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Geotechnical properties of clay soils in a brine environment.

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GEOTECHNICAL PROPERTIES OF CLAY SOILS IN A BRINE ENVIRONMENT

by

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ABSTRACT

The geotechnical properties of nineteen clay soils from Eastern, Central and Southwestern Ontario were evaluated both in their natural and brine-treated states. The main focus was on the relationship of permeability to other geotechnical properties of the soils in the two states. In addition to permeability, grain size distribution, Atterberg limits, water adsorption at 98% relative humidity and optimum compaction parameters were determined. All permeabilities were determined at optimum compaction.

The results indicated that high and significant correlations exist between Atterberg limits and water adsorption, Atterberg limits and clay content, and the permeabilities of the treated soils to brine and the untreated soils to water. Lower correlation coefficients were, however, obtained between Atterberg limits and water adsorption for treated soil samples. The results also showed that geotechnical properties of the soils in the natural state were significantly different from those in the brine-treated state. The difference has been attributed to dessication shrinking of the clay, due to sodium cation exchange. The shrinking led to the formation of near-horizontal cracks

and hence a substantial increase in the permeability of the untreated soils to brine. For most of the soils used, the ratio of permeabilities of untreated soils to brine and water was approximately 100:1. The permeability to brine was significantly reduced by pre-treating and re-moulding the soils with brine. As long as leaching of salt did not occur, the treated soils maintained reasonably low permeabilities to both brine and water. Liquid limit was shown to be more influenced by brine than plastic limit. Brine-treated soil samples showed lower liquid limits than untreated samples. Plastic limits were not significantly reduced by brine treatment. The lower Atterberg limits was explained by an increase in interparticle attractions as a result of the suppression of the diffuse double layer. Water adsorption at 98% relative humidity was significantly increased after treatment of samples with brine. Cation hydration was probably responsible for the increase.

The study demonstrates the need to pre-treat clays to be used in the construction of clay-lined brine reservoirs. Pre-treatment can take the form of moulding the clay with brine during compaction. The low permeability of the treated soils to brine suggests that seepage of brine to the subsurface would be minimized.

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NOMENCLATURE

Dimensions are given in terms of mass (M), length (L), time (T), percentage given as %.

A	Cross-sectional area of the sample	L^2
a	Cross-sectional area of the standpipe	L^2
c_o	Concentration of cations in moles per litre	L^{-3}
D	Dielectric constant	Dimensionless
D_s	Effective particle diameter	L
e	Void ratio	Dimensionless
h_o	Initial head in standpipe	L
h_1	Final head in standpipe	L
k	Coefficient of permeability	LT^{-1}
k_o	Pore shape factor	Dimensionless
K	Specific or absolute permeability	L^2
L	Length of soil sample in permeameter	L
LL	Liquid limit	%
LI	Liquidity index	Dimensionless
ln	Natural logarithm	Dimensionless
n_c	Number of cations per unit volume at the midplane between particles	L^{-3}
n_o	Number of cations per unit volume in the pore water away from the influence of clay surface	L^{-3}
PI	Plasticity index	%
PL	Plastic limit	%

Q	Quantity of flow of permeant	L^3
r	Pearson correlation coefficient	Dimensionless
S	Specific surface area	L^2M^{-1}
s	Sample standard deviation	Depends on units used
T	Temperature	°K
t_o	Time when permeant in standpipe is at h_o	T
t_1	Time when permeant in standpipe is at h_1	T
V	Darcy velocity	LT^{-1}
v	Valency	Dimensionless
$\overline{X}$	Sample mean	Depends on units used
α	Level of significance	%
γ	Unit weight of permeant	$ML^{-2}\ T^{-2}$
μ	Viscosity of permeant	$ML^{-1}T^{-2}$
μ_i	Mean of population i	Depends on units used
ν	Number of degrees of freedom	Dimensionless
σ	Population standard deviation	Depends on units used

CHAPTER I

INTRODUCTION

1.1 The Present Problem

The use of surface reservoirs to store brine is common in the oil industry. Liquified petroleum gases such as propane, butane and ethane are stored in salt caverns. The caverns are formed by injection of fresh water into an underground salt formation and dissolving out the salt formation. The resulting brine solution is returned to the surface and retained in reservoirs, once the caverns are completed. Later on, when the liquified petroleum gases are needed for use, the brine is used to displace them from the storage caverns. It is desirable that enlargement of the caverns be avoided; hence the use of brine as the displacement fluid.

Most brine storage reservoirs in Alberta and elsewhere utilize clay liners, because clay is usually the least expensive material near the site. Many petroleum companies have had costly leakage problems with brine ponds constructed from untreated clay. The present study was prompted by one such problem with a brine pond at Redwater, northeast of Edmonton, belonging to Imperial Oil Limited. The pond was constructed from glacial till consisting of some boulders in a clayey matrix. A few months after the

construction of the pond, monitoring of wells revealed that brine had leaked into the aquifer below, resulting in the contamination of the groundwater. The lining which was originally 4 feet thick was re-compacted and the thickness increased to 6 feet. The pond leaked again, six months after re-working.

Personal communication with Dr. Hudec at the University of Windsor, has indicated that in 1979 he measured the temperature of the brine in the pond. The result showed a surface layer of low salinity at 4°C, overlying concentrated brine at a temperature of 20°C. This temperature gradient was due to solar heating of the brine.

It is believed that the Redwater pond leaked, because it had attained a higher permeability, due mainly to dessication shrinking. The dessication shrinking results from the flow of fresh water from the clay into the brine by a high concentration gradient (Hudec and Sonnenfeld, 1979). The main reason for the dessication shrinkage is sodium (Na) cation exchange. The dessication shrinkage resulted in the development of cracks and hence a substantial increase in permeability. Pilot laboratory testing by Dr. Hudec in 1979 has confirmed this (personal communication). Brine ponds constructed from untreated clay are likely to leak because of the subsequent development of high permeability.

1.2 Purpose of the Study

This study was performed to investigate the effects of brine on some geotechnical properties of clay soils. The most important property under consideration was the permeabilities to water and brine of eighteen clay soils. The other geotechnical properties investigated were Attèrberg limits (liquid and plastic limits), water adsorption and compaction characteristics. It was expected that these properties would help explain the difference between the permeabilities to water and brine. Nineteen (19) clay soils from 10 locations in Southwestern, Central and Eastern Ontario were collected from clay pits and stream banks. The samples were placed in plastic bags, which were immediately sealed to preserve the field moisture contents of the samples.

Grain size analysis of the original samples was done by wet sieving on coarser fractions and hydrometer analysis on the fine portions. Atterberg limits and water adsorption at 98% relative humidity and 22°C were determined for original samples and samples treated with brine. Optimum moisture contents and maximum dry densities were determined for all the samples using the Standard Proctor method. Permeabilities of both original and treated samples at optimum compaction were determined with a specially-designed large-area permeameter, using water and brine as the permeants.

Long-term permeability to water and brine were also determined over a period of 7 days (168 hours) for six randomly selected samples. All experimental procedures are outlined in Chapter IV.

The results of the experiments and significant correlations among clay-size fraction, Atterberg limits and adsorption are presented in Chapter V. In addition, permeabilities of original and treated samples to water and brine are compared. The results are discussed with reference to the basic theoretical considerations outlined in Chapter II.

As a result of this study, a method of pre-treating and placing clay liners for brine storage is proposed.

CHAPTER II

THEORETICAL CONSIDERATIONS

2.1 Clay Mineralogy, Structure and Shape

2.1.1 Mineralogy and Structure

The term "clay," as used in soil mechanics has two meanings. It refers to all constituents of a soil smaller than some given size, usually 0.002 mm. When used as a mineral term, however, it refers to specific minerals termed the "clay minerals." Clay minerals are primarily hydrous alumino-silicates, that is, oxides of aluminium and silicon. Magnesium or iron occupy all or part of the aluminum positions in some minerals, and alkalis (for example, sodium, potassium) or alkaline earth (e.g., calcium, magnesium) are also present as essential constituents in some of them (Grim, 1968). The aluminium-oxygen combinations are the basic structural units, which are bonded together in such a way that sheets are formed. The different minerals within clay mineral groups are the result of variations in stacking of these sheets into layers, the bonding between layers, and the substitution of other ions for aluminium and silicon. Yong and Warkentin (1975) have noted that this substitution occurs for ions of

approximately the same size and is therefore called isomorphous substitution.

The two basic units in clay mineral structures are the silicon tetrahedron, a silicon tetrahedrally coordinated with four oxygens and the aluminium or magnesium octahedron, in which aluminium or magnesium ion is octahedrally coordinated with six oxygens or hydroxyls. Most clay minerals are either 2-layer sheets or 3-layer sheets, the commonest being kaolinite, halloysite, chlorite, illite or clay mica, talc, montmorillonite, and vermiculite. In these minerals, the silica and alumina components, usually and ideally, form broad sheets sharing a common interfacial plane of oxygen atoms. Figure 2.1 is a schematic representation of the silica tetrahedron and alumina octahedron. There are three major groups to which the clay minerals belong: the kaolinite group, the illite group and the montmorillonite group.

Kaolinite

Kaolinite consists of 2-layer sheets in which a silica sheet and an alumina sheet share a layer of oxygen atoms between them. This is shown schematically in Fig. 2.2. Each layer is three oxygen atoms thick. The layers are held together by hydrogen bonding (Section 2.1.3) between hydroxyls from the alumina sheet on one face and oxygens from the silica sheet on the opposite face of the layer.

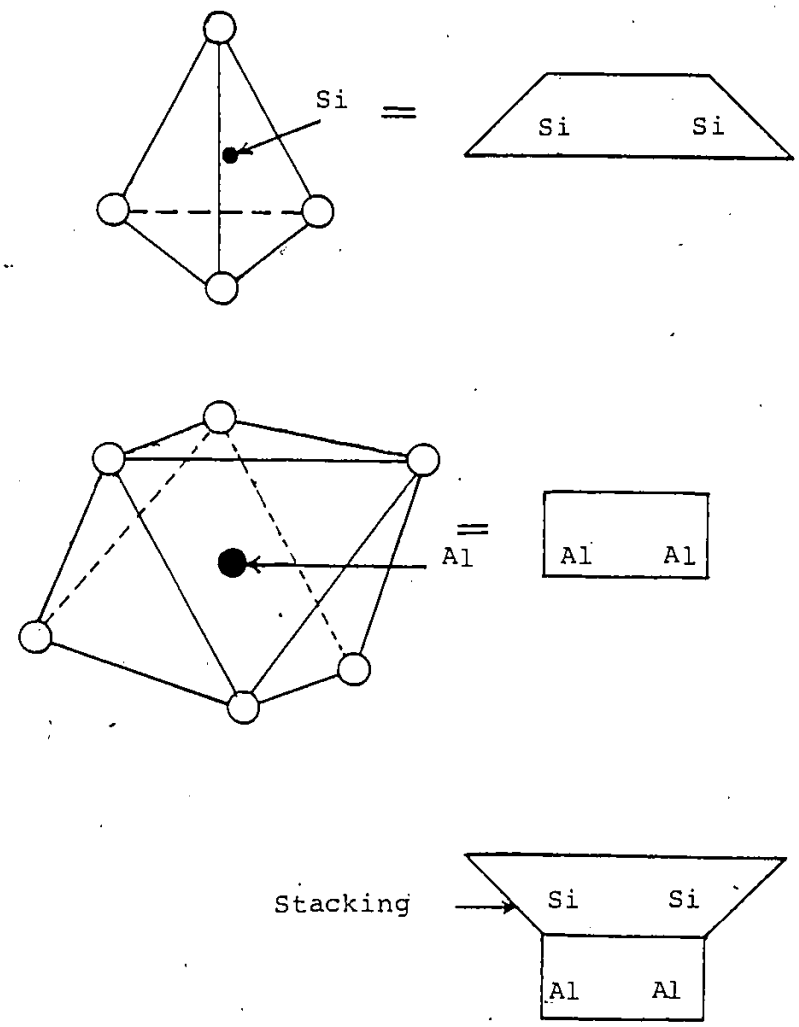


Fig. 2.1 Schematic representation of silica tetrahedron and alumina octahedron (from Yong and Warkentin, 1975).

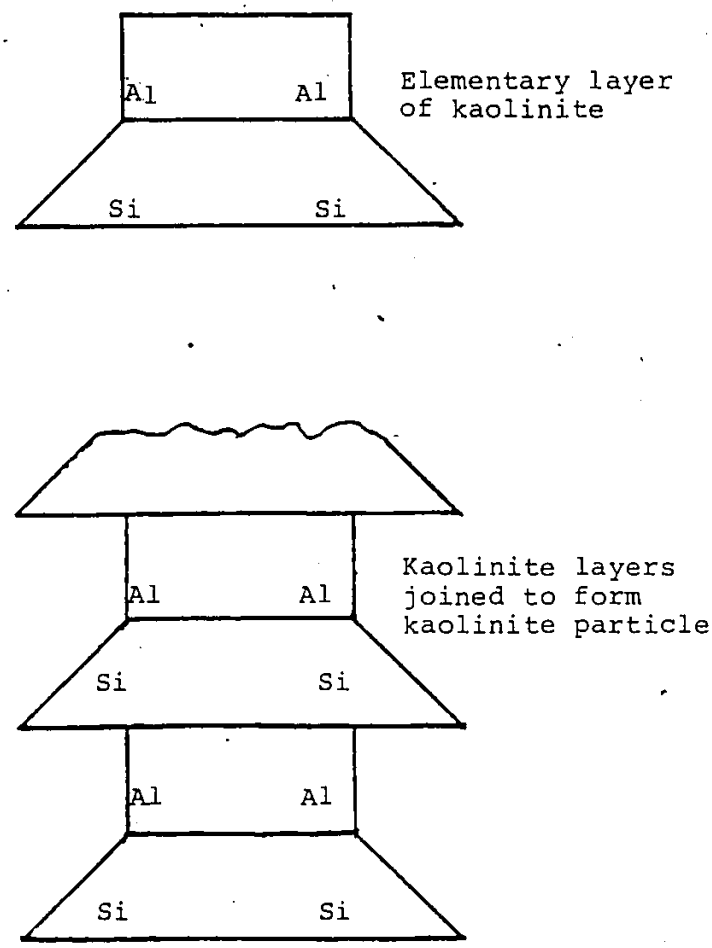


Fig. 2.2 Schematic representation of typical kaolinite structure (from Yong and Warkentin, 1975).

Mitchell (1976) was of the opinion that Van der Waals forces also contribute to the bonding between layers. In both cases, the bonding is of sufficient strength to prevent interlayer hydration and swelling, allowing many layers to build up.

Halloysite differs from kaolinite in that it can occur in hydrated form, with a layer of water molecules between the layers of the kaolinite crystal. The hydrated form found in nature loses the water readily on heating and does not rehydrate when put in water. The literature sometimes refers to the dehydrated form as metahalloysite, but Yong and Warkentin (1975) thought that a preferred usage should be halloysite with the state of hydration specified.

Illite

Illite is perhaps the commonest clay mineral encountered in engineering practice. It consists of 3-layer sheets made up of repeating layers of an alumina sheet between two silica sheets with oxygen shared to give a unit four oxygen atoms thick. The layers are bonded together by potassium ions which are just the right size to fit into the hexagonal holes of the silica sheet. Substitution of Al^{3+} for Si^{4+} provides negative charges which balance the potassium cations. Figure 2.3 represents the structure schematically. Illite differs from minerals

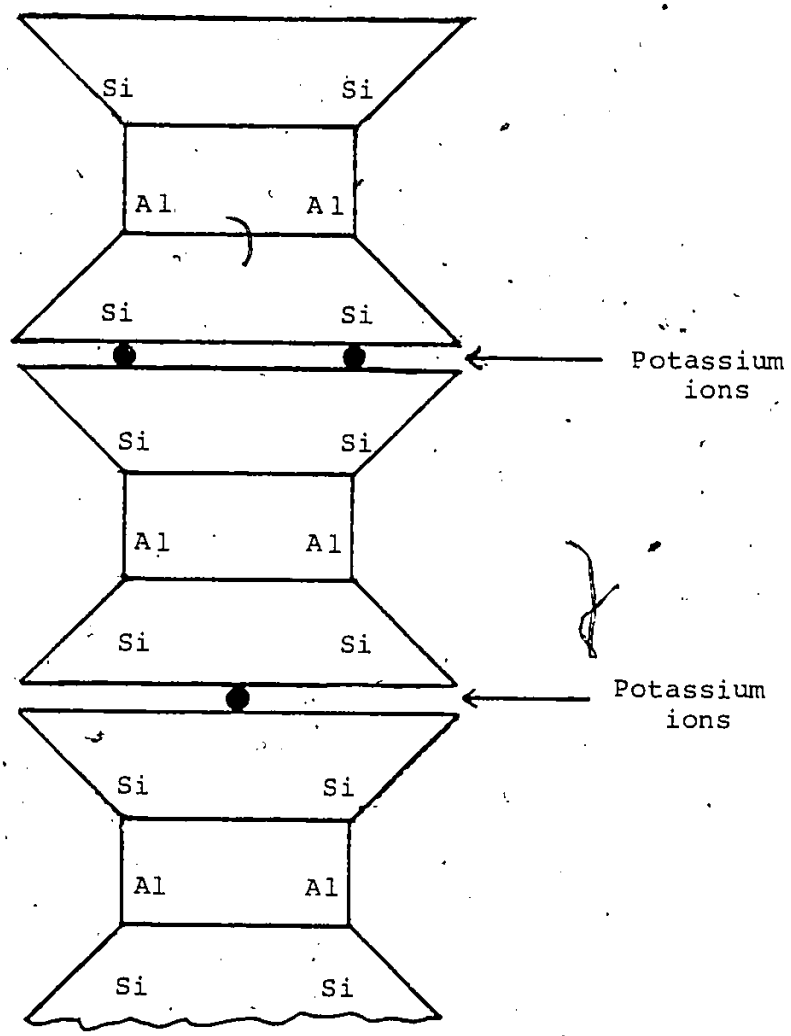


Fig. 2.3 Schematic representation of typical illite structure (from Yong and Warkentin, 1975).

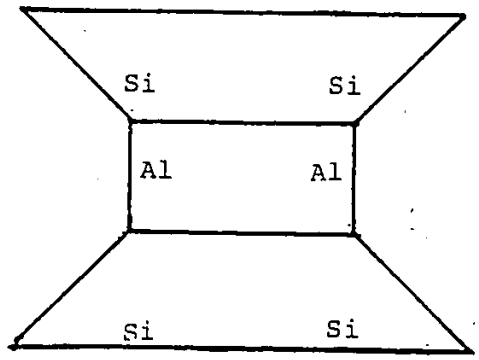
such as muscovite and biotite in having a smaller particle size, less substitution of aluminium for silicon, less potassium, and a more random stacking of layers with consequent weaker bonding between them.

Montmorillonite

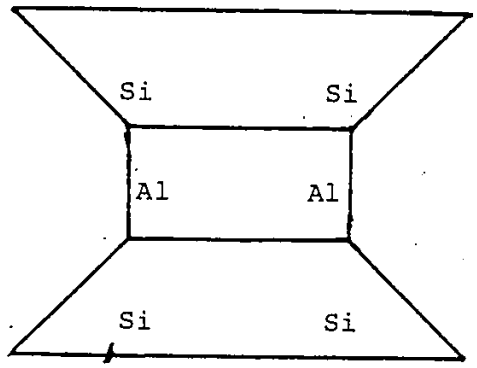
The montmorillonite mineral has the same layers as illite except that isomorphous substitution occurs mainly in the alumina sheet, with Mg^{2+} or Fe^{2+} substituting for Al^{3+} in the dioctahedral minerals. The term dioctahedral is used for clay minerals in which only two thirds of the octahedral positions are filled and trioctahedral for those in which all the positions are filled. Different substitutions of ions lead to different montmorillonite minerals. In a typical montmorillonite structure, there is no potassium to bond the layers together and water enters easily between the layers. There is no substitution in the tetrahedral sheet. A schematic representation of typical montmorillonite structure is shown in Fig. 2.4.

2.1.2 Summary of Structure and Shape of Clay Minerals

X-ray analysis has shown that the interatomic distances in the crystal lattice of clays resemble those of an ionic structure. This implies that the clay minerals are essentially an assembly of densely-packed oxygen anions, 'cemented' together by strong cations, whose high mutual



Several water layers and adsorbed cations



Several water layers and adsorbed cations

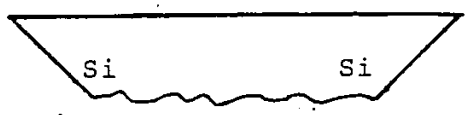


Fig. 2.4 Schematic representation of a typical montmorillonite structure (from Yong and Warkentin, 1975).

repulsions, due to their high charge, control the density of the anionic lattice. Ingles (1968) has noted that the surfaces of the clay minerals are composed entirely of oxygen (or hydroxyl) atoms.

With the aid of the electron microscope, the sizes and shapes of clay mineral crystals have been determined to a reasonable accuracy. Lambe (1958) reports that all the minerals mentioned above are sheets, that is, non-equidimensonal. Yong and Warkentin (1975) have also recognized that most clay minerals are plate-shaped or tabular. The explanation for this shape is that the layer-lattice structure results in strong bonding along two axes but weak bonding between layers. Some minerals such as attapulgite are, however, narrow fibres.

2.1.3 Interatomic and Intermolecular Forces

The various electrical forces acting between clay particles have been discussed by Lambe (1958), Yong and Warkentin (1975), and Mitchell (1976). These electrical forces act only near the particle surface and many of them are the result of discontinuities at or near the surface. This means that they become significant only when they are large in comparison to the masses of the particles themselves and the surface area per mass (that is, specific surface) is high. Lambe (1958) defined a

"colloid" as a particle with a specific surface large enough to cause the electrical forces to dominate the mass forces. Clay particles, because of their small size and large surface area, are thus essentially in the colloidal range. It follows that when two clay particles in a system are brought into close proximity their respective force fields overlap and this affects the behaviour of the system.

Lambe (1958) recognized that electrical forces acting between atoms fall into three categories: primary valence bonds, hydrogen bonds, and secondary valence bonds. Primary valence bonds hold atoms together in the basic mineral units and are the strongest. Ionic, covalent, and heteropolar bonds are the primary valence bonds most important to the soil mineralogist.

In the ionic bond, electron transfer occurs between the atoms which are subsequently held together by the opposite charge attraction of the ions formed. The covalent (or homopolar) bond involves the sharing of electrons by the linked atoms. The heteropolar bond is partly ionic and partly covalent due to unequal sharing of electrons by the linked atoms. In the hydrogen bond, an atom of hydrogen is strongly attracted by two electronegative atoms especially those of oxygen, nitrogen or fluorine. This property of hydrogen is unique and is probably due to its small size (Yong and Warkentin, 1975). Hydrogen bonding between two

oxygen atoms is very important in clay mineralogy because it bonds layers of clay minerals together, holds water at clay surfaces, and bonds organic molecules to clay surfaces.

The secondary valence forces (also called Van der Waals forces) occur between uncharged units of matter. They arise from electrical moments existing within the units. Dipoles are molecules which are neutral but have separate centres of positive and negative charge. Certain orientations of adjacent dipoles are statistically preferred and net attraction results. The instantaneous positions of electrons in atomic shells are variable so that even molecules which are not polar can be considered as instantaneous dipoles. Van der Waals forces result when these instantaneous dipoles induce in-phase dipoles in adjacent molecules. Yong and Warkentin (1975) mentioned that these forces decrease as the seventh power of the distance between atoms.

The water molecule is a permanent dipole since its polar nature arises from the position of atoms in the molecule. The positive corner of one molecule is attracted to the negative corner of another by Van der Waals forces.

The secondary valence forces or Van der Waals forces are of vital concern to the soil engineer because they contribute to clay strength and cause soils to hold water. The primary valence and hydrogen bonds are both too strong to be broken by stress imposed on a soil system during

routine engineering practice. The secondary valence forces are much weaker and can act over relatively large distances.

Another type of primary valence linkage worth considering is the electrostatic attraction or repulsion of electrically charged units of matter. This includes the attraction between negatively charged clay particles and cations, repulsion between cations, attraction between cations and anions and repulsion between anions. These electrostatic forces involve net electrical charges and can act over much larger distances than can ionic bonds (Lambe, 1958).

In conclusion, it must be mentioned that secondary valence forces (Van der Waals forces) and hydrogen bonds are of more concern to the soil engineer. This is because they are greatly influenced by applied stresses and changes in the soil-water system.

2.2 Cation Exchange Capacity

The surface of clay particles is negatively charged when suspended in a water medium and will, consequently, attract cations from the soil water, or rather soil solution, to balance the negative charge. The cations are termed "exchangeable cations" because one cation can be readily replaced by another of equal valence, or by two of one-half the valence of the original one.

The cation exchange capacity (CEC) is the number of cations bound - but only loosely - to the mineral surface

by charge deficiencies in the particular crystal lattice. It is the amount of negative charge per unit mass (or per unit surface area of the clay) and is usually expressed as milliequivalents per gram or per 100 grams. The main source of the negative charge on the surface is isomorphous substitution of Al^{3+} for Si^{4+} or of Mg^{2+} for Al^{3+}, as previously discussed. This occurs during crystallization or formation of the mineral. A second source is unsatisfied valence charges at the edges of the particles. These are known as broken-bond charges and are due to the fact that bonding at the edges of the clay crystal lattice is broken. Yong and Warkentin (1975) have noted that the broken bonds are between oxygen and silicon and between oxygen and aluminium. They suggested that the most probable plane is as shown in Fig. 2.5.

Broken bond charges have very important contribution to the CEC of the 2-layer clays such as kaolinite (Quirk, 1960). The amount of such charges per unit mass of clay increases with a decrease in particle size, because the proportion of edge to total area is increased. The broken bonds attract hydrogen ions in solution (that is, $H^{+}.H_2O^{+}$ or H_3O^{+} or hydronium ions) from the pore water. The hydrogen ion can be exchanged for other cations and the ease with which this happens is a function of the pH of the solution. The charge due to broken bond increases as pH

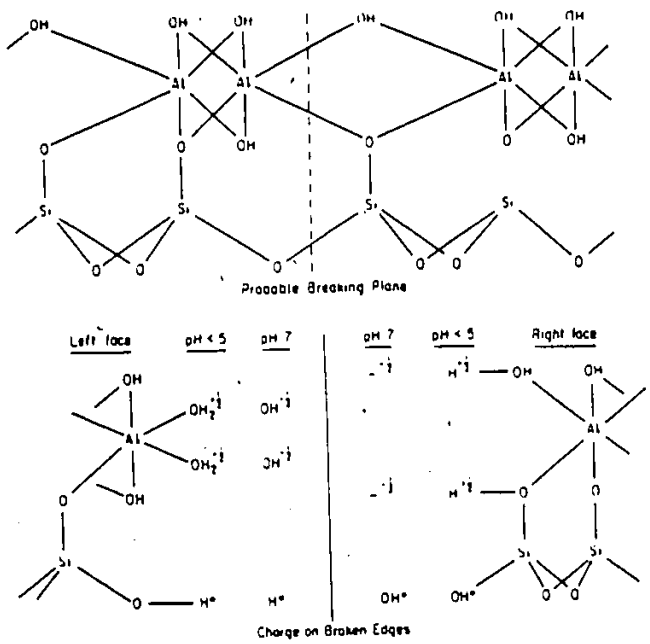


Fig. 2.5 Kaolinite structure showing probable breaking plane and mechanism for edge charge by picking up hydrogen or hydroxyl from solution to give positive charge at low pH and negative charge at higher pH. (Directly from Yong and Warkentin, 1975).

increases, that is, as the hydrogen ion concentration of the pore water decreases.

The predominant exchangeable cations in soils are calcium and magnesium. Potassium and sodium are found in smaller amounts. Geological environment and subsequent leaching determine which exchangeable cations will be present. Marine clays have predominantly magnesium and sodium while acid soils have mostly aluminium and hydrogen.

As noted above, ions of one type can be replaced by ions of another type. The replacing power depends mainly on the valence, size, and the relative abundance of the different ion types. The higher the valence the greater the replacing power, or the harder to replace if cation of higher valence is at the surface. Small cations tend to displace large cations of the same valence. A typical replaceability series is as follows (Worrall, 1968):

$$Li^{+}<Na^{+}<K^{+}<NH_4^{+}<Mg^{2+}<Ca^{2+}<Ba^{2+}<Al^{3+}<H^{+}$$

It is, however, possible to displace a cation of high replacing power, such as Al^{3+} by one of low replacing power such as Na^{+}. This can be achieved by making the concentration of Na^{+} in solution very high relative to that of Al^{3+}.

2.3 Plasticity

Plasticity can be defined as that property of a material which allows it to be deformed rapidly without rupture,

elastic rebound and volume change. This behaviour changes with a change in water content and is especially marked for clay soils. Grimshaw (1971) defined plasticity "as the behaviour of clays when they are mixed with water but in insufficient amounts to permit flow without external applied stresses." Means and Parcher (1963) have reported the occasional use of the term "plasticity" for any non-elastic deformation, such as workability, creep or yield.

Soils, at high water contents, are suspensions with the flow properties of liquids. In clay soils these flow properties will change to the non-Newtonian flow of paste-like materials, as the water content is gradually reduced. A further decrease in water content will lead to an increase in cohesion so that the soil becomes sticky and makes remoulding difficult. On further drying the stickiness disappears, and the clay can be moulded. At this water content it is plastic. As the water content decreases still further the plasticity is lost; the soil becomes harder to work and at low water contents becomes a hard and brittle solid. This means that a piece of clay soil may exist either as a brittle solid, plastic material or liquid. The transition from one state to another is gradual and an exact characterization of the transition point is not possible (Means and Parcher, 1963). Consistency is the physical condition of a soil at a given water content. It represents the resistance

of the soil to flow and gives an indication of its rheological behaviour (Yong and Warkentin, 1975).

2.3.1 Causes of Plasticity

Plastic behaviour of clay soils is due to the lubricating effect of water films between adjacent particles. It depends on the size and shape of the individual particles and the chemical nature of their surfaces. Grimshaw (1971) discussed the role of cations in solution in clay-water systems. He referred to the formation of a hydrosphere of cations surrounded by aligned water molecules at the clay surface. The size of the hydrosphere, he noted, varies according to the number, size, charge and degree of hydration of the cations. At high water contents, the hydrosphere is large so that the particles in suspension are at the maximum separation. Free and viscous flow is then possible because the thick water films permit the particles to slide past one another more easily. On the other hand, a reduction of the hydrosphere results in more cohesion because the separating water film becomes thinner.

2.3.2 Factors Affecting Plasticity

Plasticity is related to the thickness of the water films formed around each soil particle. An excess of water will permit flow with a reduction in the plastic properties. At much lower contents, the individual particles touch and

plasticity is destroyed. Plasticity is thus dependent on water content and its peak value is a function of the surface area capable of being wetted. Grimshaw (1971) has discussed the various factors affecting plasticity. He noted, in particular, that surface area of particles and the presence of electrolytes have considerable influence on plasticity. Various investigators are of the view that plasticity is due to the presence of extremely small-sized grains in a material. This is based on the fact that various materials, usually considered non-plastic may, when very finely ground with water, develop some amount of plasticity. Coarse additives, in general, are known to reduce plasticity in a well-balanced clay.

The mineralogy of the solids in a clay-water system has been shown to affect plasticity (Grim, 1962). Effects of the previous history of a clay soil may be reflected in measurements of plasticity. For example, when a clay is air-dried it loses its plasticity, but this is readily restored on mixing with a suitable proportion of water. On the other hand, a clay dried by artificial heat or exposure to tropical conditions cannot fully regain its plasticity. This is due to partial decomposition of the clay or a partial conversion of the colloidal matter into an inert material. Any treatment which alters the colloidal matter affects the plasticity of clay soils.

2.3.3 Measurement of Plasticity

About 1911, the Swedish agriculturist, Atterberg, divided the entire cohesive range of clay soils from the solid to the liquid state into four stages. He set arbitrary limits for these divisions in terms of water content as follows:

(a) Liquid limit or lower limit of viscous flow above which the soil and water flow as a viscous liquid and below which the mixture is plastic.

(b) Plastic limit or lower limit of the plastic range is the water content at which the soil will start to crumble when rolled into a thread under the palm of the hand.

(c) Shrinkage limit or lower limit of volume change at which there is no further decrease in volume as water is evaporated.

The above limits are called Atterberg or Consistency limits.

Measurements of plasticity are made on remoulded samples. In the laboratory the liquid limit is determined, according to standard procedure, by noting the number of blows and water content at which the soil flows to close a groove made in the liquid limit device (ASTM 423-66, revised 1981). Four or five water content determinations in the range of ten to forty blows are obtained. A "flow curve" (approximately a straight line) is obtained by plotting the water content against log of blows. The liquid limit is

defined as the water content at 25 blows. The plastic limit is determined as the water content at which the soil begins to crumble when rolled into a thread 3 mm or 1/8 inch in diameter, under the palm of the hand.

Lambe (1951) and Means and Parcher (1963) have discussed, in detail, the procedure and relevant mathematical expressions for the determination of the shrinkage limit. The shrinkage limit can be obtained on soils in either their undisturbed or their remoulded states. Since there is little change in volume from the shrinkage limit after complete drying out, it can be determined from the volume of an oven-dried specimen. The volume of the dry specimen can be determined by displacement in mercury. By assuming the voids of the dry soil to be filled with water and determining the ratio of the weight of this water to the weight of the dry solid, the shrinkage limit can be obtained from a knowledge of the total volume, the specific gravity of the solids, and the weight of the dry specimen.

The shrinkage limit concept finds considerable application in testing water-sensitive, expandable clays.

2.3.4 Significance of the Atterberg Limits

The plasticity index is the range of moisture content over which the soil is plastic and is defined as the difference between the liquid limit (W_L) and the plastic limit (W_p). This index depends, to a first approximation,

only on the amount of clay present. The ratio of the plasticity index to the percentage of particles smaller than 0.002 mm by weight is known as the activity of a clay soil. The activity of a clay soil is a measure of the degree to which it will exhibit colloidal behaviour (Perloff and Baron, 1976). It is apparent that a high activity indicates a soil which would be expected to give the engineer problems due to its generally high water holding capacity, cation exchange capacity, thixotropic and sensitivity properties.

The Atterberg limits provide a rough measure of the sensitivity of a clay soil by means of the liquidity index (Perloff and Baron, 1976). The liquidity index is the moisture content of the soil in excess of the plastic limit, expressed as a ratio of the plasticity index. Glanville (1951) has noted that the liquidity index merely describes the moisture condition of a soil with respect to its index limits and is of no use for classification purposes. It shows in what part of its plastic range a given sample of soil lies, knowledge which is sometimes useful in assessing the condition of a soil at its natural moisture content in the field.

Grimshaw (1971) mentioned that the Atterberg method leaves a large margin for the personal element, which is undesirable. It is possible that among a number of tech-

nicians in different laboratories, differing humidity conditions and varying interpretations of sample preparation will lead to variability in the values obtained. He also states that "the Atterberg limits are based on the idea that the plasticity of a material is proportional to the amount of water present, but as it is not the water alone, but the nature of the viscous fluids (of which the water is only one constituent), and the nature of the solid particles as well, which determine the plasticity of a mass, Atterberg's method is fundamentally unsound, but it is a simple and convenient determination for some purposes, for example, assessing the properties of soils" (Grimshaw, 1971, p. 508). It is this disturbing fact which has sometimes prompted the investigation of the measurement of the natural sorption capacity as an alternative property to the Atterberg limits.

Odell et al. (1960) studied the relationships of Atterberg limits to the amounts of some clay minerals, organic carbon, and clay size fraction for some Illinois soils. They obtained multiple correlation coefficients of 0.959, 0.887, and 0.938 between liquid limit, plastic limit, and plasticity index, respectively, and the three soil properties. Dumbleton and West (1966) investigated the relationships between the clay content and the plastic and liquid limits of natural montmorillonitic and kaolinitic soils. They concluded from the study that the

clay content has a strong influence on the plasticity.

The main engineering use of the Atterberg limits and the related indices is in soil identification and classification. Another important use of the limits, as Lambe and Whitman (1969) noted, is in specifications for controlling soil for use in fill and semi-empirical methods of design. It has been noted that all of the limits and indices, with the exception of the shrinkage limit, are determined on remoulded soil specimens. The limits, therefore, give no indication of particle fabric or residual bonds between particles which may have been developed in the natural soil but are destroyed in preparation of the specimen for the determination of the limits (Lambe and Whitman, 1969).

It has been pointed out that the chemical and mineral composition, size and shape of the soil particles influence the adsorbed water films on the particles. Geotechnical properties such as compressibility, permeability and strength as well as the Atterberg limits are all dependent on the water films. Approximate relationships between these properties and the limits, therefore, exist.

2.3.5 Clay Mineralogy and Engineering Behaviour

Ingles (1968) has discussed the engineering properties of the various clay minerals. These properties will be summarized here for kaolinite, illite and montmorillonite.

Kaolinite is non-swelling and inactive, whereas

halloysite is very plastic and troublesome when partially dried, but relatively inactive otherwise. Kaolinite has a low liquid limit. The activity of illite is intermediate between those of kaolinite and montmorillonite. Illite undergoes only partial swelling and becomes active if leached. Montmorillonite, on the other hand, has a high activity and high liquid limit. When placed in contact with water, montmorillonite swells to several times its dry volume. This swelling is the result of water adsorbed between layers, pushing the layers apart.

2.4 Water Adsorption

2.4.1 Nature of Adsorption

The forces acting at the surface of a body are slightly different from those acting within the body. In the interior of the body, attractive forces between molecules, atoms and ions are balanced; the net attractive force is equal in all directions. At the surface, however, exposed molecules and atoms are not surrounded on all sides. The net attractive force is directed towards the interior of the body. This attraction occurs in a direction normal to the surface. In response to this net attraction, the surface shrinks to the smallest area that can enclose the body. There is, therefore, an unfilled capability by the surface to attract other molecules or ions. This phenomenon is known as

adsorption. Worrall (1968) distinguished adsorption from 'absorption' by noting that the latter occurs when the interior of the body participates in the up-take of other molecules.

Lee (1941) briefly discussed adsorption. He mentioned that adsorption occurs when the pressure of a gas or the concentration of a solution in contact with a finely-divided or porous material decreases. The surface that adsorbs is known as the adsorbent and the adsorbed medium is the adsorbate. Rigbey (1980) defined adsorption as the adhesion of a thin film of molecules to a solid or liquid surface.

Adsorption is a function of surface forces. A surface will attract dipolar or ionic molecules because of its charge. The thickness of the adsorbed layer is thus a function of the surface charge (Rigbey, 1980). This surface charge arises from the unequal molecular attraction at the surface, as mentioned above, and the presence of unsatisfied molecular bonds. Clays, due to the presence of imperfections at the surface, especially at the edges, and atomic or ionic substitutions in their crystal lattice, have particularly high surface charges (Yong and Warkentin, 1975).

Worrall (1968) noted that adsorbed substances need not be ionic. He recognized the role of Van der Waals forces

in the adsorption of non-ionized molecules by surfaces and referred to this phenomenon as physical adsorption. Chemisorption, according to Worrall (1968), is the up-take of ions of one particular type by a surface, an example being the adsorption of oxygen on metal surfaces. In chemisorption, it is possible for a reaction to be complete and irreversible. The extent of a physical adsorption is, however, always controlled by an equilibrium between adsorbed and unadsorbed substance. Worrall (1968) used the Freundlich Isotherm Equation to describe adsorption of a solute from a solution, that is,

$$x/m = k.c^{1/n}$$

where x = mass of solute adsorbed

m = mass of adsorbent

c = concentration of solute at equilibrium

k = constant related to the surface area

n = constant greater than 1

Physical adsorption, unlike chemisorption, is markedly temperature-dependent and in the above equation, Worrall (1968) makes the assumption that the temperature is constant.

2.4.2 Possible Mechanisms of Water Adsorption by Clay Surfaces

Grimshaw (1971), Yong and Warkentin (1975) and Mitchell (1976) have all discussed the attraction of water to soil minerals, particularly to clays. It is known from experimental work that dried clays will adsorb water from the atmosphere at low relative humidities. Many soils swell when given access to water, and temperatures above 100°C are needed to remove all the water from a soil.

Several mechanisms have been proposed to explain water adsorption by clay surfaces. Yong and Warkentin (1975) noted that the forces holding water molecules to the clay surface arise both from the water and from the clay. Mitchell (1976) supported this view and mentioned hydrogen bonding of water molecules to exposed oxygens or hydroxyls at the clay mineral surface as a possible mechanism for clay-water interaction. Since the water molecule is dipolar, it will be oriented with its positive pole directed toward the negative surfaces of clay partices. Cations in water are always hydrated, that is, surrounded by layers of water molecules. Thus, exchangeable cations held near the negatively-charged surface hold some of the water at the surface as water of hydration of ions. The degree of hydration of the surface would vary according to the cation present.

Mitchell (1976) mentioned the role of osmosis in clay-water interactions. He recognized that the increased concentration of cations in the vicinity of the negatively-charged clay surfaces would lead to the diffusion of water molecules towards the surface to equalize concentrations. This mechanism is feasible probably because diffusion of ions from the vicinity of the surface is restricted due to electrostatic attraction.

Water molecules have, in some instances, been known to be bonded to clay surfaces by Van der Waals attractive forces. Mitchell (1976) noted that in such cases, in-phase fluctuations of electron clouds in the molecules occur. Consequently, temporary dipoles are formed which induce displacements in neighbouring molecules so that dipole-dipole attraction occurs. Such bonds would be nondirectional and, as Mitchell (1976) noted, the resulting water structure would be close-packed and more fluid than the H-bonded structure.

2.4.3 Structure and Properties of Adsorbed Water

The structure of adsorbed water has been a point of controversy among many workers. Adsorbed water has been viewed by Yong and Warkentin (1975) as layers of water molecules held successively by hydrogen bonding to the clay surface. The force, however, becomes weaker with distance because the orienting influence of the surface

on the water molecules decreases. This means that each successive layer is held less strongly away from the surface, and the bonding quickly decreases to that of free water. The properties of this water close to the surface differ from those of free water.

Anderson and Low (1958) concluded from their experimental work on Wyoming bentonite that adsorbed water is less dense than free water. Grimshaw (1971), however, noted that the density of adsorbed water is higher than in normal water. Yong and Warkentin (1975) also stated that the density of adsorbed water is higher than that of free water. They pointed out that the density decreases as further layers of water molecules are added to the clay surface. Mitchell (1976) summarized data on the density of adsorbed water on sodium montmorillonite as a function of water content. The results indicate that for water contents less than needed to give about three molecular layers on the clay surface (10Å thick layer), the density is greater than that of normal water. It is, however, less at greater water contents. He concluded that data covering specific ranges of water content can be used to support close-packed liquid models and open H-bond models, but no clear preference for either can be made.

Several authors (Grim, 1953; Grimshaw, 1971; Yong and

Warkentin, 1975; Mitchell, 1976) have observed that the bonding and structure in adsorbed water are different than in normal water. This means that there should also be differences in viscosity and ion mobility which would be reflected in measurements of diffusion and flow properties (Jurinak and Volman, 1961). Martin (1960) observed, however, that results of fluid flow and diffusion experiments are not very reliable because of difficulties in experimentation and data analysis. He concluded that such results could not provide clear evidence about the nature of adsorbed water. Low (1961) supported the view that adsorbed water has a quasi-crystalline structure, exhibits non-Newtonian flow, and higher than normal viscosity. Current evidence, however, appears to indicate that adsorbed water, whatever its structure, behaves as a Newtonian liquid. Grimshaw (1971) and Yong and Warkentin (1975) indicated that the layer of adsorbed water close to the surface has a much higher viscosity than that of normal water. Rigbey (1980) postulated that this higher viscosity can be attributed to increase in surface tension.

Adsorbed water has also been shown to consist of water molecules which are relatively free to move in the two directions parallel to the clay surface but are restricted in their movement perpendicular to, or away from the surface. Thus, the thermodynamic properties of adsorbed water is

different from those of ice, in which movement of water molecules is restricted in all three directions. Adsorption of water is both temperature and humidity-controlled and a change in humidity conditions is sufficient to alter the amount of adsorbed water. Litvan (1970) mentioned that the amount of adsorbed water decreases with temperature. Cary et al. (1964), however, cautioned that calculations of thermodynamic quantities based on the temperature dependence of systems are, in general, not reliable at low relative humidities. This is due to changes in the available surface.

Mitchell (1976) has noted that adsorbed water will supercool. Supercooling is the reduction of temperature below the normal freezing point without the initiation of freezing. This means that adsorbed water cannot attain the structure of ice (Mitchell, 1976). This property of adsorbed water had been observed by earlier workers including Litvan (1970). Litvan (1970, 1972) stated that adsorbed water remains in a liquid-like state well below 0°C and that no freezing would occur. This observation, however, pertained to less than two complete molecular layers of adsorbed water.

2.4.4 Cation Hydratability and Water Adsorption

Most cations become hydrated due to the polar nature of water. Some monovalent cations such as K, Rb, Cs, do

not become hydrated as their generated field is too weak to take up water molecules. The single positive charge is dissipated over a large ionic surface area, resulting in a low ionic potential for these cations.

The ionic radii of the alkali ions increase in the following order (Grimshaw, 1971):

$$Cs > Rb > K > Na > Li$$

The energy of hydration depends on the radius and charge of the cation in question. In the alkali ions, there is decreasing hydration with increasing ionic radius. It follows that the hydratability of these cations will increase in the order:

$$Li > Na > K > Rb > Cs$$

This means that lithium hydrates most strongly of the monovalent cations listed above. Polyvalent cations (such as aluminium) hydrate more strongly than monovalent cations. Thus hydration is seen to be partially dependent on the ionic potential of the cation. Cations with high hydration energies hydrate at lower humidities than those with lower hydration energies.

Adsorbed cations on a clay surface are surrounded by hydration shells on introduction of moisture to the system. The quantity of water adsorbed by a clay is dependent on

the nature of the surface cations. Adsorption accounts for only 10 to 20% of the total sorption capacity of the montmorillonites. Any factors influencing the internal surfaces should, however, also influence external surfaces. In their dry state, individual lattice-layers of these clays are often assumed to be bonded to one another by Van der Waals forces between adjacent surfaces and "bridging" via exchangeable cations (Michaels, 1959). In contact with water, the degree to which the layers will separate will depend primarily on the affinity of the cations for water. Michaels (1959) has noted that if the hydration-tendency of the cation is high, water molecules will "cluster" around the ion in large numbers and reduce its intersheet bridging ability. Water sorption and swelling are consequently increased. Thus, the water sorption capacity of the alkali montmorillonites will increase in the same order as the hydration energies of the corresponding cations.

With non-lattice expanding clays of the kaolinite type, exchangeable cations are taken up mainly at the surface of the particles. At a given relative humidity, the amount of water adsorbed will be directly proportional to the hydration energy of the cation.

2.4.5 Surface Area of Clays and the Atterberg Limits

It has been noted that adsorption occurs when a substance accumulates only on the surface of a solid (without

any diffusion into the inside of a lattice expanding clay). The amount of adsorption will, therefore, be directly proportional to the total surface area of the solid. The surface area has been shown to be dependent on the fineness of grain size and particle shape. The variation in specific surface area, according to Yong and Warkentin (1975), is primarily due to different thicknesses of the tabular particles.

The activity of a clay and its various colloidal properties (such as adsorption of water and formation of gels at low concentrations) increase with increasing specific surface area. Yong and Warkentin (1975) have noted that this area could be calculated if the size and shape distributions of the particles in a sample were known. This method, however, is too tedious to be practical. Surface area is, therefore, determined by measuring the amount of a liquid or gas required to cover the surface. Water vapour, nitrogen and organic liquids such as ethylene glycol are used.

Most of the differences between clay minerals in properties such as water retention, plasticity or cohesion can be explained by the difference in surface area. Surface area has been shown to have a high correlation with the Atterberg Limits. Farrar and Coleman (1967) determined total and external surface area of clay soils from measure-

ments of water and nitrogen adsorption. They obtained a high correlation between the total surface area and the Atterberg limits. A lower correlation was, however, obtained between external surface area and the Atterberg limits. The total surface area is measured by the amount of sorbed fluid. External surface area, on the other hand, is determined by the amount of fluid adsorbed.

Seed et al. (1964b) also discussed the relationship between liquid limit and surface area. They reported that the liquid limit would be expected to depend on the intensity of the net attractive forces between the clay components. The greater the surface area and the greater the attractive intensity, the greater will be the spacing between the clay particles (a thicker water layer) and, therefore, the greater the liquid limit. Fairbairn and Robertson (1957) and Gill and Reaves (1957) also obtained a high correlation between cation-exchange capacity and plasticity index. These relationships noted above, indicate that there is a direct relation between plasticity and sorption properties of clay soils.

2.5 Permeability

2.5.1 Geotechnical Significance

The significance of the permeability of soils lies in the fact that some of the major problems encountered in soil

and foundation engineering have to do with drainage. These include drainage of highways and airports, seepage through earth dams, uplift pressures beneath concrete dams and structures below ground water level, seepage pressures causing earth slides, and seepage through compacted earth material such as clay used for storage. In all of these problems, the permeability characteristics of soils have a controlling influence on the effective strength properties of the soils and on their responses under stress, and hence on stability conditions.

2.5.2 Permeability Flow

An understanding of the nature of hydraulic phenomena is required for conducting permeability tests and interpreting and using test data. Permeability flow in saturated soils (and sometimes partially saturated soils) takes place primarily under gravitational forces or a pumping head. The water everywhere in the region of flow is under a positive hydrostatic pressure. A negative pressure or suction created in high-vacuum well-point pumping systems can sometimes induce flow in the immediate vicinity of the well points. The distinguishing characteristic of capillary flow, however, is that it takes place under capillary forces mainly through a continuous interconnected system of thin capillary moisture films at grain contacts. In this case, the water everywhere is in a state of capillary tension (Burmister, 1954).

The principal realms of flow of water through soils are laminar flow or streamline flow, and turbulent flow. In laminar flow, viscous forces shape the character of flow with velocity proportional to the hydraulic gradient. Turbulence occurs in soils at much lower velocities in the form of eddies and vortices in the larger void spaces because of the effects of expansion, contraction and change of direction. As a consequence, turbulence is characterized by increased resistance to flow and larger energy losses. Flow in natural situations is initially transient and hence time-dependent. After a period of time, depending on conditions, a steady state is approached and an equilibrium free water surface is established.

The foregoing considerations lead to the conclusion that only one type of flow, designated the Darcy flow, is stable in character. The conditions under which such a flow takes place are: (a) the laminar realm, (b) the steady state, (c) flow with soil voids 100 percent saturated (no compressible air present), and (d) flow with the continuity conditions and basic equation of flow satisfied, that is, no volume changes occur in the soil mass as a result of flow.

In his work on the mechanics of flow of water through a porous medium, Darcy, in 1856, first stated the basic law of flow:

$$V = \frac{Q}{At} = k \frac{H}{L}$$

where

V is the Darcy velocity computed on the basis of the quantity of flow, Q, in a time, t, and of the entire cross-sectional area of the soil column A. The hydraulic gradient, H/L, is the ratio of the head of water, H, causing flow to the length of the soil column, L, in which the head is lost. The Darcy coefficient of permeability k is an overall value which provides an adequate, reliable, and stable basis for reference and for comparison of different conditions of flow.

Since A is the total cross-sectional area of the soil through which the water flows, the Darcy velocity Q/At is not the true velocity, but is the velocity at which the quantity of water, Q, would flow through a tube of the same area as the total area of the soil. The hydraulic gradient H/L is a dimensionless quantity so that the dimensions of the coefficient of permeability k are the same as those of velocity. In soil mechanics the units are usually cm/sec and will be so used throughout this study. k might be defined as the discharge velocity through soil under a unit hydraulic gradient. The value of k varies from 1 cm/sec for clean gravels down to 10^{-9} cm/sec for homogenous clays.

2.5.3 Factors Affecting the Permeability of Clay Soils

Lambe and Whitman (1969) have defined the coefficient of permeability as the superficial or approach velocity of the permeant for flow through soil of unit area under a unit hydraulic gradient. It follows that the permeability is a function of both the permeant and the soil. An equation reflecting the influence of the permeant and the soil characteristics on permeability was developed by Taylor (1948) using Poiseuille's law. The equation which is based on flow through a porous media, similar to flow through a bundle of capillary tubes, is as follows:

$$k = D_s^2 \frac{\gamma}{\mu} \frac{e^3}{(1+e)} C$$

where

k = the Darcy coefficient of permeability

D_s = some effective particle diameter

γ = unit weight of permeant

μ = viscosity of permeant

e = void ratio

C = shape factor which may be taken as 0.4 for a standard value with a possible error of less than 25% in computation of k.

The Kozeny-Carman equation (Lambe, 1954) which gives

an expression for the permeability of porous media states

$$k = \frac{1}{k_o S^2} \frac{\gamma}{\mu} \frac{e^3}{(1+e)}$$

in which

k_o = factor depending on pore shape and ratio of actual flow path to soil bed thickness,

S = specific surface area.

The primary considerations needed for the valid application of the Kozeny-Carman equation are (a) relatively uniform particle size, (b) laminar flow of liquid through the pores, (c) validity of Darcy's law, and (d) absence of long and short-range forces of interaction.

The Kozeny-Carman relationship represents a refinement of Darcy's law, accounting for certain soil properties and characteristics. In clay-water systems, however, the existence of interaction characteristics must be recognized. The fact that most clay particles are plate-like in shape and form fabric units means that the overall permeability between and within fabric units are different. In such systems the Kozeny-Carman equation becomes less valid.

Lambe (1954) noted that whereas the above equations express accurately the permeability characteristics of saturated sands, they can be ignored when considering fine-grained soils. First, there is the difficulty of selecting

the effective "constants" and soil characteristics. Secondly, the various terms in the equations are not independent, but interrelated in a very complex manner. In addition, even though the equations are sound, the knowledge of soils is not extensive enough to lend the equations to proper interpretation. The equations, however, aid considerably in the following examination of the variables affecting the permeability of clay soils. The variables are:

1. Permeant Characteristics
2. Soil Composition
3. Particle Size
4. Void Ratio
5. Fabric
6. Degree of Saturation.

1. Permeant Characteristics

The Taylor equation and Kozeny-Carman relationship show that both the viscosity μ and the unit weight γ of the permeant influence the value of permeability. In the literature, these two permeant characteristics are eliminated by defining the specific or absolute permeability as $K = \frac{k\mu}{\gamma}$ in which k is the Darcy coefficient of permeability or hydraulic conductivity. Since k is in units of velocity (that is, cm/sec), the unit for K is cm^2.

The polarity of permeant has some influence on permeability. (Michaels and Lin, 1954). As a result of the

electrical potential generated by fluid flow, there is the movement of permeant in a direction opposite to the net fluid flow. This electro-osmotic backflow plus the thickness of immobilized fluid immediately adjacent to the soil particles increase with fluid flow. Lambe and Whitman (1969), therefore, recommended the inclusion of some measure of polarity in the Taylor and Kozeny-Carman equations.

2. Soil Composition

The effect of soil composition on permeability is particularly marked in clays but generally of little importance with silts, sands, and gravels. Clay "composition" includes minerals, exchangeable ions, and impurities (such as free iron oxides and organic matter). The Taylor and Kozeny-Carman equations, as they appear, do not contain a term that directly covers composition or indirectly covers exchangeable ions. Such a term is strongly recommended.

3. Particle Size

The smaller the particles, the smaller the voids, which are the flow channels, and hence the lower the permeability. The Taylor equation suggests that permeability varies with the square of some particle diameter. It is more reasonable to propose a relationship between permeability and particle size in silts and sands than in clays. This is due to the fact that in silts and sands the particles are more clearly

equidimensional and very different fabrics do not occur (Lambe and Whitman, 1969).

4. Void Ratio

The Taylor and Kozeny-Carman equations indicate that a plot of k versus $e^3/(1+e)$ should be a straight line. Lambe and Whitman, (1969) recognized the existence of other theoretical equations which suggest that k versus $e^2/(1+e)$ or k versus e^2 should be a straight line. Mesri and Olson (1971) reported values of k for smectite, illite and kaolinite. Their plot of k versus e on log scales yielded straight lines. In addition, there are considerable experimental data which indicate that e versus log k is frequently a straight line. Test data from the work of Michaels and Lin (1954), however, show that a plot of k versus $e^3/(1+e)$ for kaolinite is not a straight line.

There is sufficient evidence in the literature to suggest that generally the plot of void ratio versus log of permeability approximates a straight line. This relationship holds, of course, only when all other soil characteristics are kept constant, since void ratio is a dependent variable. The influence of the nature of the moulding fluid on permeability (Lambe, 1954, p. 61) suggests that it is not only the void ratio, but also the method by which it is obtained, that affects permeability.

5. Fabric

At the same void ratio, a specimen of clay soil in the most flocculated state will have the highest permeability and the one in the most dispersed state will have the lowest permeability. The more dispersed the particles, that is, the more nearly parallel they are, the more tortuous is the flow path if the flow is normal to the particles. This increased tortuousity explains at least partially, why a soil whose structure has been improved considerably has a low permeability. The main factor, however, is that a flocculated soil has larger channels per area available for flow than a dispersed soil.

Lambe and Whitman (1969) have summarized data on a compacted clay to illustrate the extent to which fabric influences permeability. He noted, for example, that at the same void ratio and degree of saturation, a sample compacted dry of optimum and one compacted wet of optimum have a permeability ratio of approximately 60.

6. Degree of Saturation

Soil pores filled with entrapped air are not serving as channels for flowing water. A partially saturated soil is not, therefore, transmitting its maximum amount of liquid during permeation. Thus, the higher the degree of saturation, the higher the permeability. While the foregoing is a commonly accepted reasoning, it must be noted

that data correlating the degree of saturation with the permeability of fine-grained soils are not readily available. In comparison with composition, fabric, and void radio, the degree of saturation has minor influence on permeability.

2.5.4 The Variable Head Permeability Test

In this test, a small quantity of liquid flowing through a specimen is measured in a standpipe of smaller diameter than the specimen. The ratio of the area of the standpipe to the area of the specimen can be adjusted to suit the permeability of the material being tested.

Since the head of liquid during a test in the variable head method is not constant, Darcy's law cannot be used directly in computing the coefficient of permeability from test data. The derivation of an expression relating the coefficient of permeability to other variables is deferred until Section 4.2.6.6.

A relatively large permeability is required to obtain good precision with the variable head test. It is, therefore, limited to pervious soils (Lambe, 1951, p. 60). Further, the degree of saturation of an unsaturated soil changes during the variable head test; thus, it should be used only in saturated soils.

2.6 The Clay-Water-Electrolyte System

2.6.1 The Double Layer

In a dry clay, adsorbed cations cluster at the clay surfaces to neutralize the negative charges. Cations in excess of those needed to neutralize the electronegativity of the clay particles and their associated anions are present as salt precipitates. When the clay is placed in water the precipitated salts go into solution. Since the cations have much higher concentration at or near the surfaces of the particles, they tend to diffuse away in order to equalize concentrations throughout the system. This escaping tendency of the cations due to diffusion is countered by the negative electric field originating in the particle surfaces. The result is that the surrounding regions of the water medium in the immediate vicinity of the particles will be rich in positively-charged ions (cations) which counter-balance the surface charge and maintain electrical neutrality. As the distribution of ions in the system shows (Fig. 2.6a), the anions are excluded from the negative force fields of the clay particles. The negative surface and the distributed charge in the adjacent phase are together termed the diffuse double layer. The double layer is not limited to the layers of the hydrated shell in intimate contact with the surface, but extends some distances into the liquid

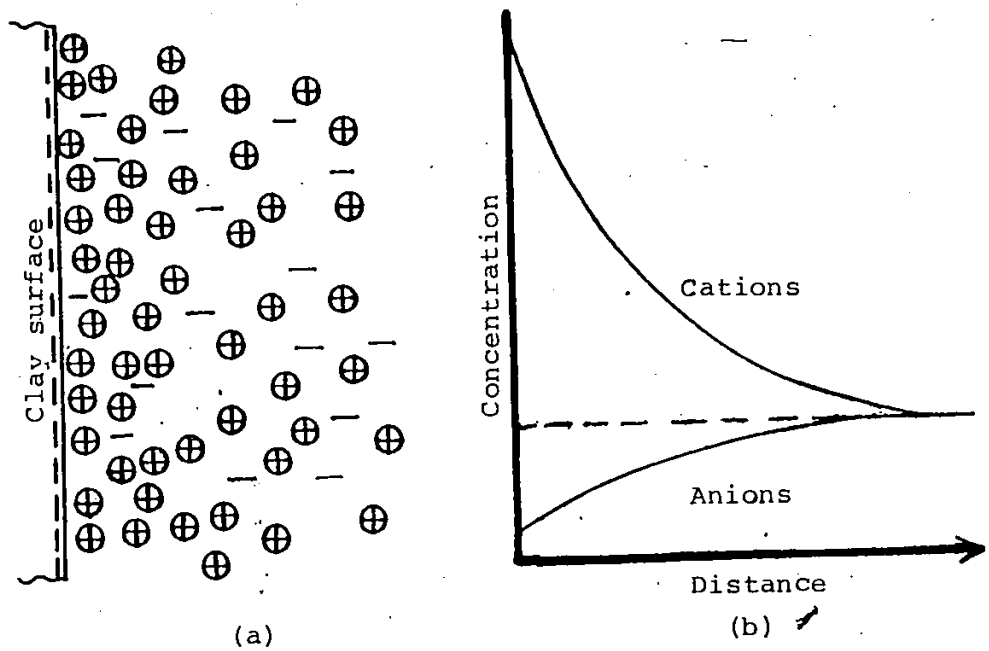


Fig. 2.6 Distribution of ions adjacent to a clay surface according to the concept of the diffuse double layer (from Mitchell, 1976).

medium. The clay particle plus the double layer is known as the micelle.

Grimshaw (1971) notes from the work of Guoy (1910) that the cationic distribution falls off exponentially as the distance from the surface is increased (Fig. 2.6b). As a result of the intense electrical forces in the vicinity of the surface, the molecules of the suspension medium are themselves oriented, giving rise to a layer of rigidly held liquid. The degree of orientation of the molecules which decreases as the distance from the surface increases, is a function of the surface and the type and concentration of ions in solution (Grimshaw, 1971).

2.6.2 Effects of System Variables on the Double Layer

Flocculation has been defined by La Mer and Healy (1963) as the 'bridging' of colloidal particles into a loose random structure, porous and three-dimensional. They defined dispersion as the permanent suspension of particles in a fluid medium, such that their size remains below that required for settlement.

Lambe (1958) and Mitchell (1976) have discussed the effects of changes in system properties on the diffuse double layer. Mitchell (1976) defined the "thickness" of the double layer as the distance from clay particle surface to the centre of gravity of the diffuse charge. He obtained an approximate expression relating the thickness,

X, to the surface charge (e), electrolyte concentraion (n_o), valence (v), dielectric constant (D) of the medium and temperature (T). That is,

$$X = \left(\frac{DkT}{8\pi n_o e^2 v^2}\right)^{1/2}$$

where k is the Boltzmann constant (1.38×10^{-16} erg/^oK).

The relationship above shows that the thickness decreases inversely as the valence, and the square root of the concentration; it increases with the square root of the dielectric constant and temperature, all other factors being constant. In other words, an increase in electrolyte concentration leads to a reduction of the double layer thickness. It follows, also, that for solutions of the same molarity and constant surface charge, an increase in valence will lead to a decrease in the thickness of the double layer.

The long range interparticle repulsive force depends on the amount of overlap or interaction between adjacent double layers. In general, a decrease in the thickness of the double layer means a reduction in the electrical repulsion and, consequently, a tendency toward flocculation. The smaller the ion plus its "shell of hydration," the close it can approach the surface of the clay particle; thus, the smaller the hydrated ion, the smaller the double

layer and the more likely is flocculation. The pH of the pore fluid influences the negative charge on a clay particle by altering the extent of dissociation of OH groups on the edges of the particle. In a high pH environment, dissociation is promoted; this increases the net charge and leads to an expansion of the double layer. This means a tendency towards dispersion.

The adsorption of anions, especially polyvalent anions, increases the negative charge on the particles and thereby tends to cause dispersion because of the increased electrical repulsion. Phosphate, arsenate and borate are examples of anions that are attracted to the clay surface. Adsorbed anions normally have about the same size and geometry as the silica tetrahedron (Mitchell, 1976).

2.6.3 Electrolyte Concentration and Dessication Shrinking

2.6.3.1 Stern Layer

The discussions presented in the last section assumed that ions in the diffuse double layer are point charges. This assumption may lead to impossibly high ion concentrations next to the clay surface. Mitchell (1976) has noted that ions are of finite size. It follows that the actual concentration of ions adjacent to the surface will be less than predicted. An improvement is made by applying the

Stern Correction. In this case, it is postulated that a fraction of the positive counter-ions are strongly adsorbed to form a monolayer close to the surface called the "Stern layer." The remainder of the cations form an atmosphere round the particle.

The physical consequence of the development of a Stern layer is a sharp fall in the cation distribution (Mitchell, 1976) and is shown in Fig. 2.7. In addition to the cations, water molecules (dipoles) are attracted and held by the charged surface as adsorbed water (Section 2.4.2). The clay particle with its layer of adsorbed water and diffuse layer of exchangeable cations can be visualized as illustrated in Fig. 2.8.

The behaviour of different cations can be explained from the above model by noting the following:

(1) Ions in solution are hydrated, that is they carry with them an envelope of adsorbed water.

(2) More electropositive ions have greater tendency to ionize away from the charged surface of the clay particle (Worrall, 1968).

The first point implies that the number of cations that can be accommodated in the Stern layer is governed by the size of the hydrated ion. Thus, the larger the ion, the thicker the layer required to accommodate the necessary number of cations and hence the greater the repulsion. This explana-

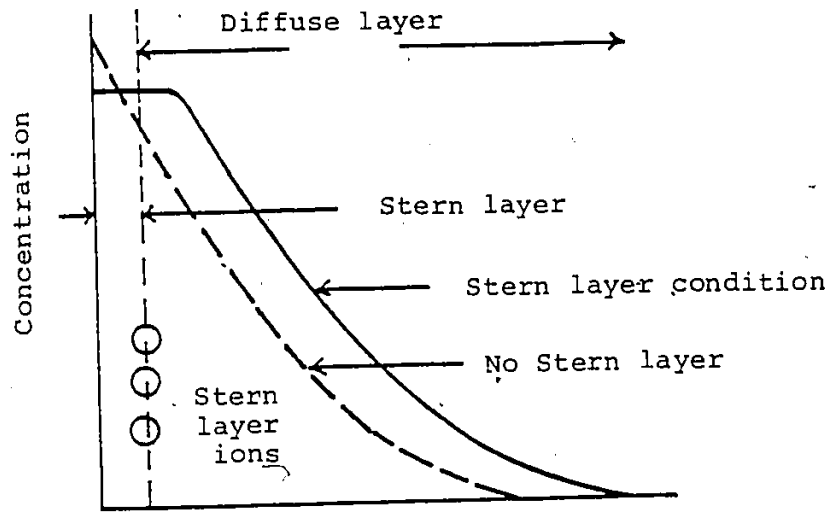


Fig. 2.7 Effect of Stern layer on cation concentration.

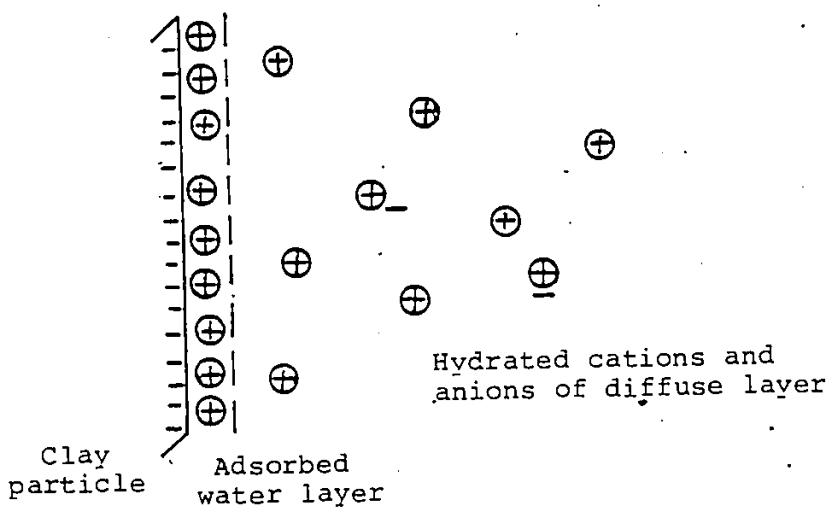


Fig. 2.8 Schematic representation of clay particle with Stern layer and adsorbed water. (After Yong and Warkentin, 1975).

tion accounts for differences between cations of the same group, such as the monovalent alkalis. It breaks down, however, when ions of different valency are compared. The second point can be illustrated by reference to a hypothetical system:

$$Na \cdot clay + [clay]^- + Na^+$$

Thus, the more electropositive the ion, the more it will tend to ionize away from the charged surface, vacating the Stern layer and leaving the surface with a residual charge. This hypothesis, for example, explains the difference between Na^+ and Ca^{2+} but does not account for differences between ions of the same group. The only conclusion to be drawn from the foregoing is that both hydration and electropositivity should be considered simultaneously in explaining the behaviour of different cations.

2.6.3.2 Dessication Shrinking

The preceding discussions have indicated that clay particles in soils are always hydrated, that is surrounded by layers of water molecules called adsorbed water. These water molecules, as noted by Yong and Warkentin (1975), are considered part of the clay surface when considering the behaviour of soils.

Soil behaviour is interpreted by considering the ion

distribution for interacting soil particles rather than the ion distribution around a single particle. Yong and Warkentin (1975) examined the case where the particles are sufficiently close to cause the individual diffuse double layers to overlap. Here, the resulting electric potential* is the algebraic sum of the potentials from single layers. At close spacing of particles, Langmuir (1938) assumed that anions can be neglected.

The resulting ion distribution can be calculated from the equation:

$$n_c = n_o \left[\frac{\pi}{0.32v \sqrt{c_o} X} \right]^2$$

where

n_c = number of cations per unit volume at the midplane between particles,

n_o = number of cations per unit volume in the pore water away from the influence of the surface,

v = valence of the cation,

c_o = concentration of cations in moles/litre away from the influence of each particle surface,

X = midplane distance from surface in Ångstrom.

The constant in the above equation has been explained by Yong and Warkentin (1975).

*The electric potential is the work done to bring a positive unit charge from the reference state to the specified point in the electric field.

When a fresh-water clay soil is placed in a saturated aqueous solution of NaCl, the high ionic strength of the solution causes Na ions to be adsorbed by the clay surface as a result of ion exchange. In the limiting case, the stern layer is 'crowded' with Na ions. The water in the pores is now saturated with Na^+. The effect of this on the fabric of the clay particles can be predicted from the above equation as follows:

At room temperature the solubility of NaCl is 35.7 grams per 100ml of solution (Weast, 1973). This is equivalent to a molar concentration = $\frac{357}{58.44} \approx 6.0$ M. The molecular weight of NaCl = 58.44 (Weast, 1973). Midplane cation concentration = $6.0 \left[\frac{\pi}{0.32 \times 1 \sqrt{6} \times 15} \right]^2 \approx 0.43$M assuming the midplane distance = 15Å.

The concentration of cations in the pore water is, here, 14 times the concentration at midplane. Water molecules held between the clay particles move in response to this concentration gradient due to osmotic activity of the ions. The diffuse double layer is suppressed and interparticle attractions due to Van der Waals forces become significant. This leads to flocculation of the soil particles. The loss of water noted above causes the clay to shrink in the same manner as drying the clay would. Syneresis cracks are consequently formed. Syneresis is the spontaneous separation

of an initially homogenous colloidal system into both a coherent gel and a liquid. Krynine and Judd (1957) have noted that the phenomenon basically involves the drawing together of particles under the action of increased Van der Waals' forces. This would invariably result in the formation of cracks. Such cracks, if induced in dry air, are referred to as dessication cracks. They can also occur under water under the influence of hygroscopic brines, such as concentrated NaCl-brines or potash-precipitating brines.

Several workers (White, 1961; Hudec and Sonnenfeld, 1979; Crooks, 1981) have observed syneresis cracks in clays suspended in salt solution. White (1961) explained that the formation of the cracks was due to the expulsion of waters from the clay-water system by internal forces. The syneresis cracks observed by him had completely random orientations ranging from vertical to horizontal, indicating that shrinkage by syneresis takes place in all directions. He noted, further, that the development of the cracks was a function of the concentration of the salt solution. White (1961) concluded that if the salt concentration is high enough to flocculate the clay, one or more of several structures including syneresis cracks and fissures should occur. The second author in Hudec and Sonnenfeld (1979) observed the formation of honeycomb-like, highly porous, clay layers in a laboratory tank

filled with hypersaline brine. The layers produced horizontal and inclined conduits and vertical fractures vaguely resembling frost action patterns. Crooks (1981) observed horizontal cracks in bentonite compacted by hand in beakers containing solution of concentration as low as 0.1M $CaCl_2$.

2.6.4 Effect of Electrolyte Concentration on Clay Properties

2.6.4.1 Atterberg Limits

The plasticity of clay masses is influenced by the presence of electrolytes in the pore water. The electrolytes may alter the interparticle force and the surface tension of the liquid medium.

Grimshaw (1971) suggested from the work of Speil (1940) and Sullivan and Graham (1940) that the concentration of cations controls the force with which water is held at the surface of the colloidal particle. He noted that the greater the cationic concentration, the more strongly will water be held. A higher proportion of bonded water is obtained with small cations of high valency (for example, Ca^{2+}, H^+, etc.) than with those of large size and small charge. In a plastic mass where the particles are very close, the absorbed water films around each colloidal grain merge together. The force required to rupture such masses, then, will depend on the degree of bonding of the water.

The plasticity in terms of yield point and breaking strength in torsion can be closely predicted by the Hofmeister series (Grimshaw, 1971).

$$H^{+}>Al^{3+}>Ba^{2+}>Sr^{2+}>Ca^{2+}>Mg^{2+}>NH_4^{+}>K^{+}>Na^{+}>Li^{+}$$

Earlier work by Baver (1948), Winterkon (1941), and Skempton and Northey (1952) indicated that consistency-limit values determined by ionized solutions are different from those determined with pure water.

Yong and Warkentin (1975) mentioned that the variation in liquid limit among different mineral groups is much greater than the variation in plastic limits. The plastic limit increases with the surface area of the clay, but not in direct proportion. For example, as reported by Yong and Warkentin (1975), the surface area of montmorillonite is 40 times as great as that of kaolinite, but the plastic limit is only 2-3 times as large.

The liquid limit is much more influenced by exchangeable cations and salt concentration than the plastic limit. Interparticle forces have a more prominent role in determining the liquid limit. The distance between particles, or between structural units of particles, is such that the forces of interaction between the clay particles become sufficiently weak to allow easy movement of particles relative to each other. The dominant interparticle force in

the high-swelling clays such as montmorillonite is one of repulsion. This force of repulsion determines the distances between particles. An increase in salt concentration or substitution of divalent for monovalent exchangeable cations will decrease the repulsion and hence the liquid limit. The type of adsorbed cation, however, has a much greater influence on the higher plasticity minerals, for example, montmorillonite, than on the low plasticity minerals, such as kaolinite.

Michaels (1959) discussed the observations made by Rosenquist (1959) on the physico-chemical properties of clay soils. He noted that the liquid limit is a rough indication of the quantity of mobile or "free" water necessary to provide a lubricating film between adjacent particles in a soil mass. In the expanding-lattice clays, an appreciable portion of the water content at the liquid limit is interlayer-water. This water is effectively immobilized (Mitchell, 1976). The amount of interlayer water in these minerals is greatly dependent on the type of interlayer cation. The greater the amount of water taken into the interlayer spaces, the greater will be the total quantity of water required to provide lubrication between particles. The liquid limit will, therefore, increase with increasing lattice-expansion. Michaels (1959) postulated that in terms of the alkali ions, the liquid limit will often increase in the order:

Cs>Rb>K>Na>Li.

In nearly saturated non-swelling clay masses, only a small fraction of the total water present appears to be entrapped or immobilized by sorption of the solid. The liquid limit, in this case, is controlled primarily by interparticle attractions rather than by a deficiency of lubricant (Michaels, 1959; Mitchell, 1976). Cations with high hydration energies would reduce interparticle attractions and hence the liquid limit.

2.6.4.2 Permeability

Lee (1941) discussed various methods of rendering clay impervious. The complete dispersion of clay through washing with sea water or impregnation with alkali salts is one of the methods he discussed in detail. The chemical process underlying this method is base exchange by which cations attached to the clay particles are replaced by sodium ions from the soil solution. The results of his work indicated that salt water considerably decreases the permeability of soil to water.

Mesri and Olson (1971) investigated the various mechanisms controlling the permeability of clays. They noted that physico-chemical variables exert greater influence on the coefficient of permeability through their influence on dispersion or aggregation of the clay particles. They showed that the coefficient of permeability are largest for

nonpolar fluids, smaller for polar fluids of low dielectric constant, and lowest for water, which is polar and has a high dielectric constant. They noted, further, that the coefficient of permeability is generally lower when the adsorbed cations are monovalent rather than divalent. Mesri and Olson (1971) observed that a reduction in electrolyte concentration, generally, tend to reduce the coefficient of permeability. The effect of electrolyte concentration, however, diminished as the valency of the cation decreased. They noted from the work of Macey (1942) that particle spacing, particle size as influenced by aggregation or dispersion, particle arrangement, adsorbed layers and interlamellar swelling all influenced the permeability. According to Macey (1942), however, the most important cause for the lower permeability of clays in water relative to the one in benzene was the anomalous viscosity of the water near the clay surfaces.

Grace (1953) demonstrated that improved dispersion by the use of certain electrolytes is the main reason for the marked reduction of permeability. Mesri and Olson (1971) concluded from their work that the most important variable influencing the permeability of clays is flocculation of the clay particles, which influences the distribution of void sizes and shapes. Lutz and Kemper (1959) observed from their experimental work that sodium clays were gener-

ally much less permeable to both water and electrolytes than were calcium and hydrogen clays. They noted that the greater the concentration of electrolyte solution the greater the permeability of clay to it. The greatest percentage change in the permeability observed by Lutz and Kemper (1959) occurred with sodium clays and sodium chloride solutions. The sodium clays had lower permeabilities to water than calcium and hydrogen clays. With one exception, they observed that all systems were more permeable to electrolyte solutions than to water.

Lambe (1969) stated that an element of flocculated soil, in general, has a higher strength, lower compressibility and higher permeability than the same element of soil at the same void ratio put in a dispersed state. He attributed the higher strength and lower compressibility in the flocculated state to interparticle attraction and the greater difficulty of displacing particles when they are in a disorderly array. The higher permeability, he explained, results from the larger channels available for flow. Glenn (1968) presented data to show that flocculation increases soil permeability whilst dispersion tends to decrease it.

The foregoing studies and others as well, indicate that the state of the clay-water system has a great influence on the permeability. This suggests that in a

system where other variables are constant, flocculation or dispersion should account for large differences in permeability.

2.6.5 Application to Brine Storage in Clay Liners

The storage of hypersaline brine in clay liners is a common practice in the oil industry. Clay is used mainly because it is the least expensive material normally available on the site and, also, because it has a low permeability, at least, to water.

Alberta Environment (1977) notes that brine storage reservoirs lined with clay are not impervious and that some amount of brine seepage into surface and groundwater systems will occur. The amount of seepage is a function of the permeability and thickness of the clay liner, the degree of saturation, the electrolyte concentrations and the depth of brine solution in the reservoir. In some situations, faulty liners have been responsible for salt water seepage from clay-lined reservoirs.

Alberta Environment (1977) has published guidelines for brine storage reservoirs. In that report it has been assumed, rather erroneously, that the only factors contributing to seepage are:

(a) Incomplete compaction of the clay liner;

(b) freeze/thaw effects on the exposed clay liner;

(c) inadequate thickness of the clay liner;

(d) improper construction of the reservoir dyke;

(e) damage to the clay liner by construction equipment; and

(f) inadequate maintenance over the operating life of the reservoir.

The preceding discussions have indicated that freshwater clay in contact with a highly concentrated electrolyte solution such as brine will flocculate and develop cracks. A solution to the problem of seepage, based solely on the above-mentioned factors is, therefore, inadequate. It must be noted that storing hypersaline brine in clay-lined reservoirs is different from storing fresh water or relatively low salinity water in the same reservoir. Leakages in brine ponds constructed from untreated clay can be explained by increased permeability of the clay due to the formation of syneresis cracks.

In Section 2.6.3 dessication shrinking was attributed to a reduction of the thickness of the diffuse double layer as a result of Na cation exchange. Hudec and Sonnenfeld (1979) have explained that apart from dessication shrinking, heat-induced shrinkage can also affect brine storage reservoirs. The heat in this case arises from two probable sources: solar heating and geothermal heating of brine as it is pumped from salt caverns. The former becomes significant when there is sufficient density stratification

(Hudec and Sonnenfeld, 1979).

In conclusion, it must be noted that salts in water contained in a clay-lined structure offer a potential cation exchange source. This can, over a period of time, alter the characteristics of the clay structure so that design features are no longer present. This must be taken into account during the design, construction and placing of clay liners.

CHAPTER III

SAMPLING AND GEOLOGY

3.1 Sampling

Loose, disturbed samples of clay soils were collected from 10 locations in Southwestern, Central and Eastern Ontario. The first set of nine samples were collected by Peter Seymour in the summer of 1980 and the second set of ten by the author during the summer of 1981. The soils which comprised till and lake clay were of glacial origin. Sampling was done from river banks and clay pits by loosening the soil in the profile face with a spade and placing it into large polythene bags. The bags were tightly sealed for transit. On the average, two samples, each weighing approximately eight kilograms were taken from each location.

3.2 Geology

The surface glacial deposits of Southern Ontario are of Late Wisconsin age (Prest, 1970). During the advance of the ice in Late Wisconsin time the till was deposited from the base of the ice. A complex of till, outwash and glaciolacustrine sediments was deposited during the retreat of the last ice from the Lake Erie and Lake Ontario basins.

When the Late Wisconsin ice moved south into the

United States the weight of the ice caused depressions in the earth's surface. The land surface was still depressed when the ice vacated the Lake Erie and Lake Ontario basins. As a result, a series of high-level glacial lakes occupied the basins. Meltwaters from the receding ice provided an influx of fine-grained sediments into the lakes. This gave rise to extensive deposits of glaciolacustrine sediments which are now found at the surface over much of Southern Ontario.

The samples taken for this study were from underlying till and overlying glaciolacustrine sediments.

Figure 3 shows the ten sample locations. These are listed in Appendix A along with the sample descriptions.

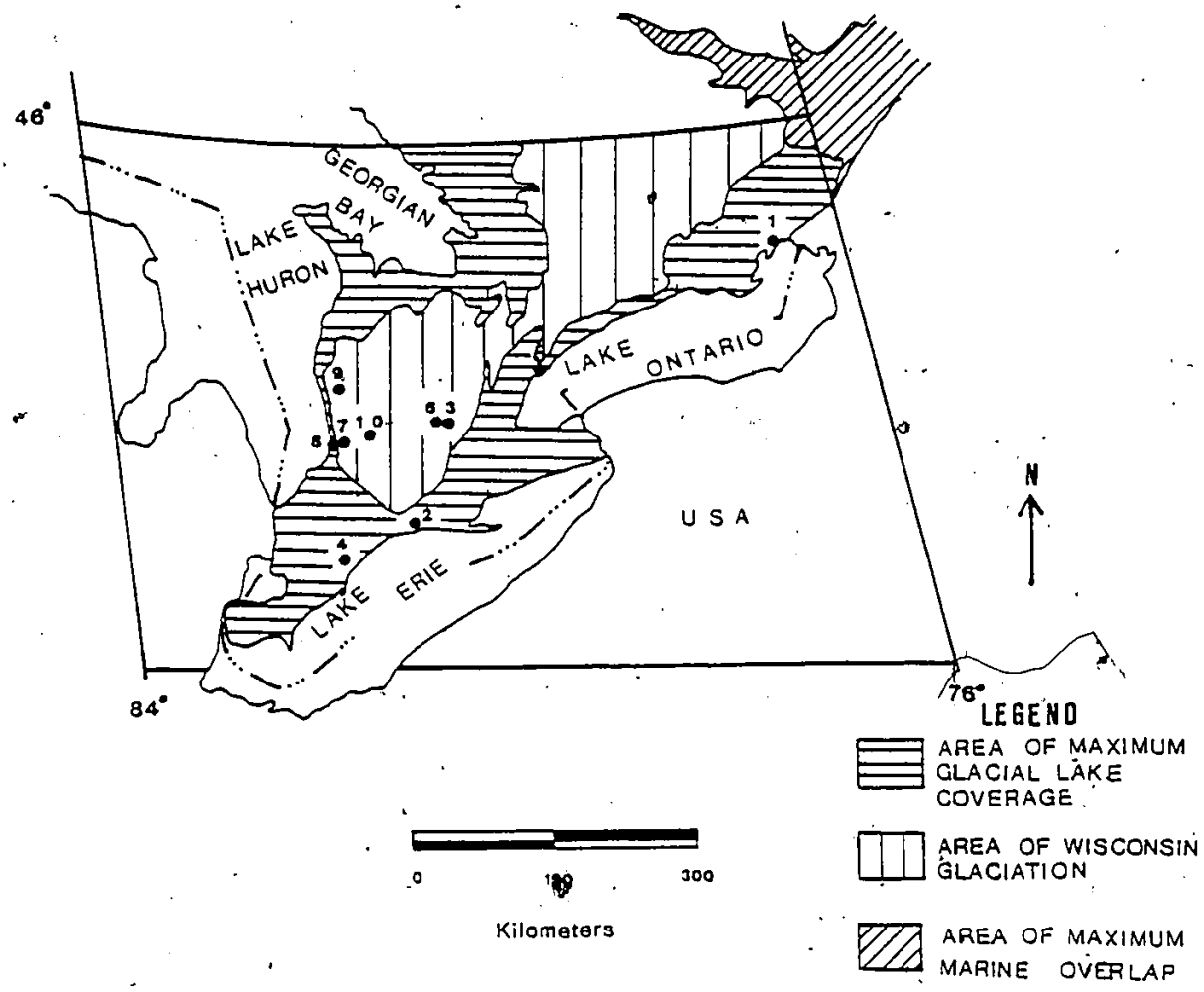


Fig. 3 Map Showing Sample Locations in Eastern, Central and Southwestern Ontario.

CHAPTER IV

EXPERIMENTAL PROCEDURES

4.1 Outline

Five series of tests were performed during the study. These were particle size distribution, water adsorption, Atterberg limits (liquid and plastic limits), Standard Proctor compaction, and permeability tests.

The natural moisture contents of 19 clay soils sampled as discussed in Section 3.1 were determined immediately on arrival in the laboratory. All the 19 samples were analyzed for their particle size distribution by wet sieving on the coarser fractions and hydrometer analysis on the portion passing No. 200 sieve.

Atterberg limits (liquid limit using the ASTM liquid limit device, and plastic limit) were determined for all samples. Fresh samples were treated by soaking in a saturated solution of sodium chloride (NaCl) at room temperature for seven days. Atterberg limits were determined for the treated samples. It has been shown that there is a relationship between the Atterberg limits and adsorption in clay soils. To verify this relationship, adsorption of water vapour at 22°C and 98% relative humidity was deter-

mined for both original and treated samples.

The optimum moisture content and maximum compaction density were determined for original and treated samples, using the Standard Proctor test.

Permeability testing was the main focus of this study as it was expected that it would help explain the leakages encountered in the storage of brine in clay-lined reservoirs. A large-area permeameter was therefore designed to simulate field conditions. Six of such permeameters were machined to speed up the tests. On the whole, five sets of permeability tests at optimum compaction were conducted. Untreated samples were first tested for their permeability to water using the variable head test. Without disturbing the set-up, water was replaced with brine (concentrated NaCl solution) and the test repeated. Permeability to brine and then to water were also determined for treated samples. Long-term permeability (over a period of seven days) to brine and water were determined for both original and treated samples. All the permeability measurements were made at room temperature.

4.2 Description of Procedures

4.2.1 Particle Size Distribution Test

4.2.1.1 Wet Sieving

About 500 grams of each of the samples were dried in a forced-air circulating oven at a temperature of 65°C for

24 hours. They were allowed to cool, then placed on the No. 200 sieve and carefully washed using tap water until the water was clear. The residue was poured, using back-washing into a weighed evaporating dish and allowed to stand for a short period of time until the top of the suspension became clear. As much of the clear top water as possible was poured off; the dish with the remaining soil-water suspension was then placed in the oven for drying.

The oven-dry residue was then cooled and weighed to obtain the weight of the dry soil retained on the No. 200 sieve. The dry soil was then passed through a stock of sieves (U.S. Sieve No. 5, 4, 10, 30, 100, and 200) by sieving, according to the procedure outlined by Lambe (1951), in a mechanical shaker.

4.2.1.2 Hydrometer Analysis

Approximately 200 grams of each sample were oven-dried at 65°C, as above. The oven-dry sample was cooled and then broken as fine as possible using a porcelain mortar and a rubber pestle. It was then sieved through the No. 200 sieve by hand shaking and exactly 50 grams of the portion passing the sieve was taken for hydrometer analysis. The procedure used in the hydrometer test was according to ASTM (422-63, revised 1981).

4.2.2 Atterberg Limits Tests

The Atterberg liquid and plastic limits were determined for the 19 samples according to ASTM (423-66, revised 1981) and ASTM (424-59, revised 1981) respectively. The tests were performed on air-dried samples passing U.S. No. 40 (0.420 mm) sieve. The liquid limit was conducted with the liquid limit device and an ASTM grooving tool.

For moisture content determinations, samples were dried in a Blue M forced-air circulating oven at 105°C for 24 hours, and weighed on CP 440 Mettler delta range balance sensitive to 0.01 gram.

The liquid limit was defined as that moisture content at which a pat of soil placed in the brass cup, cut with the ASTM groove underwent a groove closure of 1/2 inch when dropped from a height of 1 cm 25 times. The plastic limit was obtained by measuring the water content of the soil when a thread can just be formed when it is rolled down to a diameter of 1/8 inch (approximately 3 mm) by hand on a glass plate.

4.2.3 Adsorption Test

Adsorption of water vapour was determined on samples prepared in the same way as those for the Atterberg limits tests. Samples, air-dried and passing the No. 40 sieve, were placed in mason jars clearly labelled. The caps were

removed and the samples dried at 65°C in a forced-air circulating oven until approximately, constant mass was obtained for each of them. This took 48-72 hours. At the end of the drying period, the samples were removed from the oven and tightly capped as quickly as possible to minimize contact with the atmosphere. Before weighing, samples were allowed to cool to room temperature over activated silica gel in a dessicator. A Sartorius single pan balance was used for the determination of the mass to an accuracy of 0.01 gram.

The oven-dry samples were placed under 98% relative humidity conditions at 22°C in a humidity chamber (Fig. 4.1) for a period of 120 hours. The 120-hour period was assumed to be sufficient to attain close to equilibrium conditions in the samples. Mass increases were then determined as the differences between the masses of the samples after and before exposure to water vapour in the controlled chamber. Adsorption to water vapour was reported as the mass of adsorbed water in milligrams per gram of dry soil. The chamber humidity was maintained by a saturated aqueous solution of hydrated cupric sulphate ($Cu\ SO4 \cdot 5H_2O$). This solution provides a humidity of 98% at 20°C (Weast, 1973). The humidity chamber was placed in a closed, temperature-controlled room (a modified walk-in freezer). The Sartorius balance was also

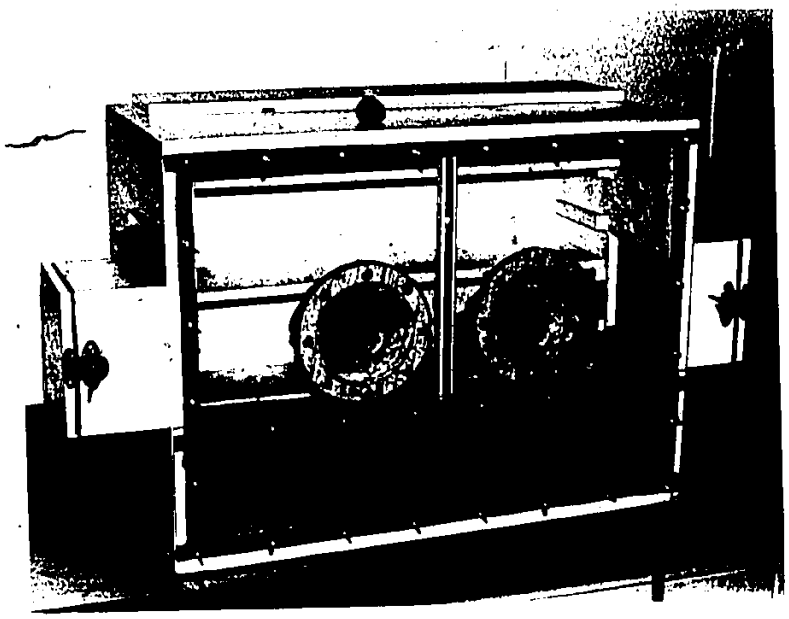


Fig. 4.1 Photograph showing humidity chamber used in adsorption test. Pans in the bottom contain saturated $CuSO_4 \cdot 5H_2O$ solution for maintenance of humidity.

placed in the walk-in freezer so that weighings could be performed at the same temperature at which adsorption had taken place.

4.2.4 Standard Proctor Compaction Test

The sample of soil passing U.S. No. 4 sieve was mixed with the required amount of distilled water and manually compacted in a mould 116.8 mm (4.6 in.) high, 101.6 mm (4 in.) diameter, with a 2.5 kg (5.5 lb) hammer and 304.8 mm (12 in.) drop. The procedure used is outlined by ASTM (D698-78, revised 1981). Moisture contents were determined in the same way as those for the Atterberg limits.

4.2.5 Treatment of Samples with Brine

Saturated brine solution was prepared by dissolving sodium chloride (NaCl) crystals in warm distilled water. At the beginning of the preparation, the temperature of the distilled water was raised to about 60°C using a hot plate. Powdered sodium chloride crystals were added and the solution stirred continuously until no more salt would dissolve at room temperature. Any excess salt was separated by pouring off the liquid portion into a clean pail.

Samples of the 19 clay soils were soaked in the saturated brine solution (prepared as above) at room temperature, covered, and allowed to stand for 7 days. It

was expected that the 7-day period was sufficient to establish equilibrium conditions in the samples. At the end of the treatment period, excess salt solution was poured off and the samples were used for appropriate tests.

Atterberg limits, adsorption, and standard Proctor compaction tests were repeated for the treated samples. Saturated brine solution was used as the moulding fluid.

4.2.6 Permeability Measurements

The measurement of permeability was the most important aspect of the laboratory testing. It was mentioned in Section 2.6.5 that leakages have been observed in brine storage reservoirs constructed from untreated clay. The permeability experiment in this study was therefore designed to simulate conditions in the field and thereby explain the leakages. In addition, equipment was needed to enhance the observation and study of the dessication cracks mentioned in Section 2.6.5. A simple, large-area permeameter was therefore designed. The tests were conducted on the first 18 samples using the variable head test. Sample No. 19 was eliminated in the permeability tests because its grain size distribution and Atterberg limits indicated that it was not a clay.

4.2.6.1 Design and Mode of Operation of Permeameter

A permeameter cell of 152.4 mm (6 in.) inside diameter, 6.35 mm (0.25 in.) wall, and 127 mm (5 in.) height

was machined from plexiglas. The cell had an overflow and inflow provided with microedge stainless steel screens of 0.406 mm (0.016 in.) diameter holes. A 6.35 mm (0.25 in.) diameter graduated standpipe (also of plexiglas) joined the cell to a source of permeant supply by means of two needle valves made from brass (see Fig. 4.2). An O-ring seal was provided at the top of the cell to prevent any leakages during flow. A threaded plexiglas top was tightened with a wrench. In this way, the sample was confined in the cell so that no movement of sample was possible during permeation.

The compacted soil sample was held between sand layers at the bottom and top of the cell. A piece of nylon cloth wrapped around a 1 mm hole fibre glass screen, cut to the same diameter as the cell, separated the sample faces from the sand. Prior to the placing of the sample in the permeameter, a complete sieve analysis was performed on a sample of dark-grey sand. The portion passing U.S. Bureau of Standards No. 16 sieve (of 1.18 mm size) but retained on No. 30 (0.6 mm) was taken, washed several times until the water was clear, dried in an oven and used as above. These sizes were chosen because it was expected that a coarser portion would have made the compaction of the sample in the cell difficult due to lack of a horizontal base. A finer fraction, on the other hand, would have introduced

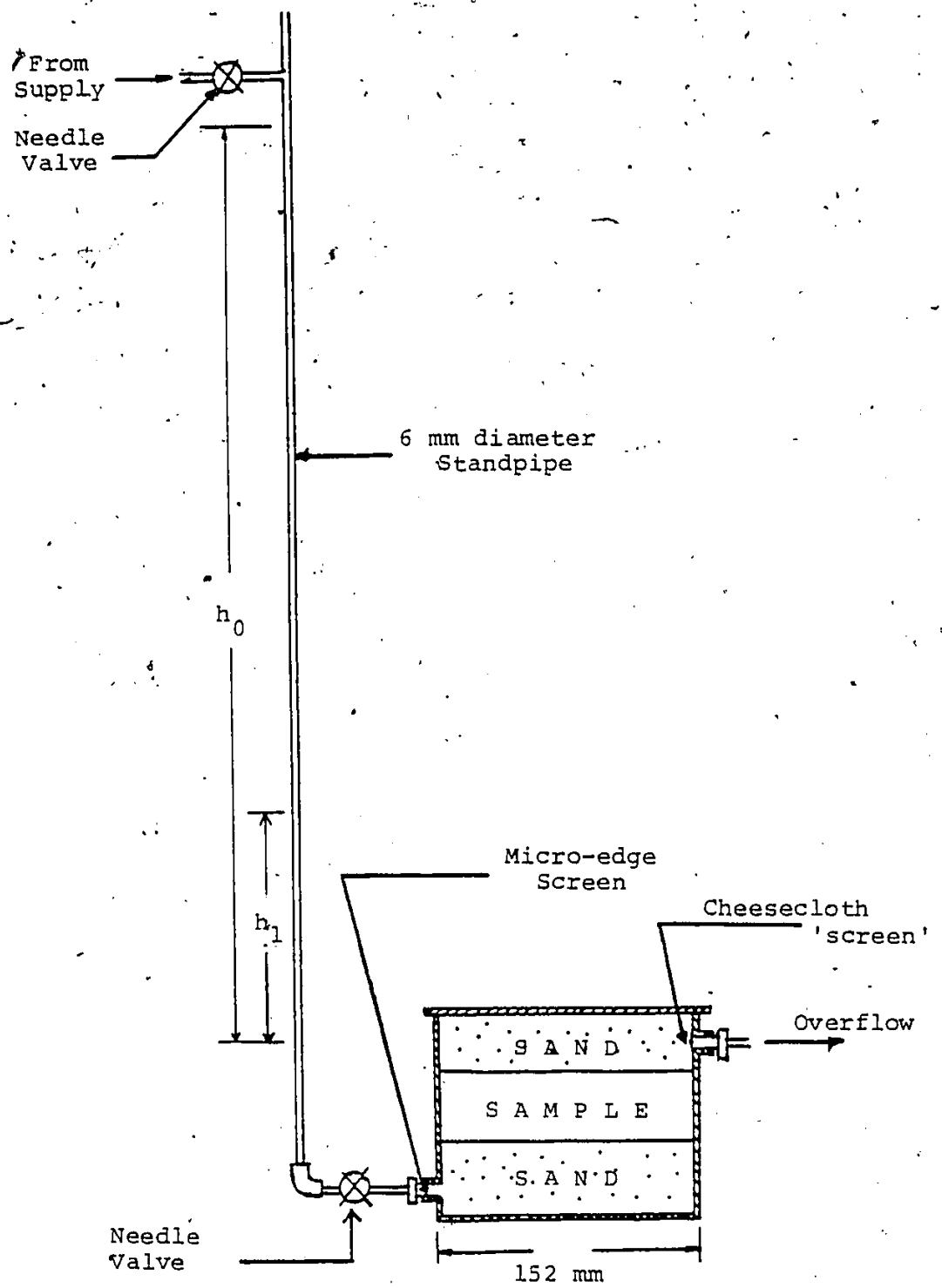


Fig. 4.2 Schematic Diagram of Equipment Used For the Permeability Tests.

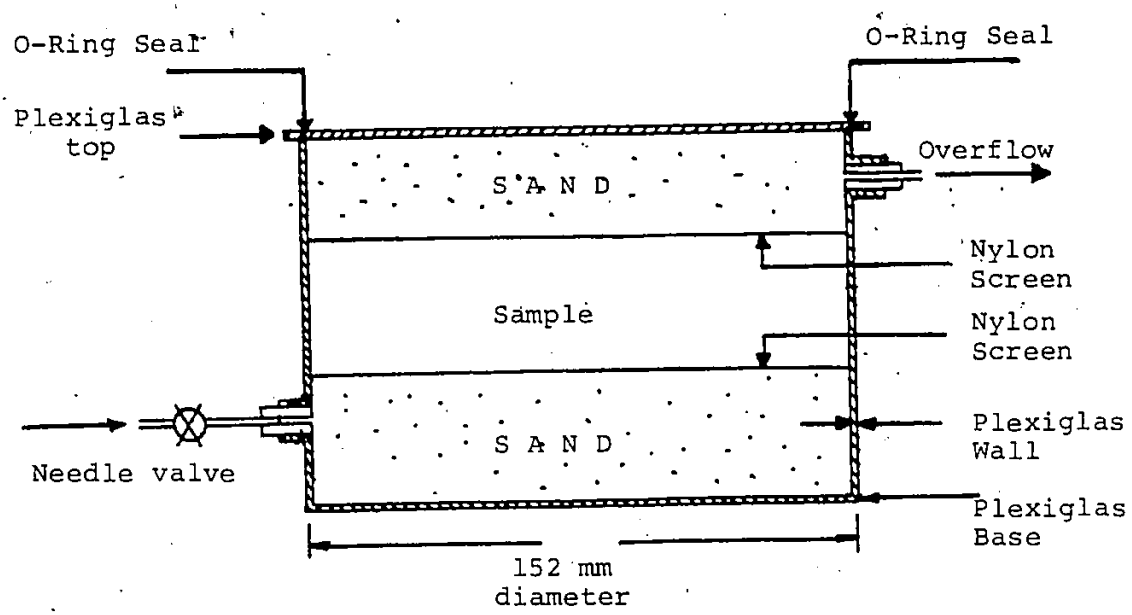


Fig. 4.3 Sectional Elevation of Typical 152 mm diameter Plexiglas Permeameter Cell.

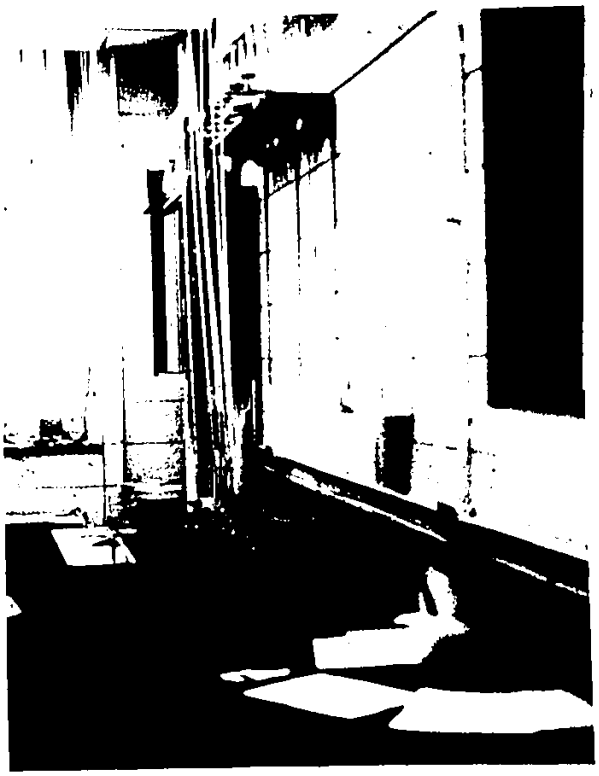


Fig. 4.4 Photograph showing Equipment Used For the Permeability Tests.

an extra complication into the value of the head drop across the sample. The use of the sand below and above the sample in the cell made it possible to vary the length of sample when necessary. Six such permeameters were used simultaneously to speed up the testing.

4.2.6.2 Sample Preparation

A sample of soil passing the No. 4 sieve was mixed with the required amount of moulding fluid to its optimum moisture content and compacted in a 152.4 mm (6 in.) mould using the standard Proctor hammer. Fifty (50) hammer blows were applied to the sample in the mould; it was then extruded and the faces trimmed so that they were horizontal. The sample was slowly placed in the permeameter cell on clean dry sand covered with 'nylon screen.' This served as a 'porous plate.' The compaction of the sample was continued with 130 blows from the Proctor hammer to approximately its maximum compaction density. In this way, the difficulty of compacting the sample directly on the sand layer was removed. Trial tests run at the start of the experiment indicated that this method of compaction gave the same compactive energy per unit volume and that the dry densities attained were within 99.0-101.0% of those obtained in the standard Proctor tests. Table 4.1 lists the two densities at the optimum moisture content for samples AU-3, SC-5, BE-6, and SH-9.

Table 4.1

Means and Variances of Dry Densities

Sample	Optimum Moisture Content (%)	Dry Density Attained by Compaction in Cell (kg/m^3)	Maximum Dry Density from Proctor Compaction (kg/m^3)
AU-3	18.0	1684.0	1701.0
SC-5	26.8	1595.1	1580.0
BE-6	18.6	1655.2	1657.0
SH-9	22.6	1579.3	1577.0
F-value = 1.51		Mean = 1628.40	Mean = 1628.75
Student's t-value = 0.009		Standard deviation = 49.43	Standard deviation = 60.75
$F_{3,3,0.01}$ = 29.46		N = 4	N = 4
$t_{6,0.01}$ = 3.143			

In the above table, the means and variances of the dry densities have been statistically compared (see Appendix D). The result indicates that the two means belong to the same population at the 1% level of significance. It follows that the two methods of compaction statistically achieved the same maximum dry densities.

A sample thickness of 2.5 cm was used throughout the tests because it gave measurable permeabilities within a reasonable period of time.

After levelling the top of the compacted sample in the permeameter to a length of 2.5 cm, the 'nylon screen' was

placed on top. Dry sand, prepared as already described, was then added until the permeameter was filled to the top. The cell was then tightly sealed using a wrench. Trial experiments had shown that sand particles tend to block the holes in the overflow screen during flow. The screen was therefore replaced with a piece of cheese cloth cut to the same size. This ensured that only permeant, but no sand, flowed out during the tests.

A potential error in the permeability results was expected to be the flow along the contact between the sample and the cell wall. To detect whether boundary leakage took place, two specimens of sample BR-2 were prepared - one to be tested in a cell coated on the sides with petroleum jelly before the placement of specimen, and the other in a cell without jelly. Nearly the same values of the coefficient of permeability were obtained for each method of testing. It was concluded that reliable tests could be performed on samples compacted in plexiglas cell and that leakage between the sample and the cell was not taking place.

4.2.6.3 Permeability of Untreated Clay

After the sample had been prepared as above, the needle valves were opened and distilled water from the supply was allowed to flow through the sample under a

pressure head of 95 cm. It was expected that the sample had achieved sufficient saturation when there was an overflow. When a constant rate of flow was obtained, the heads h_o and h_1 were located and measured on the standpipe. The top limit of h_o was selected at the upper end of the standpipe and h_1 a few centimeters above the lower end such that the head difference to be applied was 95 cm. Checks were made to ensure that there were no leakages and that there was no air in the line between the standpipe and the permeameter. Air bubbles trapped in the standpipe were removed by lowering a long steel wire down the standpipe. The standpipe was then filled with distilled water to an elevation which is a few centimetres above. After this, the valve at the top of the standpipe was closed; the timer was started as the water level fell to h_o and the elapsed time when the water level reached h_1 was recorded. The temperature of the permeant was also recorded.

The water in the supply tank was next replaced with saturated brine prepared as in Section 4.2.5. Without disturbing the sample in the permeameter, the brine was allowed to 'saturate' the sample until a constant rate of flow was established. Measurements of elapsed time, h_o and h_1 were obtained as above under the same pressure head.

4.2.6.4 Permeability of Treated Clay

The clay soil treated as in Section 4.2.5 was compacted at optimum moisture content using the method outlined in Section 4.2.6.2. First, brine was allowed to flow through the treated sample placed in the permeameter as described above. Measurements of elapsed time, h_o, and h_1 were obtained in the usual way under a pressure head of 95 cm.

Next, the brine was replaced in the supply tank with distilled water and without allowing time for the leaching of the sample, measurements of time, h_o, and h_1 were obtained as above. The test was performed on all the 18 samples.

4.2.6.5 Long-Term Permeability Measurements

Another aspect of the study was to investigate the variation of permeability with time. Six samples were randomly selected for permeability testing over a period of 7 days or 168 hours. The test was performed on both original and treated samples using brine and water as the permeants under a pressure head of 95 cm.

The long-term permeabilities of the untreated and treated samples to brine were particularly useful. First, they offered an investigation into the rate of 'deterioration' of the untreated clay in a brine environment. They also provided a possible assessment of pre-treatment of clay with brine as a means of reducing brine seepage in

clay liners.

4.2.6.6 The Variable Head Permeability Equation

The variable head equation was used to compute the coefficient of permeability k in all the above cases. The following variables and parameters are defined with reference to Figure 4.2.

a = cross-sectional area of the standpipe

L = length of soil sample in permeameter

A = cross-sectional area of the sample

t_o = time when permeant in standpipe is at h_o

t_1 = time when permeant in standpipe is at h_1

h_o, h_1 = the heads between which the permeability is determined.

A consideration of the hydraulics of the set-up indicates that as the head decreases, the velocity of permeant through the sample decreases. At some head, h, between h_o and h_1 the velocity $V = k\frac{h}{L}$. During an interval of time dt the head will fall a distance dh in the standpipe and the quantity of water dq flowing through the sample can be expressed by Darcy's Law as

$$dq = k\frac{h}{L} A\ dt$$

This quantity of permeant measured in the standpipe is equal to dh, so

$$dq = k\frac{h}{L} A\ dt = -adh \text{ (h decreases with time)}$$

Separating variables,

$$dt = \frac{-La}{kA} \frac{dh}{h}$$

Integrating from h_o to h_1

$$\int_{t_o}^{t_1} dt = \frac{-La}{kA} \int_{h_o}^{h_1} \frac{dh}{h}$$

that is,

$$t_1 - t_o = \frac{-La}{kA} [\ \ln h_1 - \ln h_o\ ]$$

$$= \frac{La}{kA} \ln \frac{h_o}{h_1} = \frac{La}{kA}\ 2.303\ \log_{10} \frac{h_o}{h_1}$$

solving for k,

$$k = 2.303\ \frac{La}{tA} \log_{10} \frac{h_o}{h_1} \quad (t = t_1 - t_o)$$

CHAPTER V

RESULTS AND DISCUSSIONS

5.1 Experimental Results

The geotechnical data obtained from the experiments discussed in Chapter IV are shown in Table 5.1. The particle size distribution curves are presented in Appendix B. The definitions of clay, silt and sand used were according to the MIT classification (Lambe, 1951). Dry density - moisture content curves obtained from the Proctor compaction test are also presented in Appendix C. The results of the long-term permeability studies are discussed in Section 5.4. Liquidity index and activity were derived from the Atterberg limits using the definitions presented in Section 2.3.4.

With the exception of samples SC-19 and AU-12, the results of the particle size distribution tests indicated that the soils used in this study were predominantly silty clays. Some of the soils, however, were appreciably sandy. Sample AU-12 had a very high proportion of silt sizes (80.2%) and sample SC-19 contained relatively high (39%) sand fractions. The percentages of clay sizes (<0.002 mm) obtained for samples KN-1 and KN-10 were very high (about

Table 5.1

Geotechnical Characteristics of Some Clay Soils from Ontario

Sample number	Natural moisture content (%)	Clay fraction <0.002 mm (%)	Clay fraction <0.005 mm (%)	Liquid limit (untreated) (%)	Liquid limit (treated) (%)	Plastic limit (%) (untreated)	Plastic limit (%) (treated)	Plasticity index (%) (untreated)	Plasticity index (%) (treated)	Liquidity index (untreated)	Activity (untreated)	Water adsorption (mg/gm of dry soil) (untreated)	Water adsorption (mg/gm of dry soil) (treated)	Optimum moisture content (%) (untreated)	Maximum dry density (kg/m³) (untreated)	Optimum moisture content (%) (treated)	Maximum dry density (kg/m³) (treated)	Permeability to water (untreated) x 10^{-8} cm/sec	Permeability to brine (untreated) x 10^{-8} cm/sec	Permeability to brine (treated) x 10^{-8} cm/sec	Permeability to water (treated) x 10^{-8} cm/sec
KN-1	58	77.0	77.6	90	47	38	28	52	19	0.38	0.68	45.0	51.0	33.0	1328	22.6	1506	0.16	0.54	0.18	0.15
BR-2	30	47.0	74.0	35	27	25	18	10	9	0.50	0.21	23.1	30.9	19.0	1597	13.5	1648	6.13	1.62	6.48	3.88
AU-3	26	24.0	44.0	26	21	18	17	8	4	1.00	0.33	13.0	22.6	18.0	1701	15.9	1782	10.14	4.20	5.80	2.91
HG-4	17	31.5	39.2	31	27	16	17	15	10	0.07	0.48	21.8	50.4	17.6	1734	13.8	1782	2.76	0.11	4.30	13.37
SC-5	25	45.5	69.0	35	31	25	18	10	13	0.00	0.22	20.3	34.8	26.8	1580	19.4	1661	30.20	19.42	32.36	21.57
BE-6	24	41.5	43.8	35	24	21	17	14	7	0.21	0.34	17.8	28.6	18.6	1657	14.2	1769	1.97	3.15	5.53	7.33
SE-7	19	39.0	56.8	28	24	18	10	10	14	0.10	0.26	16.4	24.5	18.0	1794	13.9	1840	3.69	1.08	1.05	1.94
SJ-8	32	45.5	58.0	50	30	29	13	21	17	0.14	0.46	25.2	48.2	22.8	1520	17.6	1534	4.26	2.91	4.17	2.71
SH-9	27	43.0	54.5	49	33	20	23	29	10	0.24	0.67	33.0	37.8	22.6	1577	17.8	1603	2.27	0.68	4.14	3.03
KN-10	53	71.4	84.0	75	55	35	26	40	29	0.45	0.56	36.3	51.4	40.3	1249	28.0	1434	1.29	0.88	0.20	0.18
BR-11	28	44.0	68.0	34	28	22	22	12	6	0.50	0.27	23.8	29.5	18.5	1688	17.7	1742	0.95	1.19	2.14	0.74
AU-12	27	19.5	28.5	27	20	20	18	7	2	1.00	0.04	18.2	23.9	18.7	1662	16.4	1767	120.82	5.39	100.00	50.91
HG-13	28	39.0	59.0	43	32	23	19	20	13	0.25	0.51	25.8	48.2	23.2	1598	21.6	1637	3.43	1.01	3.51	1.42
BE-14	28	27.0	34.0	38	34	20	24	18	10	0.44	0.67	22.6	35.8	20.8	1603	18.6	1647	9.70	6.47	6.51	3.89
SE-15	21	36.0	48.0	28	23	17	16	11	7	0.36	0.31	17.0	33.9	18.1	1747	15.1	1812	2.15	3.34	3.75	1.63
SJ-16	19	34.5	45.8	27	22	16	14	11	8	0.27	0.32	16.2	32.1	13.6	1874	13.4	1880	3.23	0.10	3.04	2.34
SH-17	23	20.0	26.8	23	22	15	21	8	1	1.00	0.40	18.3	22.4	15.2	1832	12.3	1877	11.65	13.05	10.35	6.90
DU-18	28	45.0	56.2	40	31	23	18	17	13	0.29	0.18	23.7	37.6	17.6	1694	18.8	1703	2.48	1.95	33.40	29.02
SC-19	7	16.0	20.5	-	-	-	-	-	-	-	-	11.6	18.3	8.4	2076	7.0	2104	-	-	-	-

Table 5.2

Grain Size Distribution
(MIT Classification)

Sample Number	% Sand Size (<0.06 mm)	% Silt Size (0.002-0.06 mm)	% Clay Size (<0.002 mm)
KN-1	2.5	20.5	77.0
BR-2	2.0	51.0	47.0
AU-3	14.0	62.0	24.0
HG-4	23.0	45.5	31.5
SC-5	4.0	50.5	45.5
BE-6	21.0	37.5	41.5
SE-7	2.4	58.6	39.0
SJ-8	5.5	49.0	45.5
SH-9	9.0	48.0	43.0
KN-10	0.8	27.8	71.4
BR-11	1.0	55.0	44.0
AU-12	0.3	80.2	19.5
HG-13	11.0	50.0	39.0
BE-14	11.0	62.0	27.0
SE-15	6.0	58.0	36.0
SJ-16	18.5	47.0	34.5
SH-17	12.0	68.0	20.0
DU-18	5.0	50.0	45.0
SC-19	39.0	45.0	16.0

two standard deviations from the group mean).

Liquid limits obtained for untreated samples ranged from 23 to 90 percent and plastic limits from 15 to 38 percent. After treatment with brine, the liquid limits ranged from 20 to 55 percent and plastic limits from 10 to 28 percent. Sample SC-19 was non-plastic.

Water adsorption at 22°C and 98 percent relative humidity was reported as the amount of water adsorbed in milligrams per gram of dry soil. The average change in adsorption after treatment with brine was 12.2 mg per gram of dry soil. This represented an increase of 53.6 percent. The untreated sample KN-1 adsorbed the highest amount of water (45 mg/gm of soil). The maximum value of water adsorption (51.4 mg/gm of soil) determined on treated samples was, however, for sample KN-10.

Mean compaction parameters (optimum moisture content and maximum dry density) obtained for untreated samples were respectively 20.6 percent and 1658 kg/m^3. With the exception of sample DU-18, the optimum moisture content showed a decrease after treatment with brine. The maximum dry density was, however, increased.

Coefficient of permeability to water calculated from the variable head equation (Section 4.2.6.6) gave a mean value of 1.1×10^{-7} cm/sec for untreated samples. With the exception of samples SC-5, AU-12 and SH-17, values of

permeability to water determined on untreated samples were an order of magnitude lower than the group mean. Sample AU-12 which represents a clayey silt gave a value of 1.1×10^{-6} cm/sec. For the untreated samples, the permeabilities to brine were, on the average, two orders of magnitude higher than permeabilities to water. Permeability to brine after treatment with brine gave a mean value of 1.1×10^{-7} cm/sec. After treatment with brine sample DU-18 showed only a slight decrease in permeability to brine.

5.2 Sources of Error

Particle Size Distribution Test

The specific gravity of most soil particles lies within the range of 2.65 to 2.85 (Lambe, 1951). Means and Parcher (1963) reported a typical value of 2.72 for sandy or silty clays. A specific gravity of 2.70 was assumed for all the samples during the particle size distribution test. The actual specific gravities of the soils used in this study could be different from this value. The difference would affect the results of the hydrometer test and hence the particle size distribution curves presented in Appendix B.

B. Atterberg Limits Test

Although variations in room temperature and humidity could have taken place during the Atterberg limits test, they were not taken into account.

Adsorption Test

It was assumed during the test that 120 hours (5 days) was sufficient time to establish equilibrium between the samples and the atmosphere in the humidity chamber. This assumption was based on the work of Orchiston (1954) who observed that this period was sufficient to obtain equilibrium for high-sorbing clay samples.

The temperature within the humidity chamber was monitored during the adsorption test. The humidity was maintained by a freshly-prepared saturated aqueous solution of hydrated cupric sulphate ($CuSO_4 \cdot 5H_2O$). This solution provides a relative humidity of 98% at 20°C within a close space (Weast, 1973). The chamber was equipped with fans to ensure an air flow over the salt solution. The temperature in the chamber observed during the test was 22°C ± 1°C. The relative humidity of 98% reported in this study could, therefore, be in error. Rigbey (1980) observed that temperature variations of less than ±1°C resulted in humidity variations of less than ±2%.

Permeability Tests

During the compaction of samples in the permeameter cells, it was possible that some of the samples attained compaction densities that were slightly less than maximum (Section 4.2.6.2). For such samples, values of permeability

presented in Table 5.1 would not be minimum.

No provision was made for the control and measurement of the degree of saturation during the permeability tests. A substantial increase in the degree of saturation of a soil is known to result in an increase in permeability.

Temperature variations of permeants up to 2°C were observed during the permeability tests. These variations are thought to have affected the viscosity of the fluid and hence the permeability of the soil.

5.3 Mineralogical Composition and Classification of Samples

Since the principal aim of the present study was to investigate the geotechnical properties of the soils used, chemical and mineral analyses were not considered necessary. Guillet (1977) has made detailed chemical and mineral analyses of samples collected from the same locations as those used in this study. In subsequent discussions, reference will be made, where relevant, to Guillet's results part of which is presented in Table 5.3.

It can be seen from the table that illite is the dominant clay mineral in these clays. Chlorite is present in minor amount and montmorillonite in minor to moderate proportion. Due to the prevalence of limestones in the Paleozoic rocks of southern Ontario, most of the clays are limy (Guillet, 1967, 1977). Kaolinite is rarely detected

Table 5.3

Mineral Analyses of Some Ontario Clays
(from Guillet, 1977)

Sample Location	Quartz %	Soda Lime Feldspar %	Illite	Chlorite	Expanding Minerals
Kingston (KN-1,KN-10)*	16	2	A	C	-
Brownsville (BR-2,BR-11)	28	4	A	C	C
Aurora (AU-3,AU-12)	23	4	B	D	C
Highgate (HG-4,HG-13)	46	2	A	C	B(I)
Scarborough (SC-5,SC-19)	24	9	A	B	C(IM)
Beeton (BE-6,BE-14)	32	7	C	C	(A(M)
Seaforth (SE-7,SE-15)	18	3	A	C	ND
St. Joseph (SJ-8,SJ-16)	31	2	A	C	C(I)
St. Helen (SH-9,SH-17	29	3	B	C	B(MI)
Dublin (DU-18)	21	1	A	C	ND

M = Montmorillonite; I = Interlayered; A = Abundant;
B = Moderate; C = Minor; D = Trace; ND = Not detected.

*Letters and number in parathenses refer to samples used in this study.

because it will not form in a limy environment (Grim, 1953).

The general uniformity in the mineralogical contents is well expressed in Fig. 5.1 in which the plasticity indices of the clay soils are plotted against the clay fractions (<0.002 mm). In the diagram, the classification of activity as proposed by Skempton (1953) is shown. It is seen that all the clays used in this study fall in the "inactive" group as they range in activity from 0.18 to 0.68.

Casagrande (1947) provided a classification scheme for cohesive soils based on the plasticity indices and liquid limits. Figure 5.2 provides a good indication of typical Atterberg limit values based on his work. The diagram is referred to as the plasticity chart. It consists of a graph relating the plasticity index (PI) and liquid limit (LL) on which the main dividing axis is a line corresponding to the equation PI = 0.73 (LL-20). This line is known as the "A" line. Seed *et al*. (1964b) have shown that inorganic clays must lie within a small zone of the plasticity chart, and organic clays must lie in a larger but lower zone of the chart, as is known to be the case. Ordinates at LL = 30 and LL = 50 further subdivide the chart.

From the plasticity chart (Fig. 5.2), it can be seen

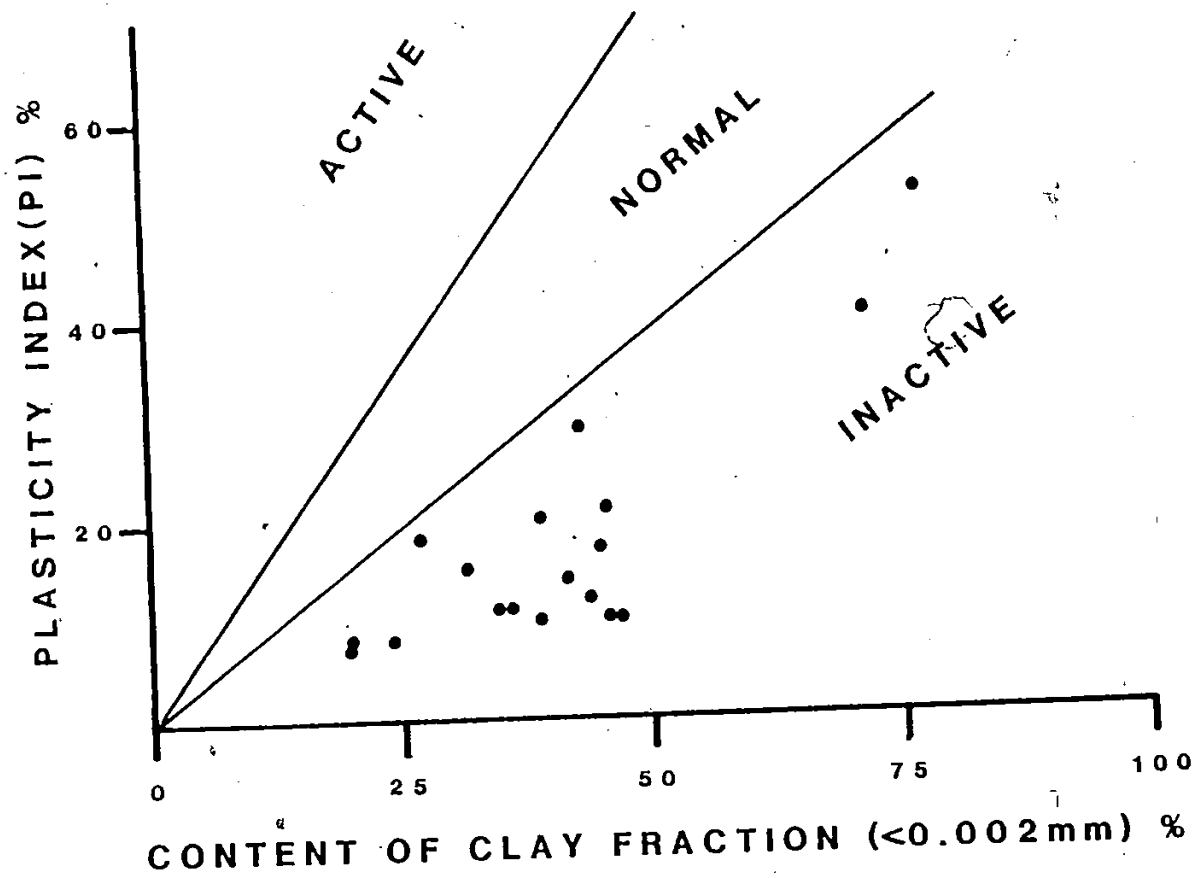


Fig. 5.1 Activity of Clays.

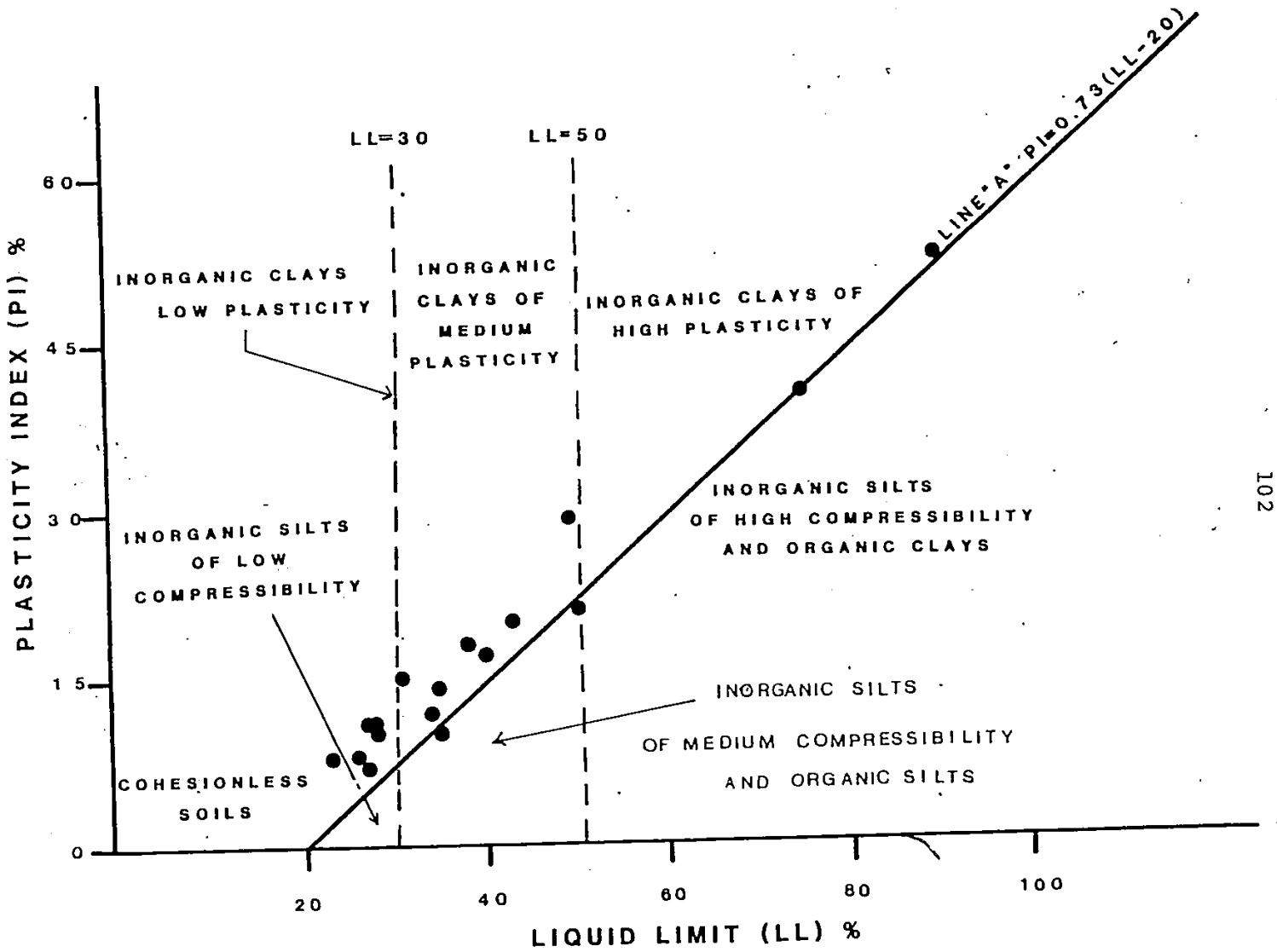


Fig. 5.2 Plasticity Chart for Untreated Samples.

that the soils used in this study were inorganic clays of low, medium and high plasticity. They lie in the same zone on the chart as glacial clays from Detroit and Canada used by Casagrande (1948). Samples BR-2, SC-5 and SJ-8 plot below the "A" line and appear to be more silty than Table 5.2 shows.

5.4 Statistical Analysis

Eighteen (18) samples were used in most of the statistical analysis. Sample SC-19 was eliminated because it was non-plastic, as shown by the results of the Atterberg limit tests. From the results of the grain size analysis, sample SC-19 is seen to contain a high proportion (39.0%) of sand-size fraction.

The geotechnical data presented in Table 5.1 were grouped together for statistical analysis. All the samples were considered as a single population. The data were analyzed by simple linear regression, comparison of means and variances using the F-test and student's t-test and factor analysis.

All statistical analyses were performed on an IBM 370 computer at the University of Windsor using SAS (Statistical Analysis System) packages. The results of the factor analysis are shown in Table 5.8. Means, standard deviations and other statistics are tabulated in Table D.2 (Appendix D). Simple linear correlations and regression

analysis are discussed in Section 5.4.1.

5.4.1 Linear Correlations and Regression Analysis

Correlations between measured variables were calculated using the Pearson 'r' correlation coefficient. This coefficient is designed as an index of association between variables that are linearly related to each other. This simply means that the 'best-fit' graphic line describing the two variables is indeed a straight line. Davis (1973), has defined the Pearson 'r' as the ratio of the covariance of two variables to the product of their standard deviations. The value of 'r' ranges from +1 to -1. A value of +1 describes a perfect direct relationship between two variables. A value of -1 indicates a perfect negative relationship that is, one in which one variable decreases as the other increases. Relationships that are less than perfect take values of 'r' between the two extremes. These include a correlation of zero which indicates the lack of any sort of linear relationship at all.

The line of best-fit or regression was calculated for each correlation. The purpose of the regression was to predict the value of the dependent variables from parameters that were more easily measured in the laboratory. Thus if the independent variable is more convenient to measure than the dependent variable, the latter

may be deleted from the test set and economy of test procedure effected.

A two-tailed student's t-test was performed to determine the goodness of fit for each regression line. The value of t obtained tests for the significance between a population with a mean of r = 0 and the sample population with a mean of r (the calculated correlation coefficient). A very small value for the significance level indicates that the regression parameters contribute significantly to the model.

Fisher (1948), has shown that, for a small sample size, the distribution of r is not sufficiently close to normality to justify the use of a standard error or a probable error to test its significance. A more accurate method has been developed by Fisher based on the distribution of t. For a correlation coefficient r,

$$t = \frac{r\sqrt{n}}{\sqrt{1-r^2}}$$

where

n = number of degrees of freedom available for estimating the correlation coefficient.

The degrees of freedom can always be taken as N-2 where N is the sample size. This is because there is a loss of one degree of freedom for each statistic calculated from

the sample in order to obtain r. These are $\bar{y}$ and b_{yx} (the regression coefficient).

5.4.1.1 Plasticity Index and Liquid Limit

The interdependence of the plasticity index and liquid limit for the untreated samples is expressed in a scatter diagram in Fig. 5.3. Here the plasticity index is the dependent variable. The calculated value of r = 0.9774 is highly significant as seen from Table D.1 in Appendix D. The regression line obtained is represented by the equation PI = 0.67LL - 9.12 or, written in another form, PI = 0.67 (LL - 13.6). This line is less steep than the 'A' line of the Casagrande plasticity chart [PI = 0.73 (LL - 20)] and has a higher intercept on the plasticity index axis. This means that most of the soils used in this study had probably lower plastic limits than those used by Casagrande (1947).

It is generally assumed that the Pearson r requires that measurements be continuous and at the least interval. Some authors, notably Senter (1969), also require that the little distributions of Y scores associated with each value of X (and the converse) exhibit approximately equal amounts of scatter (homoscedasticity). If the two extreme data points shown in Fig. 5.3 are deleted, the value of 0.8926 calculated for r is still highly significant. The equation for the regression line in this case is PI = 0.65LL - 8.64

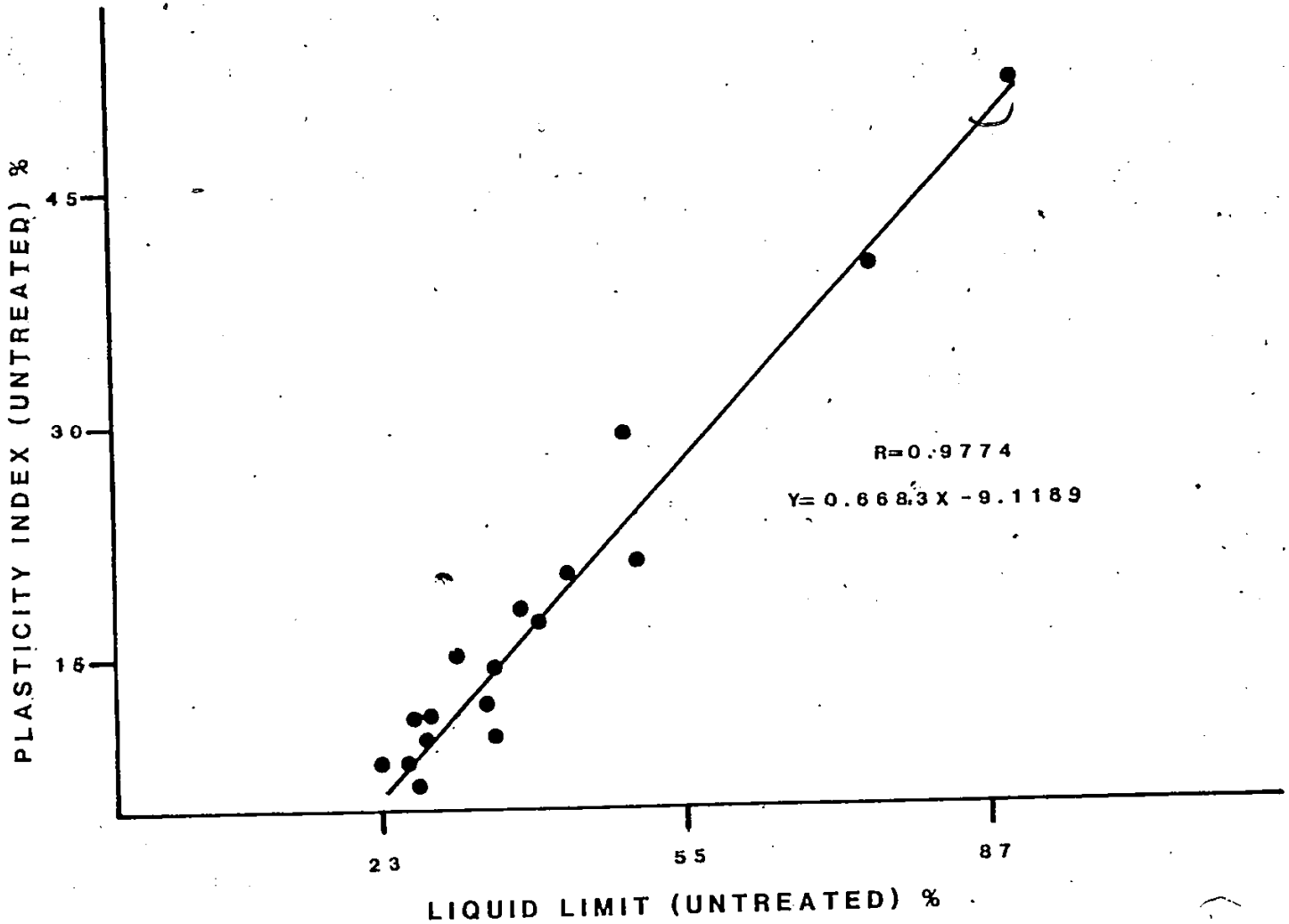


Fig. 5.3 Plasticity Index vs. Liquid Limit for Untreated Samples.

or PI = 0.65 (LL - 13.3). It is seen that the regression equation does not change much. The two extreme data points reflect the high plasticity of samples KN-1 and KN-10. These samples have, also, very high percentages of clay content (<0.002 mm), that is 77.0 and 71.4 respectively. The high values of Atterberg limit values for these samples are probably due to the high proportion of clay sizes.

Figure 5.4 shows the scatter diagram for plasticity index - liquid limit plot for the treated samples. In this case, a lower correlation (r = 0.8771) is obtained. When samples KN-1 and KN-10 are removed from the population, the calculated r value of 0.6786 is not as good but still significant. This poor correlation reflects the greater amount of scatter in the data points obtained for the treated samples. This suggests that the structure of the samples are altered after treatment with brine. This change in structure is shown by the lower Atterberg limit values obtained for the treated samples.

Table 5.4 shows the determined plasticity indices compared with those calculated from the liquid limit using the above regression equations.

The variances and means can be compared by using the method outlined in Appendix D. The results indicate that there is no statistical evidence to suggest that there is any difference in the values of the plasticity indices at

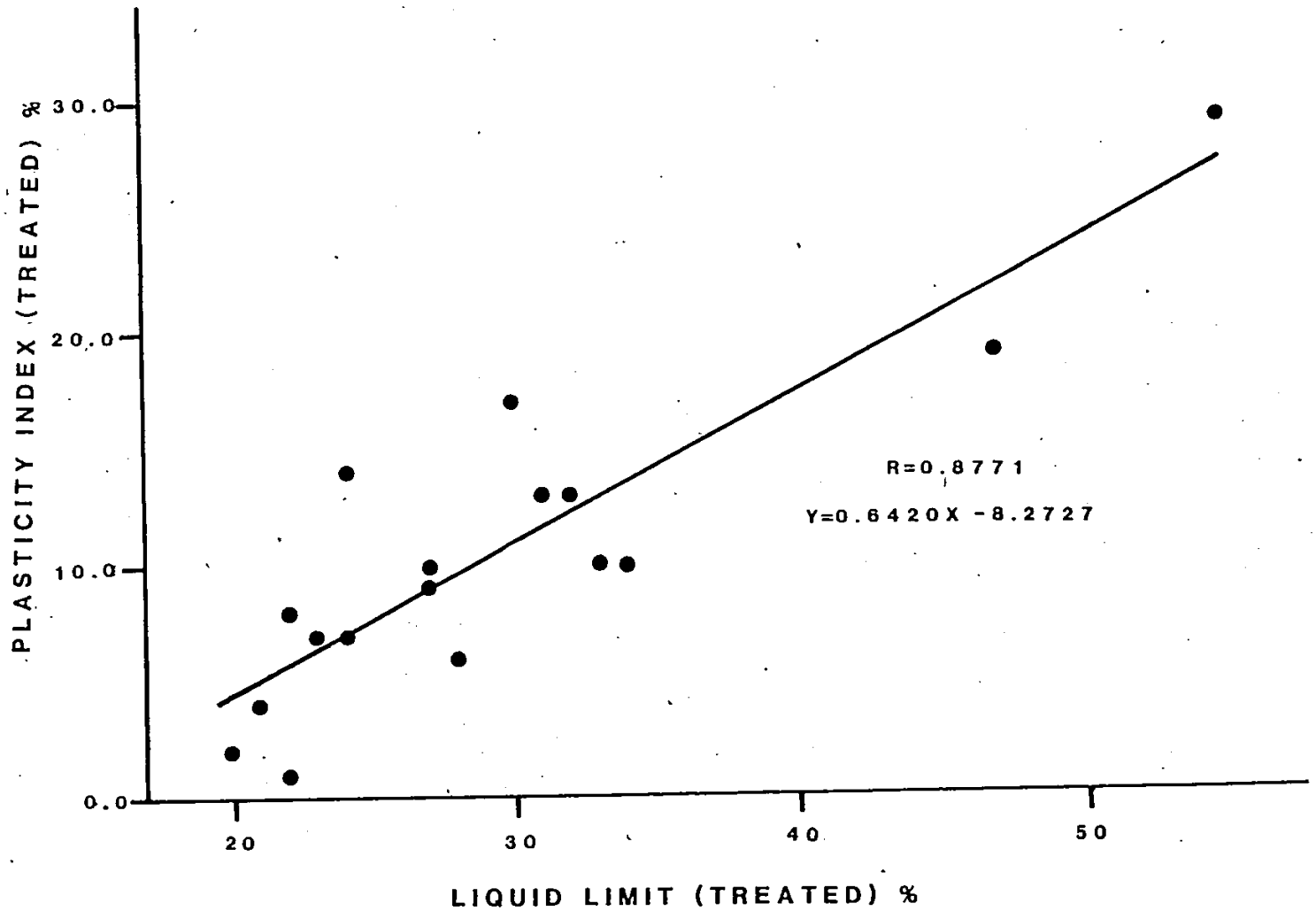


Fig. 5.4 Plasticity Index vs. Liquid Limit for Treated Samples.

Table 5.4

Determined and Calculated Plasticity Indices

Sample Number	Determined Plasticity Index Untreated Samples %	Calculated Plasticity Index Untreated Samples %	Determined Plasticity Index Treated Samples %	Calculated Plasticity Index Treated Samples %
1	10	14.26	9	9.12
2	8	8.37	4	5.12
3	15	11.64	10	9.12
4	10	14.26	13	11.73
5	14	14.26	7	7.17
6	10	9.68	14	7.17
7	21	24.08	17	11.08
8	29	23.42	10	13.04
9	12	13.61	6	9.77
10	7	9.03	2	4.56
11	20	19.50	13	12.38
12	18	16.23	10	13.69
13	11	9.68	7	6.51
14	11	9.03	8	5.86
15	8	6.40	1	5.86
16	17	17.53	13	11.73
	Mean = 13.81	Mean = 13.81	Mean = 9.00	Mean = 9.00
	Standard Deviation = 5.92	Standard Deviation = 5.29	Standard Deviation = 4.44	Standard Deviation = 3.01
	Variance = 32.90	Variance = 26.21	Variance = 18.50	Variance = 8.56
	Sample Size = 16	Sample Size = 16	Sample Size = 16	Sample Size = 16

the 1% level of significance. A value of 0.80 was obtained for the square of the correlation coefficient. This indicates that 80% of the variation in the plasticity index was associated with differences in the liquid limit. The regression equation is, therefore, suitable for predicting the plasticity index from the liquid limit determined in the laboratory. In the case of the treated samples, the variances were not different, though quite close at the 5% level of significance. At the 1% level of significance, however, it could be seen that the variances of the plasticity indices for the treated samples were significantly different. It can be concluded from the foregoing that the equation for the untreated samples is a better predictive tool for the plasticity index than the one for the treated samples.

5.4.1.2 Plasticity Index and Clay Content

Figure 5.5 shows a plot of the plasticity index (untreated) against the percentage clay sizes (fraction less than 0.002 mm). As can be seen from Table D.1 the calculated correlation coefficient of 0.8301 is highly significant. The regression line can be represented by the equation, $PI = 0.66C - 9.34$. The intercept of the line on the percentage clay size axis is 14. It follows from Fig. 5.5 and the above equation that the line of best fit for the data does not pass through the origin. Seed *et al*. (1962, 1964a, 1964b) obtained similar results from their work on

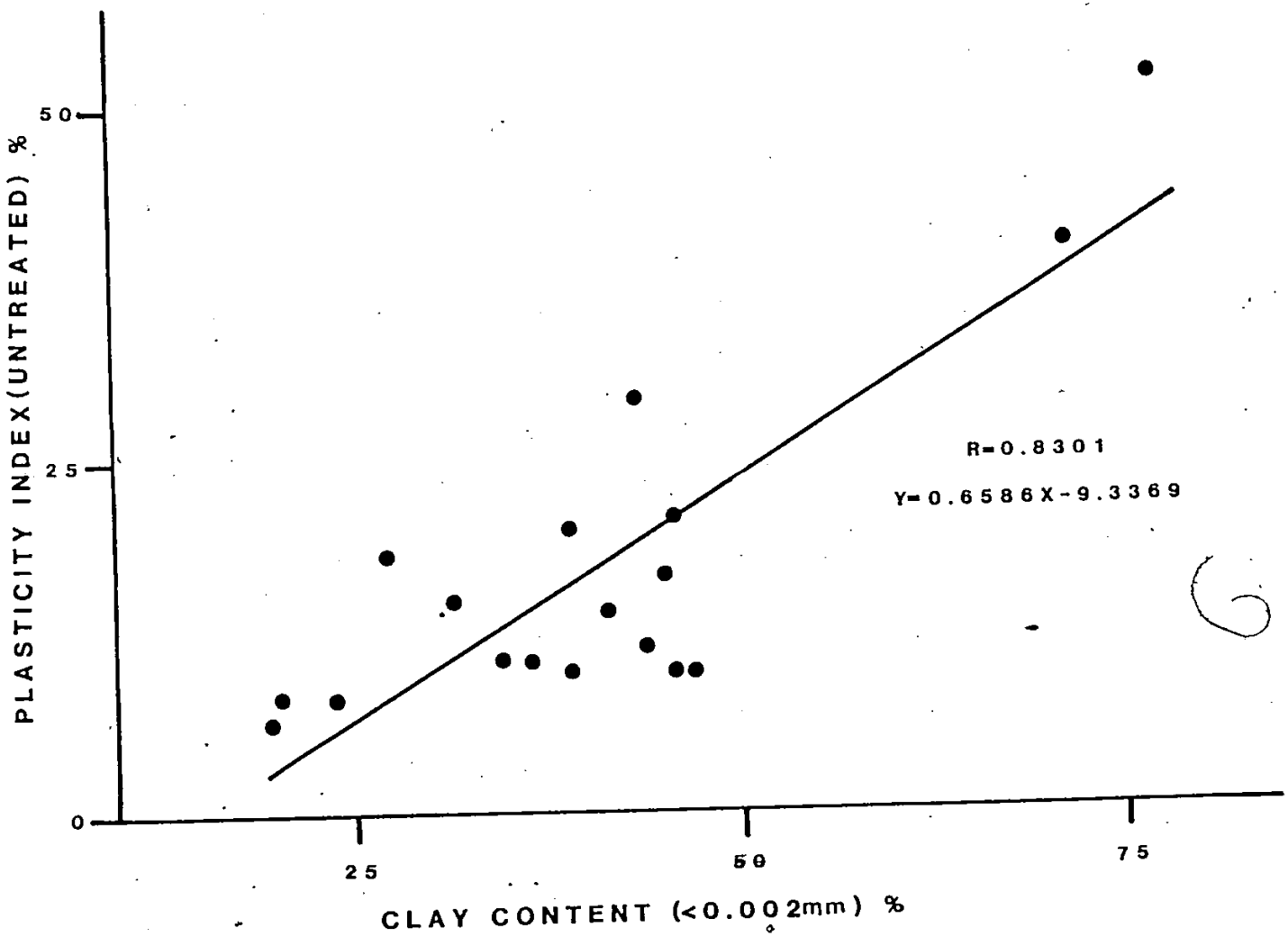


Fig. 5.5 Plasticity Index vs. Percentage Clay Sizes (<0.002 mm) for Untreated Samples.

artificially-prepared sand-clay mineral mixtures. They mentioned that intercepts as high as 10 may be obtained on the percentage clay size axis.

Activity has been defined in Section 2.3.4 as the ratio of the plasticity index to the percentage clay sizes (<0.002 mm). This definition implies that the slope of a line drawn from the origin through the data points in Fig. 5.5 should give the average activity. In situations where the regression line does not pass through the origin, Mitchell (1976) has suggested that the equation

$$A = \frac{PI}{c-n}$$

be used to calculate activity. In this equation, PI is the plasticity index, c is the percentage of material less than 0.002 mm, and n is equal to 5 for natural soils or 10 for artificial mixtures. By comparing this definition of activity with the regression equation, it can be seen that the average activity for the soil is 0.66 and n = 14.

If samples KN-1 and KN-10 are again removed from the population, a value of 0.4265 is obtained for the correlation coefficient for a sample size of 16. This poor correlation may indicate that the linear model is not appropriate for the relationship between plasticity index and clay content. Mitchell (1976) has noted that the true relationship is best described by two straight line segments; one appli-

cable for clay content over about 40% and the other applicable for clay content between about 10 and 40%. If this distinction is, however, applied to the data points shown in Fig. 5.5 the results obtained do not agree with the predictions made by Seed et al. (1964b) and Mitchell (1976). Thus, the most suitable model for the values of plasticity index and clay content obtained in this study is probably the linear equation based on all the 18 samples.

Figure 5.6 shows the plot of plasticity index against percentage of clay sizes (<0.005 mm). In this case, the correlation is poor but still significant. This means that it is more accurate to predict the plasticity index from the percentage of clay size using the size fraction less than 0.002 mm.

5.4.1.3 Atterberg Limits and Water Adsorption

The activity of a clay and its various colloidal properties such as adsorption of water increase with increasing surface area. This area could be determined if the size and shape distributions of the particles in a sample were known. This method is, however, too tedious and hence impractical. Surface area is , therefore, determined by measuring the amount of liquid or gas required to cover the surface of the soil. The Atterberg limits, especially liquid limit, is dependent upon the magnitude of the net attractive forces among the clay components. The greater the surface area and

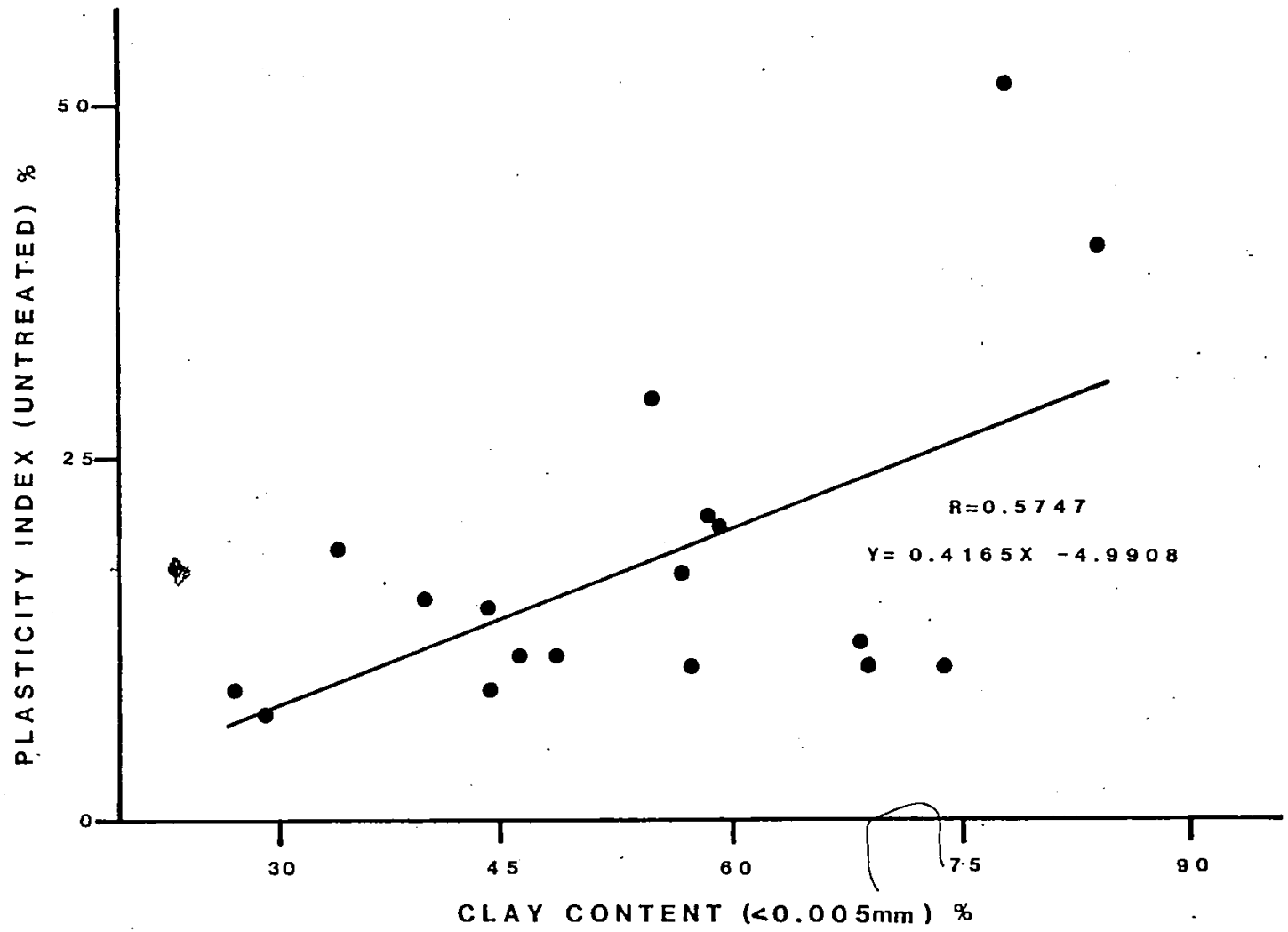


Fig. 5.6 Plasticity Index vs. Percentage Clay Sizes (<0.005 mm).

the greater the attractive intensity, the thicker the water layer held between particles. This will result in a greater liquid limit.

Figures 5.7-5.9 show the plots of the Atterberg limits against water adsorption at 98% relative humidity. The correlation coefficients calculated for the regression lines were 0.9496, 0.9514 and -0.8265 and were highly significant. These results are similar to those obtained by Farrar and Coleman (1967) and Seed et al. (1964). Farrar and Coleman calculated a value of 0.91 for the correlation coefficient at the 99% level of significance. They found that the liquid limit and the total surface area were related by the equation,

$$\text{Liquid limit} = 0.67\ (\text{surface area}) + 0.93$$

They obtained a lower correlation (within 95% confidence limits) between external surface area and the Atterberg limits.

In order to determine the suitability of the regression model for predicting Atterberg limits from water adsorption, values of the predicted liquid limits were generated. The results are shown in Table 5.5 with the liquid limits determined in the laboratory. Using the F and t-tests outlined in Appendix D, it can be seen that there is no statistical difference between the means and variances for the determined and calculated liquid limits. The square of the correlation coefficient gives a value of 0.90. This indicates

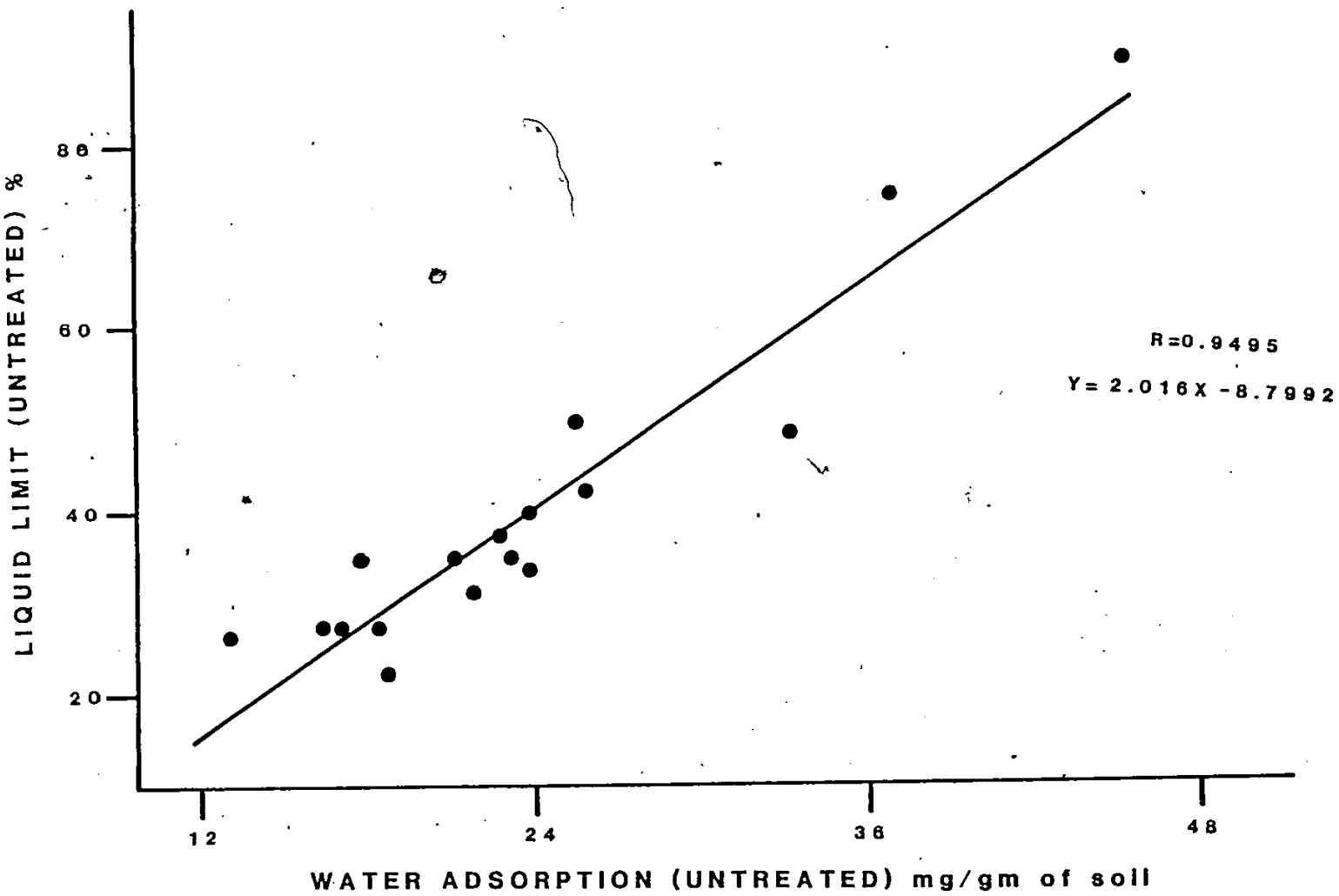


Fig. 5.7 Liquid Limit vs. Water Adsorption.

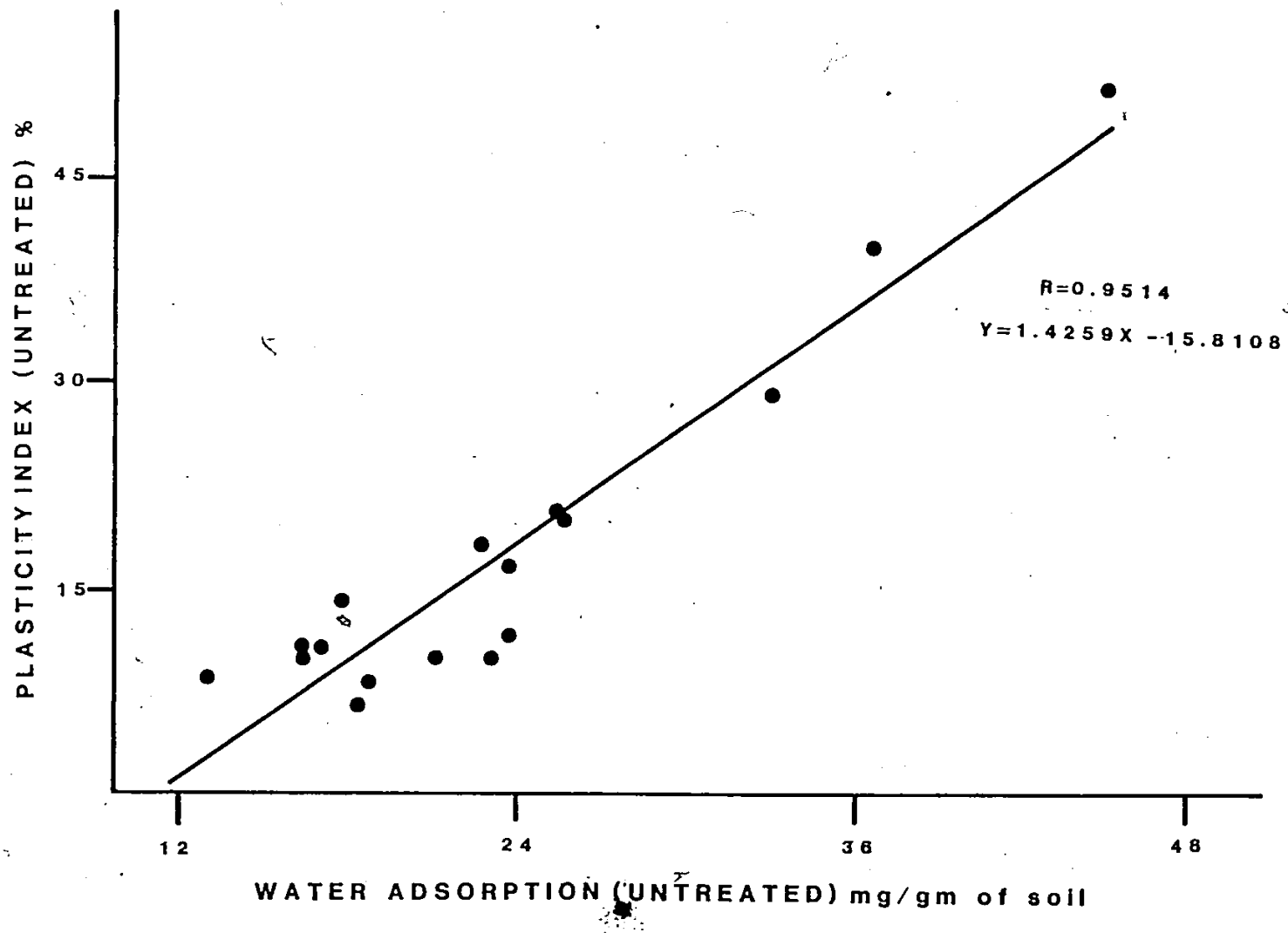


Fig. 5.8 Plasticity Index vs. Water Adsorption for Untreated Samples.

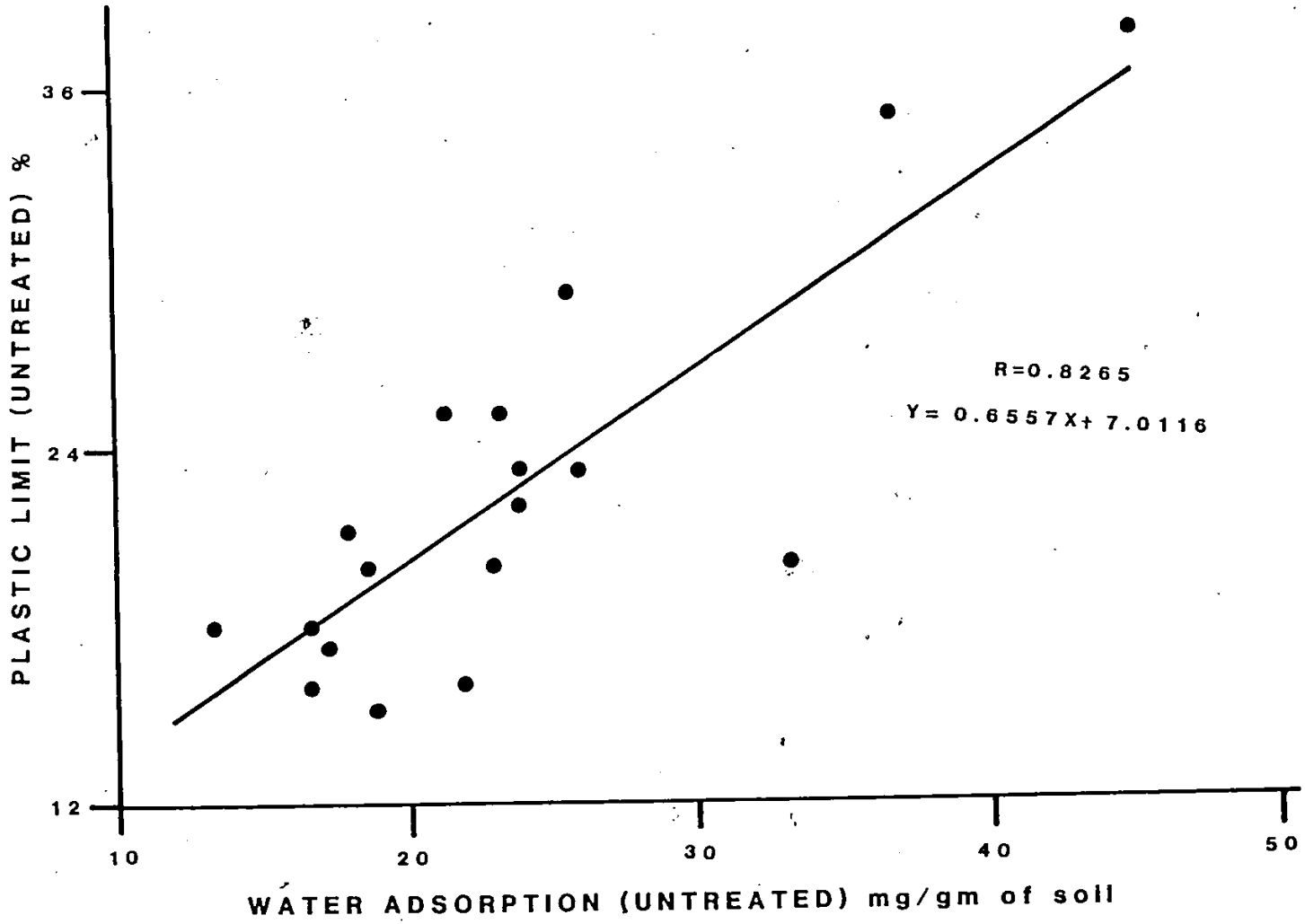


Fig. 5.9 Plastic Limit vs. Adsorption for Untreated Samples.

Table 5.5
Determined and Calculated Liquid Limits

Sample Number	Determined Liquid Limit (Untreated Samples) %	Calculated Liquid Limit (Untreated Samples) %	Determined Water Adsorption mg of water / gm of dry soil
1	90	84.87	45.0
2	35	39.29	23.1
3	26	18.26	13.0
4	31	36.58	21.8
5	35	34.71	20.9
6	35	28.25	17.8
7	28	25.34	16.4
8	50	43.66	25.2
9	49	59.89	33.0
10	75	67.80	36.8
11	34	40.74	23.8
12	27	29.09	18.2
13	43	44.90	25.8
14	38	38.24	22.6
15	28	26.59	17.0
16	27	24.92	16.2
17	23	30.33	18.8
18	40	40.53	23.7
	Mean = 39.67	Mean = 39.67	Mean = 22.67
	Standard Deviation = 17.52	Standard Deviation = 16.64	Standard Deviation = 8.21
	Sample Size = 18	Sample Size = 18	Sample Size = 18

that 90% of the variance in the liquid limit was associated with differences in the adsorption. Similar conclusions can be drawn for the plasticity index and the plastic limits.

Figures 5.10-5.12 show respective plots of the liquid limit, plasticity index and plastic limits against water adsorption, for treated samples. With the exception of the plastic limit - adsorption plot, the correlations are not as high as those obtained from untreated samples, but still significant. The correlation between the plastic limit and adsorption for treated samples is very poor. The high degree of scatter suggests that the structure of the samples are altered by treatment with brine.

5.4.1.4 Water Adsorption and Percentage of Clay Sizes (<0.002 mm)

Since the surface area of clay minerals is dependent upon the size and shape distributions of the particles, it would be expected that adsorption would correlate very well with the amount of fine sizes in clay soils. Figure 5.13 shows a scatter diagram of water adsorption at 98% RH against percentage of clay sizes (<0.002 mm) for the untreated samples. A high correlation coefficient was calculated for the linear regression model. It can be seen from Table D.1 (Appendix D) that the correlation is highly significant.

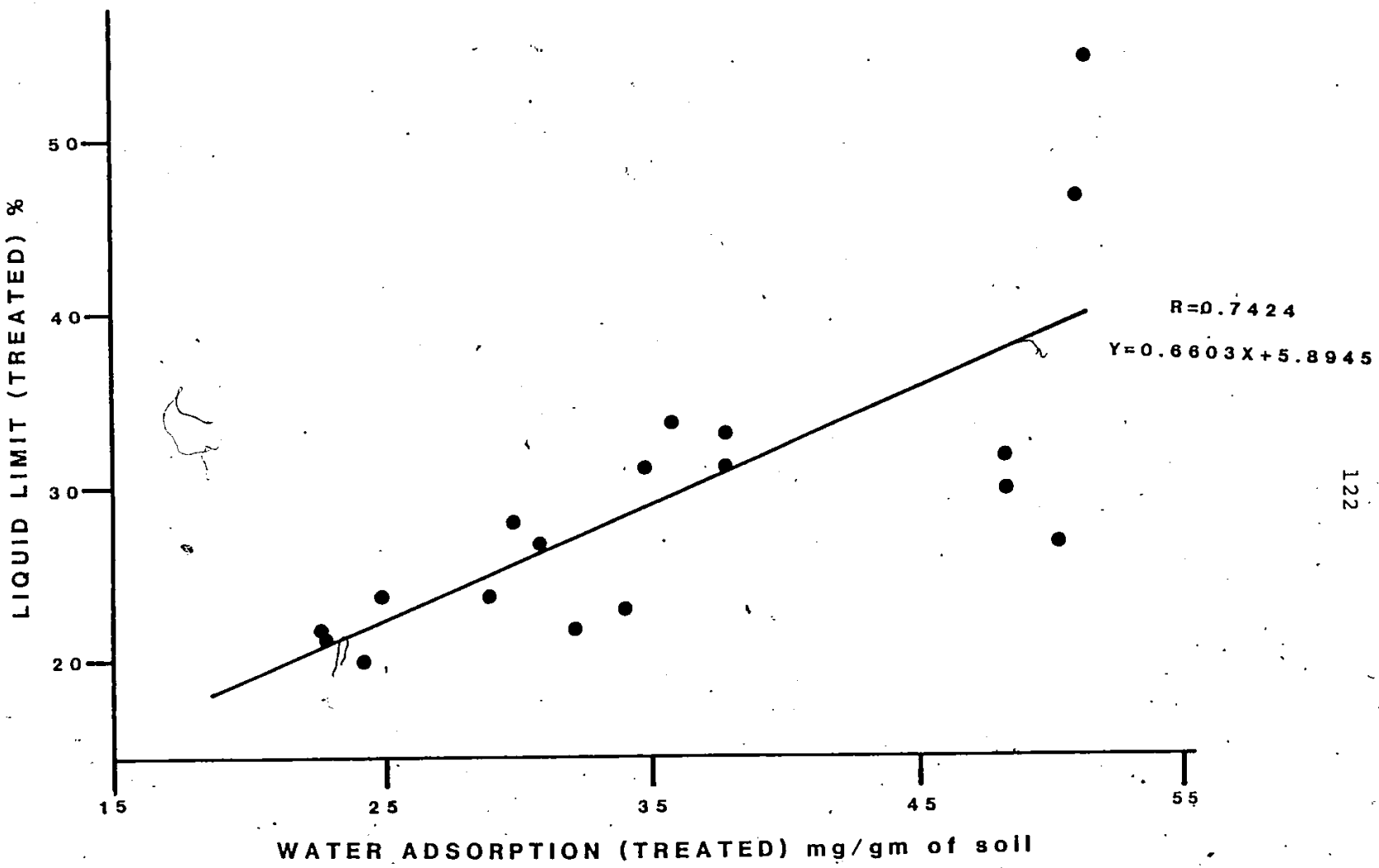


Fig. 5.10 Liquid Limit vs. Water Adsorption for Treated Samples.

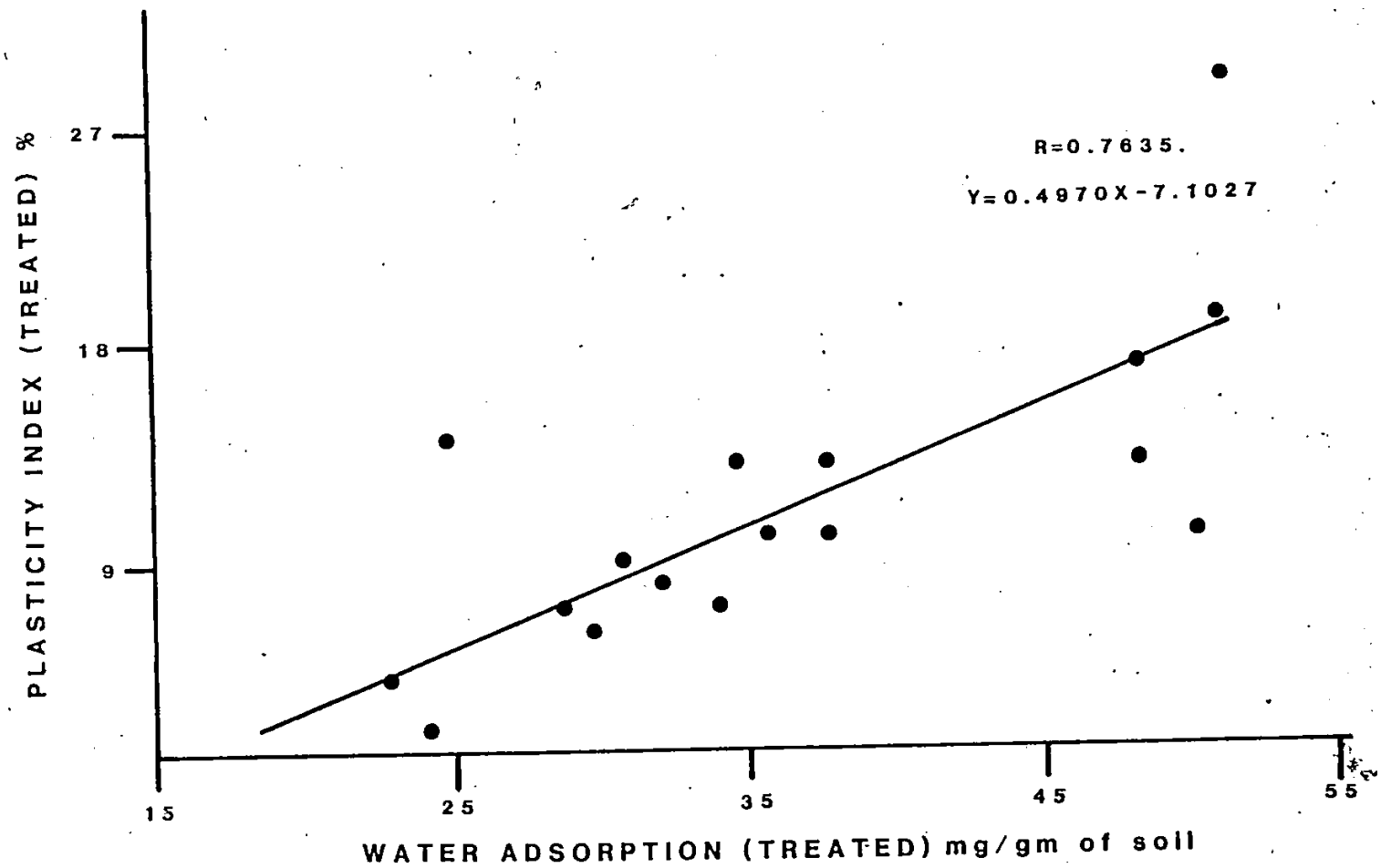


Fig. 5.11 Plasticity Index vs. Water Adsorption for Treated Samples.

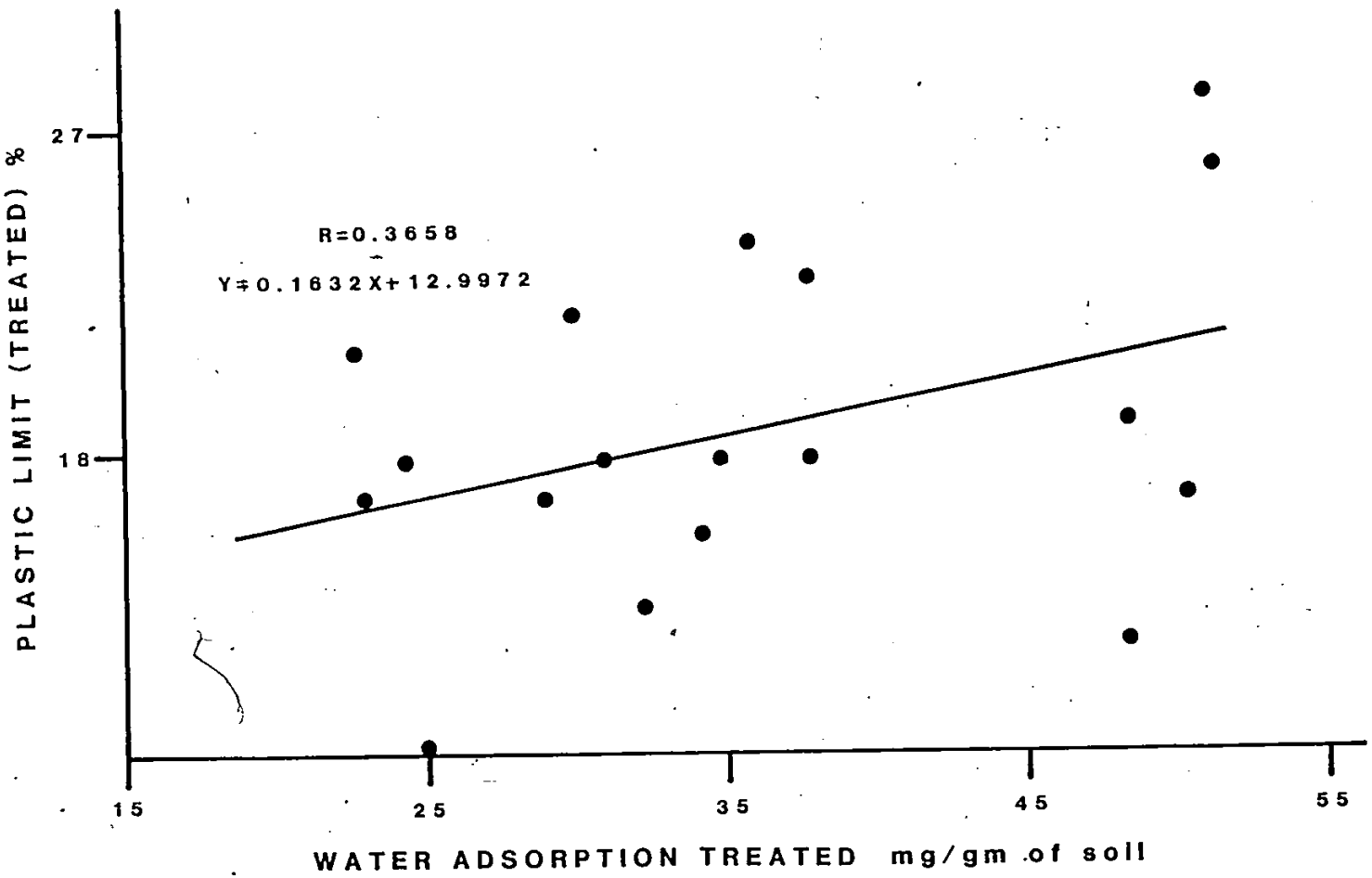


Fig. 5.12 Plastic Limit vs. Adsorption for Treated Samples.

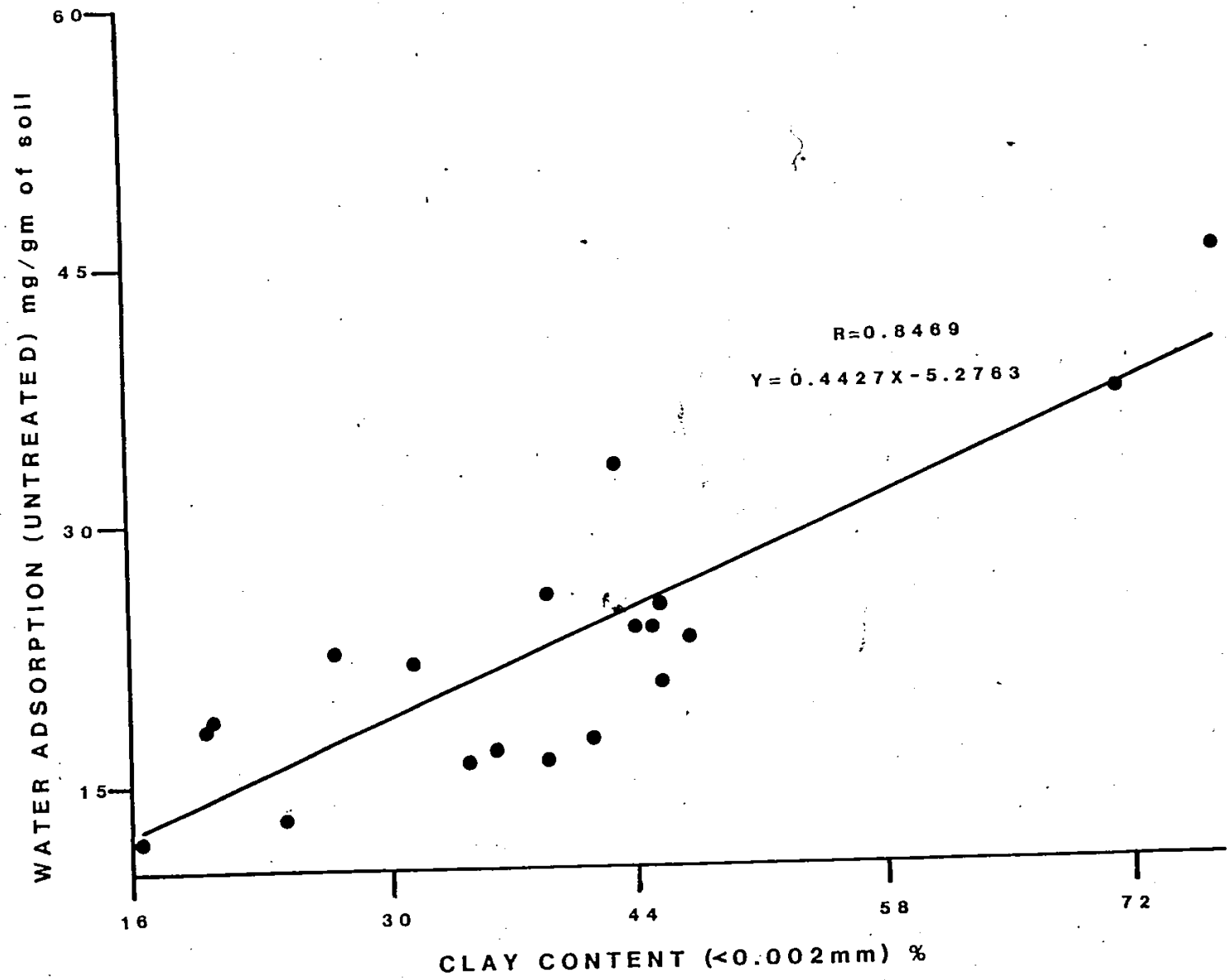


Fig. 5.13 Adsorption vs. Percentage Clay (Sizes <0.002 mm)

5.4.1.5 Optimum Compaction Parameters

In Fig. 5.14, the optimum moisture contents of the treated samples have been plotted against those of the untreated. A high correlation was obtained for the relationship. The plot of the maximum dry density (treated) against the maximum dry density (untreated) shows a higher correlation. These highly significant correlations suggest that the optimum moisture contents and the maximum compactive densities of the treated samples can be derived from those of the untreated samples. It can be deduced from the squares of the calculated correlation coefficients that more than 80% of the variation in the treated parameters were associated with differences in the untreated parameters. It can be concluded from this that the regression equations can be used to predict the optimum moisture contents and the maximum dry densities of the treated samples from the results obtained on untreated samples.

5.4.1.6 Permeability of treated samples to brine and permeability of untreated samples to water

In Fig. 5.16 a log-log plot of the permeability of the treated samples to brine against that of the untreated

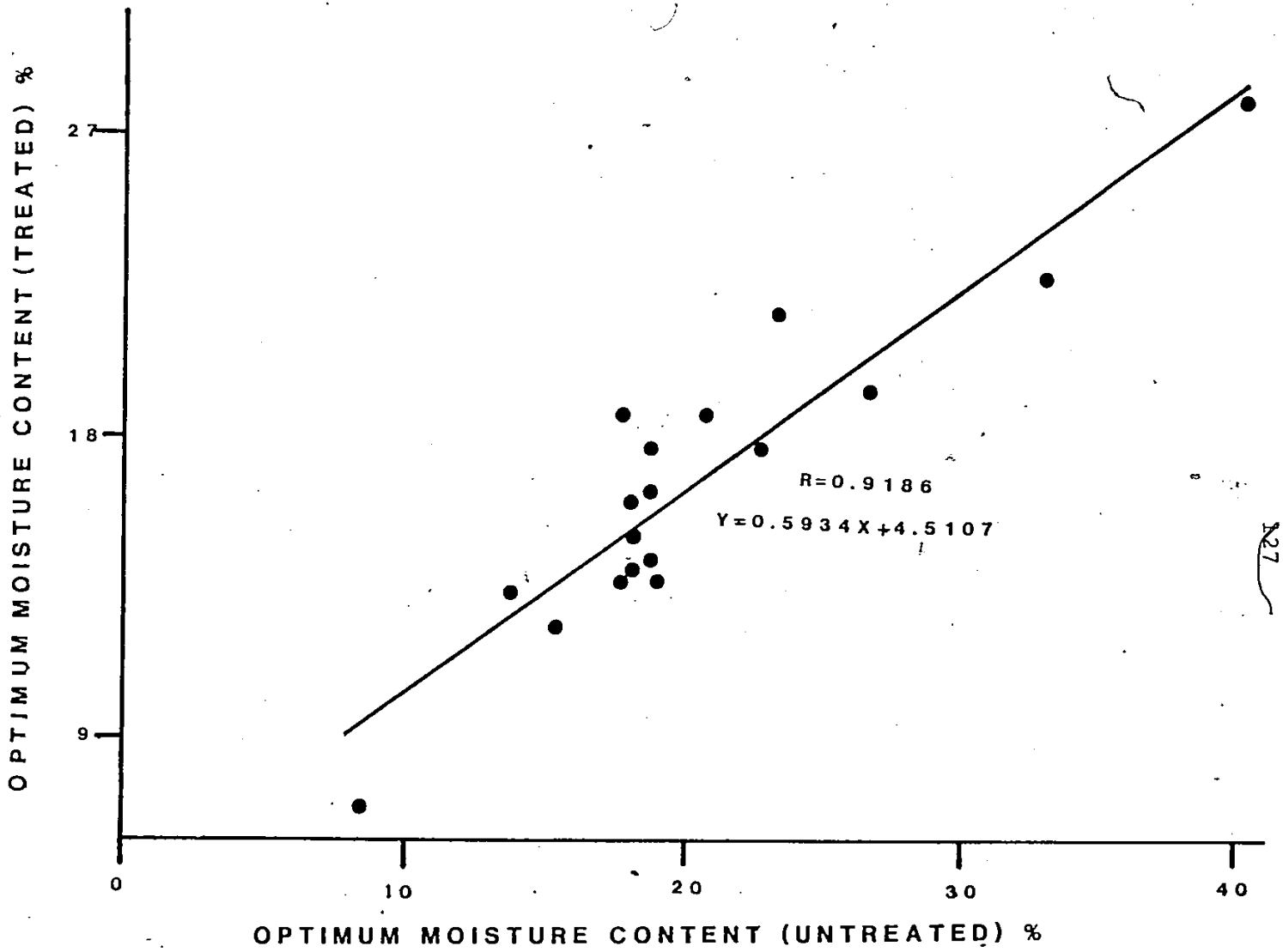


Fig. 5.14 Relationship Between Optimum Moisture Contents - Treated vs. Untreated Samples.

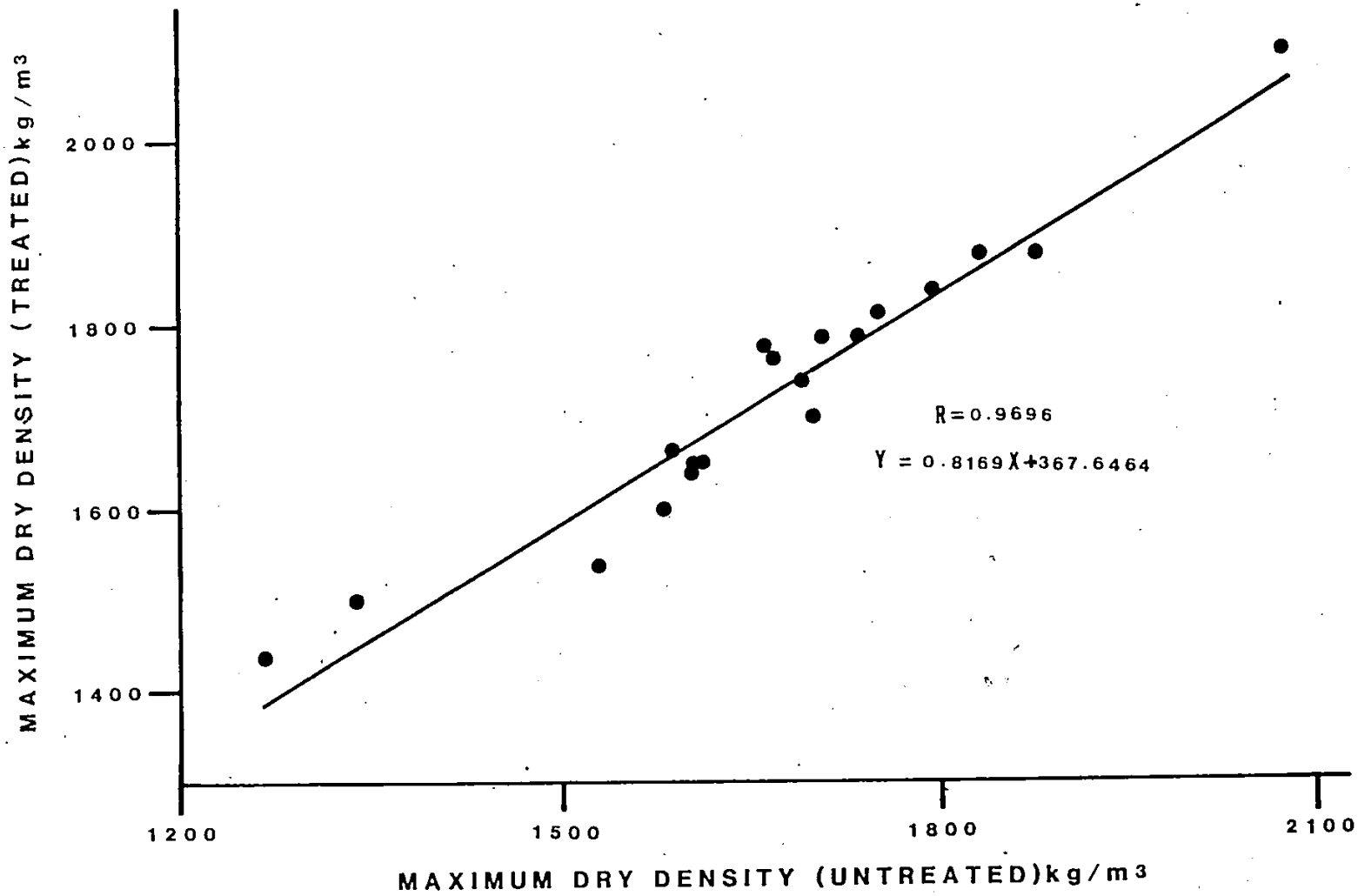


Fig. 5.15 Relationship Between Maximum Dry Densities - Treated vs. Untreated Samples.

Table 5.6

Determined and Calculated Optimum Compaction Parameters

Sample Number	Determined Optimum Moisture Content (treated) %	Calculated Optimum Moisture Content (treated) %	Determined Maximum Dry Density (treated) (kg/m^3)	Calculated Maximum Dry Density (treated) (kg/m^3)
1	22.6	24.09	1506	1452.55
2	13.5	15.79	1648	1672.31
3	15.9	15.19	1782	1757.27
4	13.8	14.95	1782	1784.23
5	19.4	20.41	1661	1658.42
6	14.2	15.55	1769	1721.32
7	13.9	15.19	1840	1833.24
8	17.6	18.04	1534	1609.40
9	17.8	17.92	1603	1655.97
10	28.0	28.42	1434	1388.01
11	17.7	15.48	1742	1746.65
12	16.4	15.61	1767	1725.41
13	21.6	18.28	1637	1673.12
14	18.6	16.85	1647	1677.21
15	15.1	15.25	1812	1794.85
16	13.4	12.58	1880	1898.60
17	12.3	13.53	1877	1864.29
18	18.8	14.95	1703	1751.55
19	7.0	9.50	2104	2063.62
	Mean = 16.72 Standard Deviation = 4.50 Variance = 19.22 Sample Size = 19	Mean = 16.71 Standard Deviation = 4.14 Variance = 16.21 Sample Size = 19	Mean = 1722.53 Standard Deviation = 154.61 Variance = 22646.2 Sample Size = 19	Mean = 1722.53 Standard Deviation = 149.91 Variance = 21290.6 Sample Size = 19

At the 1% level of significance, the mean and variances of the determined and calculated optimum compaction parameters are statistically the same.

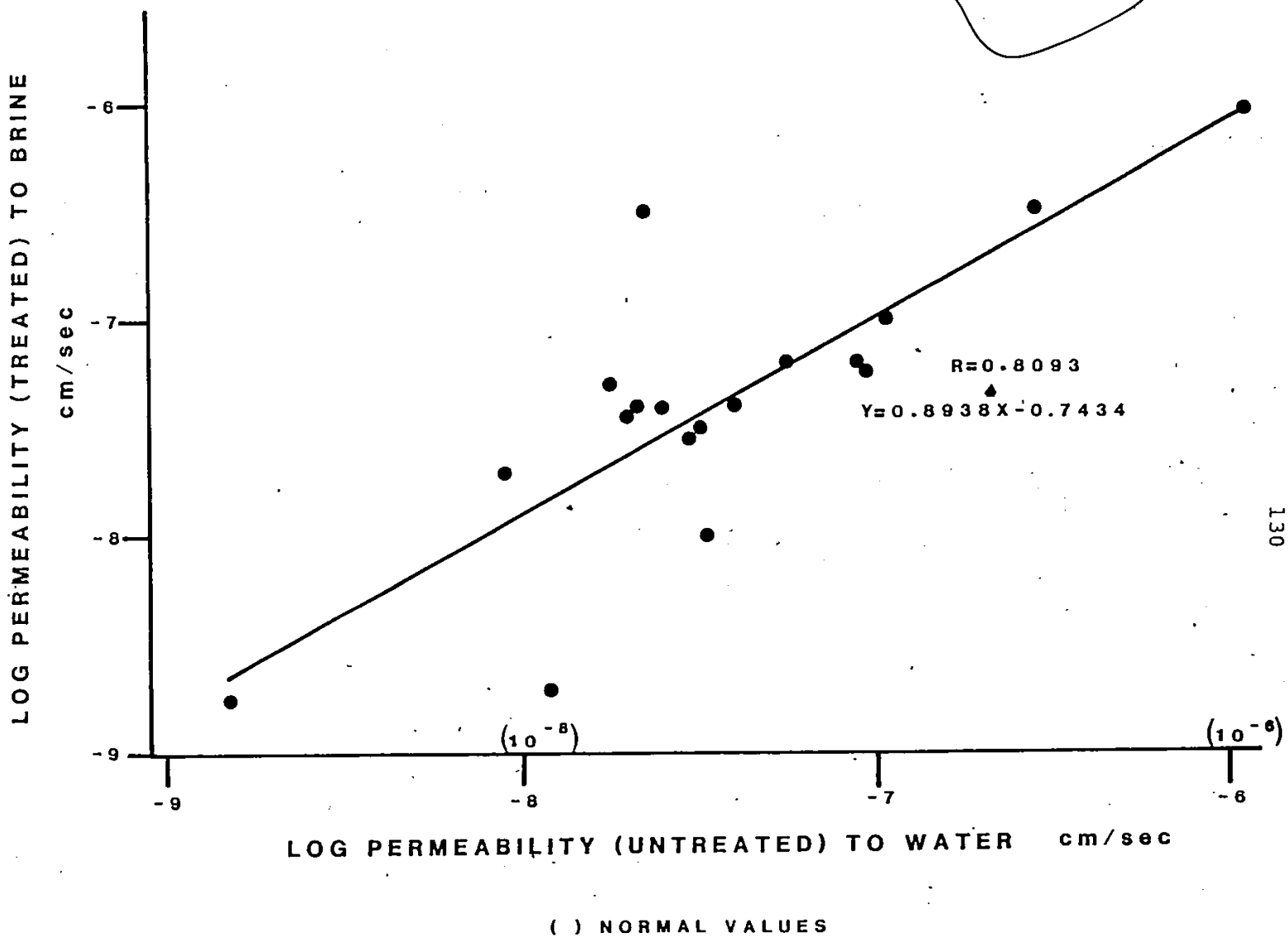


Fig. 5.16 Permeability (treated) to brine vs. Permeability (untreated) to Water.

to water is shown. A highly significant correlation was obtained, although a few of the data points were widely scattered about the regression line. The r^2 value of 0.65 obtained indicates that 65% of the variation in the permeability (treated) to brine was associated with the permeability (untreated) to water.

In Table 5.7 permeabilities to brine determined on untreated samples are compared to those predicted from the regression equation.

It can be deduced from Appendix D that the variances and means belong to the same population at the 1% level of significance. This implies that the determined and calculated permeabilities of the treated samples to brine are statistically the same. It can be concluded from this that the regression equation derived from the logarithmic plot is a suitable model. Thus, the permeability of the treated sample to brine can be predicted from permeability of the untreated sample to water.

From Table D.1 it can be seen that the correlation between the permeabilities of the treated and the untreated soils to water were significant. The distribution of data points about the regression line, as seen from Fig. 5.17 is, however, poor.

The correlation between the permeabilities of the untreated soils to water and brine (Fig. 5.18) was very poor. This

Table 5.7

Determined and Calculated Permeability to Brine of Untreated Samples

Sample Number	Determined Log Permeability to Brine (treated)	Calculated Log Permeability to Brine (untreated)
1	-8.7696	-8.6299
2	-7.2211	-7.2193
3	-7.2692	-7.0238
4	-7.3990	-7.5286
5	-6.5229	-6.6001
6	-7.2899	-7.6589
7	-8.0132	-7.4162
8	-7.4123	-7.3603
9	-7.4157	-7.6055
10	-8.7212	-7.8227
11	-7.7011	-7.9431
12	-6.0315	-6.0620
13	-7.4868	-7.4445
14	-7.2190	-7.0406
15	-7.4584	-7.6245
16	-7.5498	-7.4671
17	-6.9974	-6.9699
18	-6.5092	-7.5702
	Mean = -7.39	Mean = -7.39
	Standard Deviation = 0.68	Standard Deviation = 0.55
	Sample Size = 18	Sample Size = 18

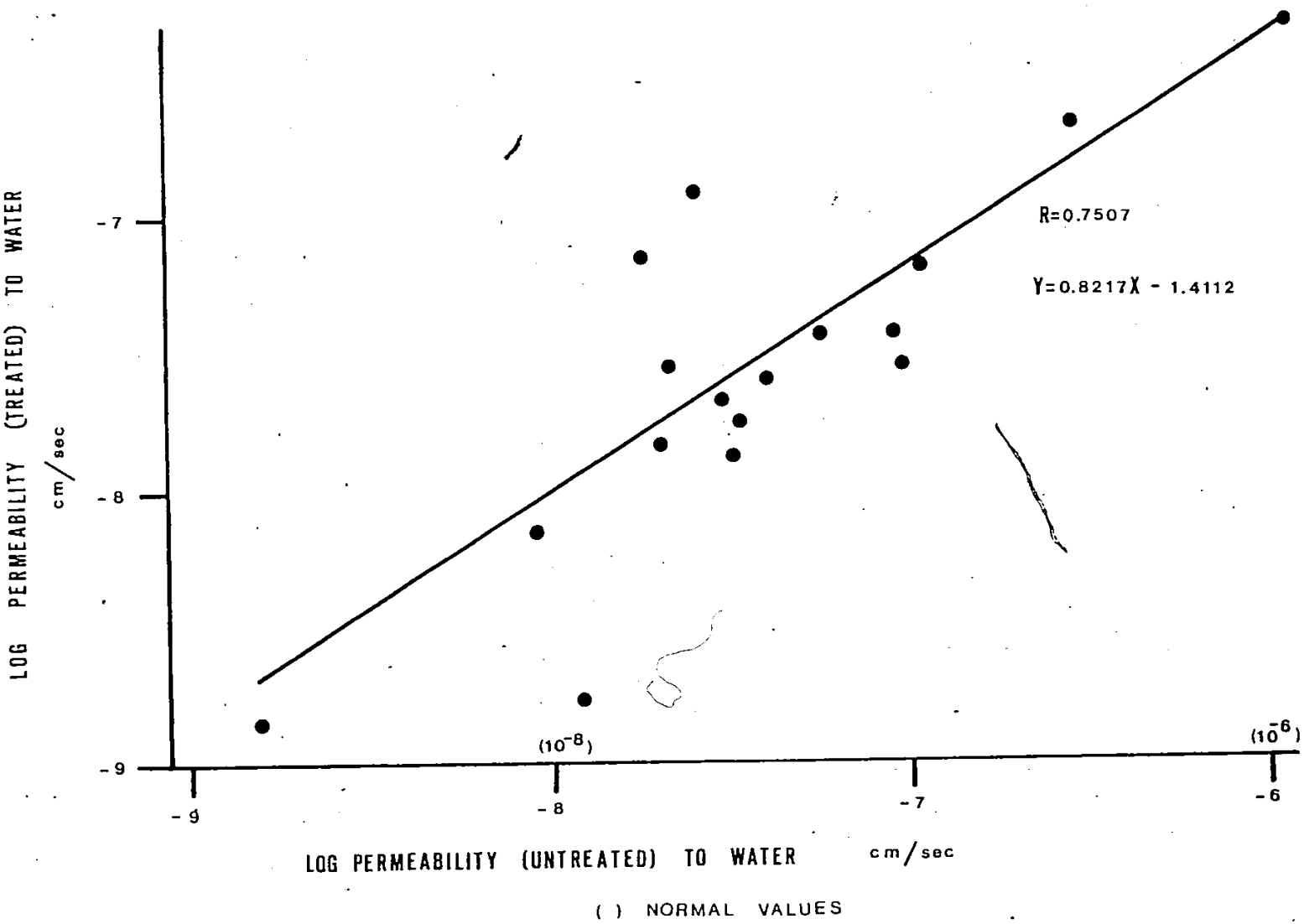


Fig. 5.17 Permeability (treated) to Water vs. Permeability (untreated) to Water.

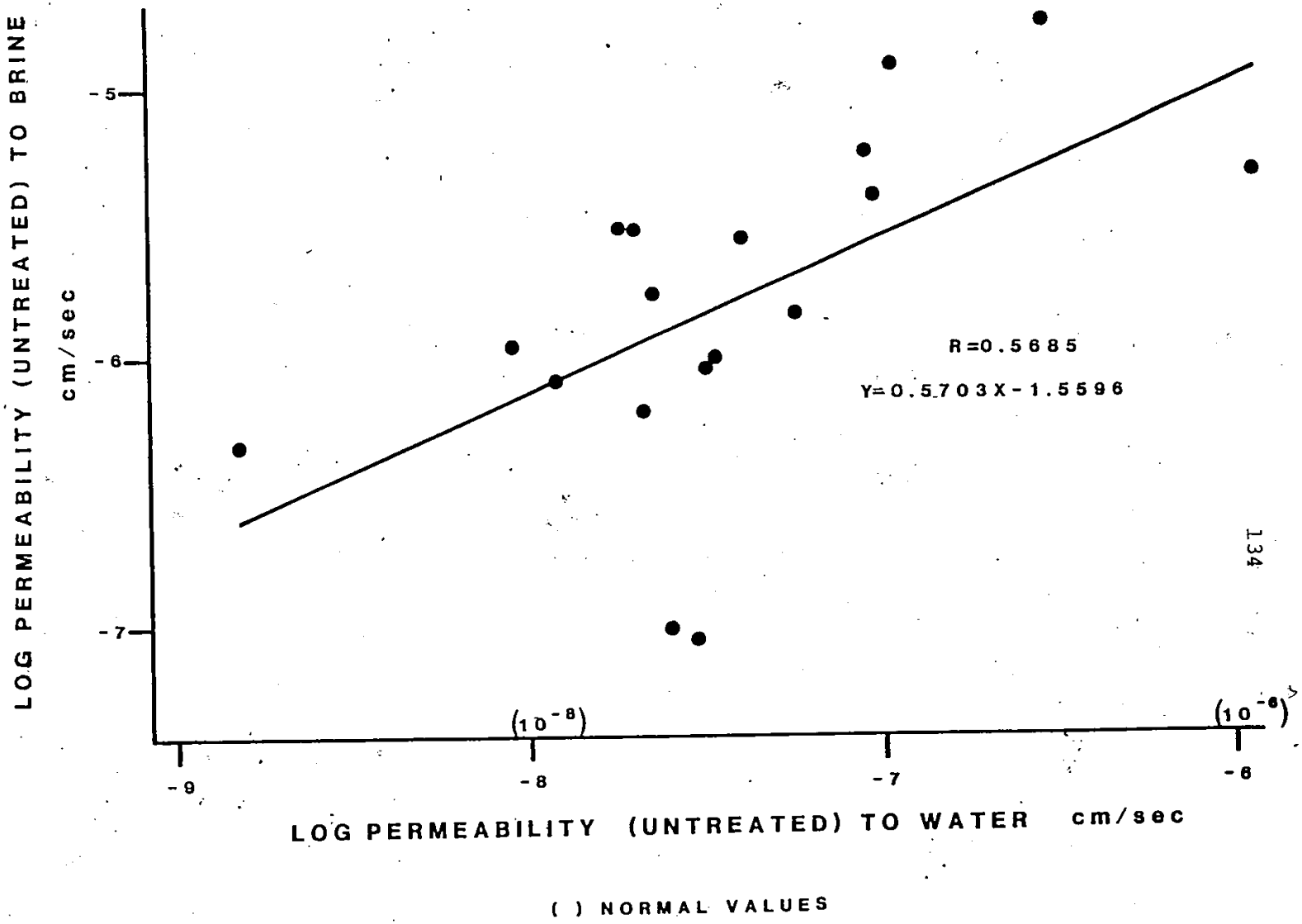


Fig. 5.18 Log Permeability to Brine vs. Log Permeability to Water for Untreated Samples.

can be seen from the low value of 0.5685 obtained for the Pearson r. This suggests that the permeability of the untreated clay soil to brine cannot be predicted from its permeability to water. In other words, the fact that the untreated soil has a low permeability to water does not imply that its permeability to brine is also low.

5.4.2 Comparison of Means and Variances

Means and variances of geotechnical characteristics determined for untreated and treated samples were compared by grouping the two types of samples separately. The variables used in the comparison were Atterberg limits, water adsorption at 98% relative humidity and permeability. The method used is outlined in Appendix D and the results are tabulated in Table D.2

At the 5% level of significance the means of the Atterberg limits (liquid limit and plasticity index) of the treated and untreated soils do not belong to the same population. This indicates that the treated and untreated samples have significantly different plasticities. The means and variances of the plastic limits, however, belong to the same population. This agrees with an earlier point made in Section 2.6.4.1 that the liquid limit is much more influenced by chemistry than the plastic limit. It implies that most of the changes in the plasticity index due to the treatment with brine was due to changes in the liquid limit.

In the brine environment, the Atterberg limits were significantly reduced. The explanation for the reduction is the dessication shrinking of the clay as a result of sodium cation exchange. A further discussion of this subject will be taken up in Section 5.4.3.2.

From Table D.2 it can be seen that the means and variances of the water adsorption of the treated and untreated soils do not belong to the same population. At the 0.1% level of significance, there is sufficient statistical evidence to suggest that the treated and untreated soil sample adsorbed different amounts of water. Water adsorption at 98% relative humidity was higher for the treated soils than for the untreated. The increase in adsorption was due to cation hydration in the presence of water vapour.

Table D.2 indicates that the means and variances of the permeabilities of the untreated samples to water and brine do not belong to the same population at the 1% level of significance. There is sufficient statistical evidence to suggest, therefore, that the permeability of the untreated clay soil to brine would not be the same as its permeability to water. With the exception of sample AU-12, which contained a very high proportion of silt, all the untreated soils used in this study were more permeable to brine than to water. The perme-

ability ratio was approximately 100:1 (Table 5.1, Section 5.1) The high permeability to brine was due to the formation of a porous, flocculated structure in the presence of the brine (Section 2.6.4.2).

It would appear from the physico-chemistry of the clay-water-electrolyte system that if the clay soils are pre-treated and remoulded with brine and compacted at optimum, the permeability to brine could be much lower. In this case, the flocculated soil is in its densest state and, more significantly, in equilibrium with its own cation. Consequently, there is very little chance of subsequent cation exchange reactions to alter the structure of the soil (Taylor, 1959). In Table D.2 the means and variances of the treated permeabilities to brine and the untreated permeabilities to water have been compared. The result indicates that the two permeabilities were statistically the same.

It can be deduced from Table D.2 that the means of the permeabilities of the treated soils to water and brine belong to the same population. By comparing variances, it can be seen that the variations in the permeability data were, however, different for the two sets of measurements.

The foregoing comparisons of permeabilities indicate that the permeabilities of the untreated clay soils to

water and brine were significantly different. The permeability to brine was, however, reduced as a result of pre-treatment with brine.

5.4.3 Factor Analysis

5.4.3.1 Introduction

Some type of quantitative method is often required to summarize and analyze information hidden in a multivariate data table. It is natural to enquire how the variables measured for a homogenous sample are related to each other and whether they occur in different combinations drawn from various relationships in the population. On the other hand, one may sometimes be interested in seeing how entities, specimens or objects of the sample itself are interrelated, with the aim of looking for natural groupings. In both cases, one is looking for structure in the data.

Factor analysis is concerned with the interpretation of the structure within the variance-covariance matrix of a multivariate data collection (Davis, 1973). This is achieved by representing the set of variables or objects in terms of a smaller number of hypothetical variables or entities called factors. This number of factors will usually account for approximately the same amount of information as do the much larger set of original observa-

tions. Thus, factor analysis is, in this sense, a multivariate method of data reduction.

There are two modes of factor analysis: R-mode and Q-mode. In an R-mode factor analysis the primary purpose is to investigate the interrelationships among variables. If the primary purpose, however, is to determine the interrelationships among samples, then the analysis is referred to as Q-mode.

For an R-mode analysis, McCammon (1975) proposes a factor model represented by the equation

$$Z_j = a_{ij}F_1 + a_{2j}F_2 + \ldots + a_{mj}F_m + a_jE_j \qquad (5.1)$$

In other words, the equation states that any variable (considered in standard form here, for convenience), Z_j, consists of a linear combination of m <u>common</u> factors plus a unique factor. The F's refer to factors and it assumed that each of the m factors will be involved in the delineation of two or more variables. Thus, the factors are said to be <u>common</u> to several variables; m is assumed to be less than n, the number of variables. The a's are termed loadings and the F's <u>factor scores</u>. The factor E_j is unique to variable Z_j and is analogous to the error term in a regression equation (McCammon, 1975).

The factor model of n variables will contain n such equations as (5.1) above. Whereas the a's are the same

for all samples, the F's change from sample to sample. Another way to view the F's is to think of them as new variables that are linear combinations of the old variables. The basic problem in factor analysis, then, is to find values for the a's and F's and m, the number of common factors. This problem can be solved by recourse to general purpose computer packages that contain factor analysis programs. Examples of these are BMD, SAS, and SPSS.

Various aspects of factor analysis have been treated by Harman (1960), Rummel (1970), Davis (1973), and Jöreskog *et al*. (1976).

The total variance of a variable Z_j is made up of the sum of the squared a's. This consists of two parts:

1. That due to the common factors termed the *communality* and symbolized by h_j^2.

2. That due to the unique factor. This is equal $1-h_j^2$ and is defined as that part of the variance of variable j that is not shared by any of the other variables.

In order to elucidate the nature of the interactions between the variables determined in this study, an R-mode factor analysis was run in which the equamax method of rotation was specified.

5.4.3.2 Interpretation of Factors

The results of the factor analysis indicate that four factors explain 86.6% of the variance of the original data. The factor matrix of the geotechnical data is given in Table 5.8. The column headed "communality" gives the fraction of the variance of each variable that is explained by the factors that have been extracted.

Factor 1 - clay content, plasticity and structure.

In this factor, clay size fraction (<0.002 mm) is moderately associated with plastic limit (untreated), plasticity index (treated), optimum moisture content (untreated), decrease in optimum moisture content, and the increase in maximum dry density after treatment. The optimum moisture content (untreated) is, in turn, antipathetically associated with the maximum dry density (untreated). This suggests that the engineering behaviour of clay soils is determined by their grain size distribution, plasticity, and the structure which the soils have at the time the behaviour is measured. This final structure depends on both the initial structure and on the changes which have occurred to the initial structure. Placement or compaction conditions determine the initial structure. The most important factors controlling the compaction of clay soils are the amount of compaction, the moulding water content, and the grain size distri-

Table 5.8

Equamax Factor Matrix

Variable	Clay Content Plasticity and Structure 1	Permeability and Activity 2	Atterberg Limits and Surface Area 3	Cation Exchange 4	Communality
Clay fraction <0.002 mm	0.74469	-0.27422	0.45250	0.45250	0.90828
Clay fraction <0.005 mm	0.79502	-0.27757	0.03804	0.34550	0.82992
Liquid limit (untr (untreated)	0.55596	-0.15728	0.61432	0.49184	0.95312
Liquid limit (treated)	0.59275	-0.18324	0.59110	0.45644	0.94267
Plastic limit (untreated)	0.75528	-0.00652	0.43020	0.38828	0.90632
Plastic limited (treated)	0.19226	-0.05099	0.89788	0.04491	0.84777
Plasticity index (untreated)	0.41338	-0.22658	0.67078	0.51384	0.93620
Plasticity index (treated)	0.67806	-0.21540	0.19214	0.59282	0.89452
Liquidity index (untreated)	-0.25450	0.32684	0.44416	-0.62269	0.76161
Activity (untreated)	-0.04587	-0.55497	0.65623	0.35256	0.86503
Adsorption (untreated)	0.43116	-0.13874	0.66232	0.51444	0.90846
Adsorption (treated)	0.28301	-0.18848	0.31202	0.78707	0.83246
Optimum moisture content (untreated)	0.76144	-0.04031	0.53824	0.29083	0.95570
Maximum dry density (untreated)	-0.67891	-0.00790	-0.59082	-0.36558	0.94370
Optimum moisture content (treated)	0.56805	0.05951	0.54057	0.43124	0.80441
Maximum dry density (treated)	-0.57931	0.03620	-0.50444	-0.51761	0.85929
Decrease in optimum moisture content	0.79841	-0.14670	0.40227	0.05514	0.82384
Increase in maximum dry density	0.64600	0.11542	0.56271	-0.16321	0.77392
Permeability to water (untreated)	-0.05011	0.92827	0.06152	-0.22667	0.91936
Permeability to brine (untreated)	0.21682	0.26429	-0.07455	-0.57576	0.45392
Permeability to brine (treated)	-0.05422	0.98796	-0.02131	-0.11809	0.99340
Permeability to water (treated)	-0.08181	0.95886	-0.11392	-0.01954	0.93947
Variance explained by each factor	6.23089	3.64891	5.13073	4.04277	

Explanation

1.00 to 0.80
0.80 to 0.60
0.60 to 0.40
0.40 to 0.00

bution (Glanville, 1951). The more the water added during compaction, other variables being constant, the greater is the repulsion between particles. Thus water weakens, or "lubricates" clay. At high water contents, low compactive densities are obtained.

The rheology of a soil-water-air system being compacted can be sharply changed with additives. When a clay soil is treated with saturated brine as described in Section 4.2.5, the net interparticle attractive forces are increased. The optimum moisture content is reduced and the maximum dry density is increased.

In Appendix C the dry density-moisture content curves of both the treated and untreated soils are presented. Compaction is dependent on lubrication of the soil particles by moisture. In compaction, the particles are made to pack more closely together through a reduction in the air-voids, generally by mechanical means. Compaction of the soil at increasing moisture contents by a given amount of work leads to a maximization of the dry density and then a decrease, giving a relationship of the type shown in Appendix C.

The association of the content of clay size fraction with the plastic limit of the untreated sample shows the dependence of plasticity on the proportion of fine particles. Yong and Warkentin (1975) have mentioned the

use of the plastic limit with particle-size distribution in evaluation and classification of soils for engineering purposes. The compaction of a soil also depends on its particle-size distribution. Soils with higher proportions of clay-size fractions have, in general, lower compactive densities. Glanville (1951) noted that a flat compaction curve, generally speaking, denotes a closely graded soil, and a curve with a pronounced peak denotes a well-graded soil.

The association of the plasticity index of the treated samples with the changes in the maximum dry density and optimum moisture content suggests that plasticity is affected by changes in structure. The moderate sympathetic associations indicate that the plasticity index is decreased and the optimum moisture content is also decreased after treatment. In a brine environment, the soil is flocculated because of the high concentration of Na (sodium) ions which decrease the net repulsive force. The clay particles approach each other more closely and plasticity is reduced. The brine solution draws water from the clay so that the moisture content is reduced. The optimum moisture content after treatment is, therefore, lower than the original value.

Factor 2 - Permeability and activity.

In this factor, the permeability of the untreated

clay soil to water is strongly associated with permeabilities of the treated clay to brine and water. This strong association suggests high positive correlations between these variables, as already discussed. The permeability of the untreated samples to water has a weak antipathetic association with the activity of the untreated sample. This suggests that the permeability of a clay soil is influenced by its activity. The activity is a function of the plasticity and the amount of clay size fraction present in the soil. Mitchell (1976) has noted that the higher the activity of a soil, the more important the influence of the clay fraction on its properties. The antipathetic association indicates that an increase in activity means a decrease in permeability of the soil, other factors being equal. The strong sympathetic associations among the permeabilities also suggest that the permeability of the treated clay soil to both brine and water can be predicted from the permeability of the untreated soil to water. It also implies that a fair idea of the magnitude of the permeability of the treated soil to both brine and water can be obtained from a knowledge of its activity. Terzaghi (1936) was the first to indicate that the Atterberg limits depend precisely on the same physical factors which determine the permeability of soils, but in a more complex manner. Such factors, according to Terzaghi, include shape

of particles, effective size, and uniformity.

Factor 3 - Atterberg limits and surface area.

In this factor liquid limit, plasticity index, and activity are moderately associated with water adsorption. The water adsorption represents the amount of water vapour in milligrams per gram of dry soil adsorbed by the clay surface. It is therefore a measure of the external surface area of the particles in a clay soil. The Atterberg limits are a function of the interparticle forces in the clay structure. These, in turn, depend on size, shape and the type of clay particles present. The surface area can be calculated if the size and shape distributions of the particles in a sample are known (Yong and Warkentin, 1975). The high and significant correlations between Atterberg limits and adsorption calculated in Section 5.4.1.3 show that the Atterberg limits are dependent on surface area.

The change in the plastic limit of the soil after treatment can be explained by the change in structure, as already mentioned. The flocculated structure is formed as a result of sodium cation exchange. A plot of the treated samples on the plasticity chart (Fig. 5.19) shows that they are more "granular" than the original samples. Since the plasticity has a strong influence on the behaviour of clay soils, it follows that cation exchange should explain the geotechnical properties of the soils in a brine environment.

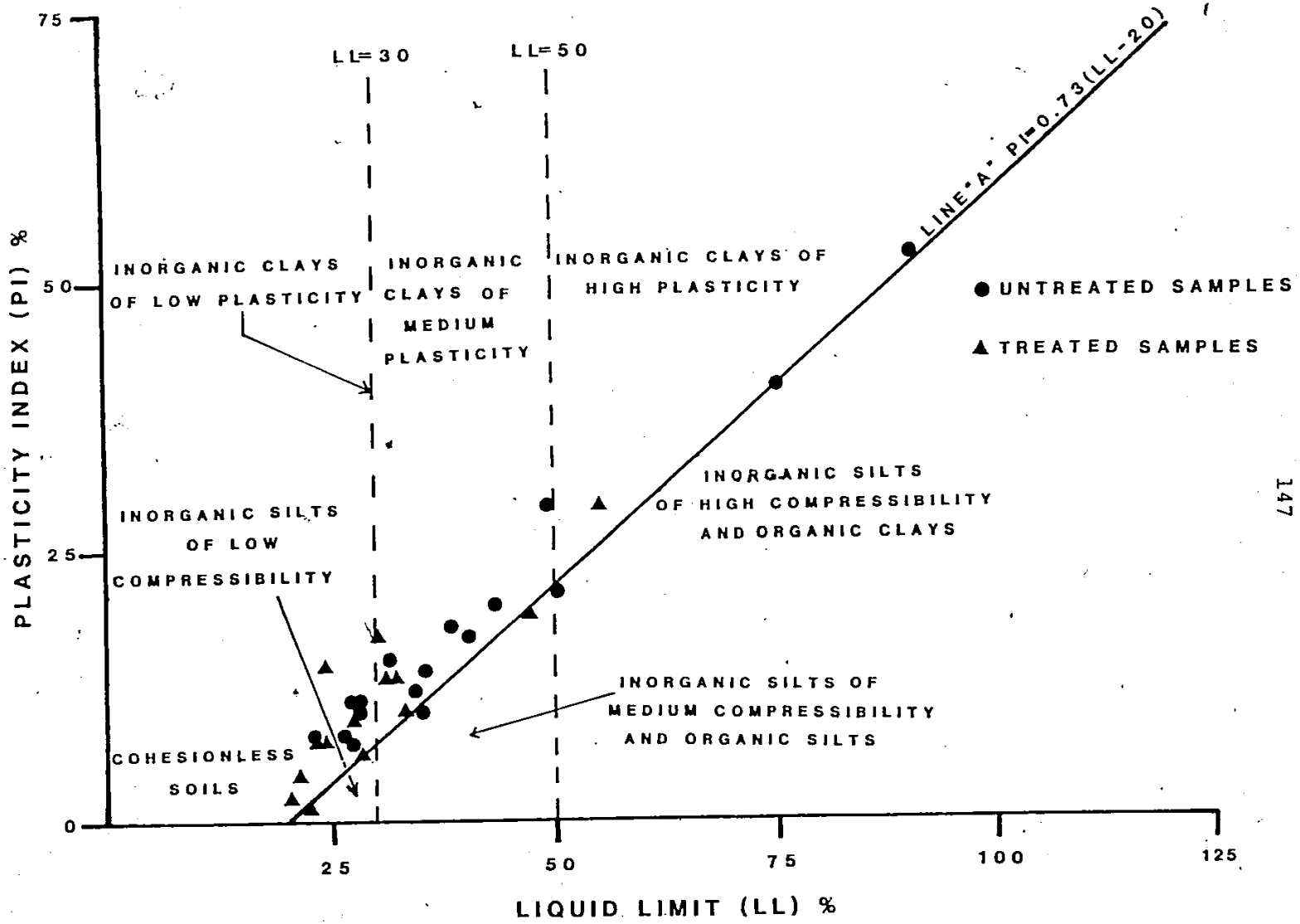


Fig. 5.19 Plasticity Chart for Treated and Untreated Samples.

Factor 4 - Cation exchange.

In factor 4 adsorption to water vapour after treatment with brine has a moderate antipathetic association with the liquidity index before treatment. It is also weakly associated antipathetically with the permeability of the untreated samples to brine. The weak association of the index with the permeability indicates that in the presence of the brine, they both increase, although not in direct linear proportion.

The weak association of the adsorption of the treated sample with the untreated permeability to brine suggests that they were both influenced by the same factor-cation exchange. Due to the high ionic strength of the brine solution, excess sodium ions (Na^+) 'crowd' into the diffuse double layer. They displace any other cation(s) originally adsorbed on the clay surface. As a result, the field of influence of the negatively-charged clay surface is diminished as shown in Fig. 5.20.

The thickness of the double layer is reduced and Van der Walls forces of attraction become significant. This leads to the flocculation of the clay particles. A flocculated clay is porous and has a higher permeability than a dispersed soil at the same void ratio (Lambe and Whitman, 1969). The increased permeability of the untreated soil in the brine environment is due to the formation of the flocculated structure.

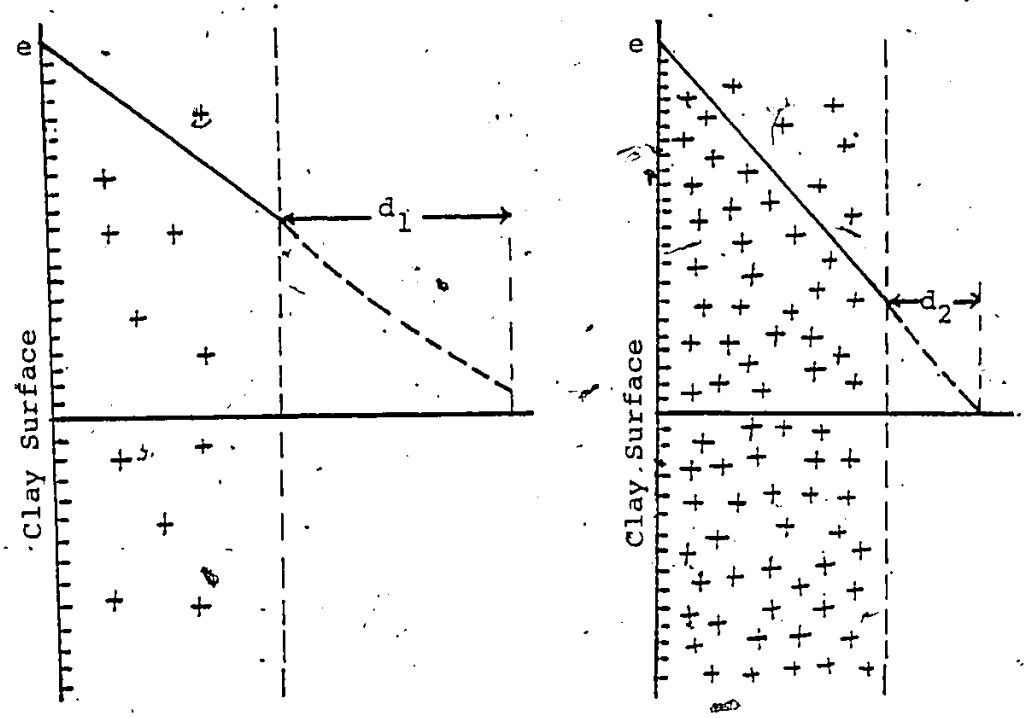


Fig. 5.20 Schematic representation of the 'crowding' of the double layer by an excess of cations (modified from Grimshaw, 1971).

In the presence of water vapour, the adsorbed sodium ions become hydrated. This is due to electrostatic attraction between the negatively-charged ends of the water dipole and the positively-charged sodium ions. As a result, the amount of water adsorbed by the clay particles is high.

5.4.4 Permeability-time Measurements

Long-term permeability studies were made on both treated and untreated samples. The results are tabulated for six samples in Table 5.9 and presented in Figs. 5.21-5.26. The 48-hour gap was the time required for equilibrium to be achieved between changing the permeant from water to brine and vice versa. The results indicate that for almost all the samples, the permeability of the untreated soil to water was appreciably constant with only occasional decreases. This decrease in permeability was probably due to changes in structure (Lambe, 1954). As flow through a soil occurs, particles tend to move to positions of greater stability to seepage forces. If particles are not washed out of the soil, this particle shifting always results in lower permeability. There was no evidence of particles being washed out of the soil during the test. Any washings would probably have been filtered by the sand layer at the top of the specimen. The reduction in permeability was particularly marked for sample AU-3. No obvious

Table 5.9

Permeability-time Measurements

Sample Number	Coefficient of permeability of untreated sample to water $\times 10^{-8}$ (cm/sec)								Coefficient of permeability of untreated sample to brine $\times 10^{-6}$ (cm/sec)						
	1st day	2nd day	3rd day	4th day	5th day	6th day	7th day	8th day	9th day	10th day	11th day	12th day	13th day	14th day	15th day
BR-2	5.88	5.75	6.05	6.09	5.39	5.40	5.39	Permeant changed from water to brine	1.19	1.73	1.83	1.94	2.16	2.05	2.27
AU-3	5.07	4.82	6.72	4.62	3.30	1.55	1.11		4.21	4.85	6.26	6.69	5.39	7.01	7.25
HG-4	2.68	2.68	2.79	2.82	2.59	2.16	1.83		1.08	2.16	2.16	1.08	2.16	2.16	2.16
BE-6	1.81	1.64	1.48	1.42	1.49	1.51	1.41		84.90	89.10	103.13	104.64	99.24	97.09	105.72
SE-7	3.69	3.67	3.68	3.73	3.68	3.54	3.43		1.19	1.08	1.08	1.08	1.08	1.40	1.19
SH-9	2.59	2.59	2.70	2.70	2.70	2.70	2.91		0.76	0.76	0.86	0.76	0.86	0.86	0.86
	Coefficient of permeability of treated sample to brine $\times 10^{-8}$ (cm/sec)								Coefficient of permeability of treated sample to water $\times 10^{-8}$ (cm/sec)						
BR-2	6.48	6.72	6.69	6.36	6.27	5.74	5.93	Permeant changed from brine to water	3.88	4.11	3.93	4.24	4.04	4.25	4.31
AU-3	5.80	5.71	4.30	3.89	6.05	5.38	4.11		2.91	3.03	3.02	2.85	3.17	4.31	6.47
HG-4	4.30	4.13	4.01	4.39	4.43	3.93	3.99		13.38	13.38	14.67	9.52	9.55	7.46	6.47
BE-6	5.53	7.22	8.07	8.00	8.62	9.48	9.66		7.33	7.75	6.90	6.54	7.33	7.39	6.67
SE-7	1.04	1.50	1.08	1.95	2.71	2.62	2.48		1.94	1.96	2.70	3.27	3.12	4.76	4.37
SH-9	4.14	3.88	2.79	3.05	3.74	3.72	3.45		3.03	3.13	3.20	3.20	3.18	3.14	3.12

Sample BR-2

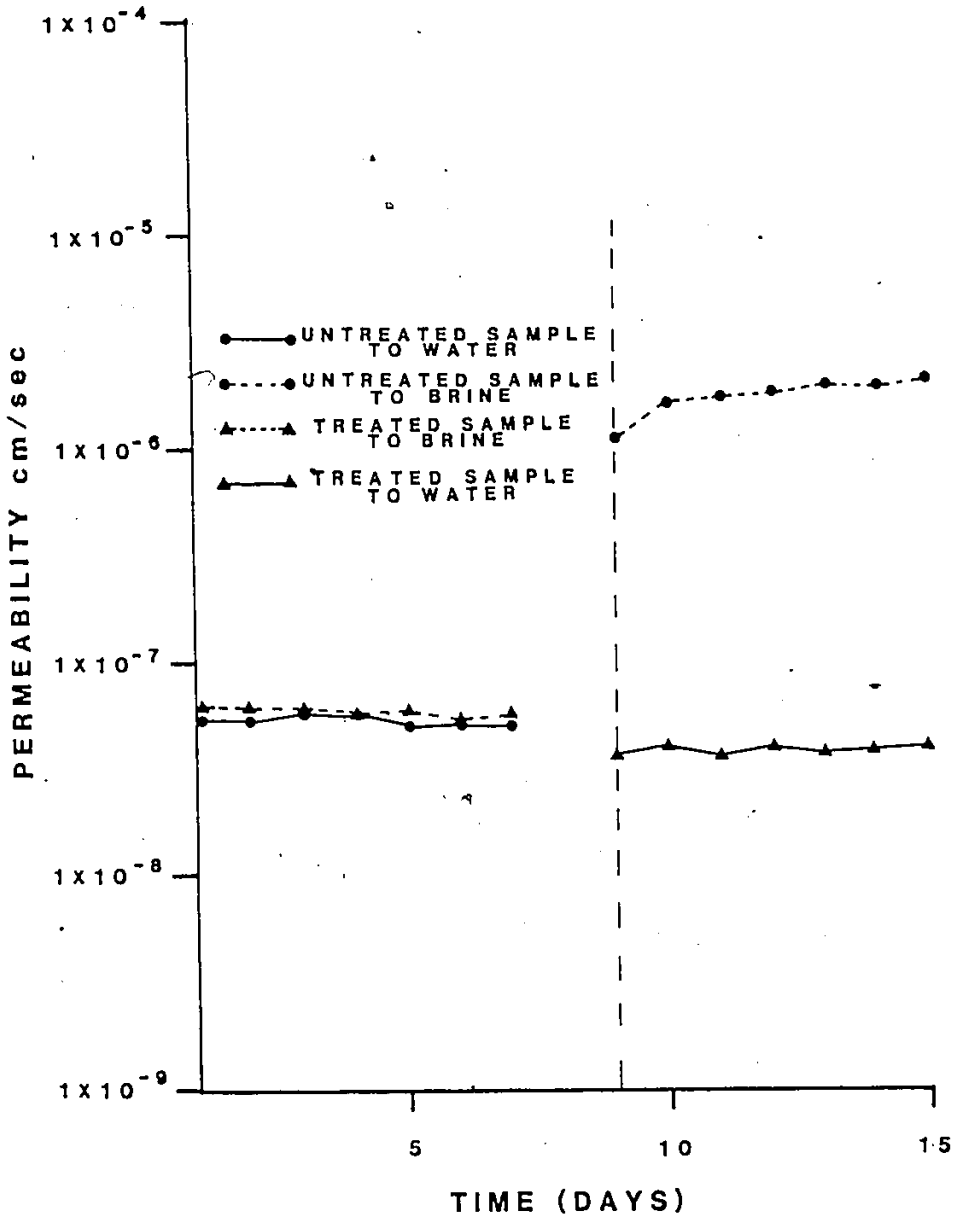


Fig. 5.21 Permeability vs. time.

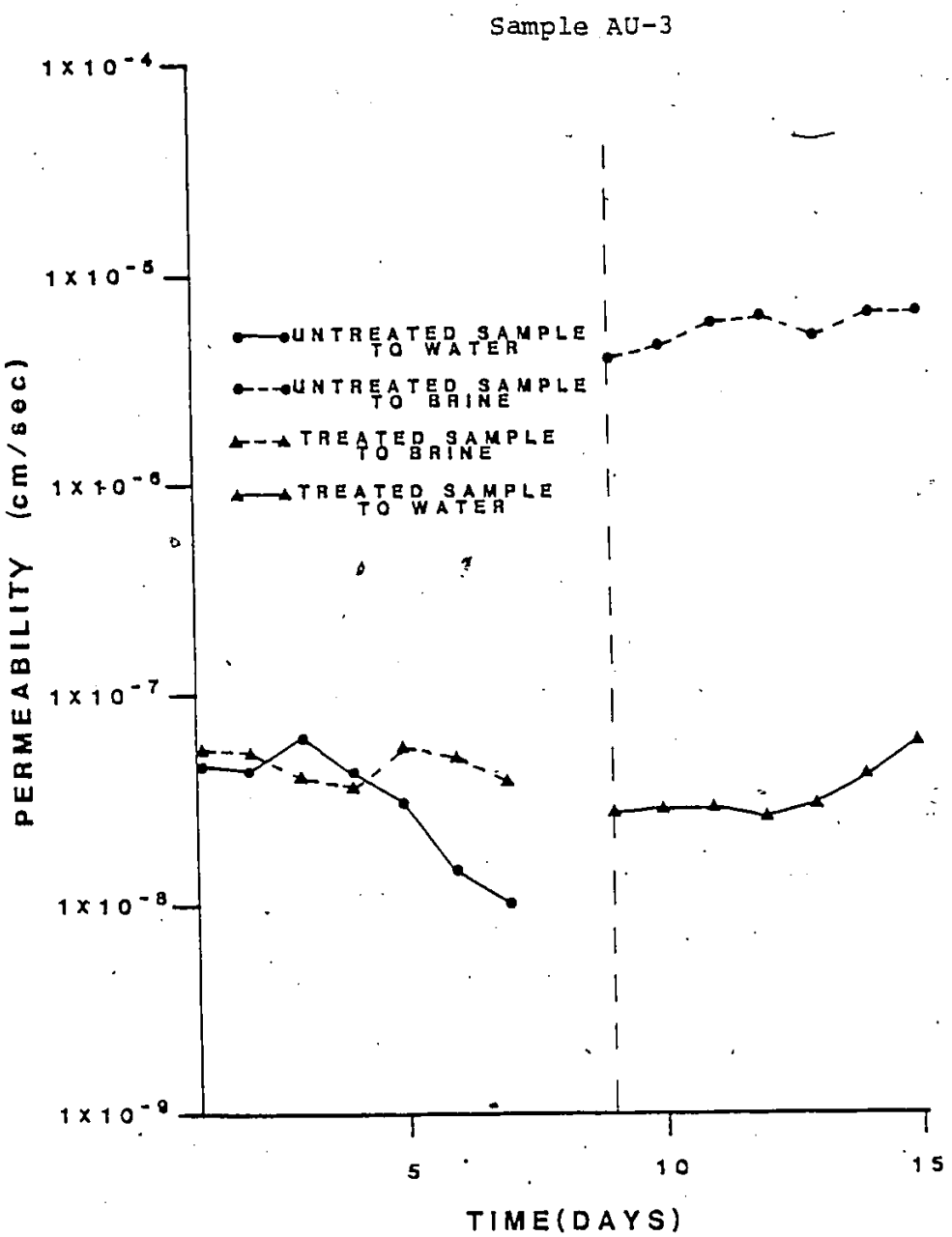


Fig. 5.22 Permeability vs. time.

Sample HG-4

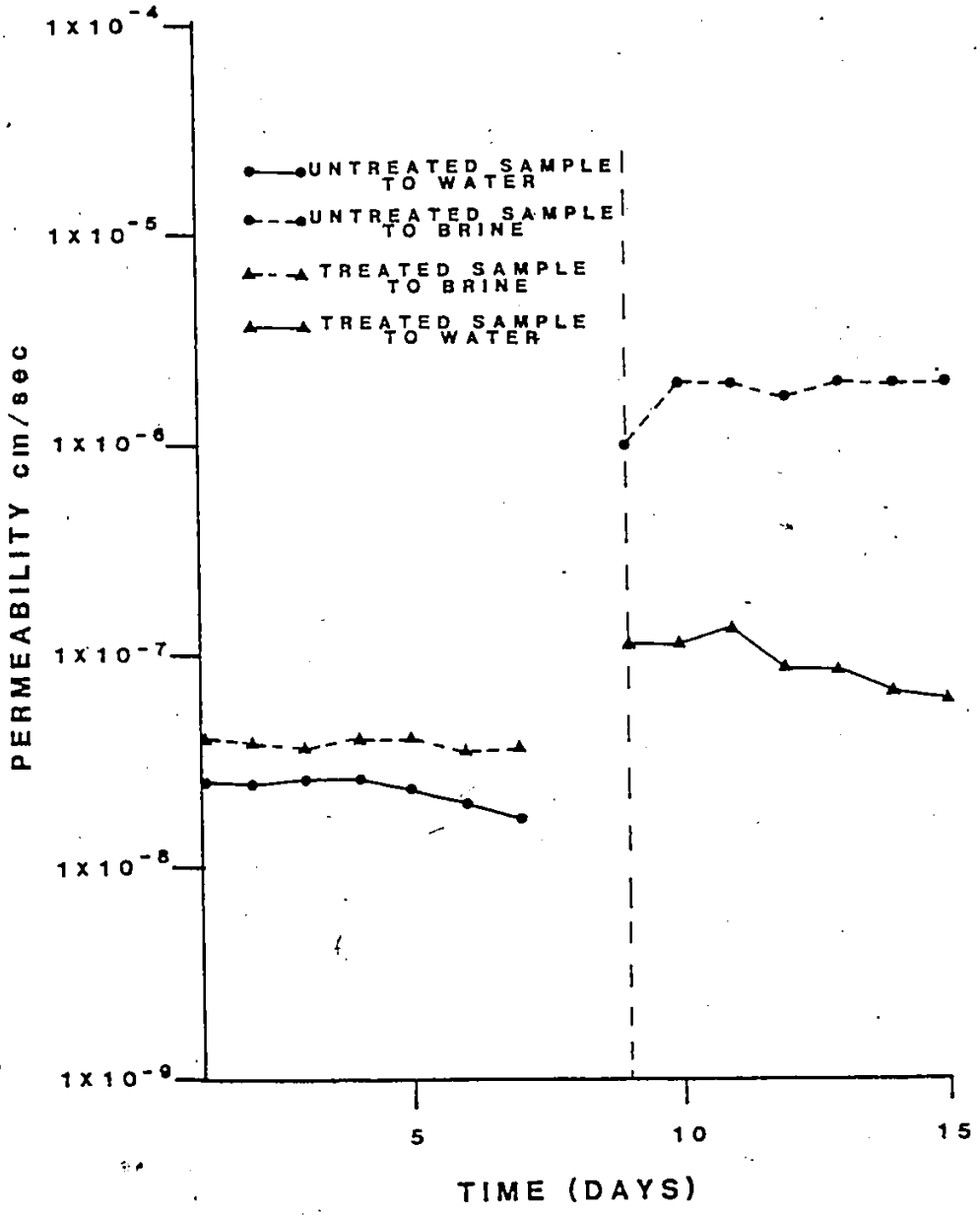


Fig. 5.23 Permeability vs. time

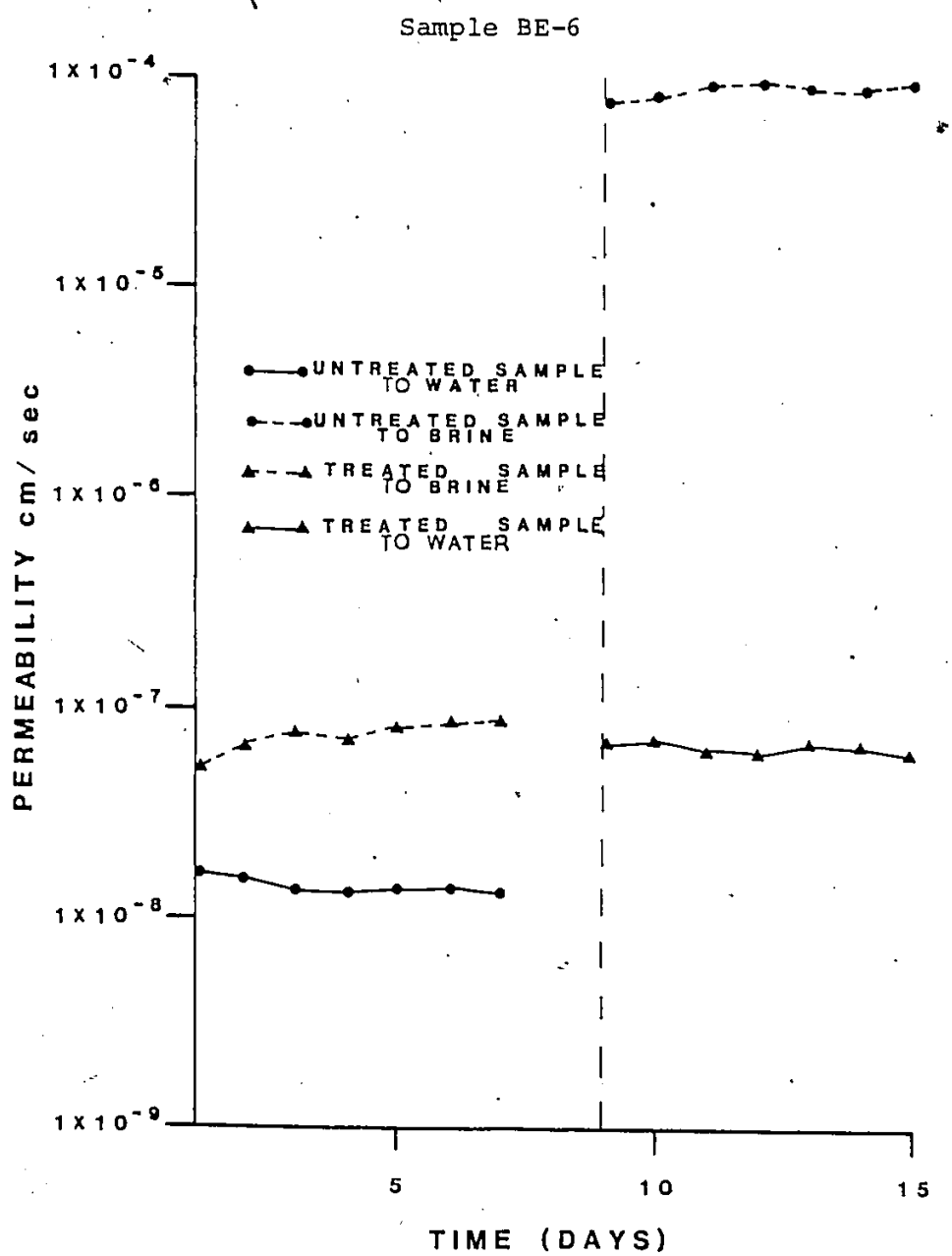


Fig. 5.24 Permeability vs. time

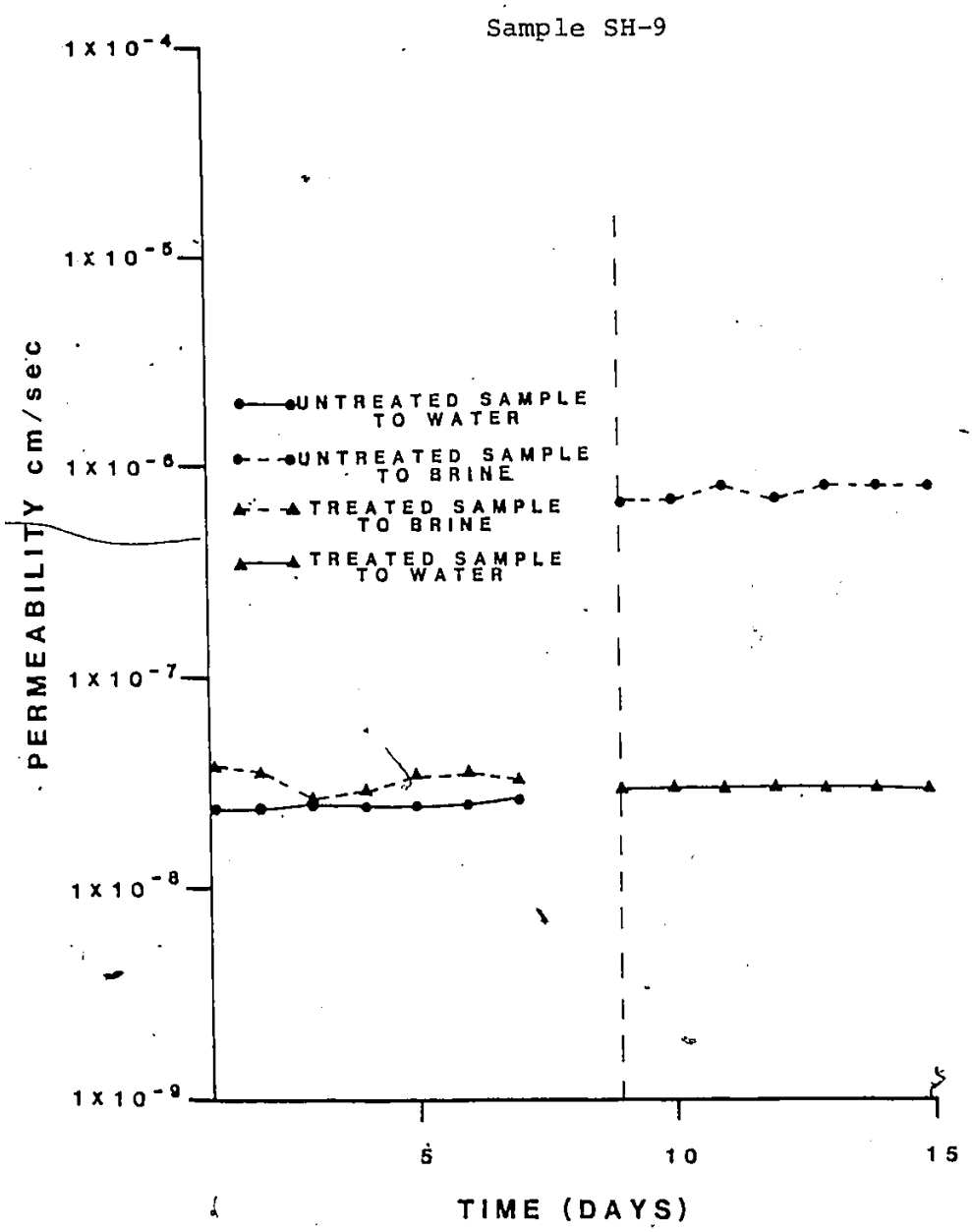


Fig. 5.25 Permeability vs. time.

Sample SE-7

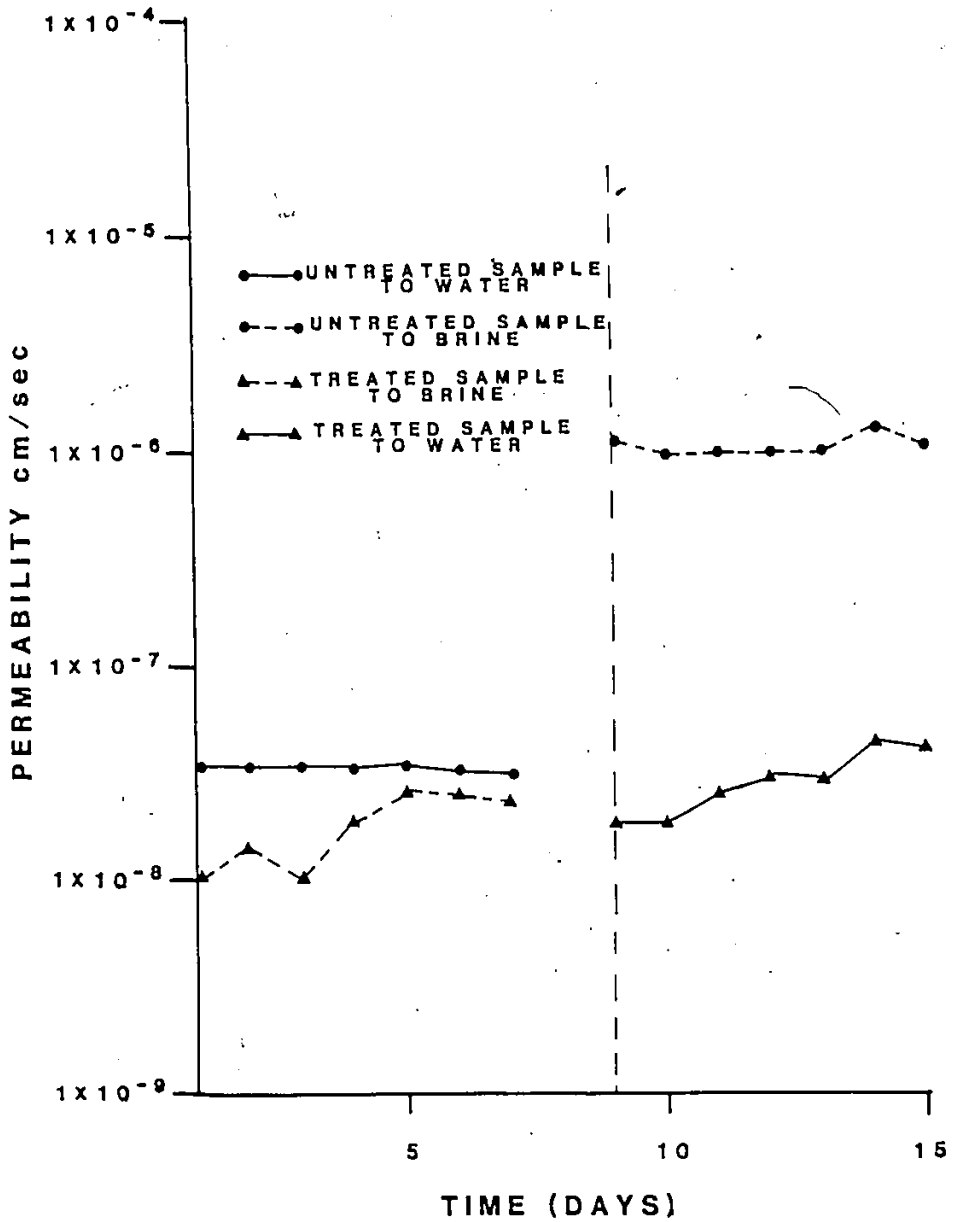


Fig. 5.26 Permeability vs. time.

explanation can be offered for this behaviour except that it would be due to error in the sample preparation.

The permeability of untreated samples showed significant increase to brine. The increase in permeability to brine exhibited by all the samples was due to the formation of dessication cracks (Section 2.6.3). Examples of the cracks are shown in Figs. 5.27 and 5.28. In samples BR-2, HG-4, AU-3 and BE-6 and most of the samples tested, the crack development tend to be fairly rapid after initiation. The cracks observed during the tests were mostly horizontal. When the specimens were examined at the end of the test, however, it was seen that they easily ruptured in the vertical direction. The formation of the horizontal cracks was probably due to reorientation of the clay particles during compaction. When clay is compacted, the plate-like particles tend to reorient themselves horizontally, that is, parallel to the face of the compacting hammer. The dessication shrinking of the clay due to sodium cation exchange is greatest in the vertical direction due to the weak bonding between layers. In the presence of the brine, the cracks, therefore, develop parallel to the plates.

The permeability of the treated samples to brine did not change much during the last two days of the test. Within the first four days, this permeability was irregular

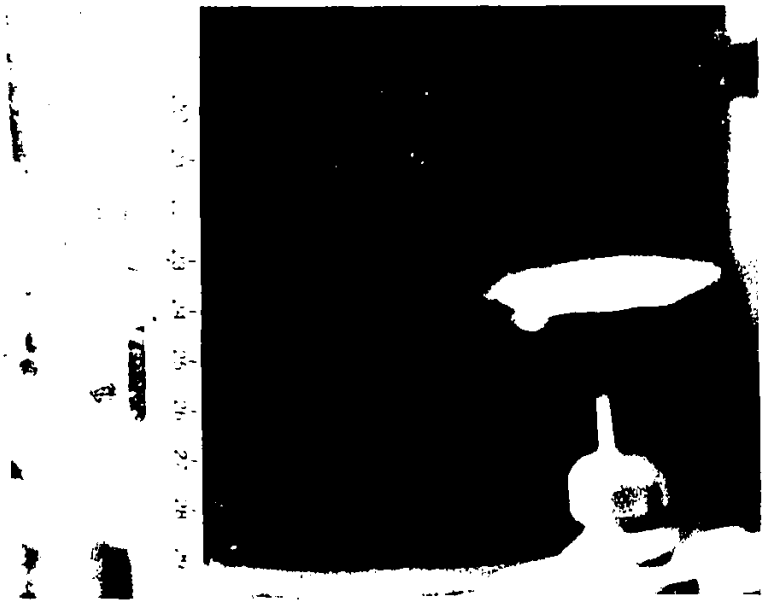


Fig. 5.27 Photograph showing the formation of cracks in sample SC-5 during testing of the untreated sample for its permeability to brine.

(a)

(b)

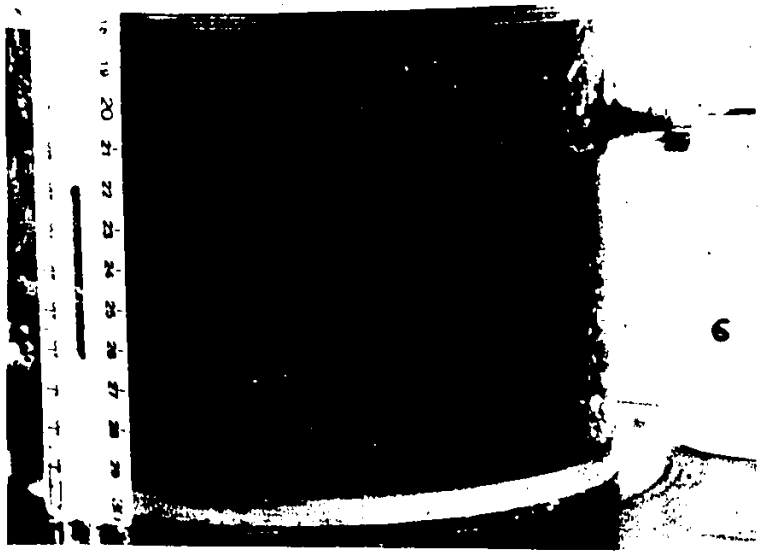


Fig. 5.28 Photographs showing similar cracks in samples: (a) BR-2, and (b) BE-6.

for samples AU-3 and SE-7. This irregularity was probably due to changes in the density or degree of saturation as a result of the particle shifting mentioned above. It can be seen from the graphs that the permeability of the treated sample decreased slightly when water was used as the permeant. A possible explanation for this behaviour is that the water molecule becomes strongly attached to the sodium ion. This is due to electrostatic attraction between the negative ends of the water dipole and the positive sodium ion. This structural arrangement at the surfaces of the clay particles prevent further entry of water. As time goes on, however, some of the sodium ions are removed by leaching and the permeability begins to increase slightly. Where the leaching was not great, the permeability remained fairly constant. The graphs also show that the permeability of the untreated samples to water and that of the treated samples to brine and water were of the same order of magnitude.

CHAPTER VI

SUMMARY AND CONCLUSIONS

6.1 Summary

The high correlations obtained in the regression analysis indicate that relationships exist between the Atterberg limits and water adsorption, the Atterberg limits and clay content, and the permeabilities of the treated soils to brine and the untreated to water. Values of the Pearson r obtained for the correlations among the treated characteristics were lower.

It was noted that the Atterberg limits of the soils were reduced as a result of treatment with brine. This can be explained by the dessication of the clay in the presence of brine. The dominant clay mineral in soils sampled from the same locations as those used in this study was reported by Guillet (1977) to be illite. Interlayer cations in clay minerals are not accessible to normal exchange reactions (Ingles, 1968). For example, the K^+ ion in illite has low hydration, large size (compared to the Na^+ ion) and surface coordination. As a result, its removal or replacement is very difficult. In the brine environment, other cations at the clay sur-

face are, however, exchanged for sodium ions due to the high concentration of the latter. In the process, the clay loses water; interparticle attractions are increased and the liquid limit is reduced. The plastic limit is also reduced but only slightly. Measurements of water adsorption of the treated samples reported in the study were significantly higher than the untreated samples. This was due to the hydration of the sodium cation.

The dessication of the clay led to shrinking and hence the formation of cracks. This resulted in the very high permeabilities to brine reported for the untreated samples. The permeability to brine was substantially reduced by pretreating and remoulding the samples with brine. As long as the treated samples were maintained in the flocculated condition with the permeation of brine, the permeability remained significantly low. With the introduction of fresh water, however, leaching took place and the permeability was increased.

6.2 Conclusions

A study of some glacial clay soils from Ontario has shown that their geotechnical properties are altered significantly in the presence of brine. Some conclusions which are believed to be of interest in the storage of brine in clay liners may be drawn from the available information:

1. The Atterberg limits are reduced as a result of dessication shrinking due to sodium cation exchange. With respect to the A-line of the plasticity chart, the majority of samples treated with brine plot in the zone of "cohesionless soils." The effect of the treatment is to reduce the plasticity of the soils (that is to move their positions on the plasticity chart to a lower position along the A-line).

2. Among the monovalent alkalis the sodium ion has a reasonably high hydration. Soils treated with brine will adsorb higher amounts of water than untreated soils due to the hydration of the sodium ion.

3. Dessication shrinking of the soils lead to the formation of cracks. Consequently, the untreated soil will be more permeable to brine than to fresh water. The ratio of the permeabilities obtained was approximately 100:1.

4. In view of the last point, brine ponds designed and constructed with specifications applicable to fresh water will not be able to hold brine.

5. The permeability to brine can be reduced by pretreating and remoulding the soil with brine. The permeabilities of the treated soil to brine and the untreated soil to water have the same order of magnitude.

6.3 Recommendations

Clay soils to be used in the construction of brine ponds should be pretreated with brine before storage. This can be done by moulding the soil with brine during compaction or mixing the soil with NaCl crystals before compaction. In the latter case, brine should still be used as the moulding fluid. At the optimum compaction, the treated soil is at its maximum density. More significantly, it is in equilibrium with its own cation; there is, therefore, very little chance for any cation exchange reactions to take place to alter the structure of the soil.

The results presented are based on only 19 samples; further investigation is needed, especially in the area of the effects of different salt concentrations on permeability. In addition, a study of a variety of soil types would be worthwhile to augment the information presented in this study.

APPENDIX A

Sample Descriptions and Locations

Sample Descriptions and Locations

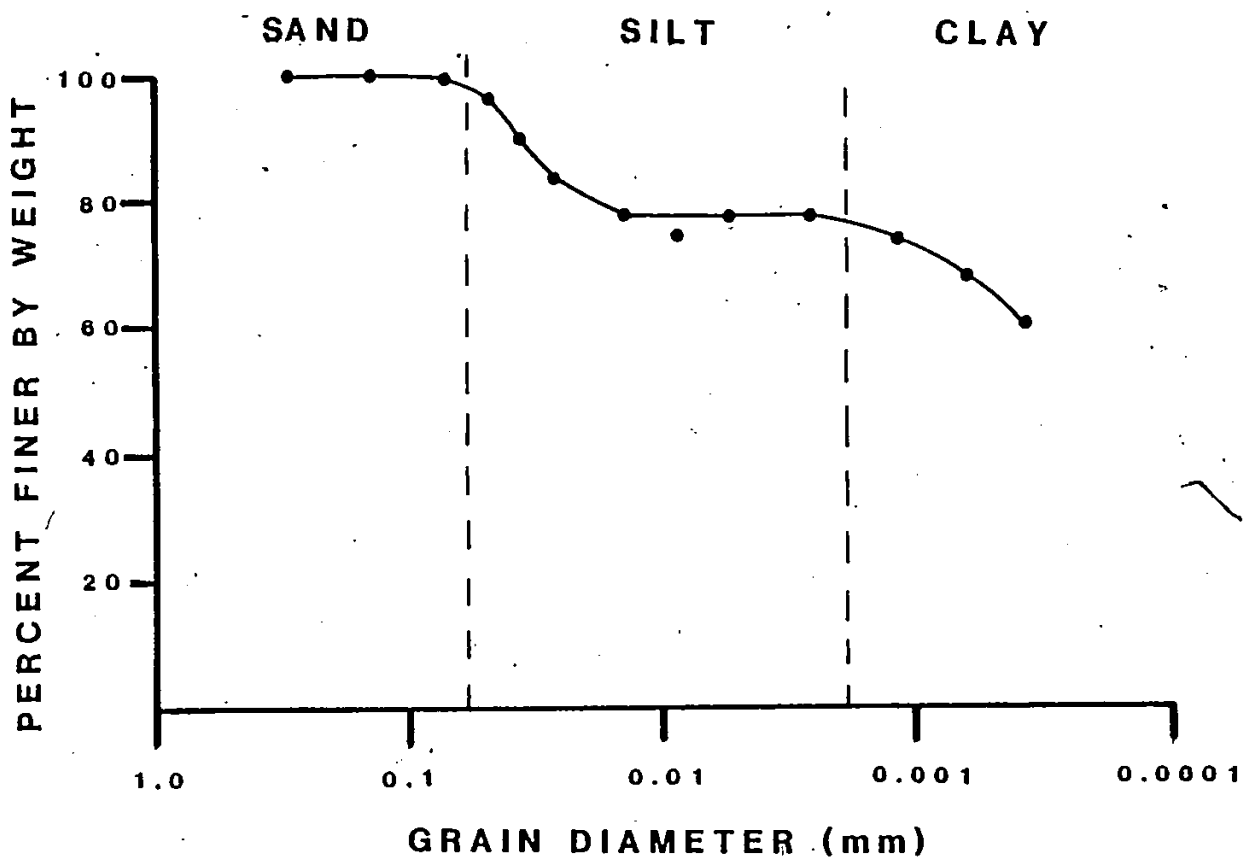
Location Number	Sample Number	Location and Description
1		Shallow bank of Little Cataraqui Creek in Kingstown Township. Concession IV, 1 km west of Country Road 10, 0.8 km north of Highway 401.
	KN-1,KN-10	Mottled, blue-grey, highly plastic clay.
2		Bank of a pool north of the Michigan Central Railway track, located south of Brownsville in Dereham Township, 13 km west of Tillsonburg.
	BR-2,BR-11	2 m of tough stoneless brown clay mottled with rust and grey, massive near the top with fine wavy laminations near the base.
3		Banks of the Holland River 1.6 km east of Aurora.
	AU-3,AU-12	Stratified buff-brown and yellow silt and silty clay.
4		Abandoned pit belonging to the former tile yard of Norman McEachen and Sons. Concession IV, Orford Township, Kent County, 2 km southeast of Highgate.
	HG-4,HG-13	Mottled grey and brown clay, tough, stoneless and gritty.
5		Scarborough bluffs about 600 m west of the mouth of Highland Creek in Scarborough.
	SC-5,SC-19	Finely laminated interglacial clay, sand, and silt.

Location Number	Sample Number	Location and Description
6		A shallow river bank 1.6 km west of Beeton in Tecumseh Township, Simcoe County.
	BE-6,BE-14	Grey brown rusty-mottled, pale grey, stoneless silty clay.
7		Southside of the Bayfield River in Tuckersmith Township Huron County, about 5 km southeast of Seaforth. North end of clay pit sampled.
	SE-7,SE-15	Coarsely, stratified brown interlayered with thin beds of yellow silt, tough, dense and stoneless.
8		Abandoned brickyard on the farm of M. L. Jeffrey in Hay Township, 2 km southeast of St. Joseph.
	SJ-8,SJ-16	Mottled grey and brown unstratified sandy clay.
9		Abandoned workings of a tile yard on the farm of A. E. Durnin at St. Helen, 8 km south of Lucknow.
	SH-9,SH-17	Mottled, brown, rusty yellow silty clay, weakly stratified, grades into grey silt and sand at the base.
10		Former tile yard of W. W. Sadler located on the south side of the Bayfield River in Hibbert Township, Perth County, 3 km south of Dublin. Upper part of river sampled.
	DU-18	Massive, tough, dark brown stoneless clay, loamy at the top.

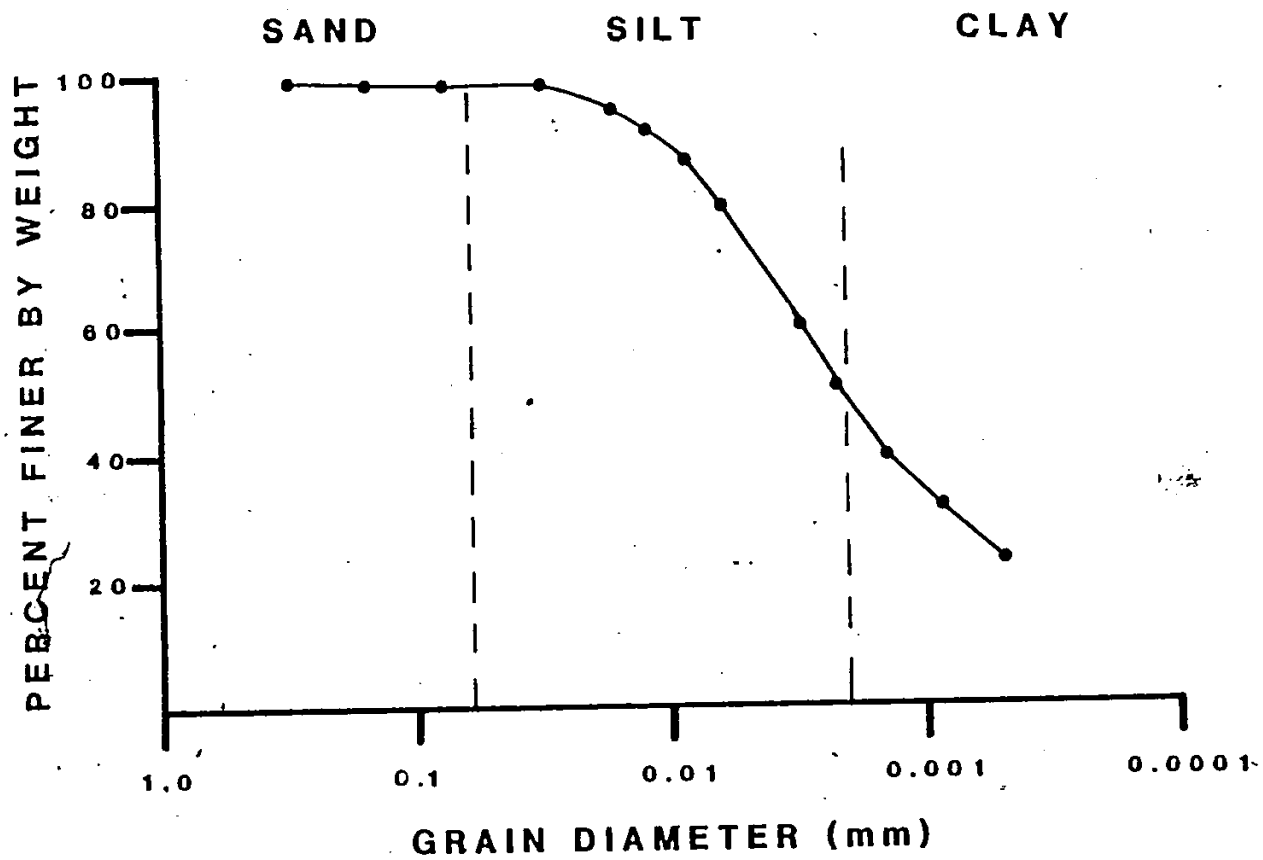
APPENDIX B

Grain Size Distribution Curves

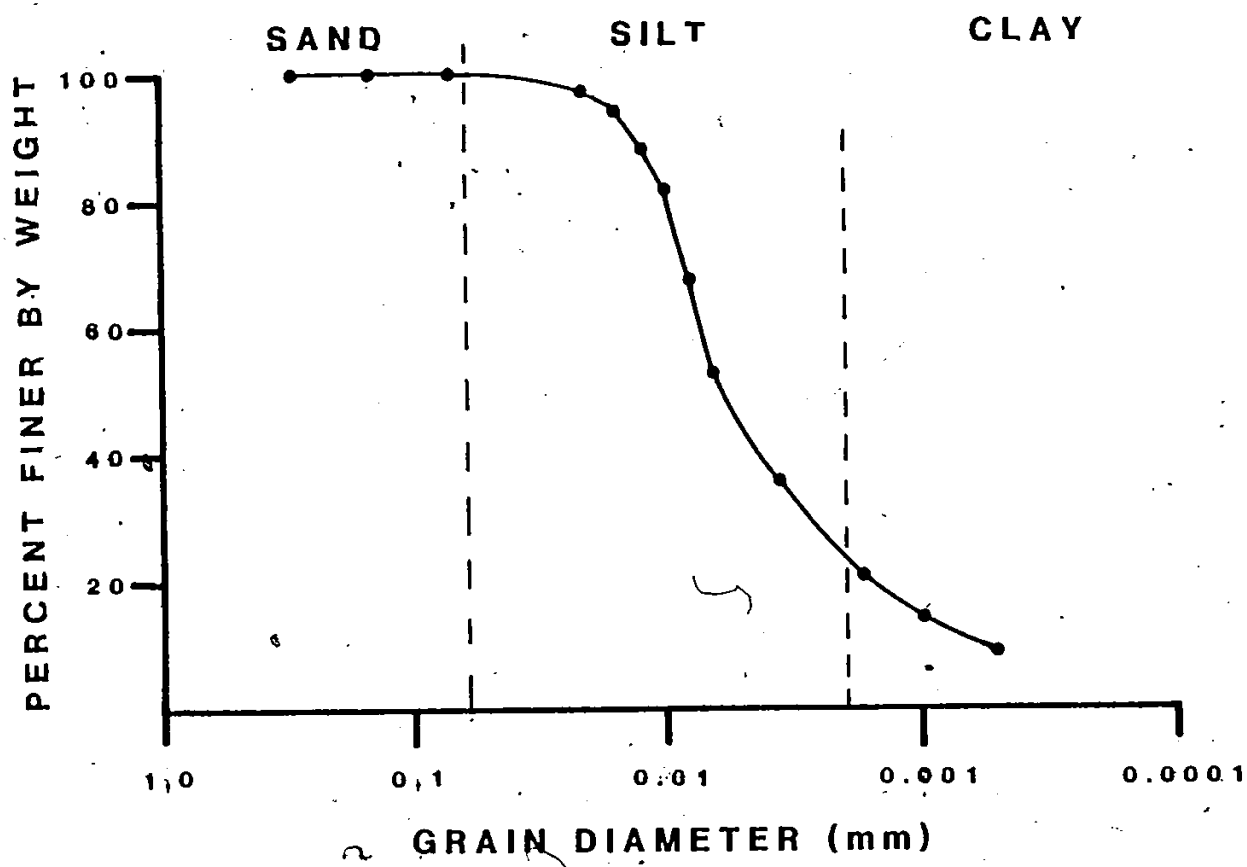
Sample KN-1


SAND
SILT
CLAY
PERCENT FINER BY WEIGHT
100
80
60
40
20
1.0
0.1
0.01
0.001
0.0001
GRAIN DIAMETER (mm)

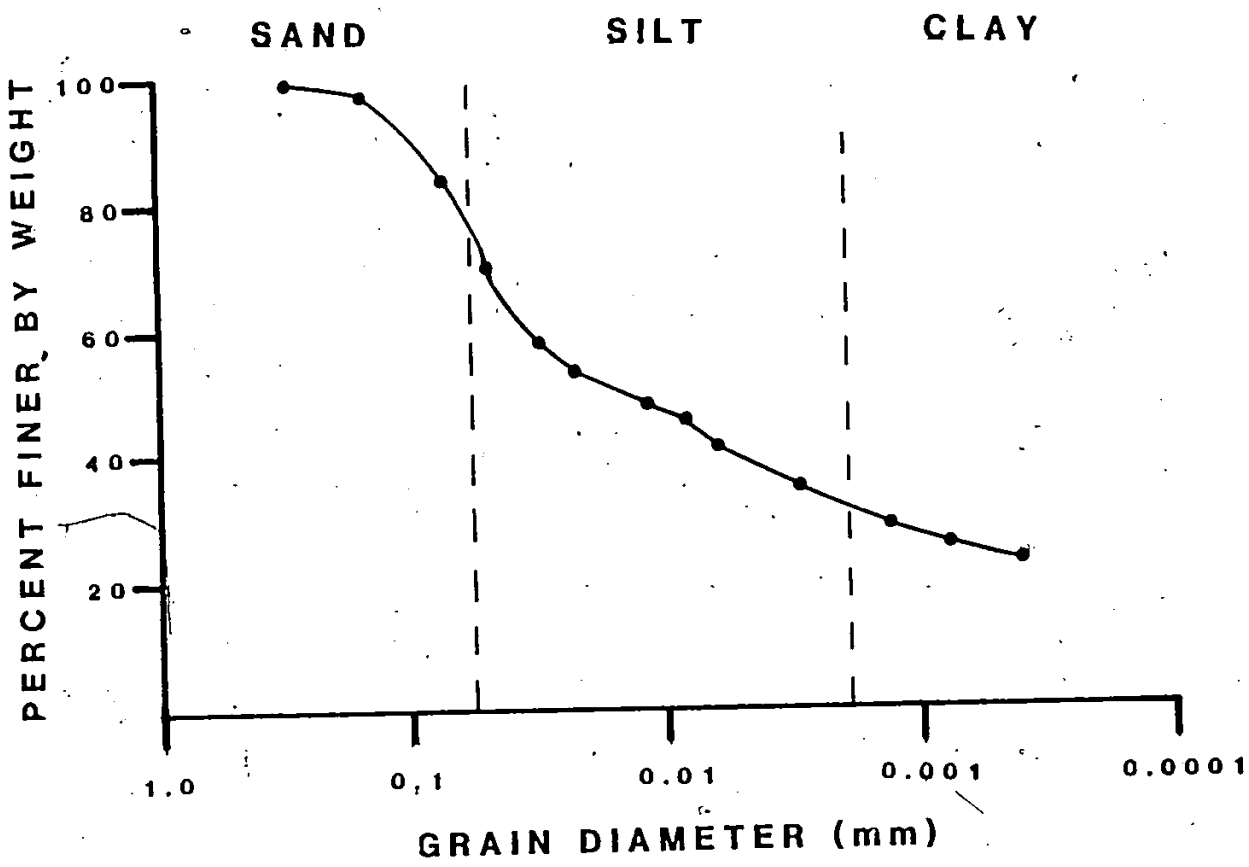
Sample BR-2


SAND
SILT
CLAY
PERCENT FINER BY WEIGHT
100
80
60
40
20
1.0
0.1
0.01
0.001
0.0001
GRAIN DIAMETER (mm)

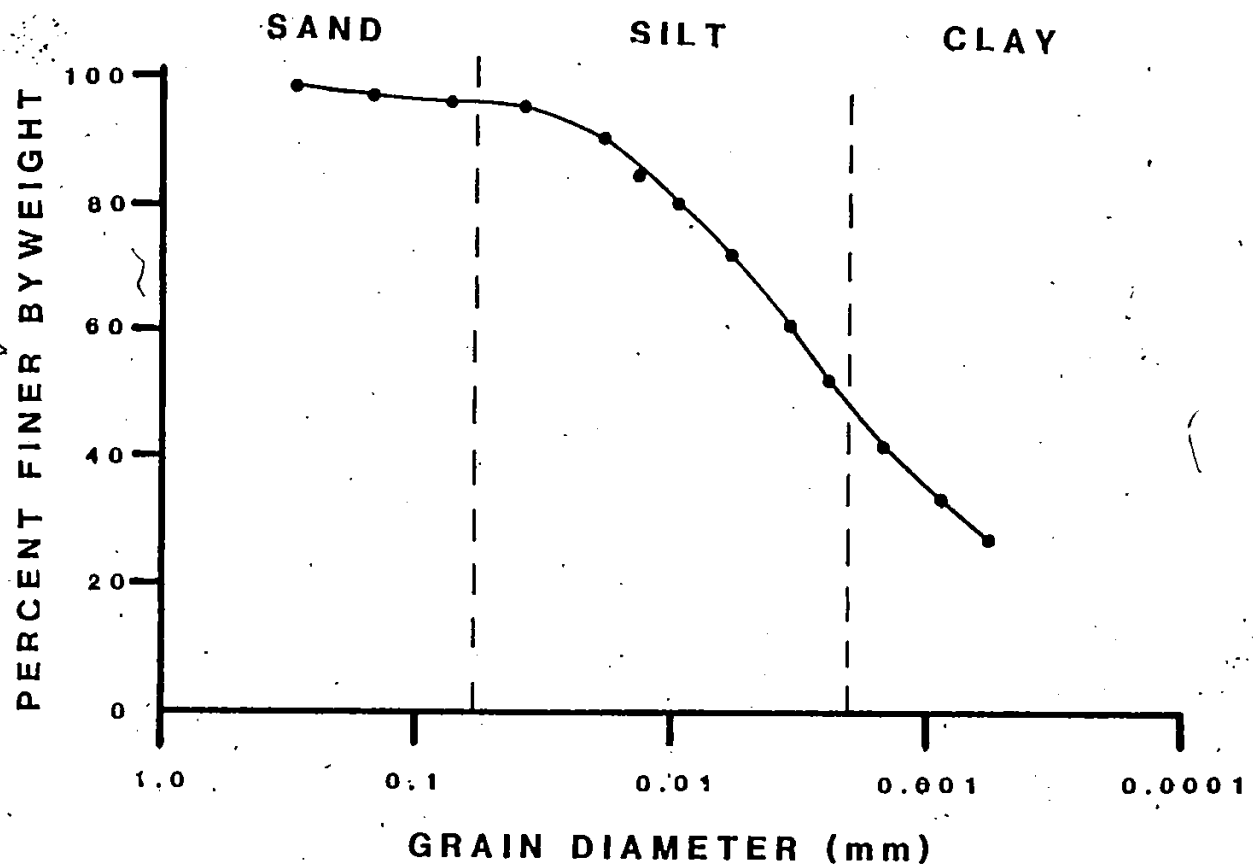
Sample AU-3


SAND
SILT
CLAY
PERCENT FINER BY WEIGHT
100
80
60
40
20
1.0
0.1
0.01
0.001
0.0001
GRAIN DIAMETER (mm)

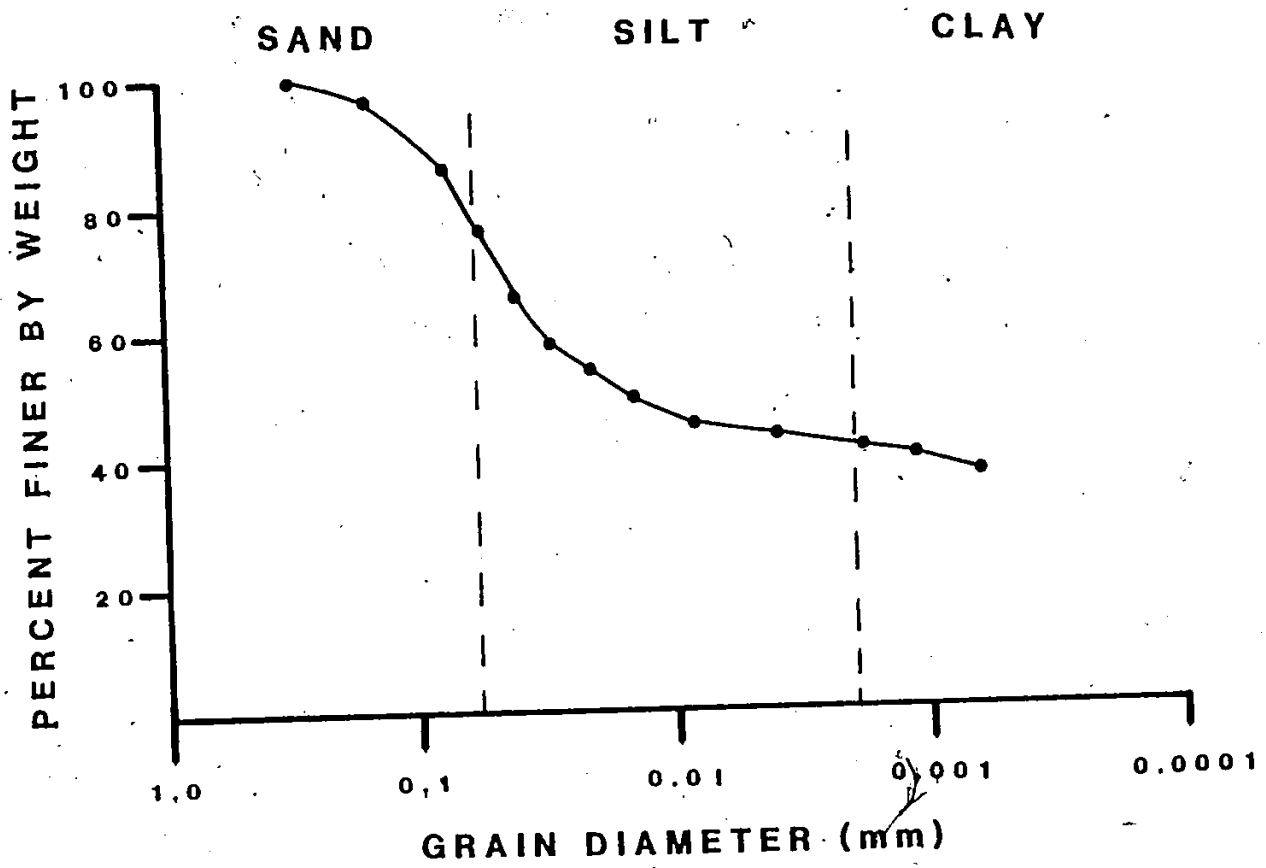
Sample HG-4


SAND
SILT
CLAY
PERCENT FINER BY WEIGHT
100
80
60
40
20
1.0
0.1
0.01
0.001
0.0001
GRAIN DIAMETER (mm)

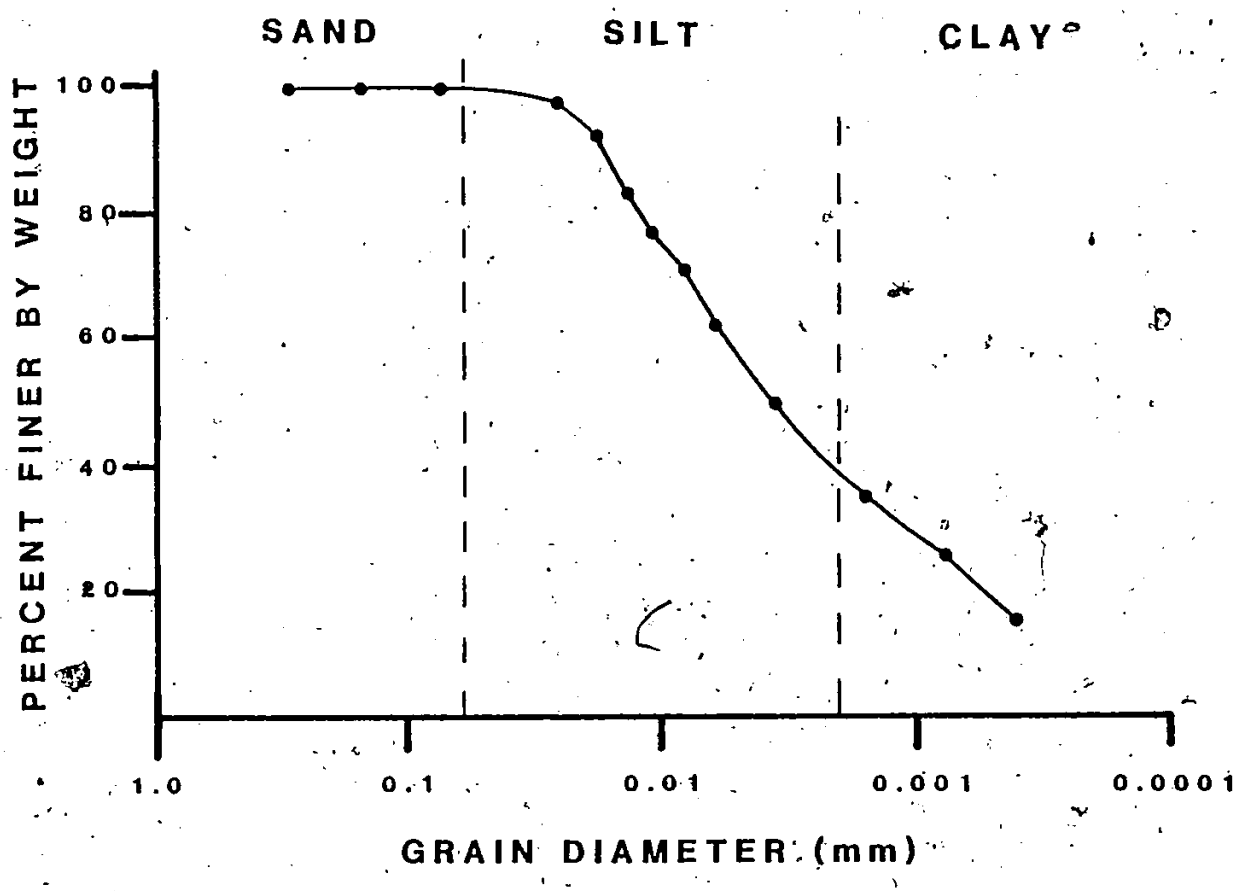
Sample SC-5


SAND
SILT
CLAY
PERCENT FINER BY WEIGHT
100
80
60
40
20
0
1.0
0.1
0.01
0.001
0.0001
GRAIN DIAMETER (mm)

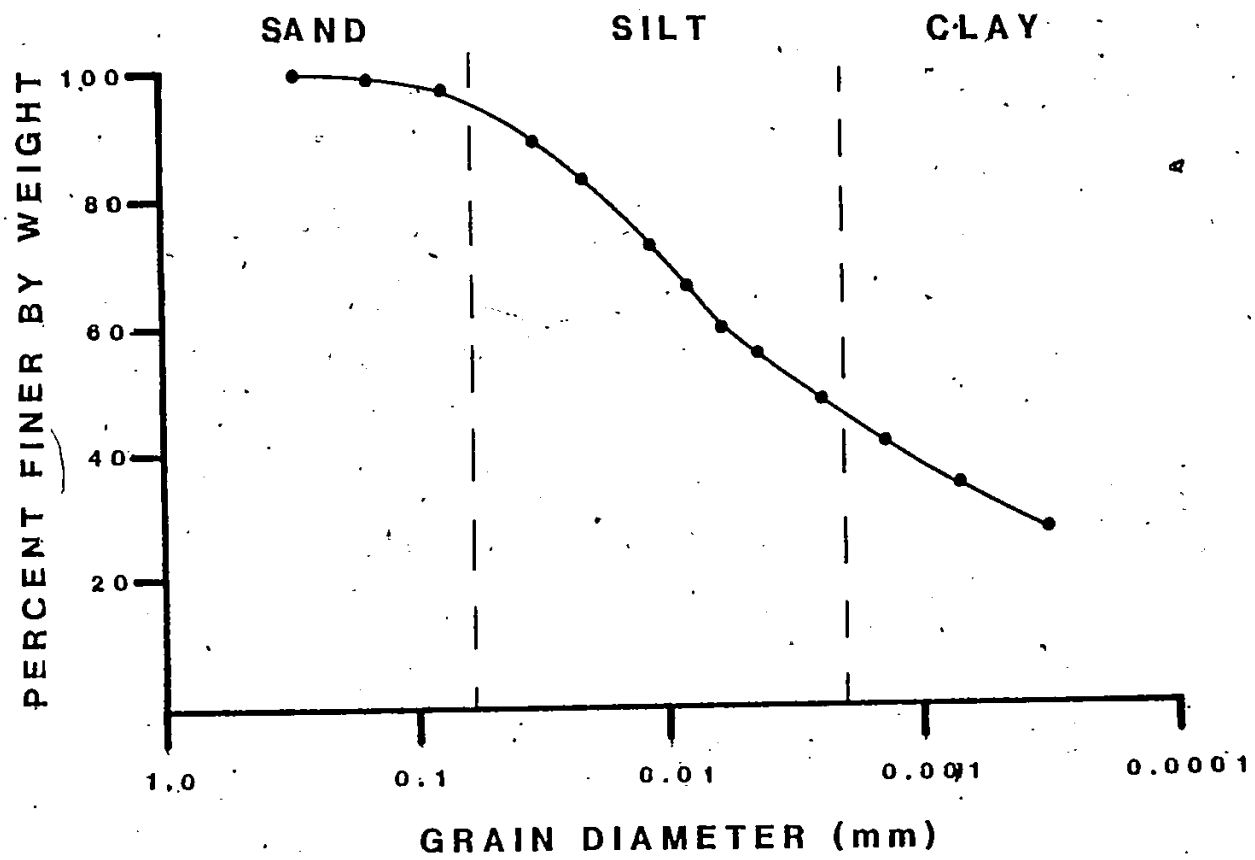
Sample BE-6


SAND
SILT
CLAY
PERCENT FINER BY WEIGHT
100
80
60
40
20
1.0
0.1
0.01
0.001
0.0001
GRAIN DIAMETER (mm)

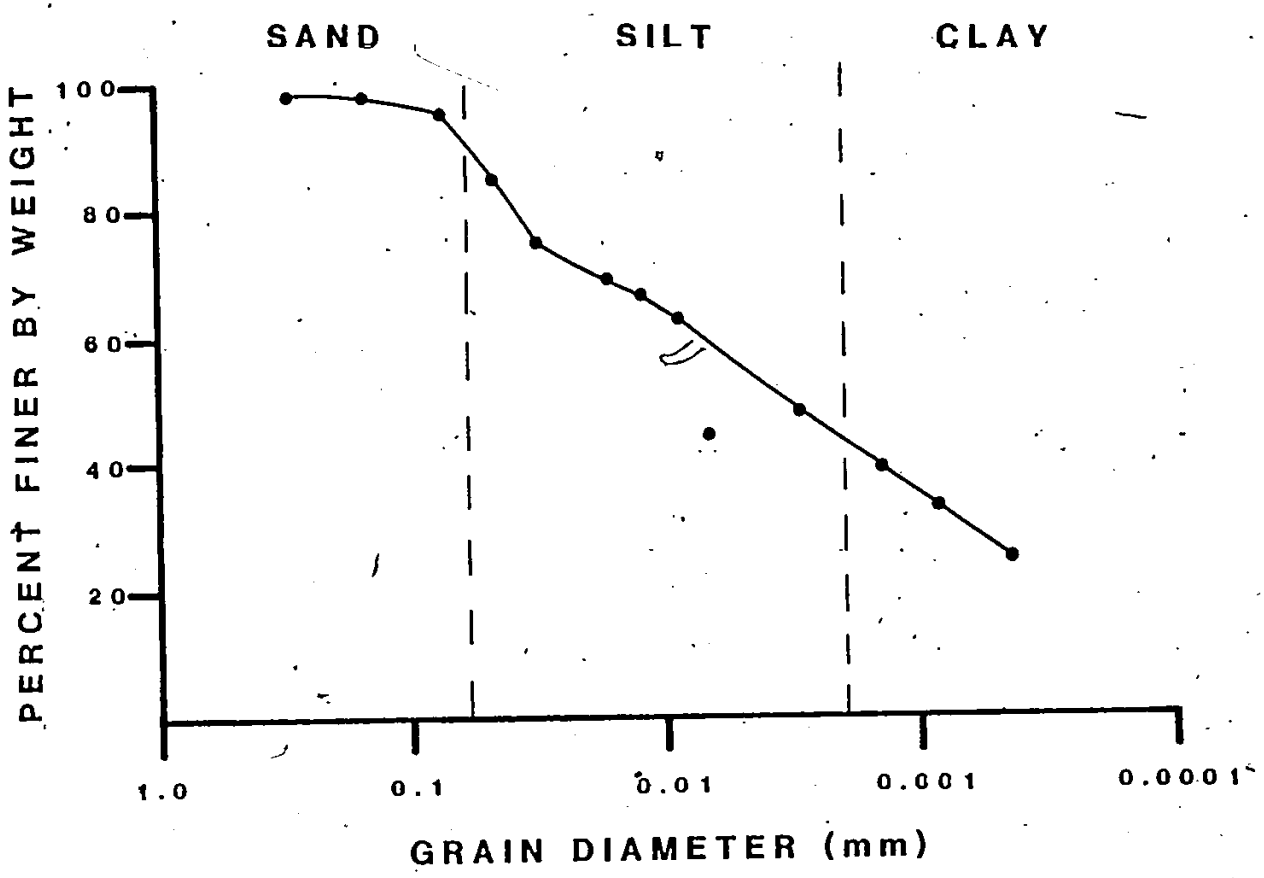
Sample SE-7


SAND
SILT
CLAY
PERCENT FINER BY WEIGHT
100
80
60
40
20
1.0
0.1
0.01
0.001
0.0001
GRAIN DIAMETER (mm)

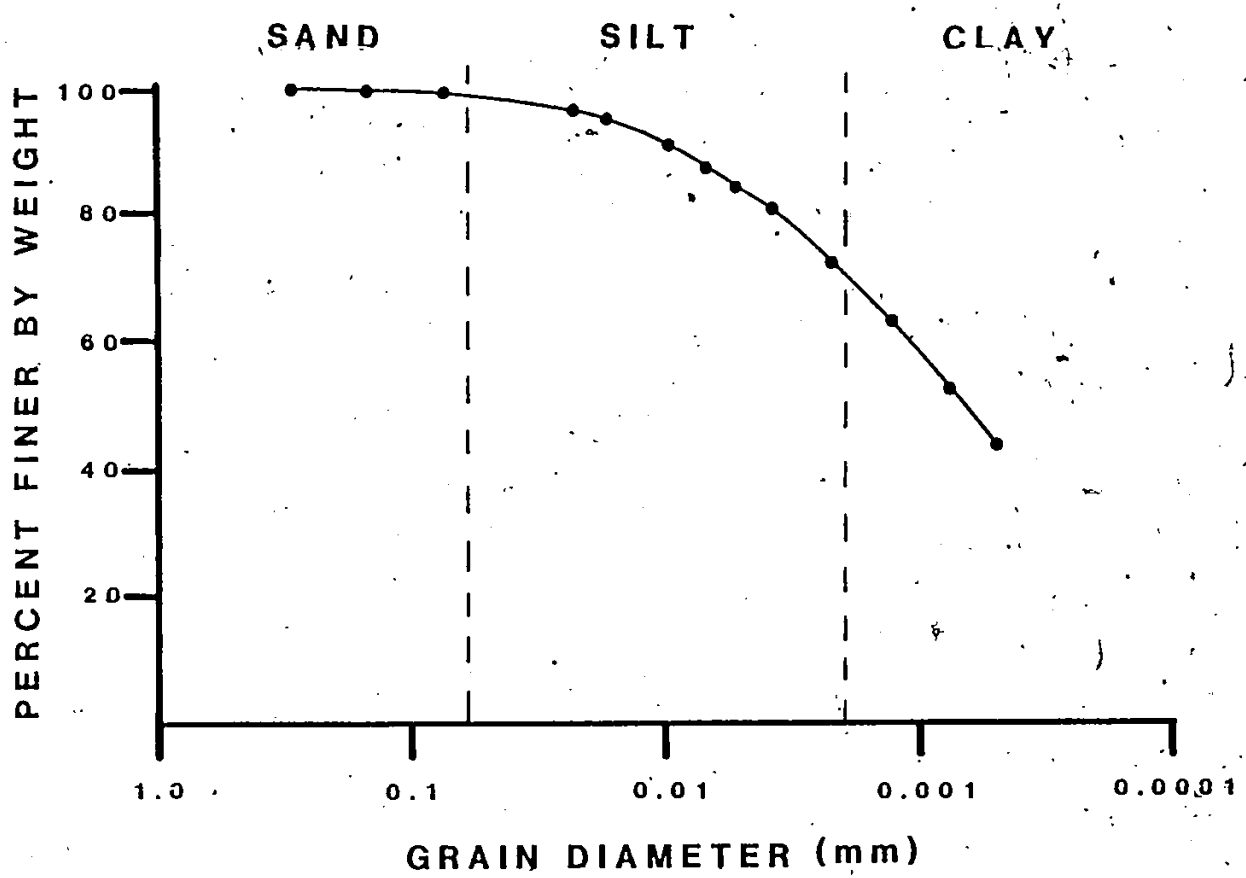
Sample SJ-8


SAND
SILT
CLAY
PERCENT FINER BY WEIGHT
100
80
60
40
20
1.0
0.1
0.01
0.001
0.0001
GRAIN DIAMETER (mm)

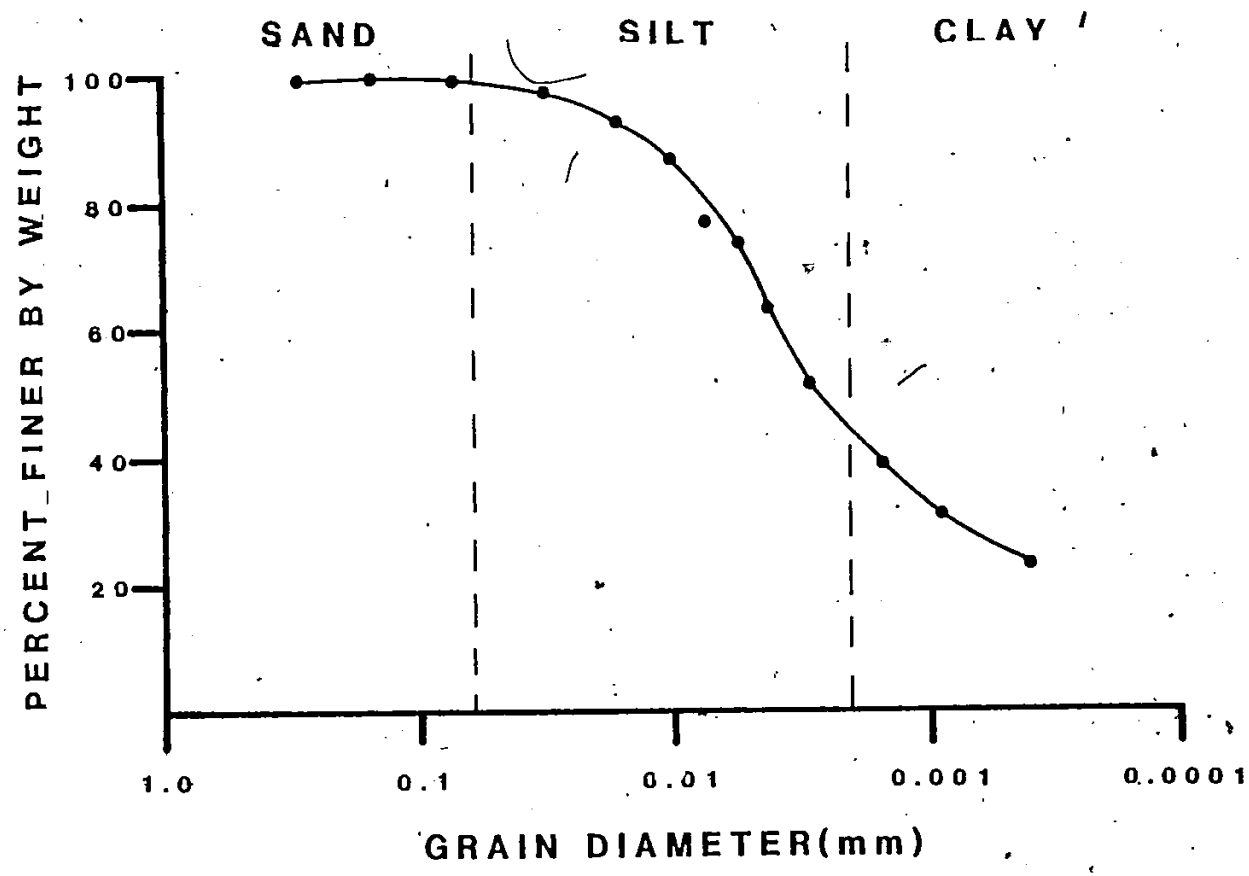
Sample SH-9


SAND
SILT
CLAY
PERCENT FINER BY WEIGHT
100
80
60
40
20
1.0
0.1
0.01
0.001
0.0001
GRAIN DIAMETER (mm)

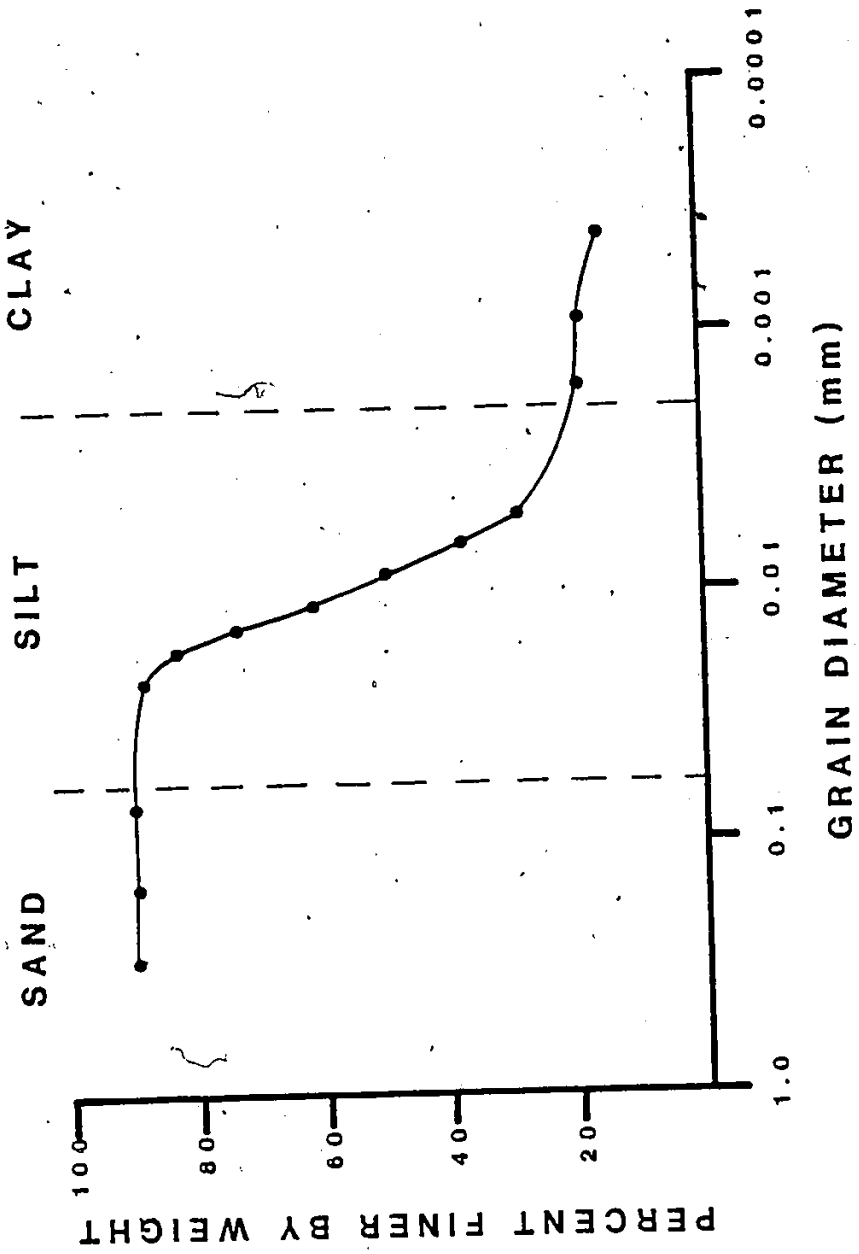
Sample KN-10


SAND
SILT
CLAY
PERCENT FINER BY WEIGHT
100
80
60
40
20
1.0
0.1
0.01
0.001
0.0001
GRAIN DIAMETER (mm)

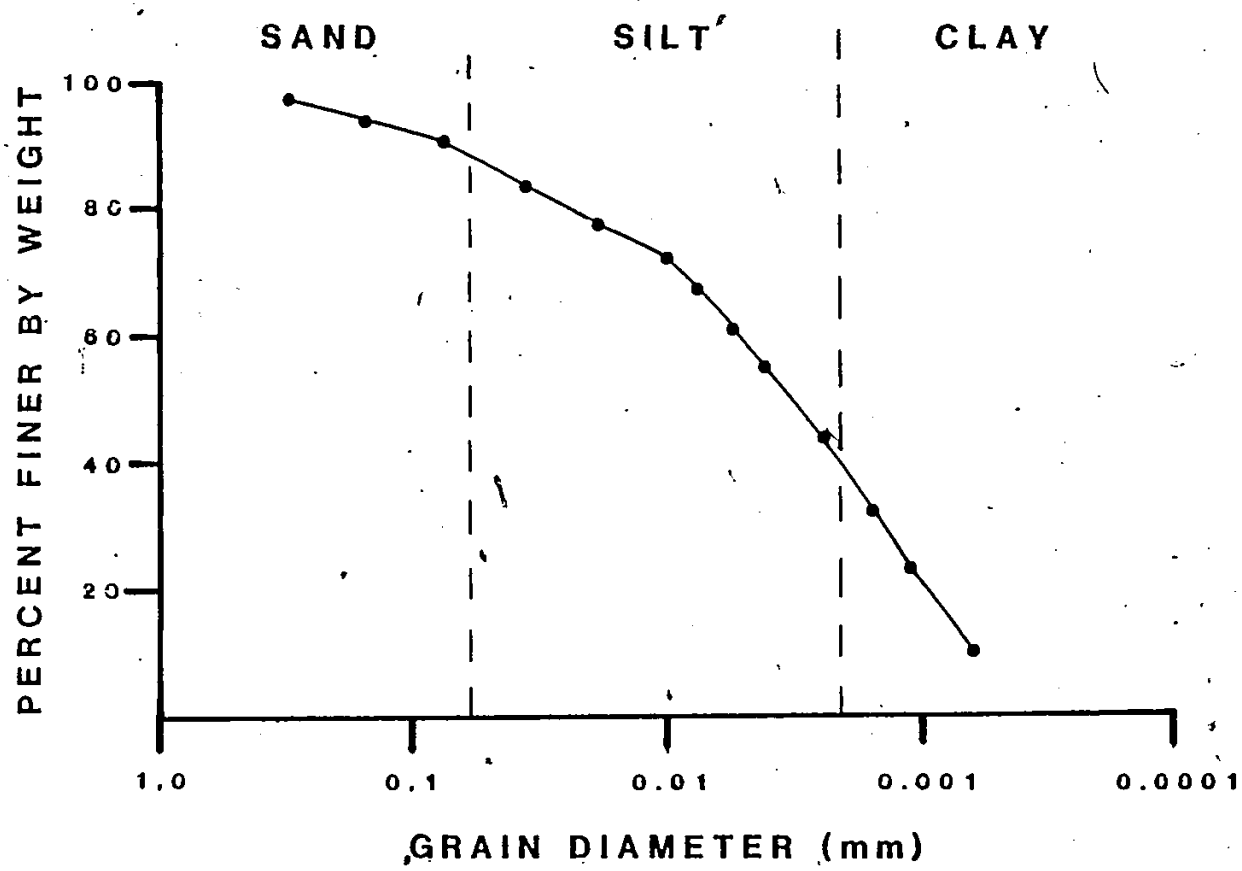
Sample BR-11


SAND
SILT
CLAY
PERCENT_FINER BY WEIGHT
100
80
60
40
20
1.0
0.1
0.01
0.001
0.0001
GRAIN DIAMETER(mm)

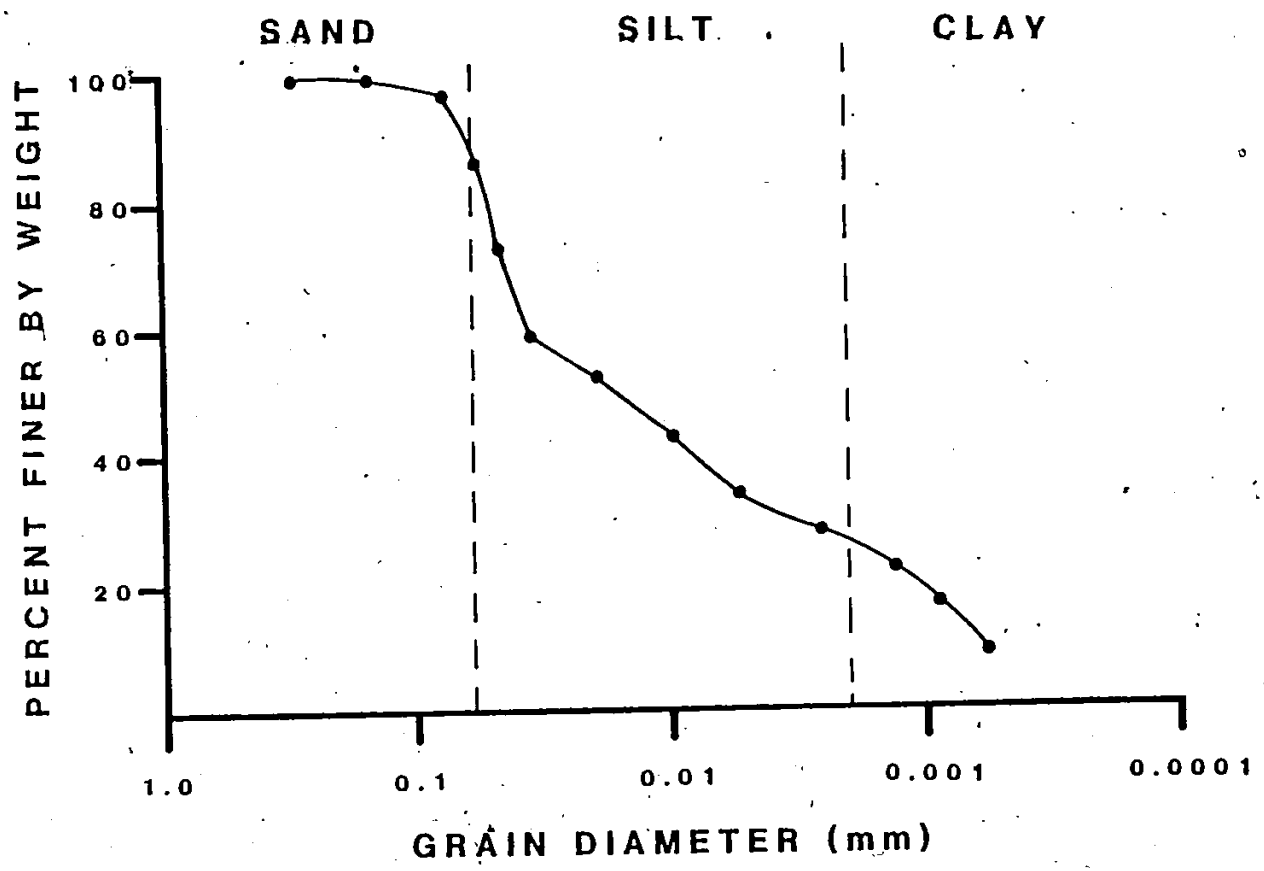
Sample AU-12


SAND
SILT
CLAY
100
80
60
40
20
PERCENT FINER BY WEIGHT
1.0
0.1
0.01
0.001
0.0001
GRAIN DIAMETER (mm)

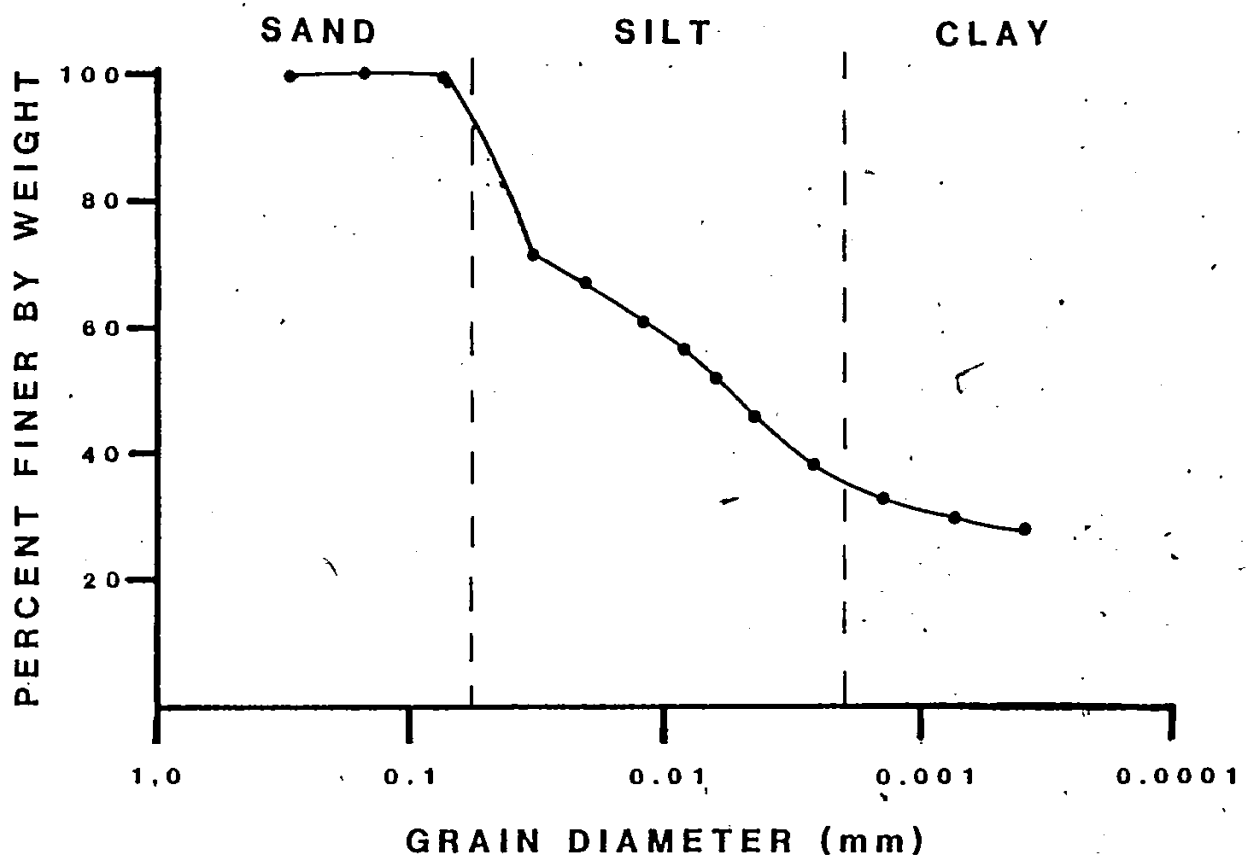
Sample HG-13


SAND
SILT
CLAY
PERCENT FINER BY WEIGHT
100
80
60
40
20
1.0
0.1
0.01
0.001
0.0001
GRAIN DIAMETER (mm)

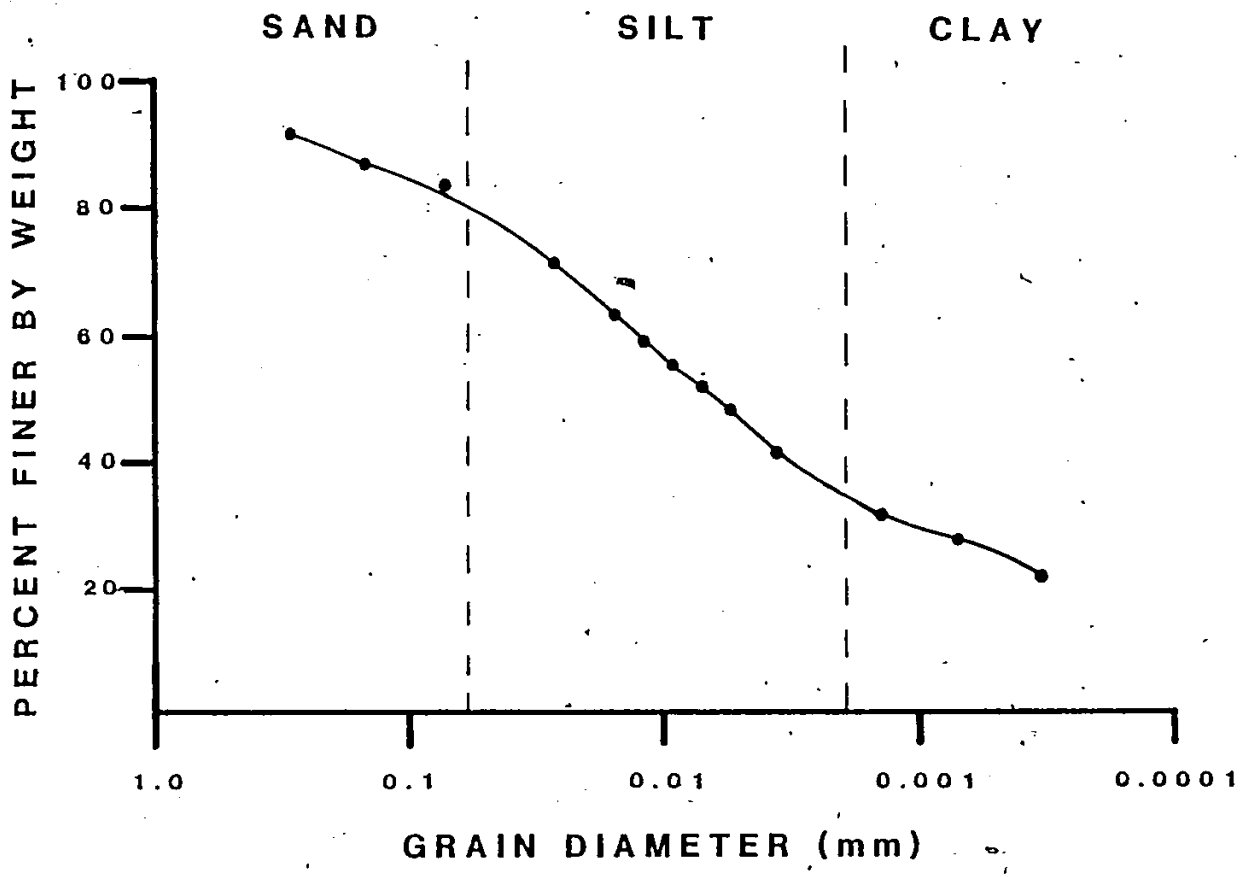
Sample BE-14


SAND
SILT
CLAY
PERCENT FINER BY WEIGHT
100
80
60
40
20
1.0
0.1
0.01
0.001
0.0001
GRAIN DIAMETER (mm)

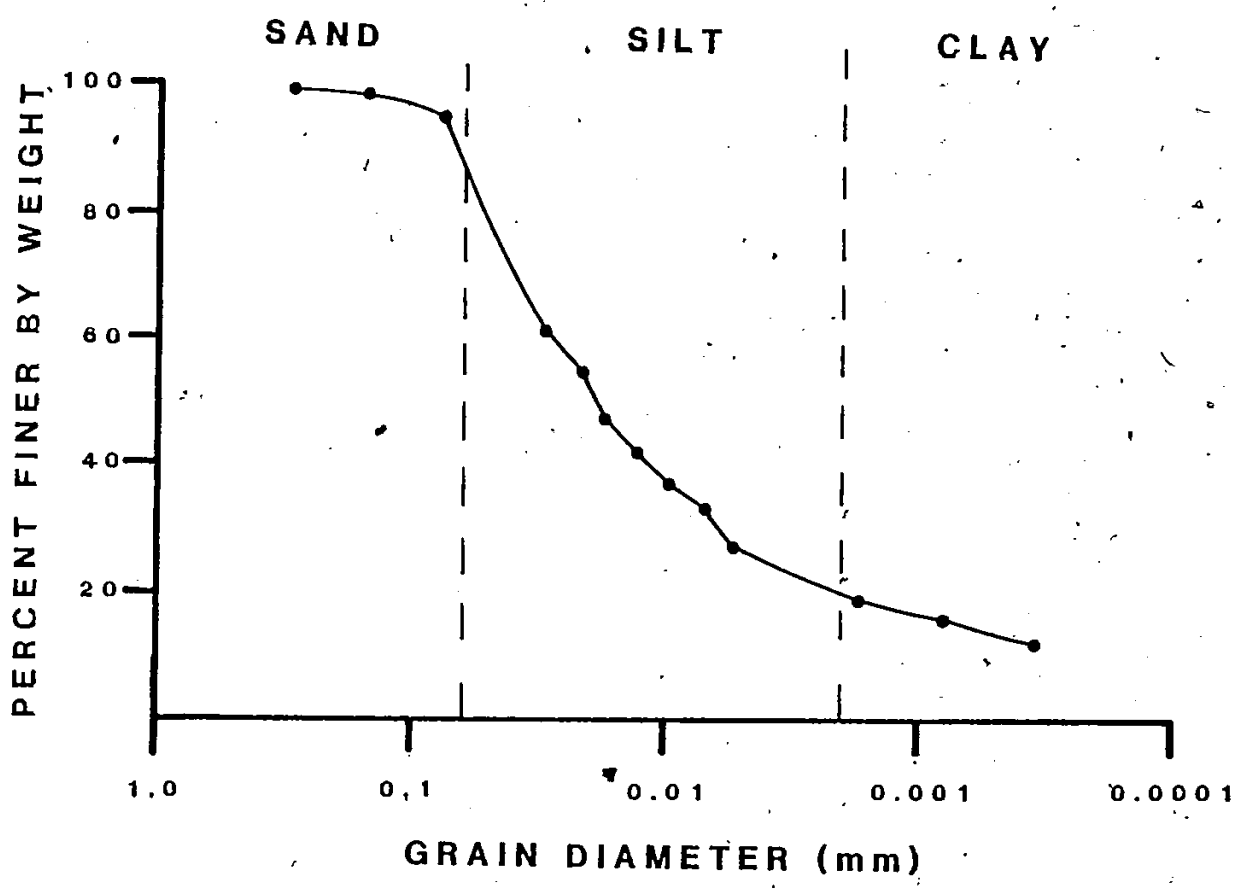
Sample SE-15


SAND
SILT
CLAY
PERCENT FINER BY WEIGHT
100
80
60
40
20
1.0
0.1
0.01
0.001
0.0001
GRAIN DIAMETER (mm)

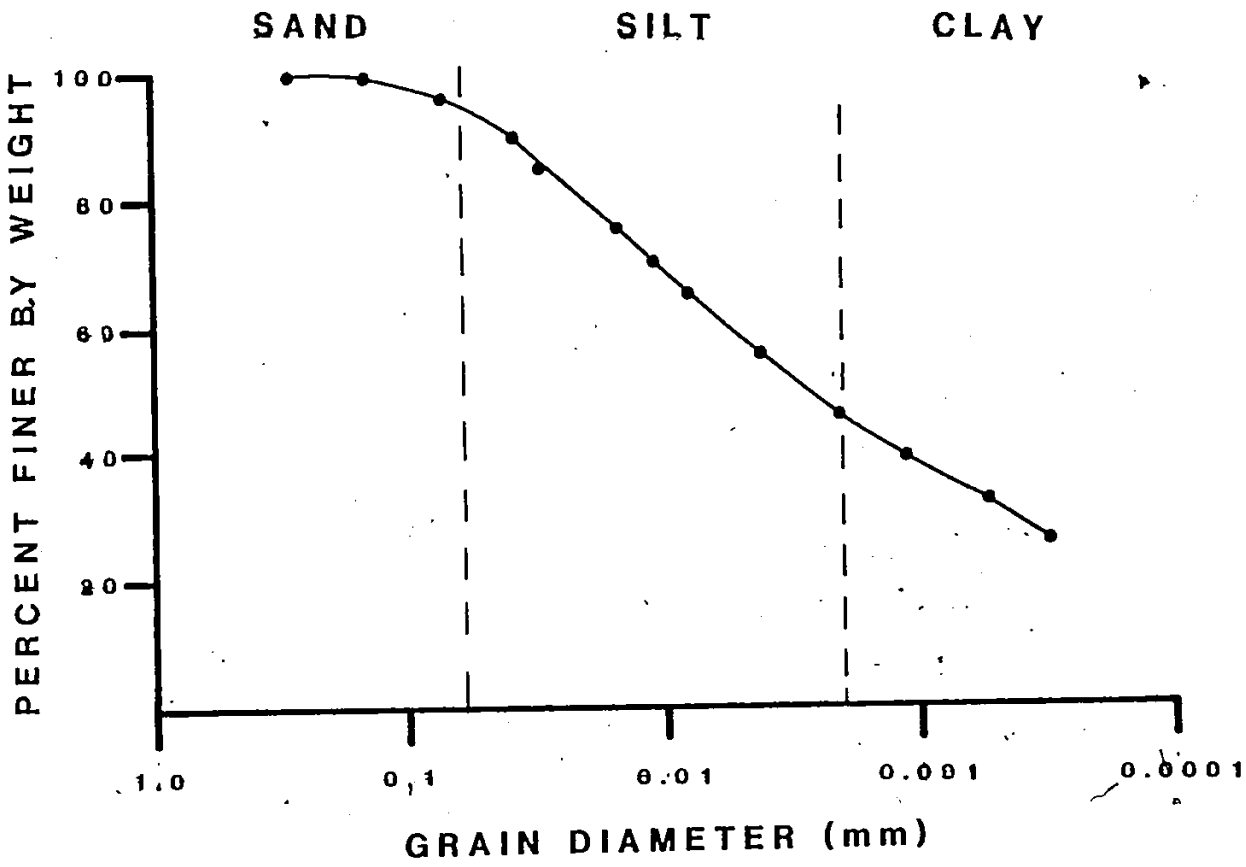
Sample SJ-16


SAND
SILT
CLAY
PERCENT FINER BY WEIGHT
100
80
60
40
20
1.0
0.1
0.01
0.001
0.0001
GRAIN DIAMETER (mm)

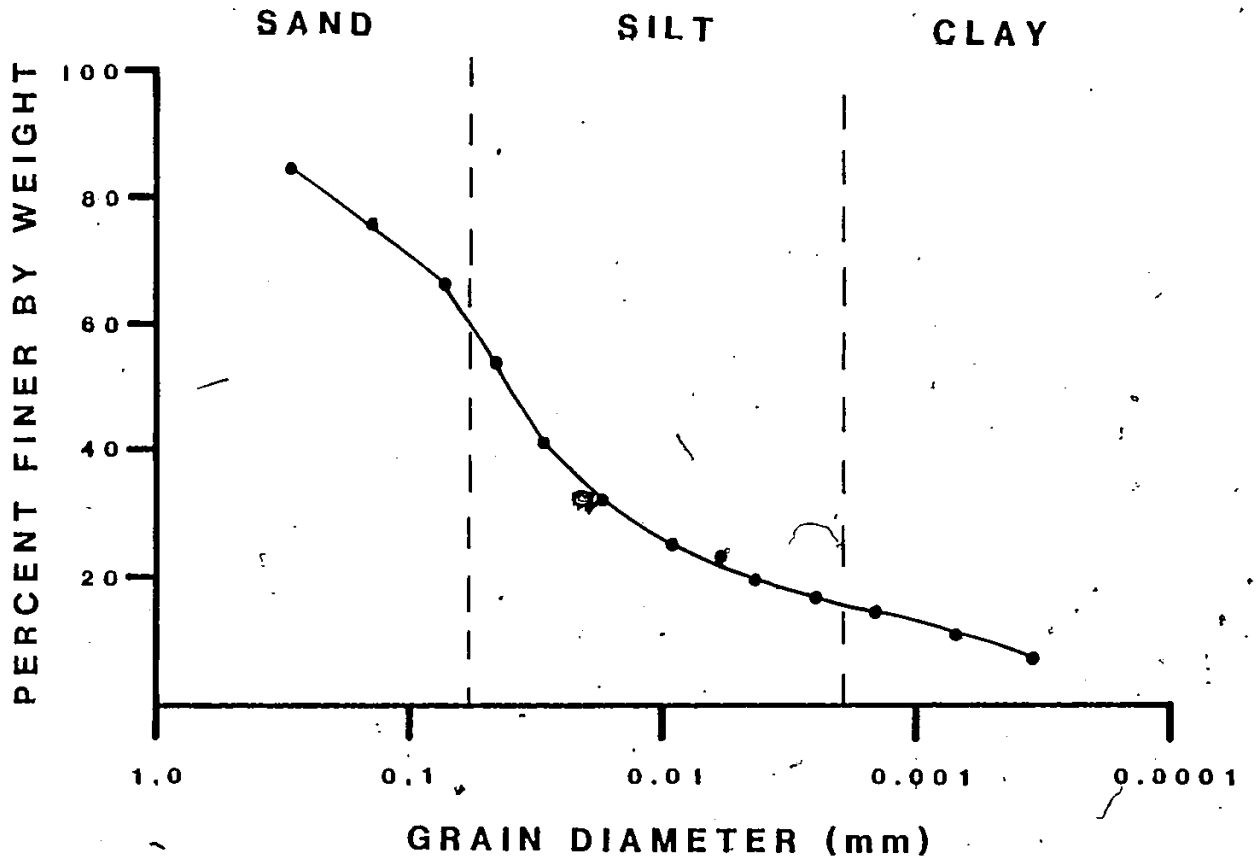
Sample SH-17


SAND
SILT
CLAY
PERCENT FINER BY WEIGHT
100
80
60
40
20
1.0
0.1
0.01
0.001
0.0001
GRAIN DIAMETER (mm)

Sample DU-18


SAND
SILT
CLAY
PERCENT FINER BY WEIGHT
100
80
60
40
20
1.0
0.1
0.01
0.001
0.0001
GRAIN DIAMETER (mm)

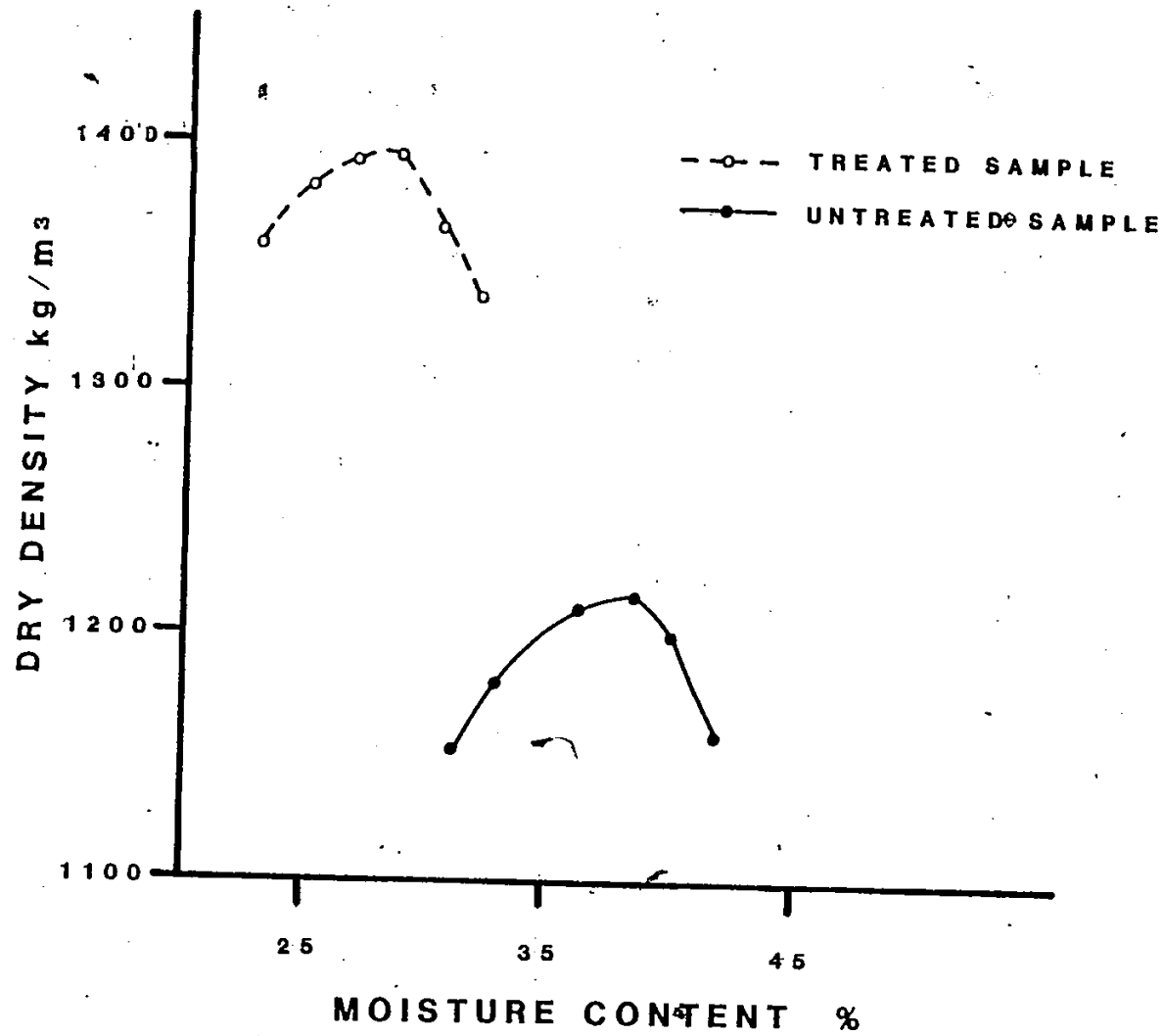
Sample SC-19


SAND
SILT
CLAY
PERCENT FINER BY WEIGHT
100
80
60
40
20
1.0
0.1
0.01
0.001
0.0001
GRAIN DIAMETER (mm)

APPENDIX C

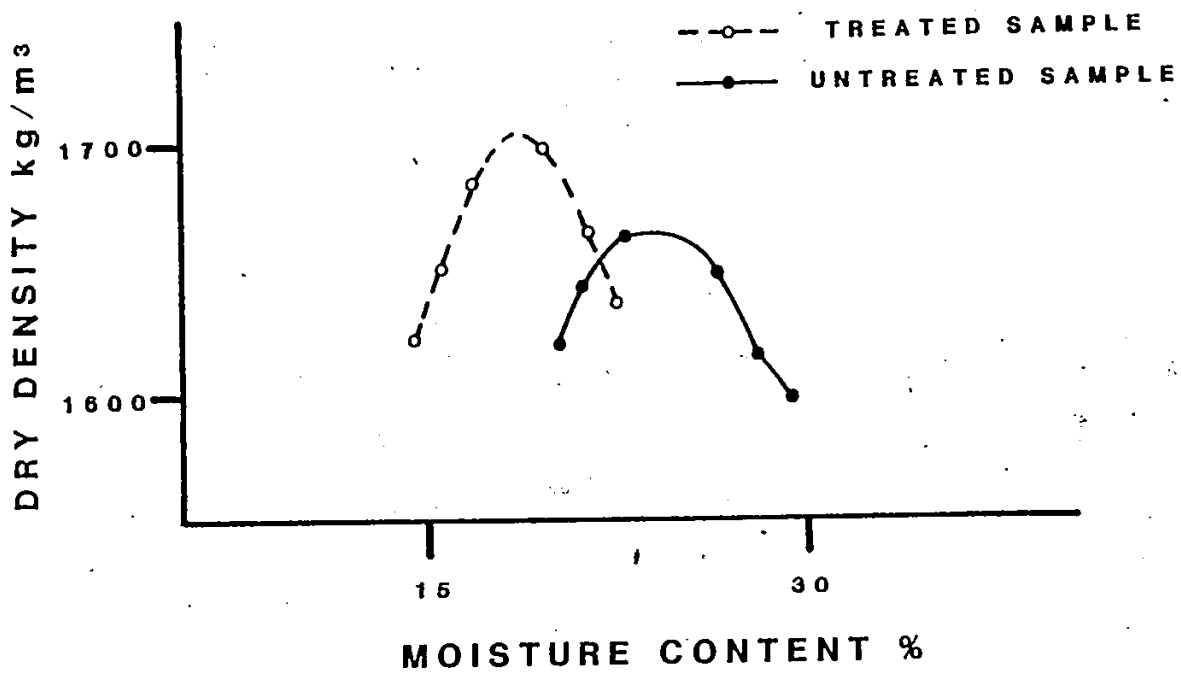
Dry Density - Moisture Content Curves

Sample KN-1

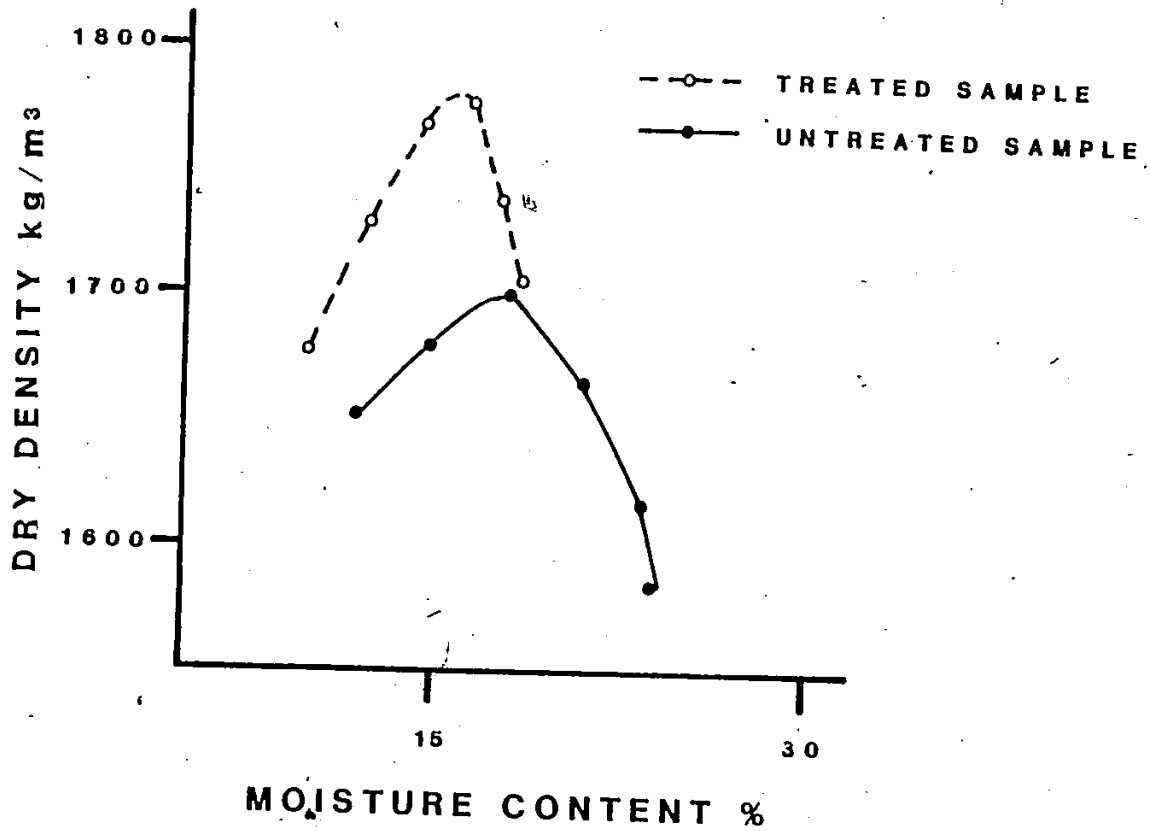

TREATED SAMPLE
UNTREATED SAMPLE
DRY DENSITY kg/m3
1400
1300
1200
1100
25
35
45
MOISTURE CONTENT %

6

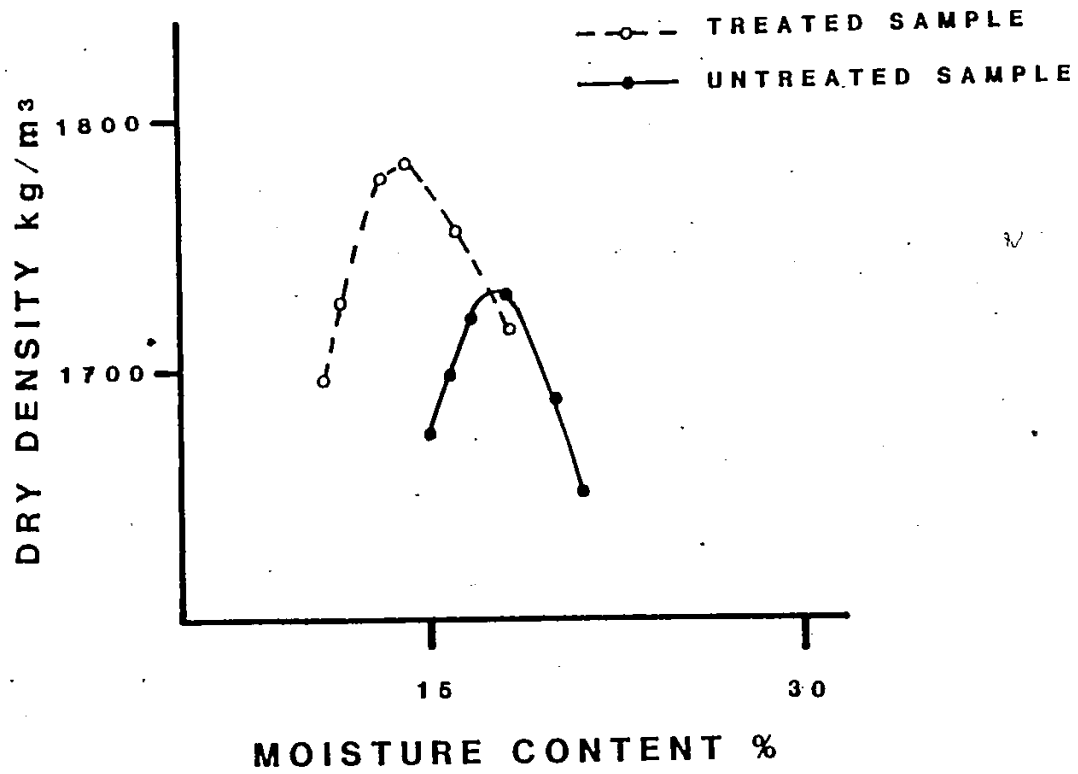
Sample BR-2


TREATED SAMPLE
UNTREATED SAMPLE
DRY DENSITY kg/m3
1700
1600
15
30
MOISTURE CONTENT %

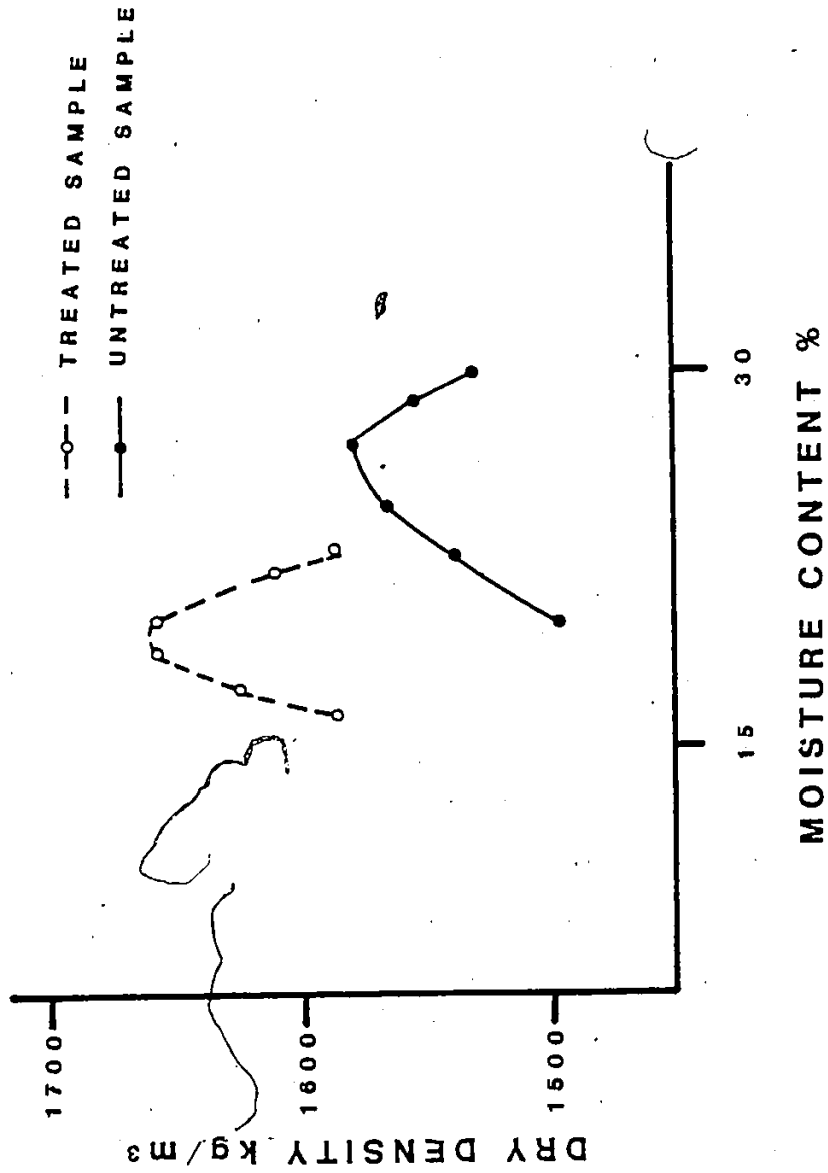
Sample AU-3


1800
1700
1600
DRY DENSITY kg/m3
TREATED SAMPLE
UNTREATED SAMPLE
15
30
MOISTURE CONTENT %

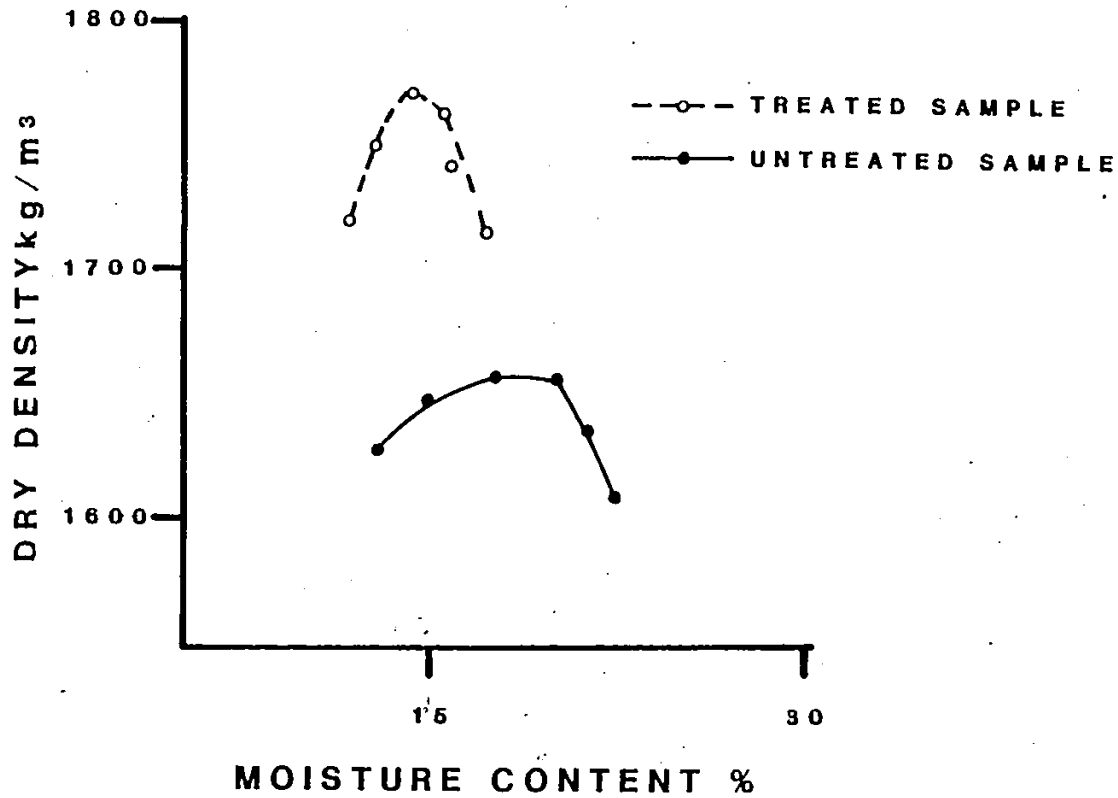
Sample HG-4


TREATED SAMPLE
UNTREATED SAMPLE
DRY DENSITY kg/m3
1800
1700
15
30
MOISTURE CONTENT %

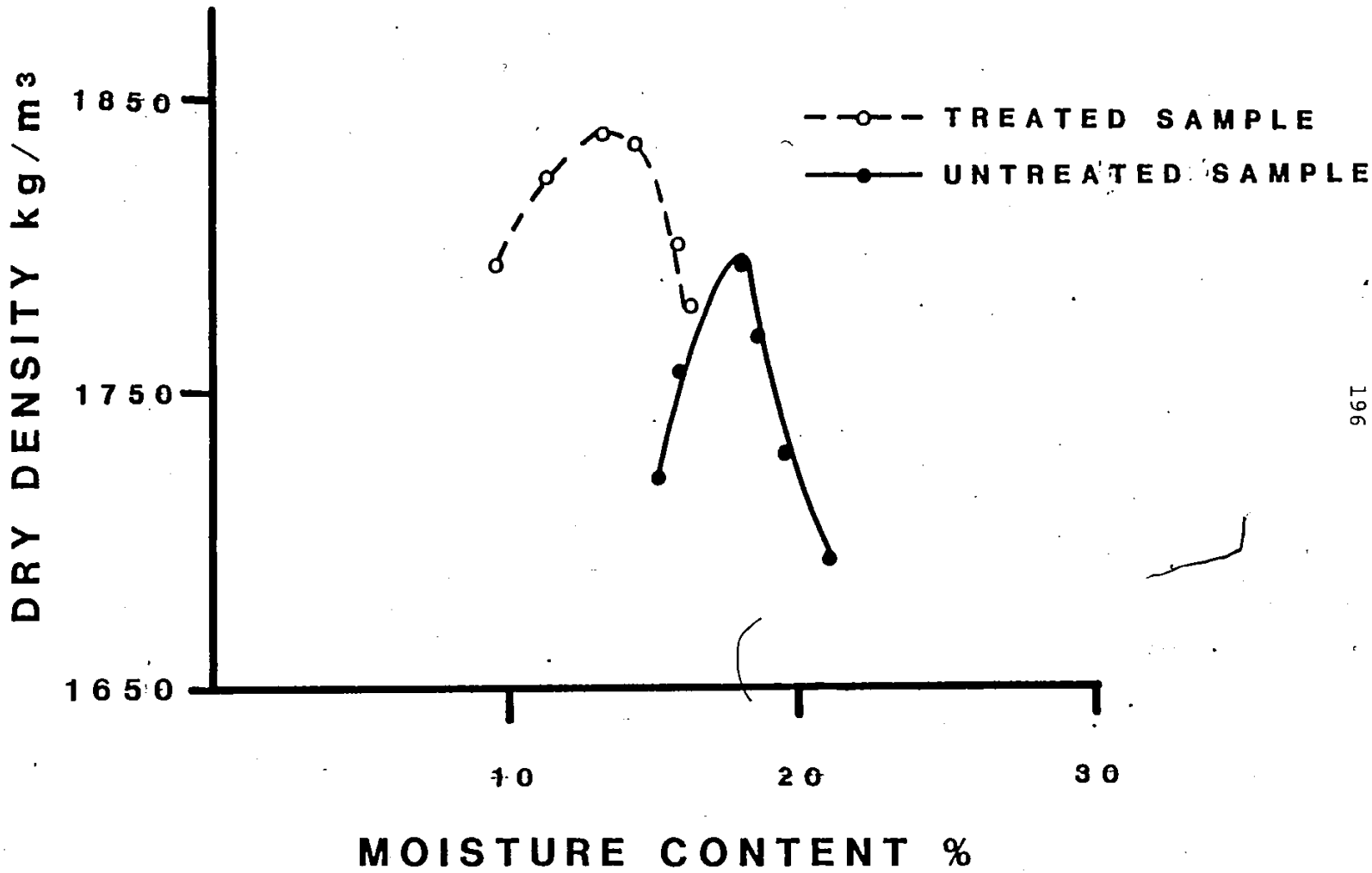
Sample SC-5


TREATED SAMPLE
UNTREATED SAMPLE
1700
1600
1500
DRY DENSITY kg/m3
15
30
MOISTURE CONTENT %

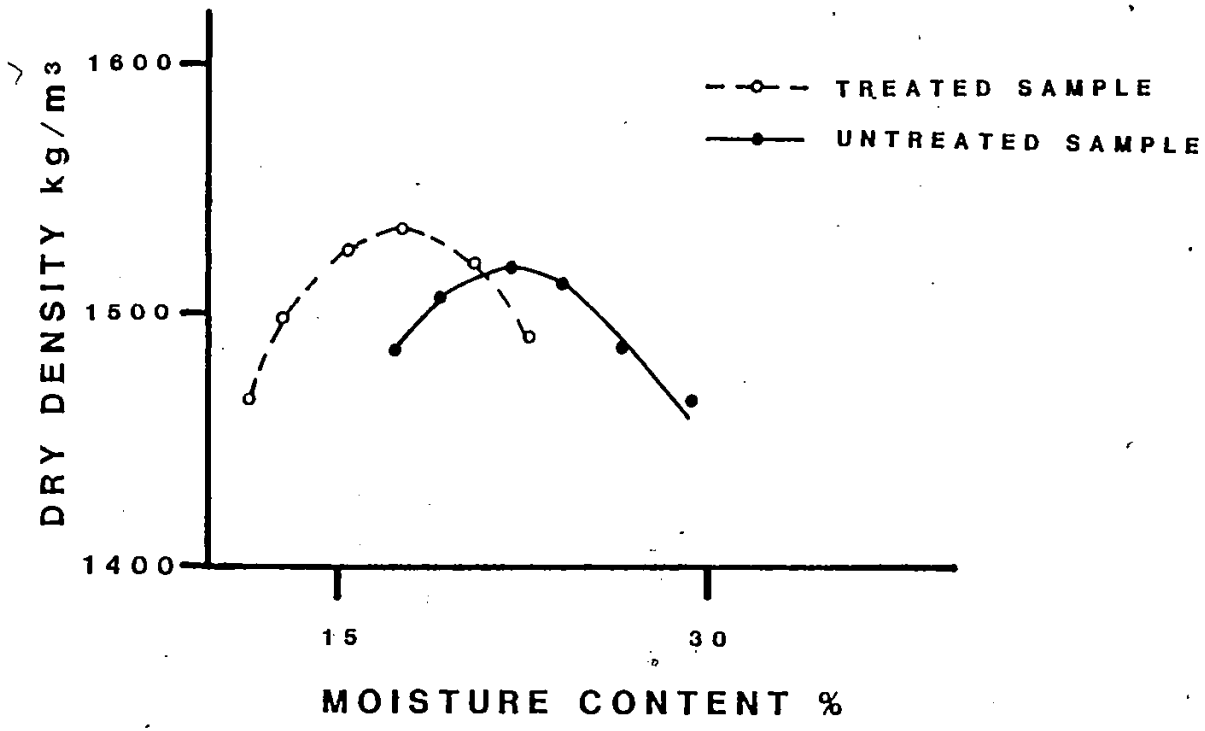
Sample BE-6


1800
1700
1600
DRY DENSITY kg/m3
TREATED SAMPLE
UNTREATED SAMPLE
15
30
MOISTURE CONTENT %

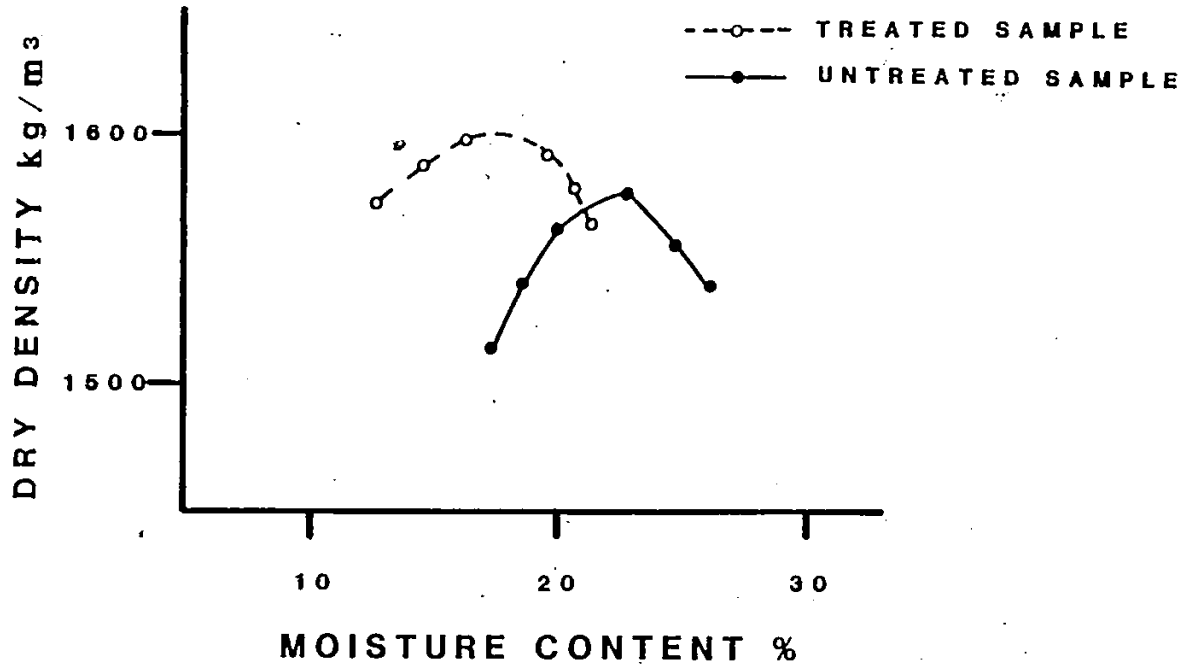
Sample SE-7


DRY DENSITY kg/m3
1850
1750
1650
10
20
30
MOISTURE CONTENT %
TREATED SAMPLE
UNTREATED SAMPLE

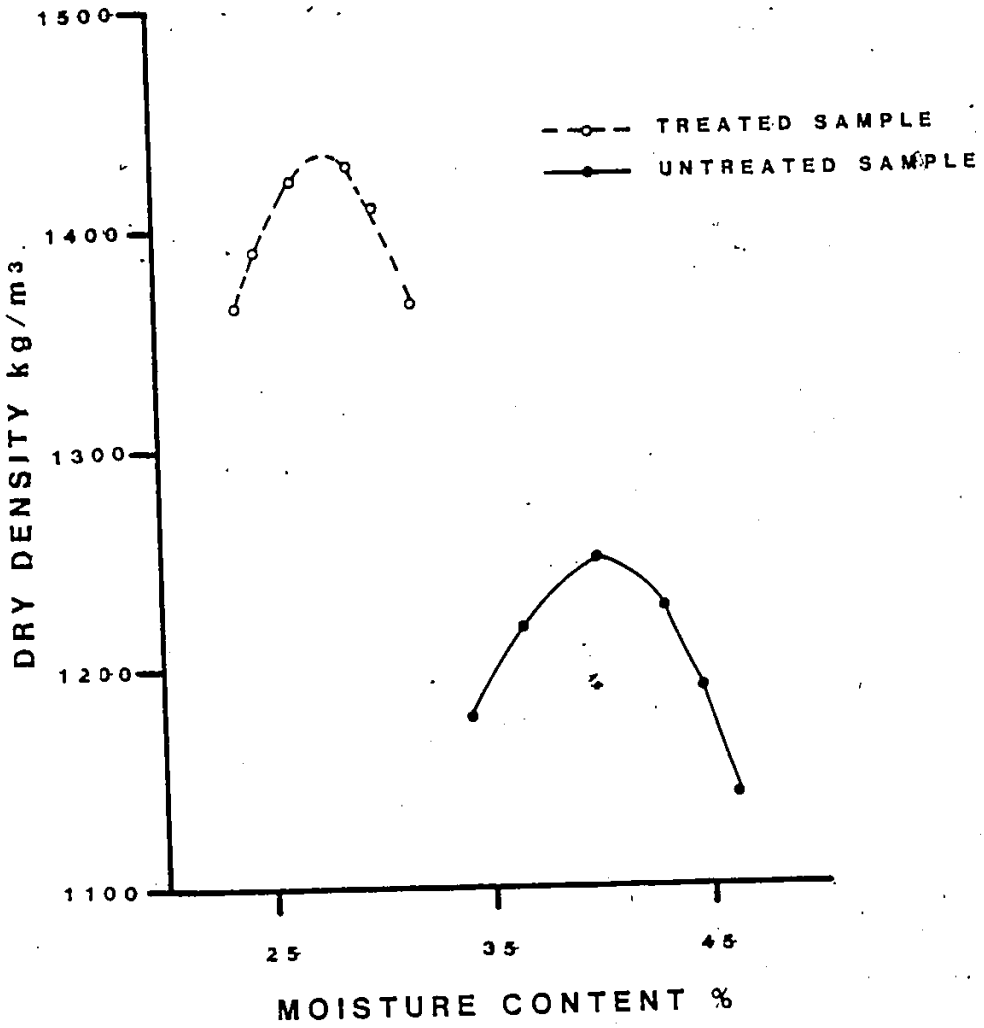
Sample SJ-8


DRY DENSITY kg/m3
1600
1500
1400
15
30
MOISTURE CONTENT %
TREATED SAMPLE
UNTREATED SAMPLE

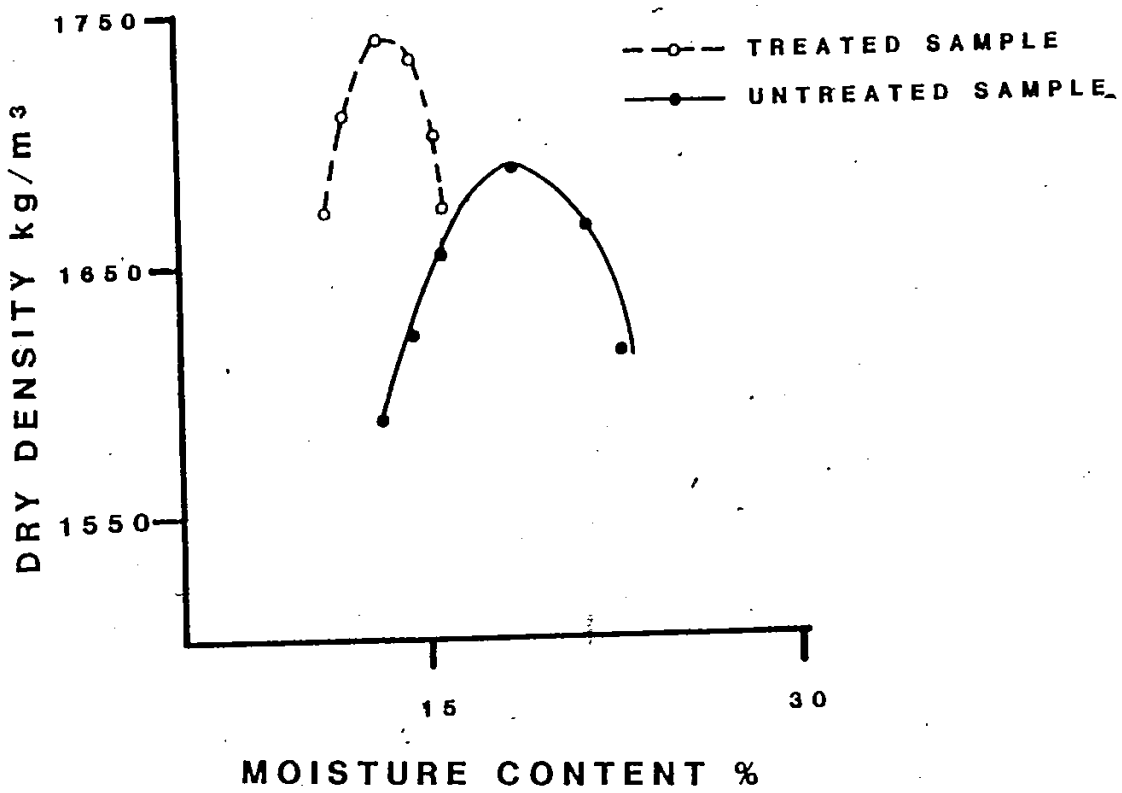
Sample SH-9


TREATED SAMPLE
UNTREATED SAMPLE
DRY DENSITY kg/m3
1600
1500
10
20
30
MOISTURE CONTENT %

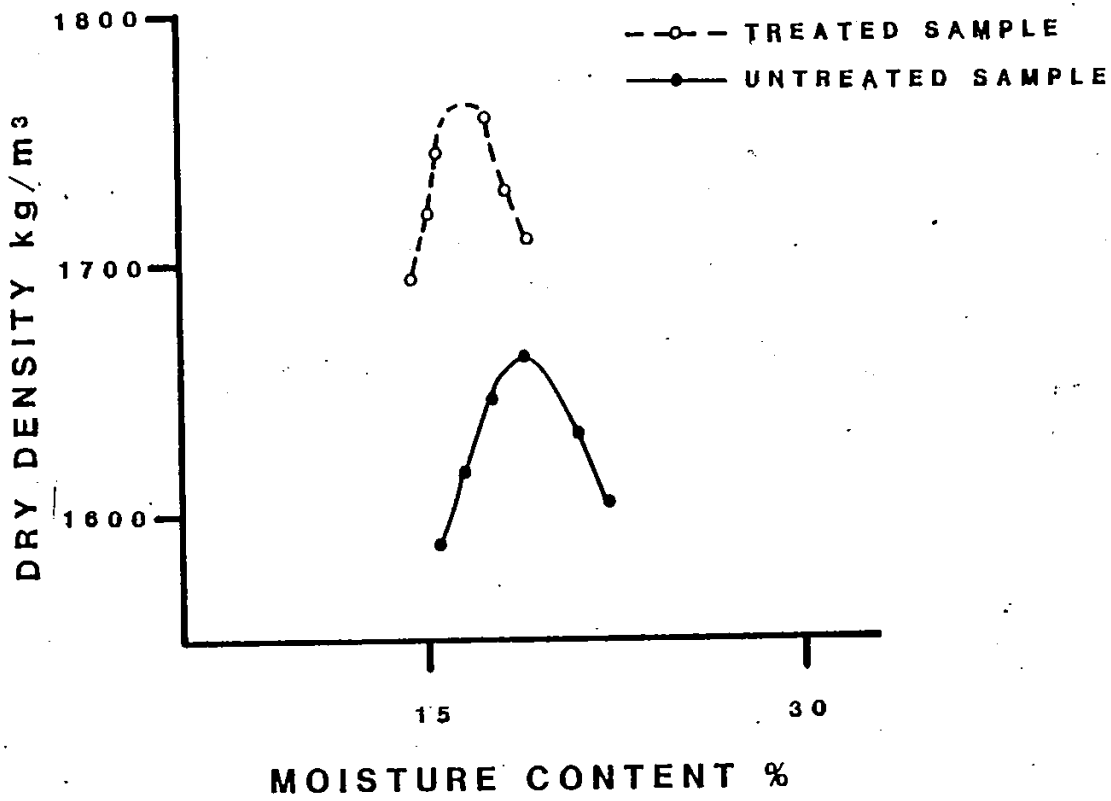
Sample KN-10


1500
1400
1300
1200
1100
DRY DENSITY kg/m3
TREATED SAMPLE
UNTREATED SAMPLE
25
35
45
MOISTURE CONTENT %

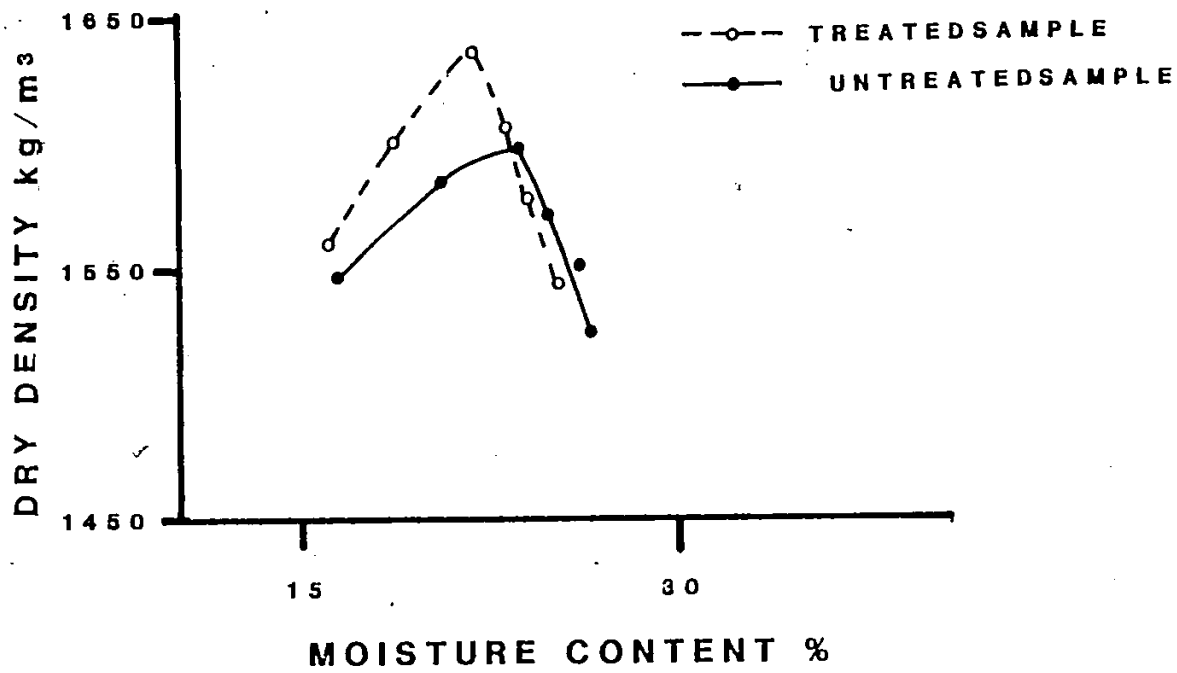
Sample BR-11


1750
1650
1550
DRY DENSITY kg/m3
TREATED SAMPLE
UNTREATED SAMPLE
15
30
MOISTURE CONTENT %

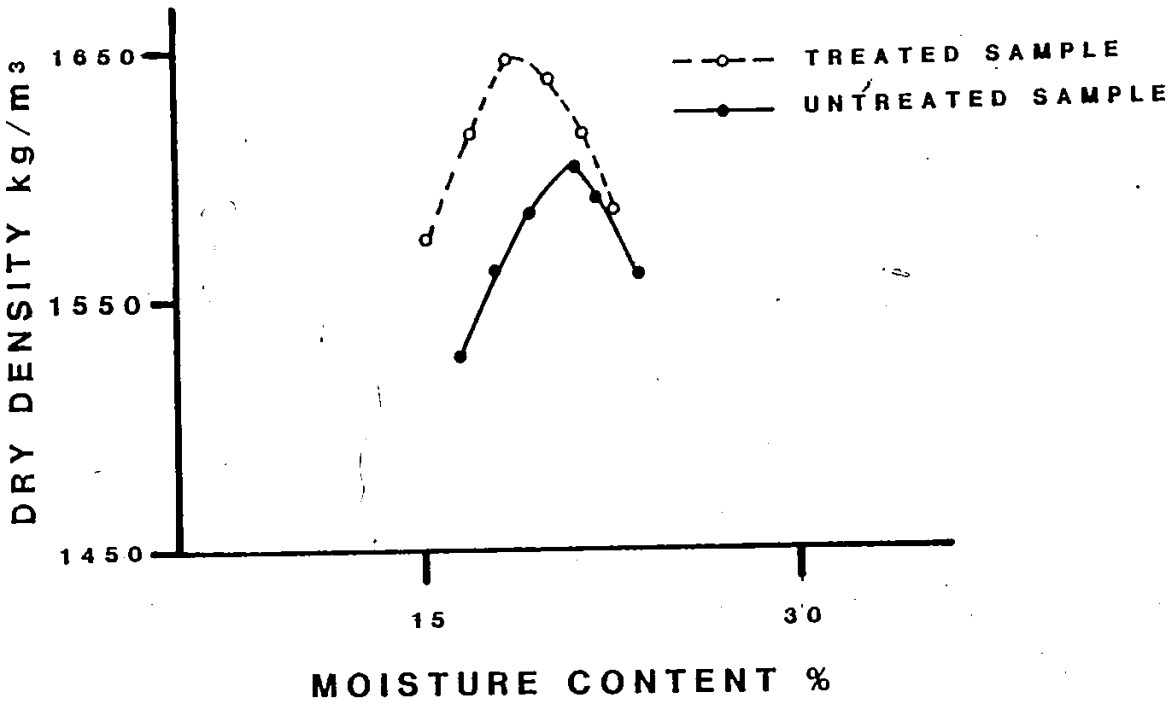
Sample AU-12


TREATED SAMPLE
UNTREATED SAMPLE
1800
1700
1600
DRY DENSITY kg/m3
15
30
MOISTURE CONTENT %

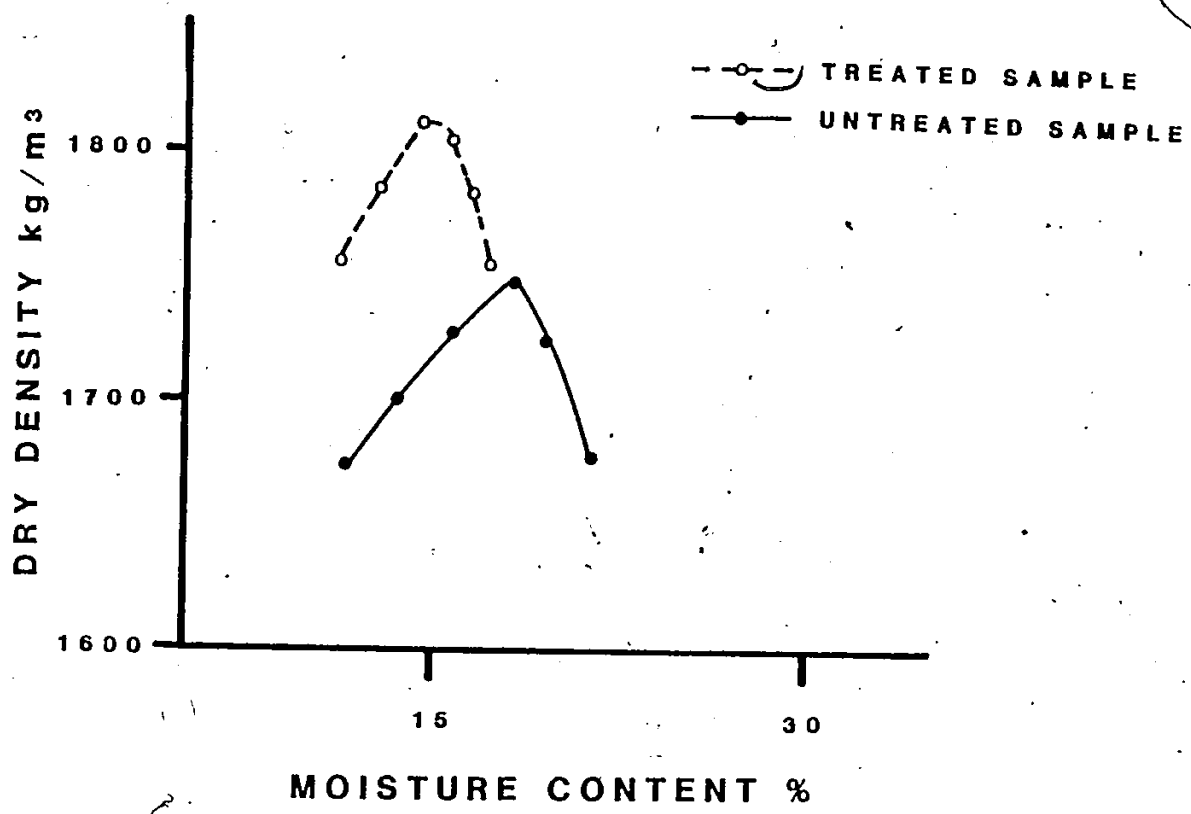
Sample HG-13


1650
1550
1450
DRY DENSITY kg/m3
15
30
MOISTURE CONTENT %
TREATEDSAMPLE
UNTREATEDSAMPLE

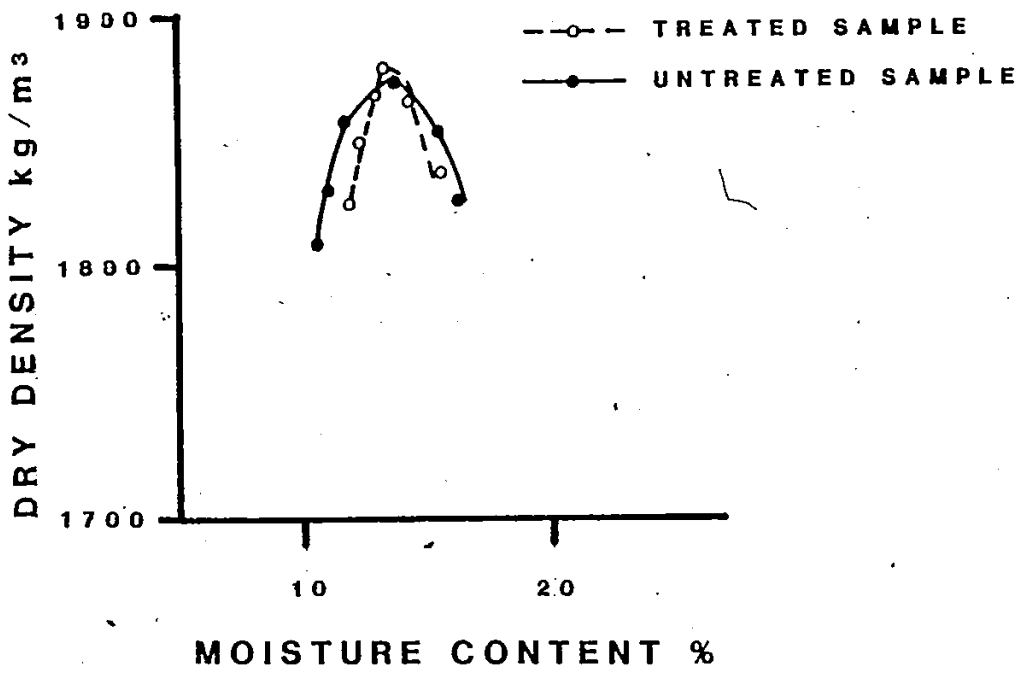
Sample BE-14


DRY DENSITY kg/m3
1650
1550
1450
15
30
MOISTURE CONTENT %
TREATED SAMPLE
UNTREATED SAMPLE

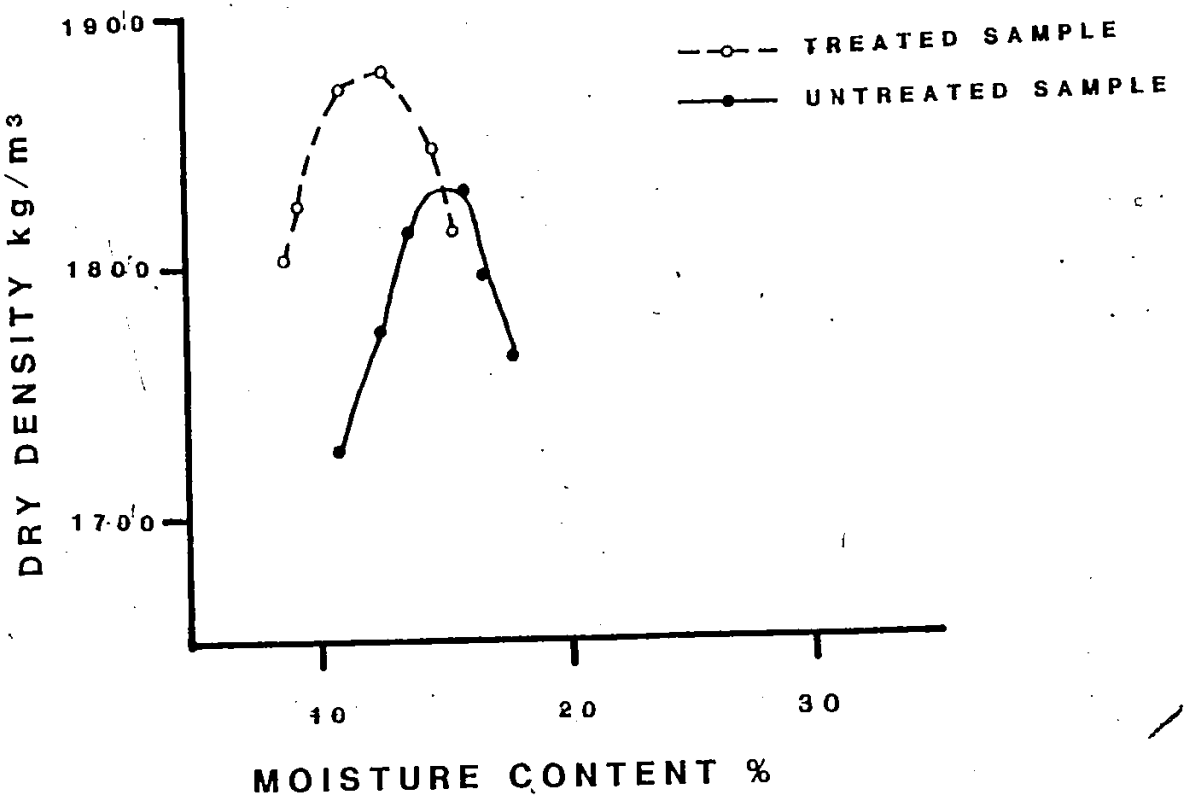
Sample SE-15


TREATED SAMPLE
UNTREATED SAMPLE
DRY DENSITY kg/m3
1800
1700
1600
15
30
MOISTURE CONTENT %

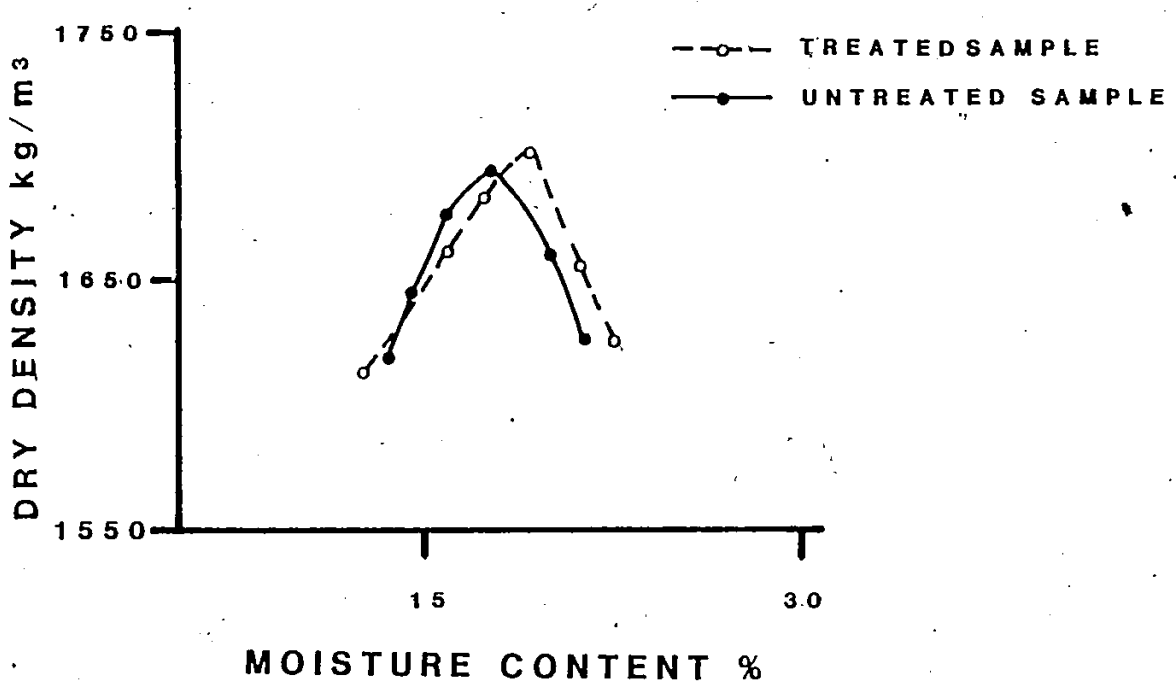
Sample SJ-16


TREATED SAMPLE
UNTREATED SAMPLE
DRY DENSITY kg/m3
1900
1800
1700
10
20
MOISTURE CONTENT %

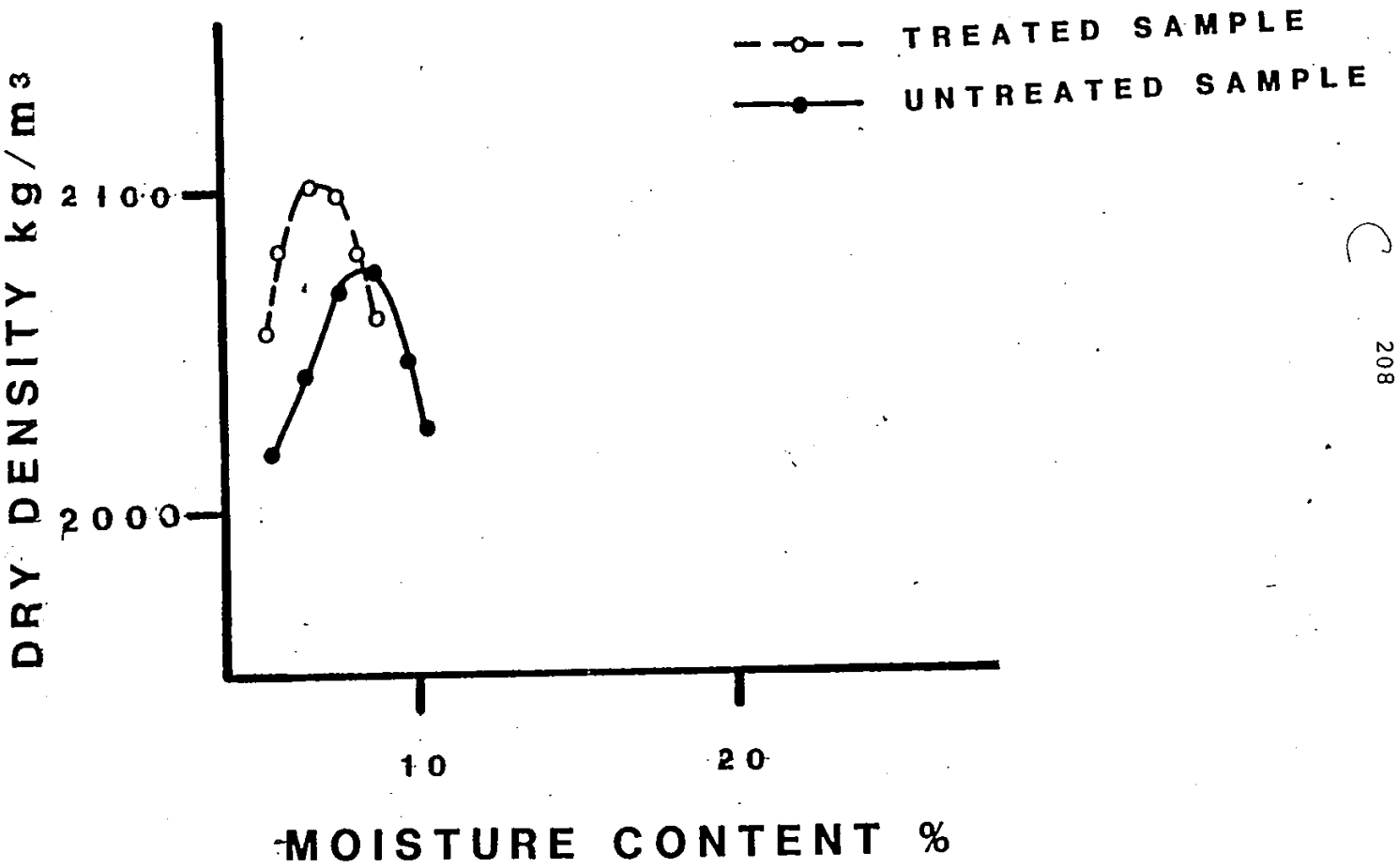
Sample SH-17


TREATED SAMPLE
UNTREATED SAMPLE
1900
1800
1700
DRY DENSITY kg/m³
10
20
30
MOISTURE CONTENT %

Sample DU-18


TREATED SAMPLE
UNTREATED SAMPLE
DRY DENSITY kg/m3
1750
1650
1550
15
3.0
MOISTURE CONTENT %

Sample SC-19


TREATED SAMPLE
UNTREATED SAMPLE
DRY DENSITY kg/m3
2100
2000
10
20
MOISTURE CONTENT %

APPENDIX D

Correlation and Regression Analysis
and Comparison of Variances and means.

Table D.1

Correlation and Regression Analysis

Figure	Variables X, Y	Sample Size N	Correlation Coefficient r	Student's t-value	Probability (Confidence) Level)
5.3	Liquid limit (untreated) Plasticity index (untreated)	18	0.9774	18.49	0.0001
5.4	Liquid limited (treated) Plasticity index (treated)	18	0.8771	7.30	0.0001
5.5	Clay fraction (<0.002 mm) Plasticity index (untreated)	18	0.8301	5.96	0.0001
5.6	Clay fraction (<0.005 mm) Plasticity index (untreated)	18	0.5748	2.81	0.0500
5.7	Water adsorption (untreated) Liquid limited (untreated)	18	0.9496	12.12	0.0001
5.8	Water adsorption (untreated) Plasticity index (untreated)	18	0.9514	12.35	0.0001
5.9	Water adsorption (untreated) Plastic limit (untreated)	18	0.8265	5.87	0.0001
5.10	Water adsorption (treated) Liquid limit (treated)	18	0.7424	4.43	0.0005
5.11	Water adsorption (treated) Plasticity index (treated)	18	0.7635	4.73	0.0005
5.12	Water adsorption (treated) Plastic limit (treated)	18	0.3658	2.33	0.5000
5.13	Clay fraction (<0.002 mm) Water adsorption (untreated)	19	0.8469	6.57	0.0001
5.14	Optimum moisture content (untreated) Optimum moisture content (treated)	19	0.9186	9.58	0.0001
5.15	Max. dry density (untreated) Max. dry density (treated)	19	0.9696	16.34	0.0001
5.16	Log perm. to water (untreated)	18	0.8093	5.51	0.0001
5.17	Log perm. to water (untreated) Log perm. to water (treated)	18	0.7507	4.55	0.0005
5.18	Log perm. to water (untreated) Log perm. to brine (untreated)	18	0.5685	2.76	0.0050

$t_{16,0.01} = 2.921$; $t_{16,0.05} = 2.120$; $t_{16,0.3} = 0.690$; $t_{17,0.01} = 2.898$;

$t_{17,0.05} = 2.110$; $t_{17,0.5} = 0.689$

Probabilities are two-tailed. Perm. = Permeability.

Table D.2

Comparison of Variances and Means

Characteristics	Sample Size	Mean	Range	Standard Deviation	Coefficient of Variation	F-Value	Student's t-value	Probability (confidence level)	Population Comparison
Liquid limit (untreated)	18	39.67	67	17.52	44.18	3.75	2.19	0.05	different
Liquid limit (treated)	18	29.50	35	9.05	30.68				
Plastic limit (untreated)	18	22.28	23	6.34	28.46	1.95	1.87	0.05	same
Plastic limit (treated)	18	18.83	18	4.54	24.11				
Plasticity index (untreated)	18	17.39	45	11.98	68.90	3.27	2.08	0.05	different
Plasticity index (treated)	18	10.67	28	6.62	62.10				
Water adsorption 98% RH (untreated)	19	22.67	33.4	8.22	36.25	1.68	3.94	0.01	different
Water adsorption 98% RH (treated)	19	34.83	33.1	10.67	30.63				
Perm. to water (untreated)	18	12.07×10^{-8}	120.66×10^{-8}	28.01×10^{-8}	232.06	320.19	3.05	0.01	different
Perm. to brine (untreated)	18	3.73×10^{-6}	19.31×10^{-6}	5.01×10^{-6}	134.47				
Perm. to water (untreated)	18	12.07×10^{-8}	120.66×10^{-8}	28.01×10^{-8}	232.06	1.38	0.07	very high	same
Perm. to brine (treated)	18	12.63×10^{-8}	99.82×10^{-8}	23.83×10^{-8}	188.59				
Perm. to brine (treated)	18	12.63×10^{-8}	99.82×10^{-8}	23.83×10^{-8}	188.59	3.29	0.45	very high	same
Perm. to water (treated)	18	8.55×10^{-8}	50.76×10^{-8}	13.13×10^{-8}	153.60				

$F_{17,17,0.01}=3.24$; $F_{17,17,0.05}=2.28$; $t_{34,0.01}=2.732$; $t_{34,0.05}=2.03$; $t_{36,0.01}=2.72$

Perm. = Permeability.

Testing the Difference of Variances and Means

A. Testing the equality of variances

Given: Standard deviations of two sample distributions to test whether the populations from which the samples were drawn have the same variances, set up the hypothesis:

Null: H_o: $\sigma_1^2 = \sigma_2^2$

Alternate: H_1: $\sigma_1^2 \neq \sigma_2^2$

The F-test then is

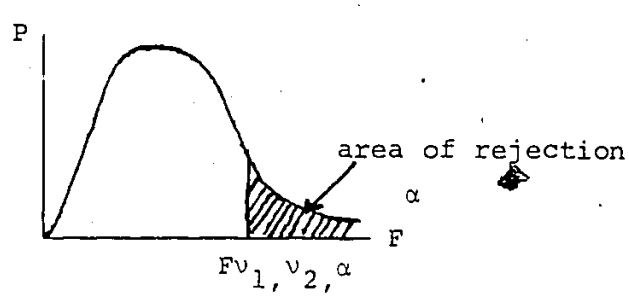
$$F = \frac{S_1^2}{S_2^2}$$

where S = standard deviation

Decision rule:

$$\text{Reject } H_o \text{ if } F = \frac{S_1^2}{S_2^2} > F_{\nu 1,\ \nu 2,\ \alpha}$$

where $\nu_1 = N_1 - 1$; $\nu_2 = N_2 - 1$, the number of degrees of freedom, and α is the level of significance or probability of rejecting H_o if it is true.



If $F = \frac{S_1^2}{S_2^2} > F\nu_1, \nu_2, \alpha$ it follows that $\sigma_1^2 \neq \sigma_2^2$.

That is, there is statistical evidence to suggest that the variances are different.

B. <u>Testing the difference of means</u>

Set up Null H_o: $\mu_1 = \mu_2$

Alternate H_1: $\mu_1 \neq \mu_2$

Since $S_1^2 \neq S_2^2$, from the result of the F-test above, calculate a 'pooled variance' defined by

$$S_p^2 = \frac{\nu_1 S_1^2 + \nu_2 S_2^2}{\nu_1 + \nu_2}$$

where

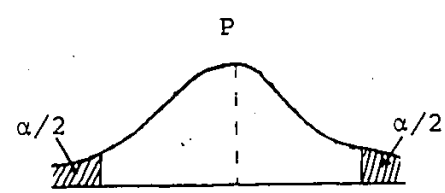
$\nu_1 = N_1 - 1$

$\nu_2 = N_2 - 1$

N is the sample size

t-statistic to be used is then given by

$$t = \frac{\overline{X}_1 - \overline{X}_2}{S_p \sqrt{\frac{1}{n} + \frac{1}{n_2}}}$$



$$P\left(-t_{\nu,\alpha} < \frac{\overline{X}_1 - \overline{X}_2}{S_p\sqrt{\frac{1}{n_1} + \frac{1}{n_2}}} < t_{\nu,\alpha} \middle| \mu_1 = \mu_2 \right) = 1 - \alpha$$

Decision rule:

Accept H_o if $\left| \frac{X_1 - X_2}{S_p\sqrt{\frac{1}{n_1} + \frac{1}{n_2}}} \right| < t_{\nu,\alpha}$

otherwise reject H_o

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