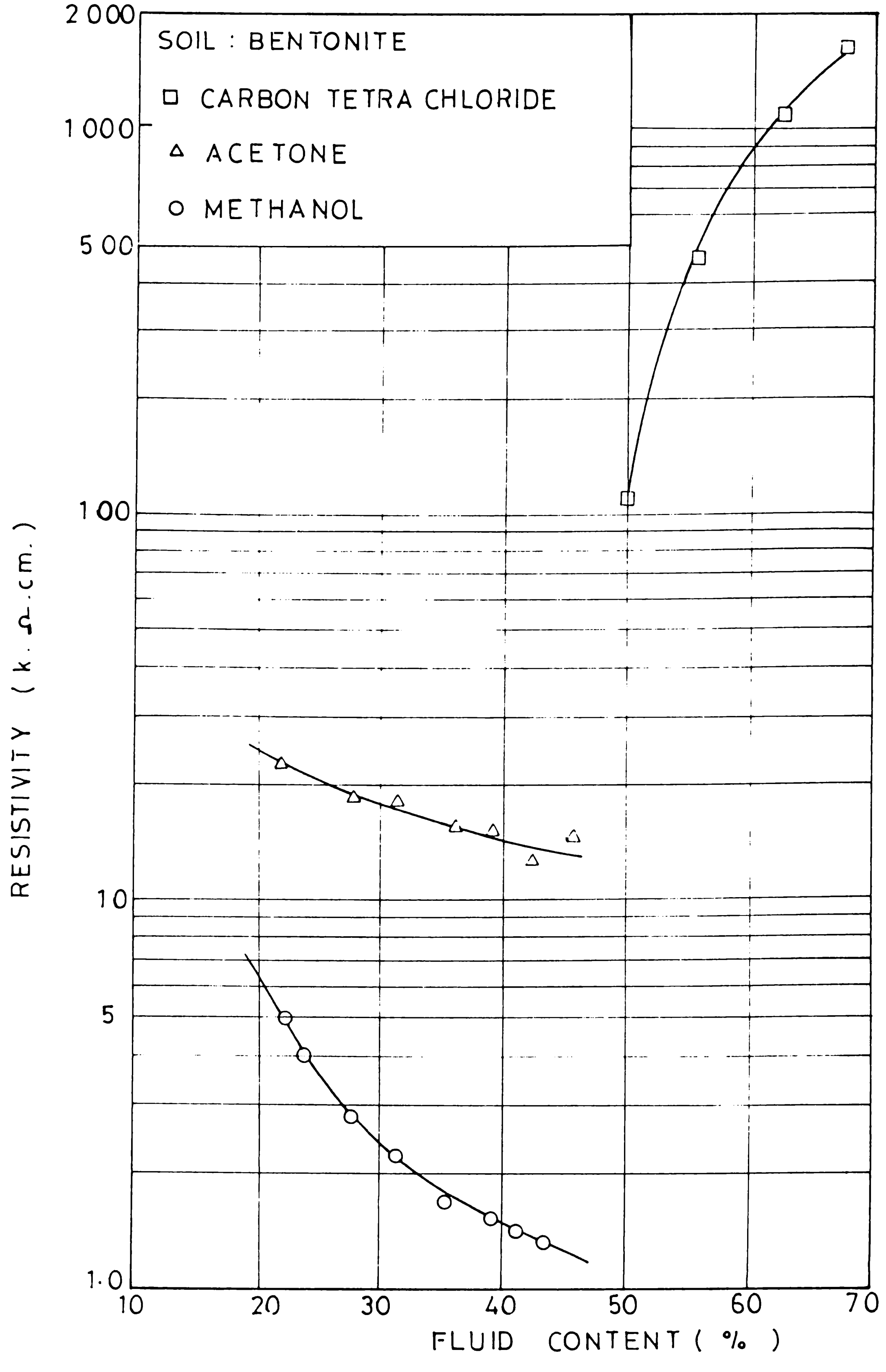

Fig. 7.2.2.3 : RESISTIVITY vs FLUID CONTENT.

of length L and area of cross section A by the equation (Chauhan & Kate, 1983).

$$\rho = \frac{R.A.}{L} \qquad (7.3.1)$$

It has been already pointed out that the value of ρ is not a constant as it should have been if it were a property of the material viz. soil. But it has been shown already that the value of ρ depends on the length of the sample and the area of cross section. This expression gives the absolute resistivity of a conducting medium. Soil is a particulate medium and the resistance offered by soil, even if fully saturated, is considerably higher than the values normally permitted for conducting media. Thus the studies on the electrical characteristics of the soil treating it as a dielectric medium will be more meaningful. The resistivity measurements on a dielectric medium has very serious limitations. The fact that soil is more a dielectric medium than a conductor can be brought out by a series of tests on soil specimens.

In a conducting medium, the value of resistance is a constant and it can be measured instantaneously. But in the case of soil, the resistance is found to vary with time.

Fig.7.3.1 shows a curve B_1 which is a plot of the resistance of a bentonite sample having 10 cm x 10 cm cross section and 2 cm height. It had a water content of 200%. When a direct current was passed through the soil specimen the first reading indicated a resistance of 120 ohms. The value of R was found to increase with time and after about 120 seconds it reached a peak value of 150 ohms. Curve B_1 is the plot of the readings. When the direction of the current was reversed by interchanging the terminals from the d.c. supply, the value of resistance instantaneously was found to drop to just 40 ohms from 150 ohms. The value was then found to increase with time as shown by curve B_2. The maximum value of resistance in both direction is found to be almost same.

Curve M_1 in Fig.7.3.1 shows the plot of the resistance values with respect to time for a marine clay specimen of 4 cm x 4 cm x 7.5 cm. The maximum value of resistance observed was 660 ohms which took about 330 seconds. When the current direction was reversed abruptly, the resistance shown by the multimeter was as low as 60 ohms. The value then improved with time in about 620 seconds and reached a maximum of 630 ohms.

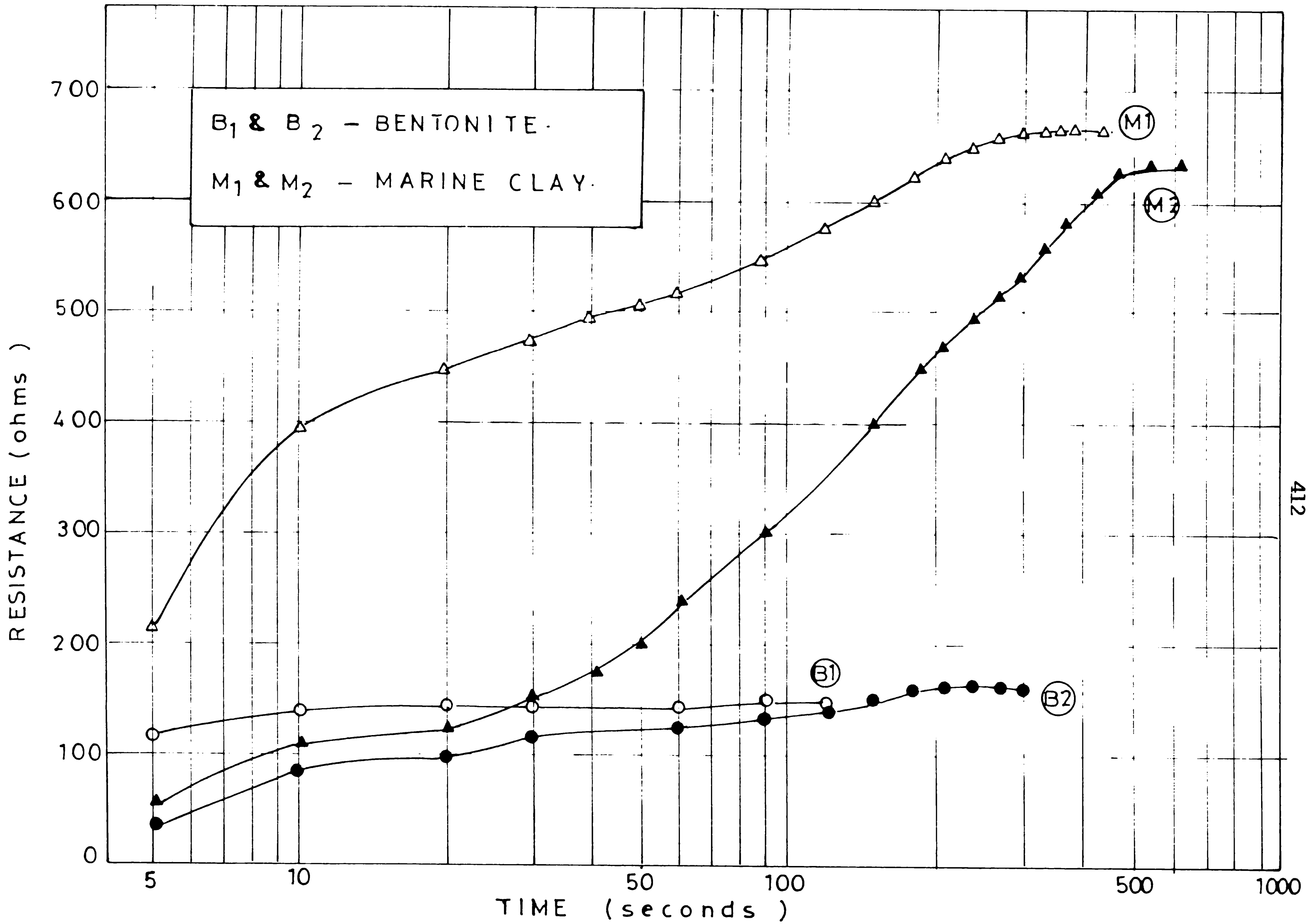

Fig. 7.3.1. RESISTANCE vs TIME.

These two experiments show that the resistivity measurements of soils treating it as a conducting medium has serious limitations. The change in resistance with respect to time is due to the polarisation of water molecules caused by the direct current. As the current flows through this medium, resistance is built up. When the direction is reversed, the water molecules being a dipole take some time to reorient themselves. Therefore the initial resistance on reversal of direction of flow of current is very low. As the material gets polarised and reoriented the resistance increases. The time taken by the material for coming back to normal orientation by itself is called the relaxation time. When current is passed in the opposite direction the relaxation time is cut down. These observations clearly indicate that soil should preferably be treated as a dielectric medium than a conducting medium.

The effects of polarisation is brought out by two cycles of direct current flow in Fig.7.3.2. When direct current was applied, a plot between resistance and time was obtained as curve 1. The initial reading was 180 ohms and it reached the maximum value of 445 ohms in 60 seconds. When the current was reversed, the resistance readings with respect to time gave curve 2. Immediately after the reversal, the resistance was as low as 120 and it reached 440 ohms

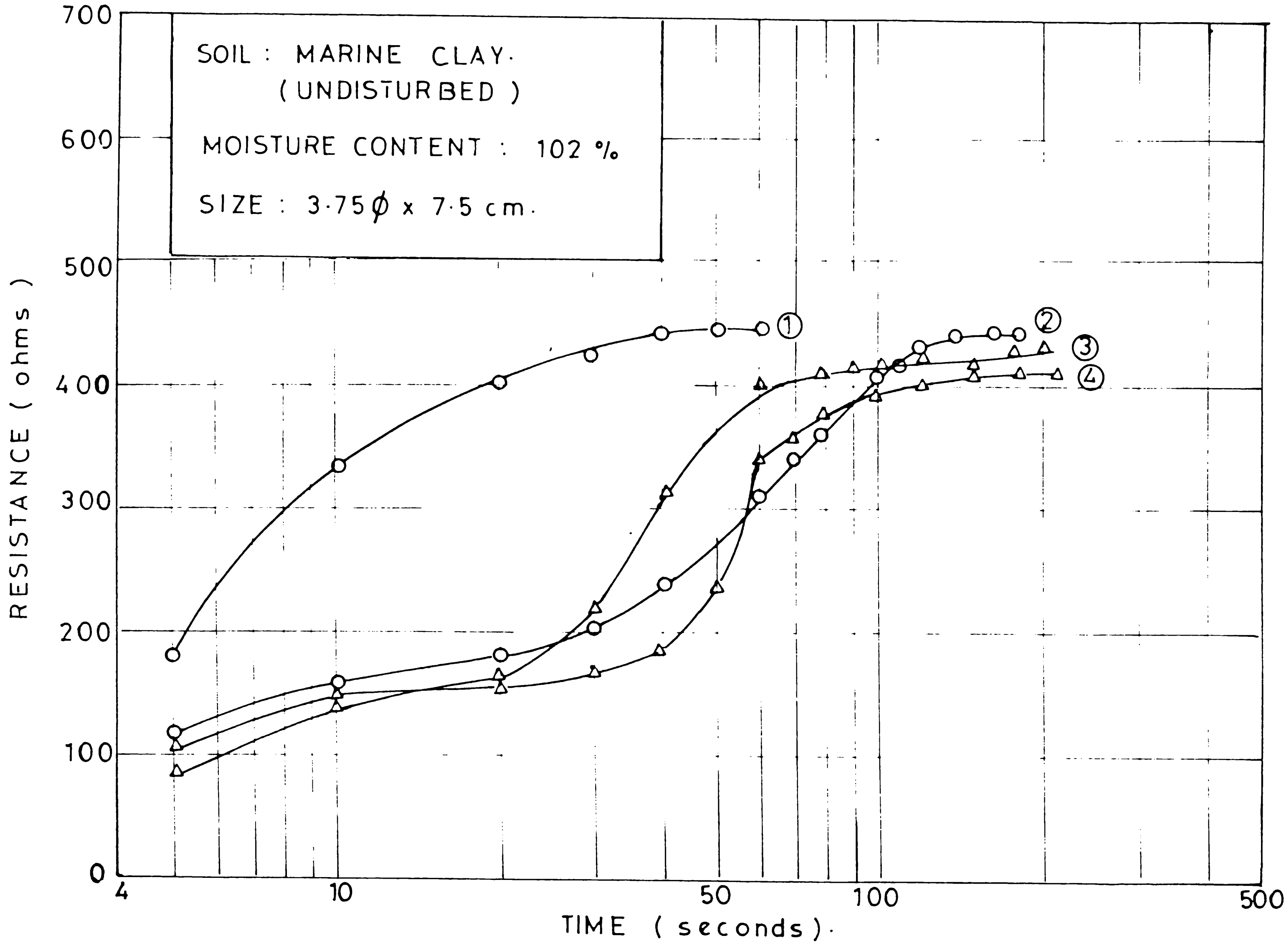

Fig. 7.3.2 : RESISTANCE vs TIME.

in 180 seconds. When the current was again reversed plot 3 was obtained and on the fourth test, reversing the flow again, curve 4 was obtained on plotting the readings. This clearly shows that soil is a dielectric medium which can be repeatedly polarised in either direction. These tests indicate that the resistivity measurements and the correlations based on them are not unquestionable due to the anomaly in the basic assumption that soil can be treated as a conducting medium. Hence the possibility of making use of the dielectric medium for interpretation of geotechnical properties of Cochin marine clays need to be explored.

Whenever there is flow of current in one direction water molecules get oriented in the direction of the field. This is called dielectric polarisation. Since the material here is loosely bound, these polarised dipoles get reoriented according to its relaxation time. This reorientation can be attained faster if a reverse field is applied. At this time it results in a large displacement current and hence a reduction in the resistivity. This explains the phenomenon observed. The relaxation time is inversely proportional to dipole moment.

7.4 RELAXATION TIME AND DIELECTRIC CONSTANT OF PORE FLUID

It has already been shown that resistivity by d.c. measurements are not very reliable as they are dependent on the polarisation of the pore fluid contained in the soil. Since polarisation is influenced by dipole moment of the pore fluid, the latter can influence the resistivity values.

To examine this possibility, two specimens were prepared by mixing carbon tetrachloride and water with oven dried marine clay. Since carbon tetrachloride has zero dipole moment, the resistance should show a reading which is not time dependent. Since water has a dipole moment equal to 1.84 debyes, it should show a relaxation time.

Fig.7.4.1 shows the resistance of a sample of oven dried marine clay mixed with 10% of carbon tetrachloride. The dimensions of the sample were 4 cm x 4 cm x 7.8 cm. When the direct current was passed for the first time, plot F_1 was obtained. It showed a steady value equal to 80 kilo-ohms immediately. When the supply was reversed the resistance showed a value of 61.5 kilo-ohms. When the supply was reversed again, the resistance was found to be 51 kilo-ohms and the resistance was found to be only 41 kilo-ohms when the direction was changed again.

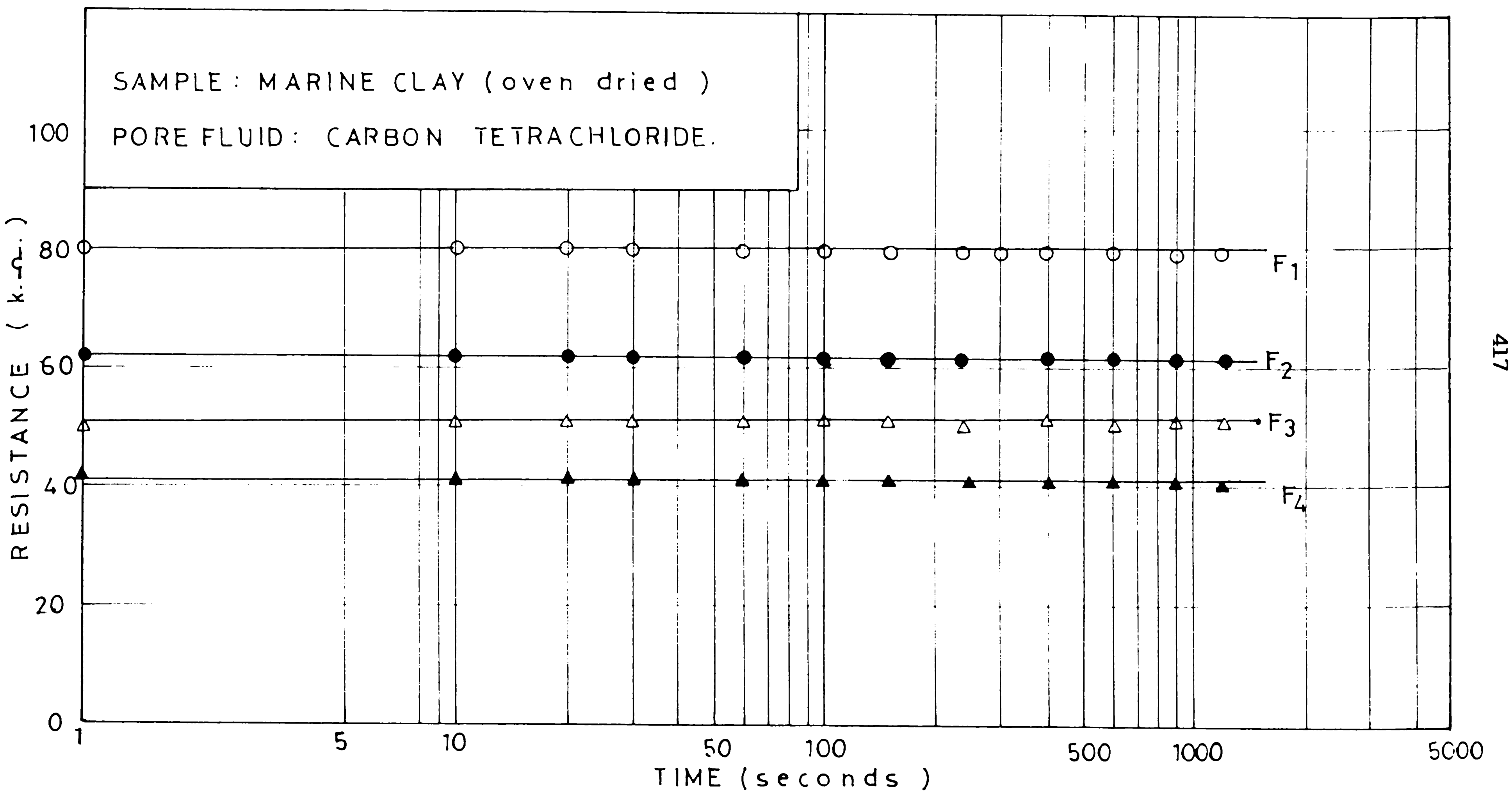

Fig. 7. 4. 1. RESISTANCE vs TIME.

As expected there was no relaxation time in any of the four tests. Fig.7.4.2 shows the resistance vs. time for three samples with the same dimensions. The first sample was prepared by adding carbon tetrachloride and, as noted earlier, the reading showed an unchanged resistance with respect to time as shown by curve 1. When current was passed through an oven dried marine clay with a water content of 20%, the initial reading was 360 kilo-ohms. The value steadily increased and reached the maximum value in about 180 seconds. Curve 3 shows the resistance vs. time plot for a sample of oven dried marine clay with a water content of 40%. The initial reading was 55 kilo-ohms which increased to the maximum value of 200 kilo-ohm in 600 seconds. These curves show that the polarisation is caused by the dipole moment of the pore fluid. Further, the figure also shows that the relaxation time is higher for a sample with higher water content.

Fig.7.4.3 shows the plot between resistance and time in seconds for a sample with water content of 20%. When the current was passed in one direction, the variation of resistance with time is given by plot F_1. When the current was reversed the plot between resistance and time curve R_1 was obtained which showed a higher maximum value.

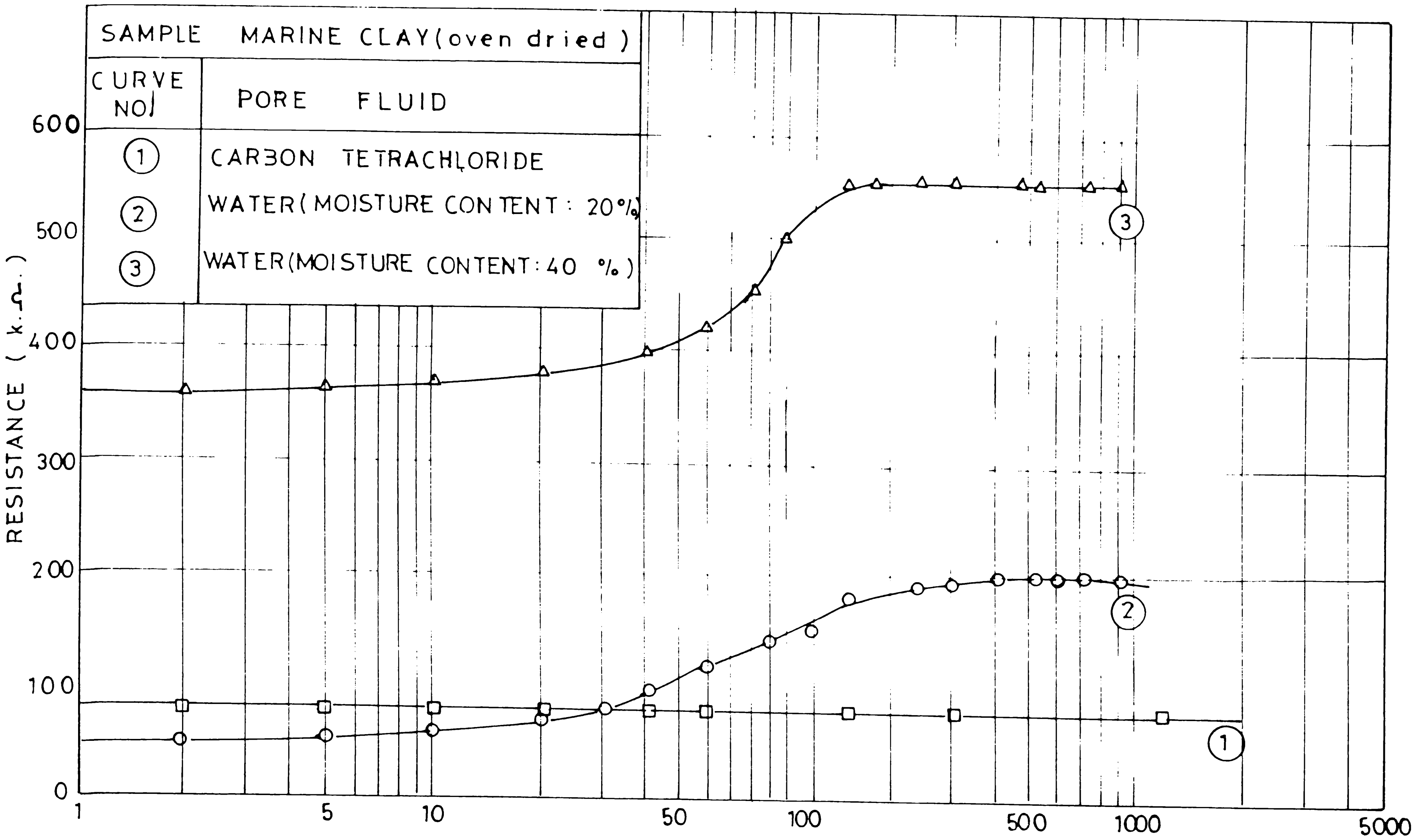

Fig 7.4.2. RESISTANCE vs TIME.

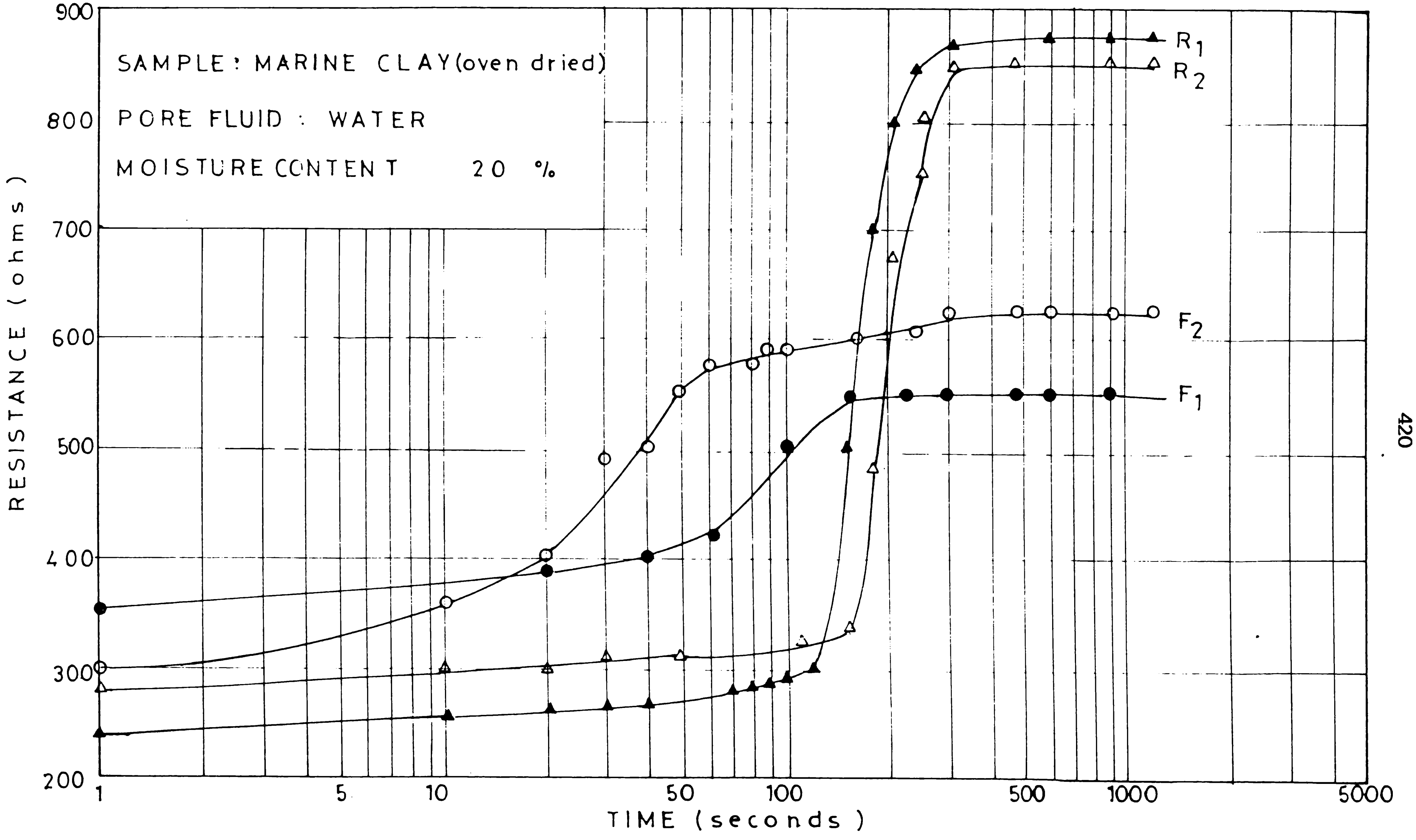

Fig. 7.4.3. RESISTANCE vs TIME.

When the current was reversed again, which was the first direction of flow of current, curve F_2 is obtained. When the current was again reversed, plot R_2 is obtained. It may be noted that the time to reach the maximum value of resistance either in the forward direction or in the reverse direction is more or less unique.

Eventhough polarisation has been pointed out as a drawback of the resistivity measurements using direct current, this property perhaps could be made use of for determining the water content of soil samples. As resistivity values are influenced by both dry density and water content, it is difficult to establish a correlation with any one of the parameters independently with the prevailing practice. But the relaxation time is dependent only on water content and this could perhaps be developed and perfected as a quick method for determination of water content especially in field control of compaction.

7.5 RESISTIVITY AS AN INDICATOR OF LIME-SOIL REACTION

Resistivity measurements of lime treated specimens show spectacular increase in resistivity values for a short spell after mixing with lime. Fig.7.5.1 shows the resistivity of three lime treated specimens of Cochin marine clay from Parur from a depth of 8 metres, treated with 6% lime. The samples prepared for unconfined compression strength had a diameter of 38 mm and height of 75 mm. The resistance of the specimens was measured with the help of a multimeter as indicated in section 7.2 The values of resistivity measured for 20 days from the date of mixing are plotted in Fig.7.5.1. Resistivity of all the three specimens sharply increases initially and comes to a more or less regular value by the 10th day.

Lime stabilisation is a process by which the Na^+ and K^+ ions are replaced by Ca^{++} ions. The affinity of a clay for reaction with lime is called the lime reactivity of the clay. The lime reactivity is a measure of the potential of the soil to react with lime and the gain in strength is proportional to lime reactivity.

Lime reactivity is defined by Thompson (1966) as the difference between the maximum compressive strength of the lime treated soil with the strength of the untreated soil. A better definition for the term would have been the ratio of the maximum unconfined compressive strength or laboratory vane shear strength of the treated sample to that of the untreated sample.

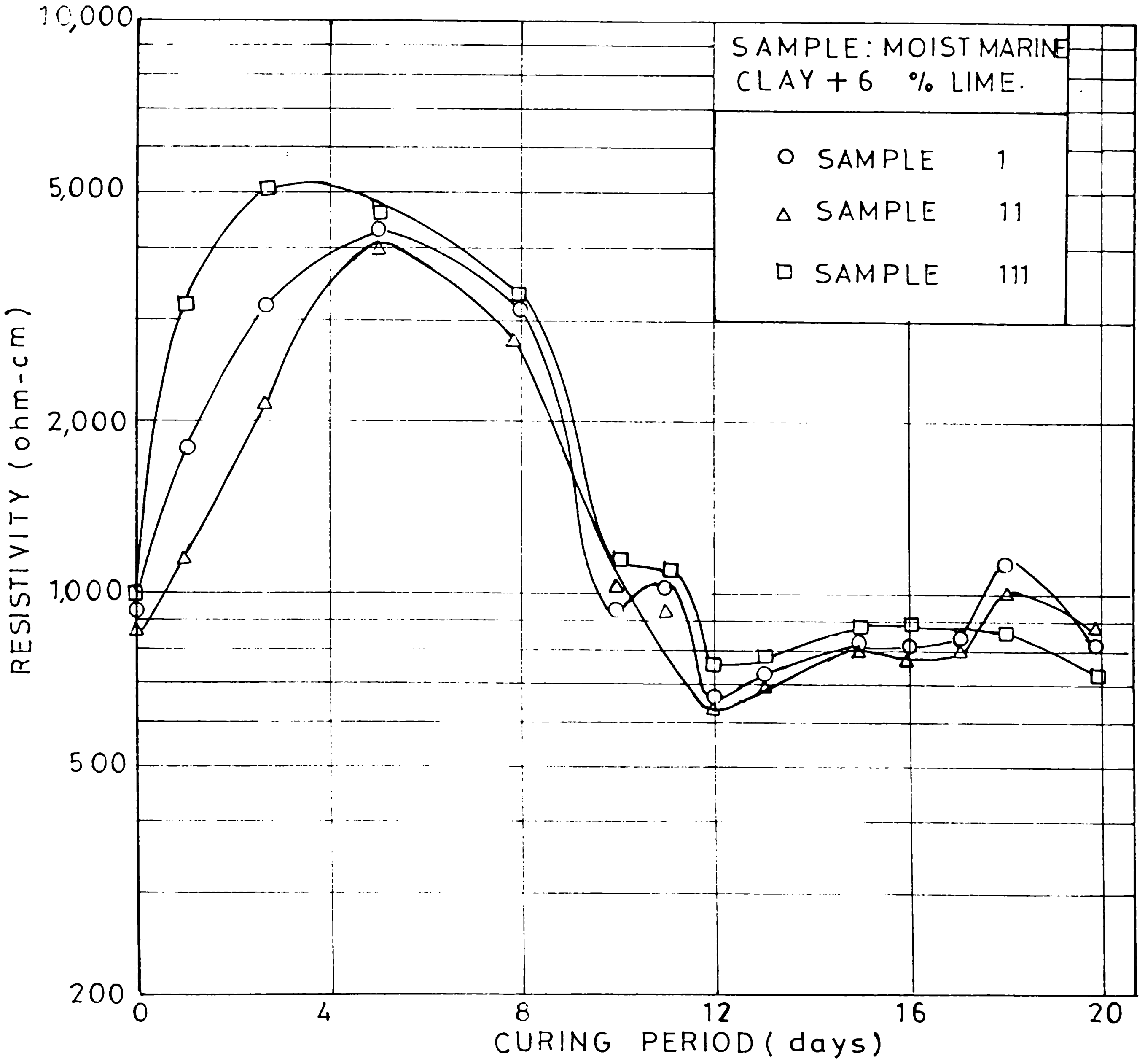

Fig · 7.5.1 RESISTIVITY vs CURING PERIOD.

At present the lime reactivity of a soil can be determined only by the destructive test of a series of treated samples, varying parameters like lime percent and curing period. Since we have to allow sufficient time for curing the specimens, the series of tests will take considerable time. Instead, resistivity measurements can be adopted for assessing the potential for lime to react with soil.

It can be seen from the figure that the resistivity of all the samples was around 900 ohm-cm initially. It shoots upto even 5000 ohm-cm and drops to steady values by the 10th day. The variation in resistivity for the last 10 days is marginal. The sudden variation in resistivity is due to the complex chemical reactions between lime and soil and hence this can be utilised to measure the lime reactivity of soils. It has the added advantage that it is a simple non-destructive test which effects considerable saving in the number of samples to be prepared.

Fig.7.5.2 shows the resistivity - curing period relationship for two samples of marine clay treated with 3% lime. The samples were 7.5 cm long and 3.8 cm in diameter. As in the previous case the value of resistivity sharply increases on the first day and comes back to steady values after nine days.

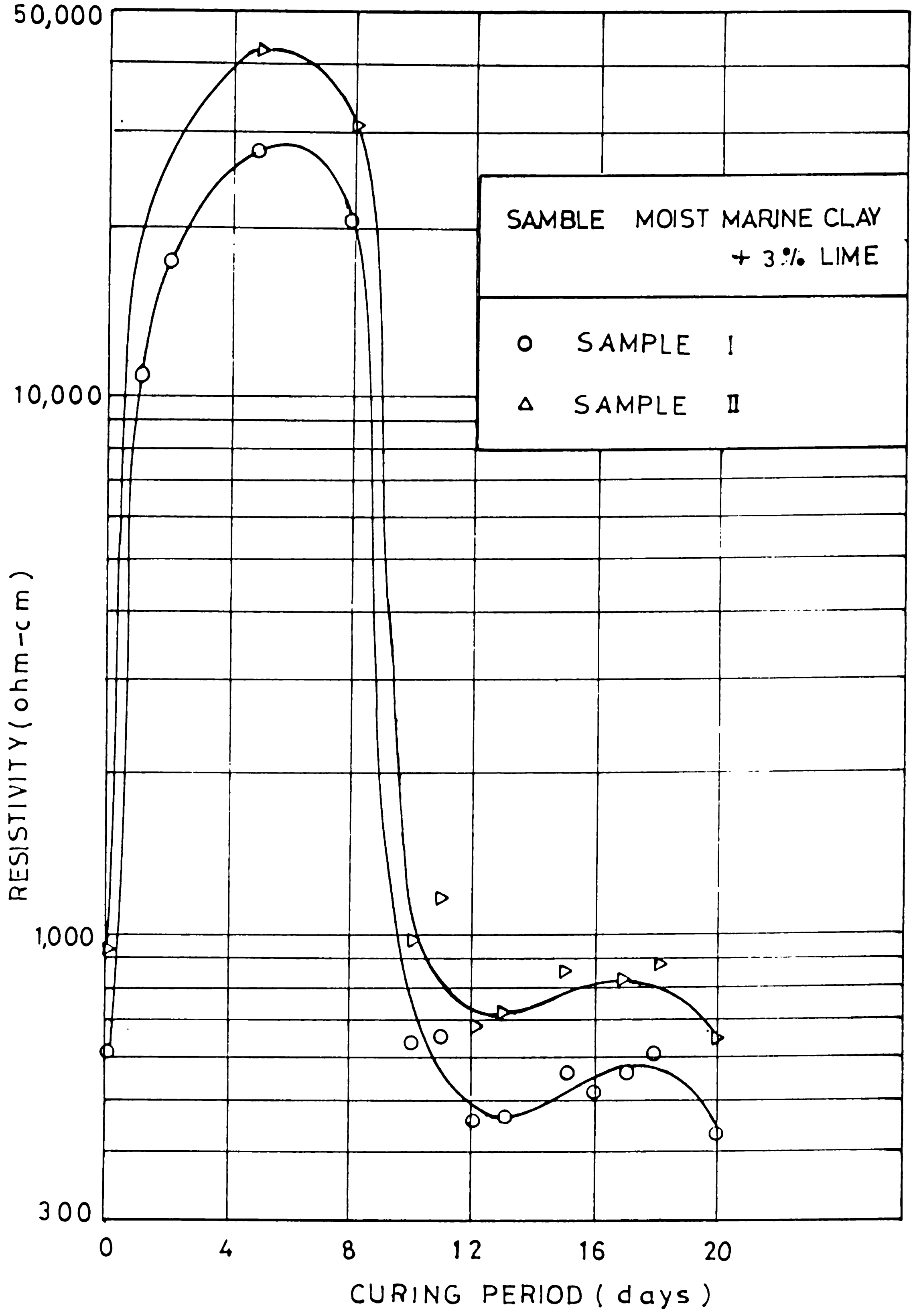

Fig. 7.5.2 : RESISTIVITY vs CURING PERIOD.

7.6 RELATION BETWEEN RESISTIVITY AND SHEAR STRENGTH

Resistivity measurements, if perfected by detailed investigations, can be of great help in the determination of geotechnical properties. The greatest advantage of the resistivity method is that it is a non-destructive testing method, which can make considerable saving in the number of samples used for strength studies. To explore this possibility, attempts were made to study the development of shear strength of lime treated specimens.

Moist soil from Parur was treated with 6% lime and the samples of 75 mm length and 38 mm diameter were prepared and kept for curing. Three samples were taken at regular intervals and their resistivity was measured and shear strength determined. Fig.7.6.1 shows the plot between the resistivity of the samples and shear strength. For the first few days the samples showed very high resistivity and a regular correlation could be obtained after a few days of curing.

It can be seen from the graph that the resistivity decreases or conductivity increases with strength. This is quite understandable as the conductivity can increase

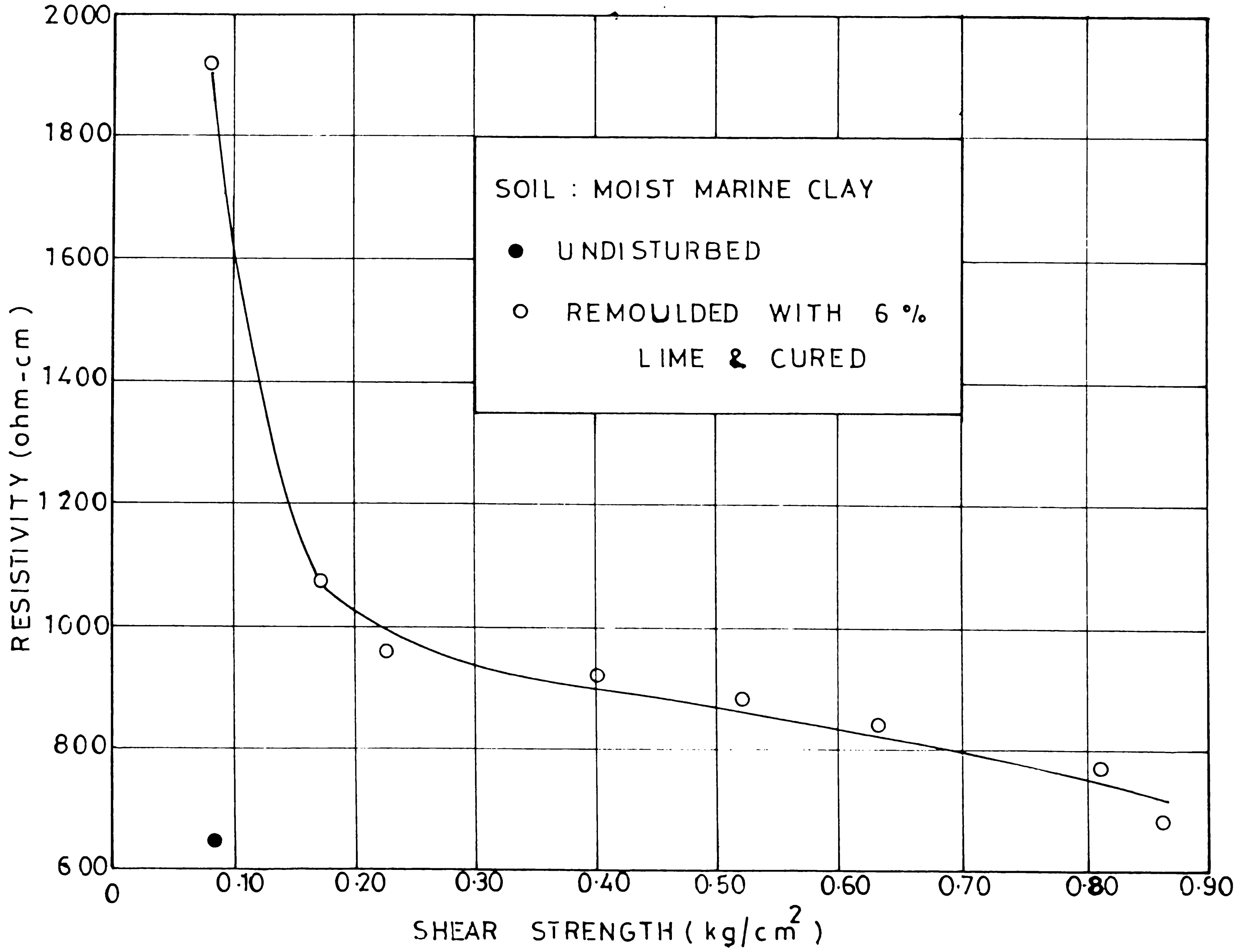

Fig. 7.6.1 RESISTIVITY vs SHEAR STRENGTH.

as more and more bonds are developed within the soil mass. Since the study was not persued further, firm conclusions cannot be drawn at this stage. But there is considerable scope for further work in this direction.

7.7 RESISTIVITY BY A.C. MEASUREMENTS

It has already been pointed out that the measurement of resistivity with a multimeter using direct current has certain inherent drawbacks due to polarisation. The reading itself takes some time to stabilise and the resistivity in the opposite direction is very low initially and takes considerable time to reach a steady reading. This value will normally be at considerable variance with the original resistivity measured. Thus the resistivity measurements can be considerably influenced by the passage of direct current prior to the test as the water particles with dipole moment might have oriented themselves in such a way to increase the value of resistivity or decrease it.

The variation in the resistivity value due to d.c. measurements can be avoided by measuring the resistivity against alternating current. Due to the frequency of oscillation, there will not be any polarisation and consequent variation in resistivity.

To study the advantages of resistivity measurements using a.c. and to compare the results of both measurements on the same sample, a circuit was designed where both a.c. and d.c. could be passed through the soil and resistivity could be measured. Fig.7.7.1 shows the circuit adopted.

Soil sam ple was taken in a container with a square base of 6 cm x 6 cm and a height of 6 cm. Both ends were covered with copper plates. To ensure the contact between the top copper plate and soil, the plate was loaded by applying a contact pressure of 20 gm/cm^2. The resistance of the sample was found out by measuring the voltage across the end plates and the current passing through the sample.

By Ohm's law

$$R = \frac{V}{I} \tag{7.7.1}$$

where

R - resistance of specimen (ohms)
V - voltage (volts)
I - current (amperes)

$$\text{Resistivity} = \frac{R.A.}{L}$$

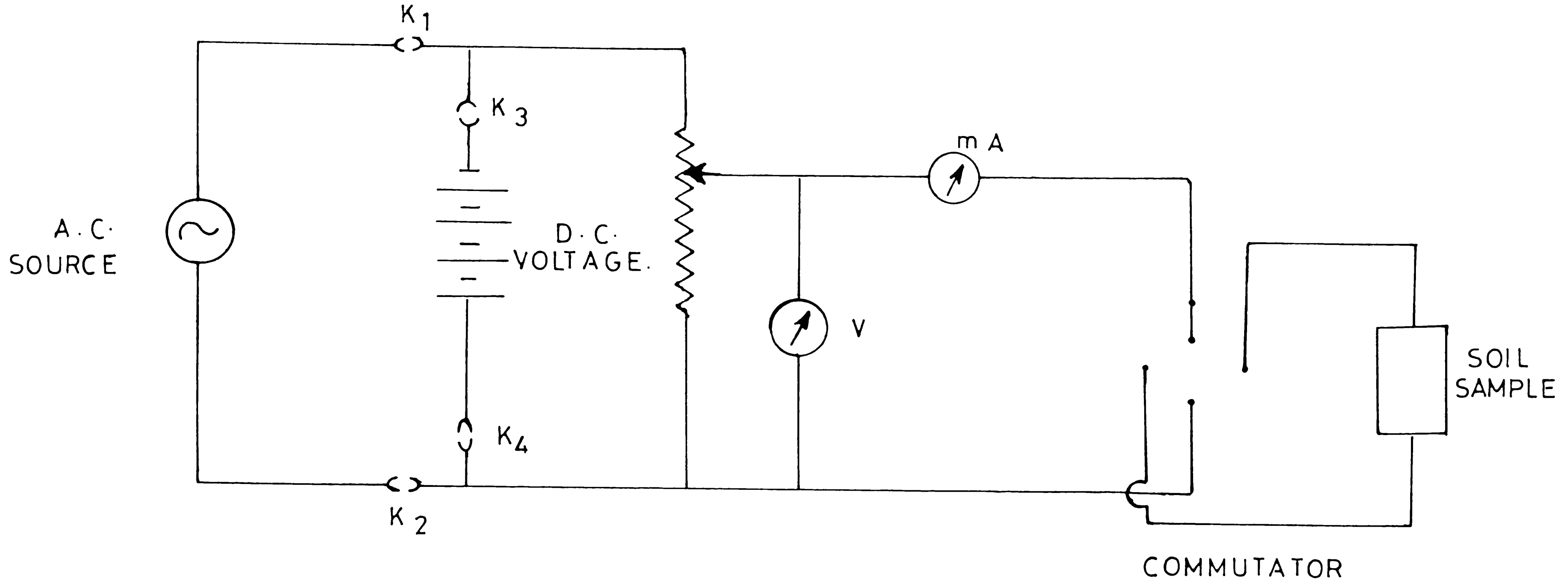

Fig. 7.7.1 : CIRCUIT DIAGRAM.

The results of a typical test is given in Fig. 7.7.2. Resistance against a.c. supply was found to be a constant equal to 11.32 kΩ cm for a voltage of 8 V. The supply was then changed to d.c. using a commutator and the resistance was measured taking readings every 15 seconds, for five minutes. The resistance values were found to be steady by this time. The plot F_1 in Fig.7.7.2 shows the variation of resistivity with respect to time. The direction of the d.c. was then reversed and plot R_1 represents the resistivity values for a duration of five minutes. The resistivity was found to increase marginally. When the direction was reversed again, the value was very low initially. But it steadily increased to about three times the original value. Curve F_2 shows these variations. Curve R_2 is the resistivity time plot obtained when the direction was reversed. Curves F_3 and R_3 are the plots of resistivity values obtained while changing the direction of the direct current.

These curves show that there could be wide variations in the value of resistivity measured by direct current whereas the a.c. measurement gave very steady values. Since polarisation influences the values significantly, resistivity measurements using alternating current will be preferable.

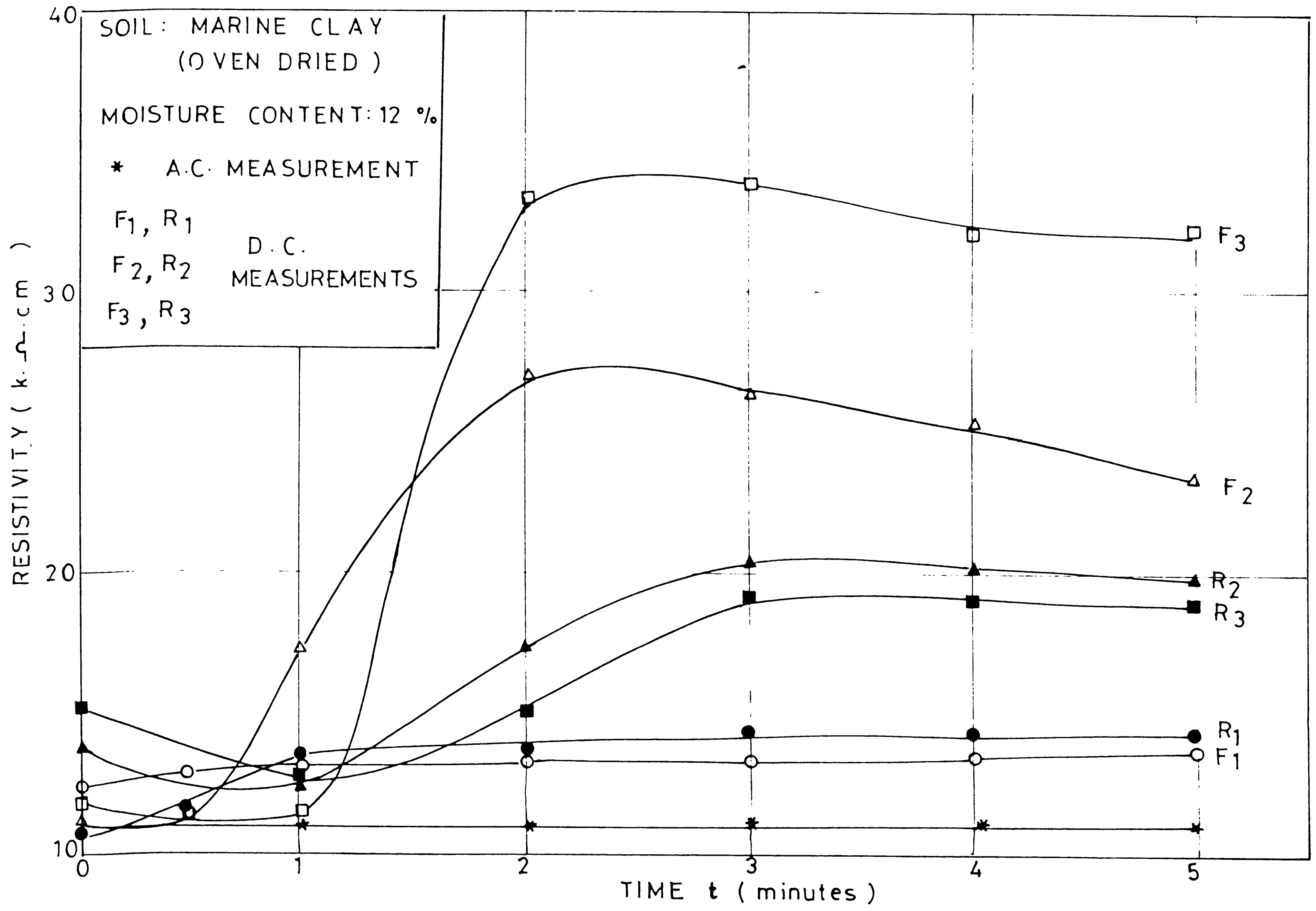

Fig. 7.7.2 : RESISTIVITY vs TIME

Chapter VIII

CONCLUSIONS

Based on the detailed investigations carried out in this thesis, the following major conclusions have been arrived at.

Marine clays, well known for its high compressibility and poor shear strength, cover extensive areas in Greater Cochin and these clays are of the same geological origin as indicated by the similar results obtained from sam ples collected from locations separated by as much as fifty kilometres.

Extreme care has to be taken in the preparation of marine clay specimens for different physical and engineering properties. Since Cochin marine clays are susceptible for significant changes in test results depending upon their initial condition, it must be stated that all the properties need to be investigated under natural moist conditions.

Drying before carrying out any test either in the oven or under atmospheric conditions will significantly affect the properties. For example liquid limit and clay content reduce significantly by drying. While plastic limit and free swell index decrease upon drying, shrinkage limit marginally increases. These changes are attributed to aggregation of particles on drying.

Since the initial conditions of the sample significantly influence the test results, initial conditions need to be specified for carrying out Atterberg limits, shrinkage limit, free swell index and grain size analysis tests. Suitable amendments should be incorporated in the I.S.Codes so that marine clays receive special attention during preparation of samples.

Since all the clay fraction in a marine soil are likely to exist in the form of flocs, deflocculation is more important compared to other clayey soils. Tests with eight different deflocculating agents showed that the dispersing agent (Sodium hexametaphosphate + Sodium carbonate) recommended by I.S. Code is the most suitable one for Cochin marine clays also.

While determining the value of coefficient of consolidation C_v, it was found that the rectangular hyperbola method is the most versatile. There were instances where both Taylor's and Casagrande's methods could not be used whereas rectangular hyperbola method could be used without difficulty.

Careful examination of X-ray diffraction patterns indicated that the primary clay mineral in Cochin marine clays is montmorillonite. Some secondary peaks show the presence of traces of kaolinite also.

Air and over drying prior to testing brought about notable changes in the physical properties of Cochin marine clays. Upon oven drying, the liquid limit reduced by 42-58%, plastic limit by 12-30% and free swell index by 65-77%. The clay content reduced by 9-33% on air drying and 45-55% by oven drying.

The following useful correlations were obtained between index properties for 'moist' samples of Cochin marine clay.

$$PI = 0.62(LL - 2.0) \qquad r = 0.92$$

$$PL = 0.473(I_f + 65.9) \qquad r = 0.80$$

$$LL = 3.40(C - 10) \quad r = 0.83$$

$$SL = 31.1 - 0.276\,C \quad r = 0.83$$

$$\text{Free Swell} = 0.12(C - 1.2) \quad r = 0.89$$

$$\text{Free Swell} = 0.043(LL - 1.7) \quad r = 0.89$$

where

PI = Plasticity Index
PL = Plastic Limit
LL = Liquid Limit
SL = Shrinkage Limit

Similar correlations have been obtained for air dried, and oven dried soils and for all three types of samples put together.

Investigations on washed samples indicated that reductions in salt content increases the liquid limit and plastic limit marginally. Clay content and free swell index also increase to a limited extent.

Soaking the sample for about two years showed that the Atterberg limits increase with soaking. But the liquid limit of oven dried marine clay could increase only to 62% of its original liquid limit of moist clay. A similar increase was noted in case of plastic limit also. But the increase in free swell index was more appreciable.

A very useful correlation could be established between shear strength and natural water content of Cochin marine clays which could be extended to other soils also. Vane shear strength of Cochin marine clays at liquid limit water content was 14 to 19 gm/cm^2. A unique relationship as shown below could be obtained between shear strength and water content ratio (ratio of natural moisture content and liquid limit) of Cochin marine clays.

$$WCR = 0.231 - 0.414 \log \tau$$

$$r = 0.987$$

This relation for clays of same geological origin will be useful as the shear strength can be estimated from natural water content and liquid limit without going in for time consuming shear tests. Similar semi-logarithmic relations could be obtained for other soils also. The slope of the lines can give more information about the behaviour of the soils as kaolinite soils have a flatter slope than montmorillonite soils.

Dielectric constant of the pore fluid influences the liquid limit and free swell index values of the soil. For montmorillonitic clays the liquid limit and free swell index increase with the increase in dielectric constant. Kaolinite clays show an opposite behaviour. Oven dried marine clays show a behaviour similar to montmorillonitic clay eventhough it is very much subdued as the soil has been oven dried.

The long series of consolidation tests helped to bring out the compressibility characteristics of Cochin marine clays. As test results, especially those taken from greater depths, indicated that the preconsolidation pressure obtained was consistent with depth, the Cochin marine clays can be classified as normally consolidated clays. It has been found that when clays of different plasticity are subjected to consolidation tests, the e - log p curves arrange themselves in the order of their liquid limits.

It has been shown that initial drying of soil improves the compressibility characteristics of marine clays considerably. The value of compression index C_c or the value de/d(log p) which is defined as the slope of the e - log p curve for any particular pressure increment, is brought down by initial drying. The value of de/d(log p) got reduced to around 50-55% for air dried soils and 35-40% for oven dried soils.

Correlations have been obtained between de/d(log p) and liquid limit values of undisturbed and remoulded specimens for two different pressure increments eventhough the correlation coefficients are not of a high order. The values of de/d(log p) are about 20-25% higher than those suggested by Terzaghi (1948) and Skempton (1944) for undisturbed and remoulded clays respectively. This is attributed to the compression of the flocs.

The liquidity index for these clays varies over a range of 0.46 to 0.87 indicating that these clays can be taken as medium to very sensitive as per Bjerrum's classification (Bjerrum, 1954).

Many workers have suggested that $C_c/1+e_o$, called the compression ratio, can represent the compressibility characteristics of clays better, compared to C_c alone. The present results also indicate that compression ratio varies linearly with log p. The value of de/d(log p) decreases as duration increases.

The C_c values obtained from consolidation tests for Cochin marine clays, around 1.5, is quite high and total settlements computed based on this value are considerably higher than the actual settlements realised in the field. It has been shown that this is due to an overestimation of C_c by I.S. method. A new term called segmental compression index, C_c' defined as the slope of the segment of the e - log p portion between the abscissa p_o and $p_o + \Delta p$, gives more realistic results from the equation for settlement. Another advantage with segmental compression index method is that a settlement of over-consolidated clays can be determined in the same way whereas the method suggested by I.S. Code is cumbersome and time consuming.

The resistance to compressibility was found to increase with duration of the load increment. Reserve strength was found to develop with secondary consolidation.

For the same pressure, especially at higher pressures, e - log p curves with longer duration show higher equilibrium void ratio, indicating development of resistance to compressibility under long duration of constant stress. At lower pressures the effect of duration is marginal.

When duration increases, the ratio of secondary compression to total compression increases as expected.

The rebound curves obtained during unloading to seating load, follow a regular pattern wherein the test with longest duration shows the minimum rebound and the one with shortest duration occupies the highest position. This might be the result of stronger bonds developed during consolidation for longer duration.

One of the major findings from the studies on consolidation is the development of a new method for determination of p_c, called the log - log method. Casagrande's method for p_c involves personal judgement of point of maximum curvature. The other two methods--Burmister's and Schmertman's--are cumbersome and time consuming. Compared to these three, the log - log method, where value of p_c is obtained from the intersection of two straight lines drawn through the initial and final sets of points plotted in log - log scale, is far simpler and more accurate. The success of the proposed method lies in the development of a technique to obtain the actual value of p_c for purpose of comparison with the value obtained from log - log method.

The significant improvement in compressibility characteristics due to air drying can be taken advantage of in the construction of embankments and formation of cuttings and subbases. The shear strength also improves by air drying and this provides a cheap method to improve the geotechnical properties of marine clays. For Cochin marine clays the optimum moisture content is around 30% and maximum dry density obtained is 1.33 gm/cc.

Studies on the compressibility behaviour of oven dried marine clays using pore fluids of varying dielectric constant indicated certain interesting features in case of oven dried samples. Attractive and repulsive forces exist at microlevel and the former controls the compressibility of kaolinite while the latter is responsible for the resistance to compressibility offered by montmorillonite. Consolidation tests with different pore fluids indicate that oven dried marine clay sample behaves like a montmorillonitic clay during consolidation till 0.5 kg/cm^2 and like a kaolinitic clay for pressures above 0.5 kg/cm^2.

After having studied the physical and consolidation characteristics of Cochin marine clays, successful attempts were made to improve their engineering properties. Out of eighteen different additives tried, lime was found to be the most potential stabilising agent. Shear strength tests on samples treated with different percentages of lime showed that 6% was the optimum lime content. Within one weak of curing the shear strength improved ten fold and after a month it improved by about eighteen times.

Physical properties of marine clays were found to improve considerably by lime treatment. The liquid limit was found to behave a little erratically but plastic limit was found to increase from 56.5% to 83% after one week. The reduction in plasticity index makes the samples less compressible but more friable. Test results on physical properties agree with the findings of Sherwood (1967) that only small percentages of lime are required to bring these changes in physical properties.

The shrinkage limits tend to increase with lime content which could be attributed to flocculation.

The free swell index increases from 6.4 to 8.3 cc/gm on mixing with lime. But on curing, the value slowly decreases.

Through a series of consolidation tests, it has been verified whether the phenomenal gain in strength due to lime treatment is equally valid for the compressibility behaviour of Cochin marine clays. The compression curves for treated soils are above that for untreated soil. They arrange themselves with the specimen with maximum period of curing, occupying the highest position.

Due to the development of bond strength which will be seen as a preconsolidation pressure p_c in the e - log p curves, the compression at lower pressures will be ¼ to 1/3 of the actual compression without the additives. But once the applied pressure crosses p_c, the compression of treated soil will be about 80% of values for untreated soils at higher pressures. Thus consolidation can be considerably reduced by keeping the maximum applied pressure lower than the p_c value developed.

The consolidation curves also show that the treated soil present a brittle behaviour while the untreated soil fail gradually.. For specimens cured for longer periods, point of maximum curvature in the e - log p curve gets shifted to the right indicating the development of bond strength with period of curing.

It has been shown by both laboratory vane shear tests and consolidation tests that the rate of development of bond strength in treated soil is inversely proportional to the lime content. Eventhough higher lime contents will show higher maximum strength, the lower lime contents will show a faster rate of development of strength.

The value of de/d(log p) decreases with increasing curing period and becomes steady after about 80 days of curing. For a specified curing period and stress level, there is a unique lime content which gives the lowest value for de/d(log p). When the curing period is short, lower lime contents will show lower de/d(log p) values as they have a higher rate for development of shear strength. But for longer durations de/d(log p) is inversely proportional to lime content.

Coefficient of consolidation, C_v increases with curing period for all lime contents. But for a specific curing period and stress range, there is an optimum lime content which will give the maximum value for C_v. The reduction in C_v with higher lime content may be explained by clogging of voids by the reaction products formed. The secondary compression coefficient, C_α decreases with increase in lime content and curing period.

For a given curing period, there is an optimum lime content which will give maximum bond strength. Both the optimum value of lime content and corresponding maximum bond strength increases with curing period.

The lime reactivity of black cotton soil is higher than that of Cochin marine clays.

Unlike untreated marine clays, the bond strength does not increase with longer durations allowed for each load increment. Bond strength does not develop under constant strain. But under sustained pressure there is an increase in preconsolidation pressure or bond strength proportional to the sustained period.

Eventhough electrical resistivity surveys are very popular as a geophysical method, resistivity measurements have not been fully exploited by geotechnical engineers in spite of the fact that it is a quick and inexpensive method. Since in geoprospecting the measurements give only the weighted average resistivity of the different layers, soil profiling can be done only through comparison with absolute resistivity values of the individual layers. Hence laboratory measurement of absolute resistivity is important.

Laboratory measurements of resistivity is done with the help of a multimeter. Satisfactory correlations have been obtained for water content and dry density. As expected, the resistivity reduces with both water content and dry density. The reduction is faster with moisture content.

The limited literature on resistivity measurements in the laboratory shows that there are no standards prescribed for the soil specimens used for measurements. But this is important as the sample dimensions influence the resistivity values considerably. From a series of tests, it has been shown that resistivity reduces with length of the sample, while it increases with area of cross section of sample. Thus the need for standardisation of the test specimen has been brought out, so that the data generated can be compared with other data. It is recommended that a standard size of 37.5 mm in diameter and 75 mm in length may be adopted.

It has been shown that the resistivity of a soil sample is also a function of its pore fluid. Using four pore fluids viz. carbon tetrachloride, acetone, methanol and water which possess a wide range of values of dielectric constants, it has been brought out that the resistivity reduces with fluid content in case of acetone, methanol and water. But with carbon tetrachloride the resistivity was found to increase.

Literature shows that the resistivity measurements in the laboratory are done using the equation

$$\rho = \frac{R.A.}{L}$$

But resistivity obtained from this equation is not a unique value. This may be due to the fact that soil is not a conducting material but a particulate medium.

Studies on the characteristics of the soil treating it as a dielectric medium will be more meaningful and the resistivity measurements using the above equation have serious limitations.

Through a series of time vs. resistance plots, it has been shown that resistivity measurements are considerably influenced by polarisation of water molecules caused by the direct current. The resistance to flow of current in either direction is also different from each other. The time taken by the molecules to orient themselves is called the relaxation time and it is inversely proportional to the dipole moment of the pore fluid.

For pore fluids with zero dipole moment, the relaxation time is zero. Hence for samples with carbon tetrachloride as pore fluid, the resistivity values instantaneously show the maximum value of resistance without any time lag. Samples with other pore fluids take some time to show the maximum value of resistance. Eventhough polarisation or relaxation interferes with the resistivity measurements with direct current, the fact that relaxation time is proportional to the fluid content can possibly be made use of for water content measurements.

When lime is mixed with moist marine clay, the chemical reactivity generated indicates the possible increase in strength that can be gained by lime stabilisation. The intensity of the reaction between lime and soil can be measured by the resistivity which gives some peak values during the initial period.

Similarly in case of lime stabilised soils, bonds are developed during curing and this reduces the resistivity. Instead of determining the shear strength by destructive testing of a large number of specimens, it might be possible to monitor the gain in strength by measuring the resistivity.

Since polarisation, which interferes with resistivity measurements, is caused by direct current, it can be avoided by a.c. measurements as shown by the test results. The directionality of measurements is also avoided. Hence a.c. measurements will be more reliable and faster.

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APPENDIX II

LIST OF PUBLICATIONS FROM THESIS

I. PAPERS PUBLISHED

1. Babu T.Jose, Sridharan, A. and Benny Mathews Abraham. (1987). "A Study of Geotechnical Properties of Cochin Marine Clays", Marine Geotechnology, New York, Vol.7, No.3, pp.189-209.

2. Babu T.Jose, Sridharan, A. and Abraham, B.M. (1988). "Physical Properties of Cochin Marine Clays", Indian Geotechnical Journal, Vol.18, No.3, pp.226-244.

3. Babu T.Jose, Sridharan, A. and Abraham, B.M. (1989). "Log-log Method for Determination of Preconsolidation Pressure", ASTM Geotechnical Testing Journal, Vol.12, No.3, pp.230-237.

4. Babu T.Jose, Sridharan, A. and Abraham, B.M. (1987). "Engineering Properties of Cochin Marine Clays and its Stabilisation with Lime", 9th South East Asian Geotechnical Conference, Bangkok, Thailand, pp.5-115 to 5-126.

5. Babu T.Jose, Sridharan, A. and Punnoose, K.K. "Geotechnical Properties and Electrical Characteristics of Cochin Marine Clays", Institution of Engineers--National Convention of Civil Engineers--Annual Paper Meeting, Calcutta, Jan.18-20 (1990).

II. PAPERS COMMUNICATED

1. "Determination of Clay Size Fraction of Marine Clays", Communicated to ASTM Geotechnical Testing Journal, Philadelphia.

2. "Variability of Geotechnical Properties of Marine Clays due to Test Techniques", Communicated to Marine Geotechnology, New York.

3. "Low Cost Ground Improvement Techniques for Construction of Highways over Soft Marine Deposits", Communicated to Ground Engineering, London.

4. "Prediction of Shear Strength of Clayey Soils based on Natural Moisture Content", Communicated to Geotechnique, London.

5. "Prediction of Settlement of Foundation - The Need for a Relook into I.S.Code Provision", Communicated to Journal of Institution of Engineers, Calcutta.

6. "Improved Methods for Estimation of Preconsolidation Pressure", Communicated to Geotechnique, London.

7. "Resistivity Measurements in Geotechnical Engineering", Communicated to Indian Geotechnical Journal, New Delhi.