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THE INFLUENCE OF EXCHANGEABLE IONS AND THEIR
CONCENTRATION IN THE PORE FLUID ON PLASTIC
AND STRENGTH PROPERTIES OF COHESIVE SOIL

by

Mahmoud H. Abd-el-Aziz

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CONCENTRATION IN THE PORE FLUID ON PLASTIC
AND STRENGTH PROPERTIES OF COHESIVE SOIL

by

Mahmoud H. Abd-el-Aziz

A dissertation submitted in partial fulfillment
of the requirements for the degree

of

DOCTOR OF PHILOSOPHY

in

Irrigation and Drainage Engineering

Approved:

Thesis Director

Major Professor

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1965

ABSTRACT

ABD-EL-AZIZ, MAHMOUD H.; 1933; The Influence of Exchangeable Ions and Their Concentration in the Pore Fluid on Plastic and Strength Properties of Cohesive Soil; Department of Civil Engineering; Dr. A. A. Bishop, major professor.

A study was conducted to find out both the effect of different exchangeable cations and the concentration of ions in the soil pore fluid on strength and plastic properties of a sample of cohesive soil taken from the bank of an open drain ditch at the Utah State University Irrigation and Drainage Farm.

The experimental results of this study may be summarized by the following points:

1. A general decrease of liquid limit and an increase in both shear strength and modulus of elasticity, with increasing salt concentration in the pore fluid, were observed for all samples, with the exception of HCl-treated samples for which shear strength was independent of HCl concentration in the pore water.

2. An increase in both shear strength and modulus of elasticity with decreasing exchangeable sodium percentage was definite.

3. The increase of soil shear strength caused by different exchangeable cations with distilled water in the soil pores was in the order $\text{Na}^+ < \text{Ca}^{++} < \text{Mg}^{++} < \text{K}^+ < \text{HCl-treated soil}$, whereas the modulus of elasticity increased in order $\text{Na}^+ < \text{K}^+ < \text{Ca}^{++} < \text{HCl-treated soil} < \text{Mg}^{++}$.

4. A decrease in the liquid limit with decreasing exchangeable sodium was found.

5. The influence of the different exchangeable cations used on soil liquid limit was in the order $\text{Na}^+ > \text{HCl-treated sample} > \text{Mg}^{++} > \text{Ca}^{++} > \text{K}^+$.

From this study it could be concluded that:

1. The strength and plastic properties of cohesive soils are affected greatly by the type and concentration of adsorbed ions as well as by the ions concentration in the pore fluid.

2. The failure of the sloping sides of drain ditches may be attributed at least in part to increasing exchangeable sodium percentage.

3. For satisfactory and economical design of earth structures, a knowledge of chemical as well as physical properties of cohesive soils is essential. The boundary conditions which may change the soil physico-chemical properties with time should be considered.

4. Strength properties of HCl-treated soils are controlled by aluminum rather than hydrogen ions. Exchangeable aluminum was liberated as a result of the breakdown of the clay fraction in soils when treated with HCl.

5. The experimental results are not completely explained in terms of the Gouy-Chapman theory of double layers. However, the Stern theory which considers the specific adsorption of ions on clay surfaces is a better alternative and does explain the results adequately.

ACKNOWLEDGMENT

I am indebted to Dr. Irving Dunn for guidance during this study, for reading the dissertation, and making many helpful suggestions. Sincere appreciation is due Dr. A. Bishop for his guidance of my academic program. Appreciation is also expressed to Dr. S. A. Taylor, Dr. G. Watters, and Professor F. Keifer for acting as advisers and graduate committee members.

The Egyptian Government and the Desert Institute in particular are gratefully acknowledged for the financial support necessary for the completion of my second Ph. D. degree.

I am deeply grateful for the encouragement and support rendered by my wife, Yosria, during five years of challenge and sacrifice. She has done a most admirable and unselfish job of being mother and substituting as the father of our children when I could dedicate hardly any time to them during the period of my studies.

Mahmoud Hassan Abd-el Aziz

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INTRODUCTION

At all locations where the ground surface stands at an inclination with the horizontal, gravitational forces tend to cause movement of soil particles from top to bottom of the slope. As a result of this movement, alteration of the slope may take place gradually or suddenly in the form of landslides. The force of gravity is aided by seepage of water through the slope which reduces the resistance to mass movement. Only when the resistance to movement is great enough to withstand the downward moving forces will the slope remain stable.

In the design of embankments, earth dams, and other earth structures, the most frequent and most serious problems which the soil engineer faces are those of stability; i. e., those requiring knowledge of the shear strength of the soil. A great amount of research concerning soil stability and measurements of shear strength of undisturbed soil samples has therefore been undertaken during the past few years. Much of this work, however, was largely concerned with the size and arrangement of the different particles rather than with the chemical and physical characteristics of the soil. The elements of shear strength of complex compacted cohesive soils (constituting the main portion of earth structures) are more complicated than for the simple cohesionless soils. Since cohesive soils have a large specific surface, their behavior may be influenced considerably by surface forces. These forces are controlled

by the physico-chemical properties of the soil and may undergo gradual change under particular conditions.

Several researchers (Berger, 1951; Marsland, 1957; Casagrande, 1959) called attention to the fact that some slopes have failed when conventional analyses have indicated that they should be stable. Casagrande (1959) stated that it is doubtful if progressive failure is the cause of the discrepancy, and further suggested that a reduction of soil strength with time may be the reason. The reduction in strength, the writer thinks, may be due to a change in the physico-chemical properties of the soil.

For satisfactory predictions of soil stability, and for a better understanding of the soil-forming materials, a knowledge of the physico-chemical characteristics of cohesive soils might be useful. It seems to be generally agreed by many investigators (Rosenquist, 1957, 1959; Lambe, 1960b; Michaels, 1959) that the strength of cohesive soils is a function of the interparticle forces. These, in turn, are affected by several variables in the soil-water-electrolyte system.

Stability of slopes in open drains is a special problem in which the physical and chemical properties of both soil material and drainage water should be of major importance. The influence of exchangeable ions may have greater effect on the stability of drainage ditch banks than is commonly realized. The problem of bank sloughing that frequently occurs in open drains may be aggravated by a reduction in strength could be a result of the continuous process of replacing the

ions on the clay fraction of the soil by that in the drainage water.

It is the purpose of this investigation to find out how much the stability (strength properties) of an embankment made from a certain cohesive soil will be affected by the type and concentration of exchangeable ions and their concentration in the pore fluid.

REVIEW OF LITERATURE

There are two main approaches to an understanding of the strength properties of cohesive soils. The first of these, and the one to which the major efforts have been devoted, is the examination of soil shearing strength. The second approach concentrates on the study of the aspects of interparticle behavior relating the macroscopic strength characteristics of the soil to the chemical and physical properties of its constituents including the pore fluid.

To obtain insight into the complexities of the shearing process in cohesive soils, it may be revealing to examine the nature of the material under consideration both physically and chemically from the microscopic point of view.

Nature and Structure of Cohesive Soils

A cohesive soil consists of an accumulation of particles ranging from larger granular constituents to the much smaller size particles; i. e., clay. It is now known in soil mechanics that clay particles are usually of a size less than two microns. The individual clay particles are, in general, sheet mineral crystal, made up of various combinations of silica, gibbsite, and brucite type structures. Their lateral surfaces are oxides or hydroxides. The crystal structures of the common clay minerals have been illustrated in the literature (Grim, 1953). Clay

particle surfaces generally possess a net negative electrical charge. This net negative charge is balanced by adsorption of cations at the surfaces of the particles. These ions are readily exchangeable with other ions (Grim, 1953). The edges of the particles, on the other hand, may be electrically positive, negative, or neutral, depending on the nature of the mineral and the environment with which it is in contact (Seed, et al., 1960).

A soil in its natural state may have single-grained or compound structure. In the single-grained structure, each particle acts independently of all others and is supported by contact with several other particles. In the compound structure, large voids are enclosed in a skeleton of arches, either of individual fine grains forming a honeycomb structure, or of colloidal sized particles aggregated into chains or rings, making a granular structure (Casagrande, 1932). Compound structure is the result of interacting particles which are small enough to exhibit appreciable surface activity.

Lambe (1953, 1960a) has suggested that clays which were flocculated during sedimentation or compacted at water contents less than optimum develop a random orientation of the clay particles called a "card-house" type structure. Conversely, clays which were dispersed during sedimentation or compacted at a water content greater than the optimum develop a parallel orientation of the particles. Possible variations from these general rules are discussed by Seed and Chan (1959b), who also found that although the structure may have a pronounced

influence on the deformation characteristics of compacted clays, it has relatively little influence on the maximum strength because a reorientation of the particle in the failure zone may take place during a shear or a triaxial test.

Rosenquist (1959) has developed techniques for obtaining stereographs of the structure of undisturbed clay by means of an electron microscope. He found that the structure of marine clays in general is a random orientation of the particles and resembles the card-house structure suggested by Tan (1957) and Lambe (1960a). Salas and Seratosa (1953) have used x-ray diffraction to investigate the relative orientation of clay. They found that the particles in remolded and uniaxially reconsolidated clays are oriented perpendicularly to the direction of the principal consolidating stress. Mitchell (1956) also investigated the structure of many undisturbed clays. He found that some clays have a completely random orientation of the particles and that others have various degrees of a preferred particles orientation. The degree of parallel orientation of the particles is uniform in some clays; in others this degree of orientation varies from point to point or zone to zone.

Stresses

Soil as an engineering material may be visualized as a compressible skeleton of solid particles enclosing voids, which in saturated soils are filled with water and in partially saturated soil with both water and air.

Consider the soil skeleton as an ideal homogeneous isotropic material; stress at a point is a force per unit area on a plane surface containing the point (Gibbs, et al., 1960). For each of the infinite number of planes containing the point, the stress can be resolved into a component stress at right angles to the plane, i. e., normal stress; and a component stress parallel to the plane, i. e., shear stress (Taylor, 1962, p. 315). The normal and shear stresses are symbolically referred to as σ and τ respectively. The equilibrium equation for the stresses on the soil skeleton are (Taylor, 1962, p. 316)

$$\sigma = \sigma_1 \cos^2 \theta + \sigma_3 \sin^2 \theta \quad 1$$

$$\tau = (\sigma_1 - \sigma_3) \sin \theta \cos \theta \quad 2$$

where σ_1 and σ_3 are the major and minor principal stresses at a point respectively, and θ is the angle between the planes on which τ and σ_1 act. Under the action of stresses, the soil skeleton undergoes elastic deformation, and its structure is altered by particle rearrangement.

Shear stresses are carried only by the skeleton of solid particles except at very high strain rates (Bishop, et al., 1960). On the other hand, the normal stress at any plane is in general the sum of two components, the stress carried by the solid particles $\bar{\sigma}$ and the pressure of the fluid in the void space u (Terzaghi, 1943, p. 268). For saturated soils, the effective normal stress $\bar{\sigma}$ is expressed mathematically as

(Terzaghi, 1943, p. 12)

$$\bar{\sigma} = \sigma - u \quad \dots \dots \dots \quad 3$$

where σ denotes the total normal stress. The case of partially saturated soils, however, calls for special treatment since the pore space contains two fluids, air and water. An expression for effective stress $\bar{\sigma}$ for the general case was put forward by Bishop (1959), which is

$$\bar{\sigma} = \sigma - u_a + x(u_a - u_w) \quad \dots \dots \dots \quad 4$$

where u_a denotes pressure in the gas and vapor and u_w denotes pressure in the pore water. The value of the parameter x is unity for saturated soils and is zero for dry soils. Intermediate values will depend primarily on the degree of saturation, but will be influenced also by factors such as soil structure and the cycle of wetting and drying.

Hilf (1956) has made an extensive study of pore water pressure in compacted soils. He assumed, for degrees of saturation greater than about 25 percent, that the water is present as a continuous film covering the grains and forming menisci of a single curvature. Considerations of surface tension effects in water show that the pressure in the water is always less than the pressure of the air in contact with the water; e.g., the capillary pressure u_c is always negative if the air is at atmospheric pressure. Consequently, Hilf showed that the pore pressure

of water u_w is the algebraic sum of two terms, the pore air pressure u_a and the capillary pressure u_c . An equation for pore pressures in compacted soil has been derived by Hilf with the aid of Boyle's law of compressibility of ideal gases and Henry's law of solubility of air in water. He considered that when a partially saturated soil is loaded under undrained conditions, the air is compressed by the full amount of the volume change and some of it dissolves in the pore water. The pore pressure developed in an unsaturated soil as a result of such a load application is given by Hilf (1956)

$$\Delta u = \Delta u_a + \Delta u_c \quad \dots \dots \dots 5$$

where

$$\Delta u_a = \frac{u_{a0} \Delta e}{e_{a0} + h e_w - \Delta e} \quad \dots \dots \dots 6$$

u_{a0} = initial air pressure

Δe = change in void ratio ($e_0 - e_1$)

e_{a0} = initial volume of air in soil mass of volume $1 + e_0$

e_w = volume of water in soil mass of volume $1 + e_0$

h = coefficient of air solubility in water by volume

Soil Shear Strength

Studies of the shearing resistance of soils have generally centered around the use of the expression first proposed in 1776 by Coulomb

(Hough, 1957, p. 162)

$$s = c + \sigma \tan \phi \quad 7$$

in which s is the shearing resistance, σ denotes the normal stress on the shearing plane, ϕ is the friction angle, and c is a term which denotes cohesion.

The conventional concept of shear strength as made up of the friction and cohesion terms, gives a simplified picture of the various complicated factors contributing to the shear strength of a soil. Recent studies have indicated, however, that such a relationship (Equation 7) greatly oversimplifies the picture. The parameters c and ϕ are not, for a given soil, constant for different loading conditions, stress history, testing conditions, or soil structures, but vary over a considerable range (Hvorslev, 1960).

In general, the shearing strength of a soil may be attributed to a combination of physical factors and physico-chemical factors.

Physical factors

The physical component of shear strength in soil is attributed to frictional resistance; i. e., the resistance to sliding of one surface on another, and interlocking between particles (Rosenquist, 1959). For two randomly oriented particles in contact with each other, the friction resistance is made up of different components (Rosenquist, 1959): macrodilatent friction, which is a force working against the applied normal stress caused by the lifting of a particle on the one side of the

shear plane when the shear movement takes place; and microdilant friction, which is a force required to move a particle up and over a surface irregularity of another particle.

The frictional resistance is proportional to the effective normal stress on the failure plane, and is of significance primarily between the granular (greater than clay size) particles.

Dilatency may have different effects on the shearing resistance of soils, depending on the test conditions. In drained shear tests, work is done against the normal stresses in increasing volume or by the normal stresses in decreasing volume. Corrections for these effects have been suggested by Bishop (1950). In drained shear tests the tendency toward volume changes induces excess pore pressure which may be either positive or negative. Pore pressure coefficients were proposed by Skempton (1954) to evaluate these effects.

One additional term which is classed as a physical factor is called by Rosenquist (1959) the nondilant friction. Here, the resistance to shear may be regarded as a physico-chemical effect since it is due to mass forces between particles. When two particles of such size that their behavior is independent of surface forces (i. e., sand or silt particles) are pressed together under the action of an effective stress, they will contact at a point since their surfaces are not completely smooth. The very close proximity of the contacting surfaces gives rise to attractive Van der Waal forces. To slide the two particles

relative to each other, a shearing resistance develops with a magnitude proportional to the strength of the adhesive bond. The total contact area developed is a function of the elasticity properties of the mineral; but if proportionality is assumed between stress and strain, then the contact area depends directly on the magnitude of applied effective stress (Seed, et al., 1960). Bowden and Tabor (1954) have concluded that the area of actual contact between two solid bodies is very low (e.g., under normal loads the area of contact between two steel surfaces will be of the order of $1/10^5$ of the apparent area of contact), and that the real area of contact is nearly independent of the total area and only dependent upon the load. The real area of contact a is expressed mathematically as (Rosenquist, 1959)

$$a = \frac{w}{P_m} \dots \dots \dots 8$$

where w is the total load and p_m is the yield value of the material. As the nondilatent friction is regarded to be due to the adhesion on the area of contact, an approximation could be made (Rosenquist, 1959) such that

$$f = as \dots \dots \dots 9$$

where f is the friction force and s is the shear strength of the junction. The friction represents the shear strength and the adhesion tensile strength of the bonds in the contact area.

In practice, the physical component of shear strength is expressed by $(s = \bar{\sigma} \tan \phi)$. The term ϕ is a material function which represents the magnitude of adhesion between areas of real contact.

If the effective normal stress $\bar{\sigma}$ is removed from a material which develops its strength in the manner indicated above, then the elastic stress within the particles will be relieved and the particles will rebound to their original shapes, reducing the contact area to point contacts and resulting in a loss of almost all adhesion (Seed, et al., 1960).

The other important physical factor contributing to the mobilization of shear strength is the interlocking between particles, particularly as reflected by the tendency for volume change during shear deformation. Rosenquist (1959) has divided interlocking into two parts: (1) a large scale interlocking between particles which necessitates appreciable movements of particles normal to the shear plane accompanied by volumetric expansion in order that failure might occur; (2) a small scale interlocking due to particle surface roughness, necessitating only small movements normal to the shear plane in order that failure might occur.

It is apparent that the magnitude of friction and interlocking; i. e., the physical factors contributing to shear strength in soils, is dependent on the effective normal stress. However, the nature of the minerals present and their surface characteristics will determine the magnitude of the true friction (Seed, et al., 1960).

Physico-chemical factors

This component of soil shear strength is attributable to physico-chemical conditions in the soil and is often referred to as cohesion. Cohesion in a soil refers to that part of strength which is present independently of any applied stresses and would remain, though not necessarily permanently, if all applied stresses were removed. In other words, cohesion is a bonding of particles by physico-chemical mechanisms of an interatomic, intermolecular, or interparticle nature (Seed, et al., 1960).

Although the precise nature of cohesion and the mechanism by which it is developed have not been clarified, it seems to be generally agreed that it is a function of the net interparticle forces (Rosenquist, 1959; Lambe, 1960a, b; Michaels, 1959). These in turn are affected by several variables in the soil-water-air-electrolyte system as reviewed below.

Interparticle forces

The appearance and behavior of soils alter as the particle size decreases. The changes in behavior are due to the increasing effect of the forces between adjacent particle surfaces as the size decreases. In the silt or sand sizes, the ratio of the area of the surface to the volume of the sample (specific surface area) is relatively small. In a small particle, the molecules forming the surface constitute a large proportion of the total number of molecules and the forces associated

with these surface molecules have an appreciable effect on the behavior of the particle and hence on the mass of the soil (Lambe, 1960a).

Interparticle forces may be divided into attractive and repulsive forces. Attractive forces between clay particles result from several mechanisms, the most important of which is attraction due to the Van der Waal forces.

As a result of the movement of electrons in their orbits around atoms, any molecule possesses an associated electric field which is capable of interacting with the field of nearby molecules giving rise to Van der Waal's attractive force between the molecules (Casimir and Polder, 1948). Derjaguin (1960) showed that the Van der Waal force between two flat parallel surfaces varies inversely as the cube of the distance between them if the plates are very close together, and inversely as the fourth power of the distance at large separation. In this connection the distance between the plates is considered small or large in relation to the wavelength of light absorbed by the materials in question and is always large in comparison with interatomic distances.

Several investigators (Michaels, 1959; Rosenquist, 1959) are of the opinion that the Van der Waal forces are of sufficient magnitude to more than account for cohesion in clays. Additional interparticle attraction may arise, however, from the electrostatic attraction between negative clay surfaces and positive clay edges. This situation can lead to a non-salt flocculated structure (Lambe, 1960a). For some cases attractive forces may be caused by hydrogen or potassium bonds or may

result from cementation by organic or inorganic compounds.

Interparticle repulsive forces arise from the electrostatic repulsion between two surfaces or between two edges of adjacent particles.

Since the shear plane in clay will pass through the water between particles, the characteristics of the water in the force field between clay particles must be considered. This water is known to possess properties different from those of normal free water. However, the literature is controversial as to the precise nature of the adsorbed water structure and the mobility of this water. The work by Low (1959) has shown the viscosity of adsorbed water to be higher than that of free water. Martin (1959) concluded that the first few layers of adsorbed water on clay surfaces has a remarkably high freedom of mobility in direction parallel to particle surfaces.

Water is strongly attached to clay surfaces by any or all of several mechanisms as outlined by Low and Lovell (1959). It appears that its role is that of a filler separating particles and resisting close approach. Thus, a lesser attractive bond is formed than would exist if water is removed from clay.

The interparticle forces in the soil-water system may in some cases undergo thixotropic changes, which are defined by Freundlich (1935) as isothermal, reversible, sol-gel transformations; i. e., the strength of a clay may be decreased during rapid deformation, but is gradually regained

when the deformations cease or the rate of deformation decreases. The phenomenon may be visualized as a disturbance and subsequent re-establishment not only of the arrangement of the clay particles but also of the structure of the bound water with consequent changes in the transmission of interparticle forces. Michaels (1959) stated that any contribution to shear strength resulting from water viscosity is negligible in comparison to the contribution of the interparticle attraction forces. He concluded that despite the fact that the failure plane passes through the water separating soil particles, the resistance to shear is attributable to the bond caused by attractive forces originating in the particles themselves and holding the particles in position.

Goldstein (1957) suggested that the interparticle forces may be divided into two groups, one producing elastic bonds, and the other forming viscous bonds. However, it has not yet been possible to identify the basic forces and conditions which cause formation of the two types of bonds. Borowicka (1959) has proposed that changes in free energy and corresponding forces at the interface of water and soil particles can produce tension in the bound water and a corresponding cohesion at any close spacing of the particles. It has not been verified experimentally that such a tension in the bound water exists. Tan (1957), on the other hand, questions that cohesion could be attributed to tension in the soil water because clay possesses cohesion even where positive pore pressure could be measured.

Externally applied compressive stresses undoubtedly influence the development of cohesive bonds, since they help to determine the particles' orientation and spacings and thereby affect the interparticle attractive and repulsive forces. Since such compression is not a reversible phenomenon (Seed, et al., 1960), the particle spacing under a condition of decreasing external forces will depend also on the previous stress applications and thus on the stress history of the soil.

In general, the magnitude of the resultant interparticle forces depends on the type of clay mineral, the size and corresponding specific surface of the particles, the type of ions adsorbed on the surfaces of the clay crystals, the type and concentration of ions in the pore fluid, particle spacings, and on temperature.

In a typical cohesive soil containing granular particles and clay size particles, the total strength will evidently be the sum of the contributions of physical and physico-chemical components. Lambe (1960b) suggested that if the clay particles are in contact, the shear strength s can be thought of as a function of the net interparticle forces

$$s = f(\bar{\sigma} + A - R + I) \dots \dots \dots 10$$

where $\bar{\sigma}$ is the externally applied intergranular stress, A is the electrical attraction, R is the electrical repulsion, and I is a repulsive stress acting only when particles are in contact.

The origin of various types of electrical charges carried by soil colloids is discussed in terms of the theory of the diffuse double layer (Warkentin, et al., 1957). The nature and mathematical expressions of conditions within the double layer have been treated in detail by Overbeck (Kruyt, 1952, p. 115). However, before reviewing the literature which has likened the electrokinetic properties of clays with other aspects of their engineering properties, it is important to discuss some general relationships in colloidal chemistry.

A clay suspension is composed of negatively charged micelles surrounded by diffuse envelopes of positively charged ions which make up the diffuse double layer. Associated with each envelope of charged ions there will be a gradient of electrical potential whose intensity decreases with increasing distance from the surface of the particle. Until recently, the theoretical description of the double layer formed on planar interfaces has been based entirely on the theory of Gouy-Chapman (Kruyt, 1952, p. 128). If we assume a clay particle to be a plane surface and the charge uniformly distributed over the surface, the charge distribution (according to the Gouy-Chapman theory) is obtained by the combination of the Poisson and Boltzmann equations, which leads to

$$\nabla^2 \psi = - \frac{4\pi}{D} \sum_i z_i \epsilon_{io} n_i e^{-\frac{z_i \epsilon \psi}{kT}} \dots \dots \dots 11$$

where ψ is the electrical potential at a distance x from the surface

of the clay particle, z_i is the valency of the ion in the bulk solution, e is the charge on an electron, k is the Boltzmann constant, ∇ is the Laplace operator, and D is the dielectric constant of the solution. This equation has been solved by Verwey and Overbeek for several cases.

As the influence of ions is greatest near the colloidal particle surface, Stern (Kruyt, 1952, p. 132) applied the Gouy-Chapman theory with the first layer of ions not immediately at the surface but at a distance δ away from it. Stern further considered the possibility of specific adsorption of the ions and assumed that these ions were also located in the plane δ . This layer of adsorbed ions is called the Stern layer.

In general terms, the double layer theory states that a tendency toward flocculation is usually carried by increasing electrolyte concentration, ion valence and temperature, and decreasing dielectric constant, size of hydrated ion, pH, and anion adsorption. The influence of the first four variables follows from the Gouy-Chapman theory of the diffuse double layer as a decrease in the double layer thickness reduces the electrical repulsion forces which in turn causes a tendency toward flocculation. The smaller an ion plus its shell of hydration water, the closer it can approach the colloidal surfaces. Thus, the smaller the hydrated ion, the smaller the double layer thickness and the more likely is flocculation. The pH of the pore fluid affects the net negative

charge on a soil particle by altering the extent of dissociation of OH^- groups on the edges of the particle. High pH encourages the dissociation and increases the net negative charge, thus expanding the double layer. The adsorption of anions, especially polyvalent anions, increases the charge on the particles and thereby tends to cause dispersion (Schofield and Samson, 1953, 1954).

Bolt and Miller (1955) showed that the classical theory of the electric double layer formed on planar surfaces provides an acceptable means of predicting the osmotic pressure of clay suspensions for various particle spacings and electrolyte contents. The results obtained with a compression apparatus for a number of illite samples were consistent with those predicted with the theory. Taylor (1959) stated that the Gouy-Chapman theory of the diffuse double layer provides a satisfactory basis for the study of many types of clay soils, and may be used to derive useful information about their mechanical behavior. Low (1961), on the other hand, showed that the double layer theory cannot be depended on for an accurate description of ionic exchange, clay swelling, and mechanical behavior of soils.

In engineering soils, we are mostly concerned with large masses of natural soils rather than with dilute suspension from which the theory of double layer derives its basis. Lambe (1960a) discussed several factors which are neglected in colloidal theories. He concluded that the force system acting between soil particles is influenced not only by

the forces arising from the nature of particles (i. e., considered in colloid theories), but also by those arising from noncompositional sources (forces derived from a source outside of the individual particles).

Since stable soil aggregates can only be formed after the clays are flocculated, it is expected that the strength properties of clays are influenced by any change in the net electrical forces between the particles. Thus, any factor which causes the expansion of the double layer (i. e., increases the repulsive forces between adjacent particles) will result in reduction of the shearing strength. Michaels (1959) observed that with montmorillonite and to a lesser degree with illite clays, the liquid limit, which is a rough indication of strength, decreases with the following order of adsorbed cations



On the other hand, with kaolinite, the liquid limit will often increase with the same order of exchangeable ions. Experiments carried out at the Norwegian Geotechnical Institute by Rosenquist (1959) proved that, if a sodium montmorillonite is consolidated and kept under constant consolidation load and then percolated by a KCl solution, the undisturbed shear strength increases considerably. However, in all reported investigations, the results were presented only in terms of the general influence of a particular factor on some of the mechanical

properties of pure clay minerals, with no indication of how the effects could be extrapolated to natural soils explaining actual failure phenomenon. The shear strength of a soil measured in a laboratory may depend considerably on the electrolytic characteristics of the water with which the soil was remixed, or on the type and concentration of the exchangeable ions, or on the temperature at which the soil was stored and on the storage time, as well as many other factors of physical or chemical nature. In the majority of reviewed literature, these details of testing were omitted.

In view of the current literature, it is apparent that the physico-chemical properties of soil may have greater effect on its engineering properties than it is commonly realized. Hence, a more thorough knowledge regarding the factors influencing strength properties of soil is needed.

EXPERIMENTAL METHODS AND PROCEDURE

The experimental procedures are discussed under four headings: (1) Soil sampling; (2) Chemical, mechanical, and mineralogical characteristics; (3) Engineering properties; and (4) Preliminary experiments.

Soil Sampling

A soil sample (about 200 pounds) was obtained from a drainage ditch bank that exists on the Utah State University Irrigation and Drainage Farm, northwest of Logan, Utah. The drainage farm area is classified as Salt Lake silt loam by the United States Department of Agriculture, Soil Conservation Service. Soils are humic gley of mixed lake sediment origin and are considered poorly drained. The sample was taken from the clay part of the drain bank (about 6 feet from the surface). Sloughing phenomenon was observed in the drain slopes.

Chemical, Mechanical, and Mineralogical Characteristics

Chemical analysis

Chemical laboratory tests consisted of determining the pH for both a saturated paste and a mixture of one part soil to five parts of water by weight (Richards, 1954), total soluble salt concentrations (Cheng and Bray, 1951), electrical conductivity of the saturation extract

(Campbell, et al., 1948), true specific gravity (Taylor, 1962), total exchangeable capacity, exchangeable sodium, exchangeable potassium (Bowers, et al., 1952), lime content (Reitemeier, 1943), and organic matter (Walkley, 1947). All tests were conducted on duplicate samples, and the results are listed in Table 1.

Mechanical analysis

The pipette method of mechanical analysis was used (Olmstead, et al., 1930). This method consists mainly of three parts: (1) Soil cleaning which includes the removal of CaCO_3^* with diluted HCl and removal of organic matter with hydrogen peroxide; (2) Soil dispersion using sodium hexametaphosphate solution (calgon); (3) Different particle size determination using different times of settlings and a volumetric pipette connected in such a way that the strength and time of suction is always constant.

Results of this test are shown in Table 2.

Mineralogical analysis

Soil fractionation. Mineralogical examination of the soil by x-ray diffraction or other methods of characterization requires fractionation of the soil. The procedure used was that of Kittrick as given by Miller (1956). This method involves: (1) The removal of carbonates by dilute

* Mechanical analysis was done on the sample twice, once without the removal of CaCO_3 and another time after CaCO_3 was removed. This was done to determine the grain sizes of the lime in the soil.

Table 1. Summary of chemical characteristics of the sample

Characteristic	Value
1. True specific gravity	2.41
2. pH of saturated paste	9.20
3. pH of one part soil to five parts of distilled water	8.20
4. Total soluble salts determined gravimetrically (percentage)	0.53
5. Electrical conductivity of the saturation extract	7.42
6. Cation exchangeable capacity (milliequivalent per 100 grams)	21.60
7. Exchangeable sodium (milliequivalent per 100 grams)	7.83
8. Exchangeable potassium (milliequivalent per 100 grams)	0.67
9. Exchangeable sodium percentage	36.00
10. Lime content, percentage	41.00
11. Organic matter content, percentage	0.77
12. Soluble salts in one part soil to five parts distilled water (milliequivalent per 100 grams)	
Ca ⁺⁺	0.10
Mg ⁺⁺	0.20
Na ⁺	7.60
K ⁺	20.00

Table 2. Particle size distribution of the sample (in mm.)

Sample treatment	Sand fractions, %					Silt %	Clay %	Sand %
	(1) 2.00-1.00	(2) 1.00-0.50	(3) 0.50-0.25	(4) 0.25-0.10	(5) 0.10-0.05			
Lime is removed	-	0.02	0.01	0.60	0.18	40.10	58.80	0.29
With lime	-	0.03	0.03	0.25	0.70	36.78	61.20	1.01

(1) Very coarse sand

(2) Coarse sand

(3) Medium sand

(4) Fine sand

(5) Very fine sand

HCl; (2) The oxidation of organic matter with hydrogen peroxide; (3) Dispersion of the soil by sodium hexametaphosphate.

X-ray diffraction. X-ray diffraction was performed on a North American Norelco x-ray diffractometer equipped with a wide angle goniometer, a copper target x-ray tube, a proportional counter, and a scanning speed of two degrees per minute. A nickel filter was used to eliminate κ_{β} radiation from the incident beam.

Preparation of samples for x-ray diffraction

The soil clay fraction was separated into two different sizes; i. e. , 0-0.2 microns, and 0.2-2 microns. The two different size fractions were then prepared as 3 percent suspensions in distilled water. The 0-0.2 μ fraction was suspended in a similar manner in glycerol-water solutions (glycerol was used at rates recommended by Jackson (1949)). The glycerol treatment was used to distinguish between montmorillonite and illite groups.

Sample slides were prepared by dropping about 0.5 ml of the suspension containing from 10-15 mg of clay on a one-inch square area of a microscope slide. The slides were air-dried before analysis.

Interpretation of x-ray patterns

Each crystalline substance has a characteristic trace of diffraction peaks. It should be realized, however, that peak heights are

not always proportional to clay mineral quantities. Therefore, the problem of quantitative determination by x-ray diffraction made it necessary to compare the characteristic peaks of the experimental specimen with other characteristic peaks of pure clay minerals; i. e., kaolinite, montmorillonite, and illite (Figures 1, 2, 3). The pure clay minerals were prepared and analyzed in the same manner as the experimental samples.

From a cursory glance at the x-ray traces of the 0-0.2 μ and 0.2-2 μ fraction samples and that of the sample treated with glycerol (Figures 4, 5, and 6), it could be seen that the montmorin^{*} type of clay minerals is the most dominant; however, small amounts of mica^{**} and little or no kaolin^{***} are indicated. The characteristic peak of montmorillonite at 18 $\overset{\circ}{\text{A}}$ is not as high in the glycerol treated sample (Figure 6) as that of pure montmorillonite (Figure 2). This may be due to poor crystallinity or to a considerable amount of amorphous material in the experimental sample.

* Montmorin refers to 2:1 lattice expanding minerals having x-ray diffraction basal spacing of approximately 18 $\overset{\circ}{\text{A}}$.

** Mica refers to minerals having x-ray diffraction basal spacing of about 10 $\overset{\circ}{\text{A}}$. This would include mica and illite.

*** This refers to kaoline-like minerals having x-ray diffraction basal spacing of about 7.2 $\overset{\circ}{\text{A}}$.

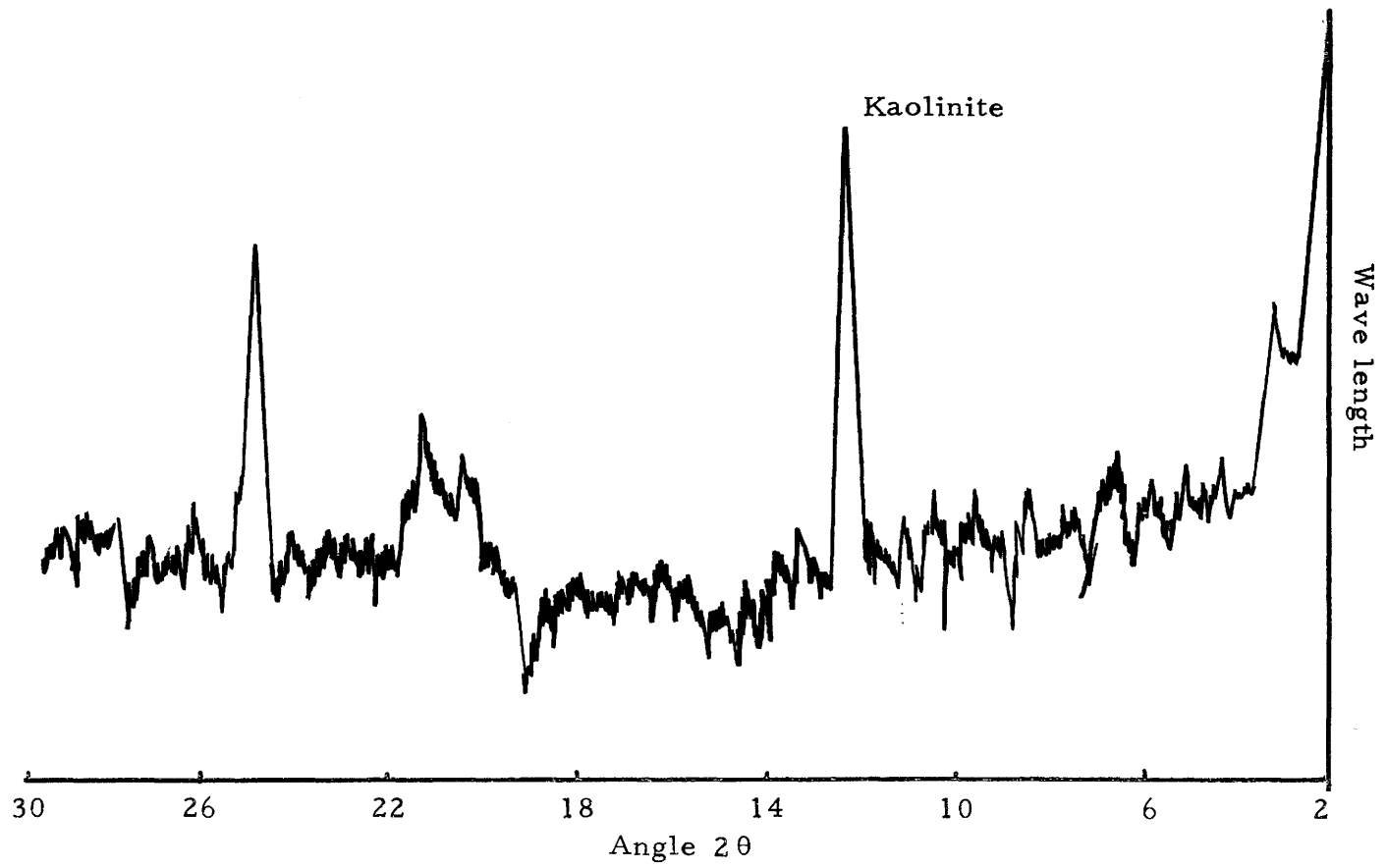


Figure 1. X-ray traces of kaolinite clay mineral

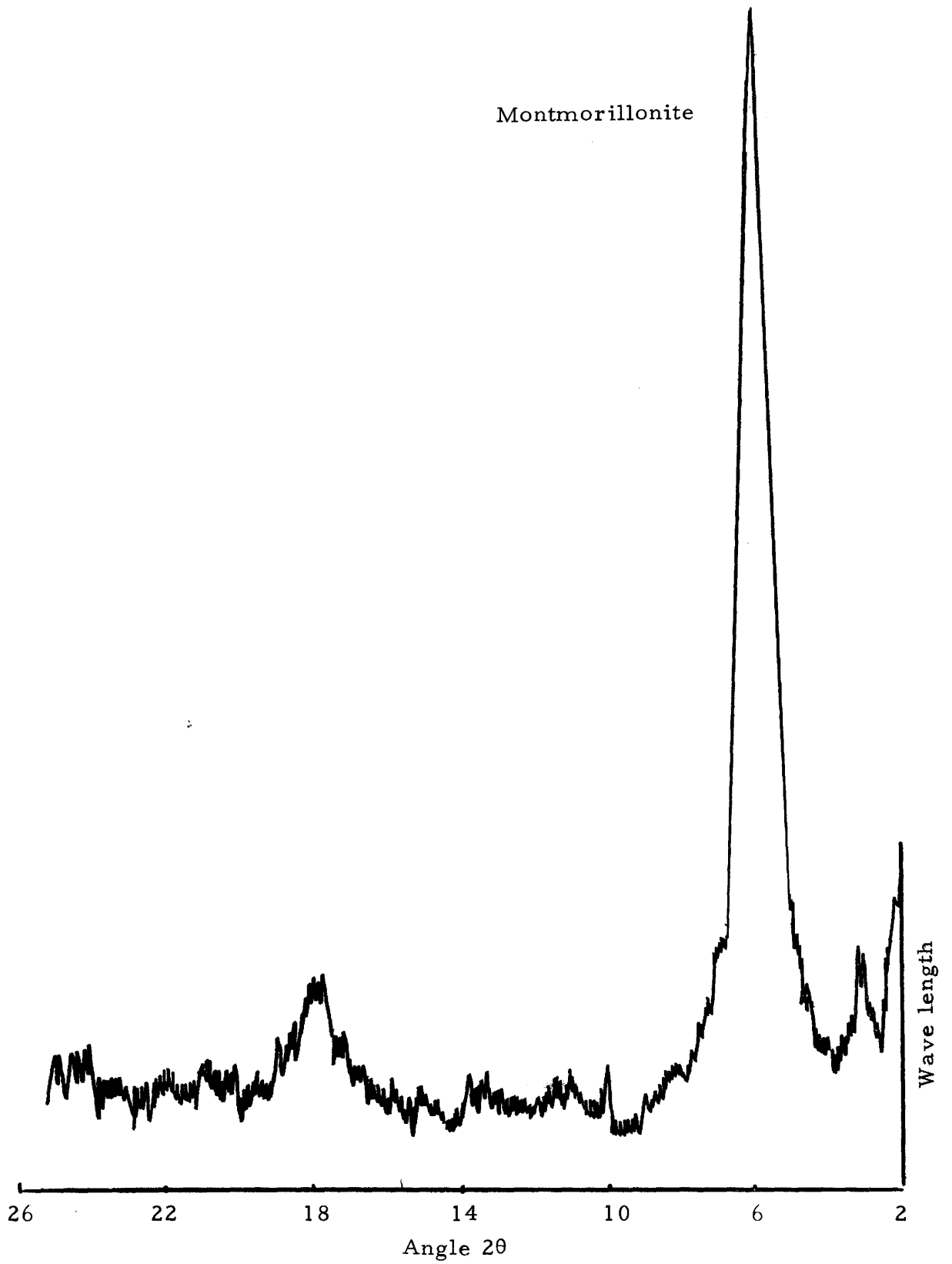


Figure 2. X-ray traces of montmorillonite clay mineral

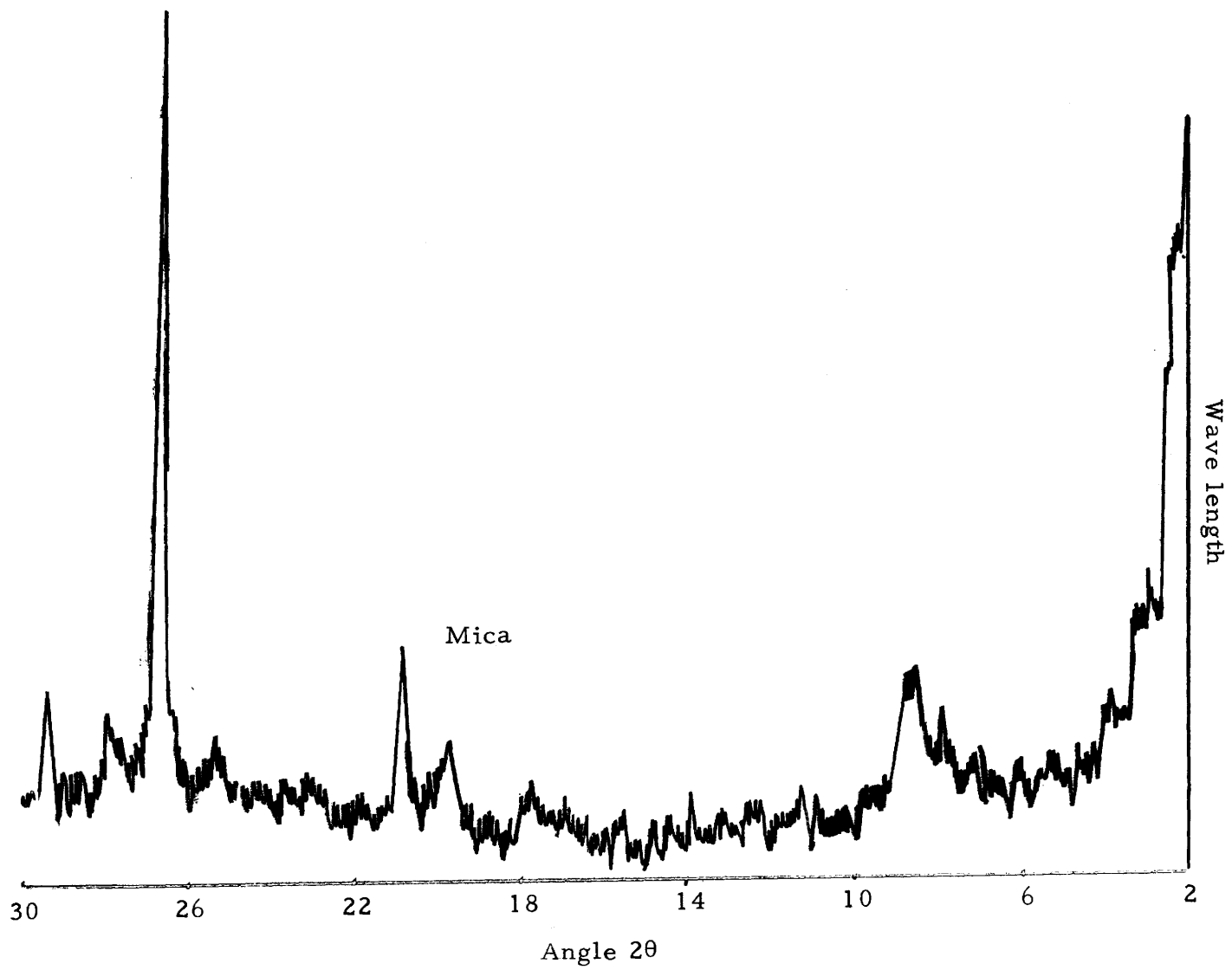


Figure 3. X-ray traces of illite clay mineral

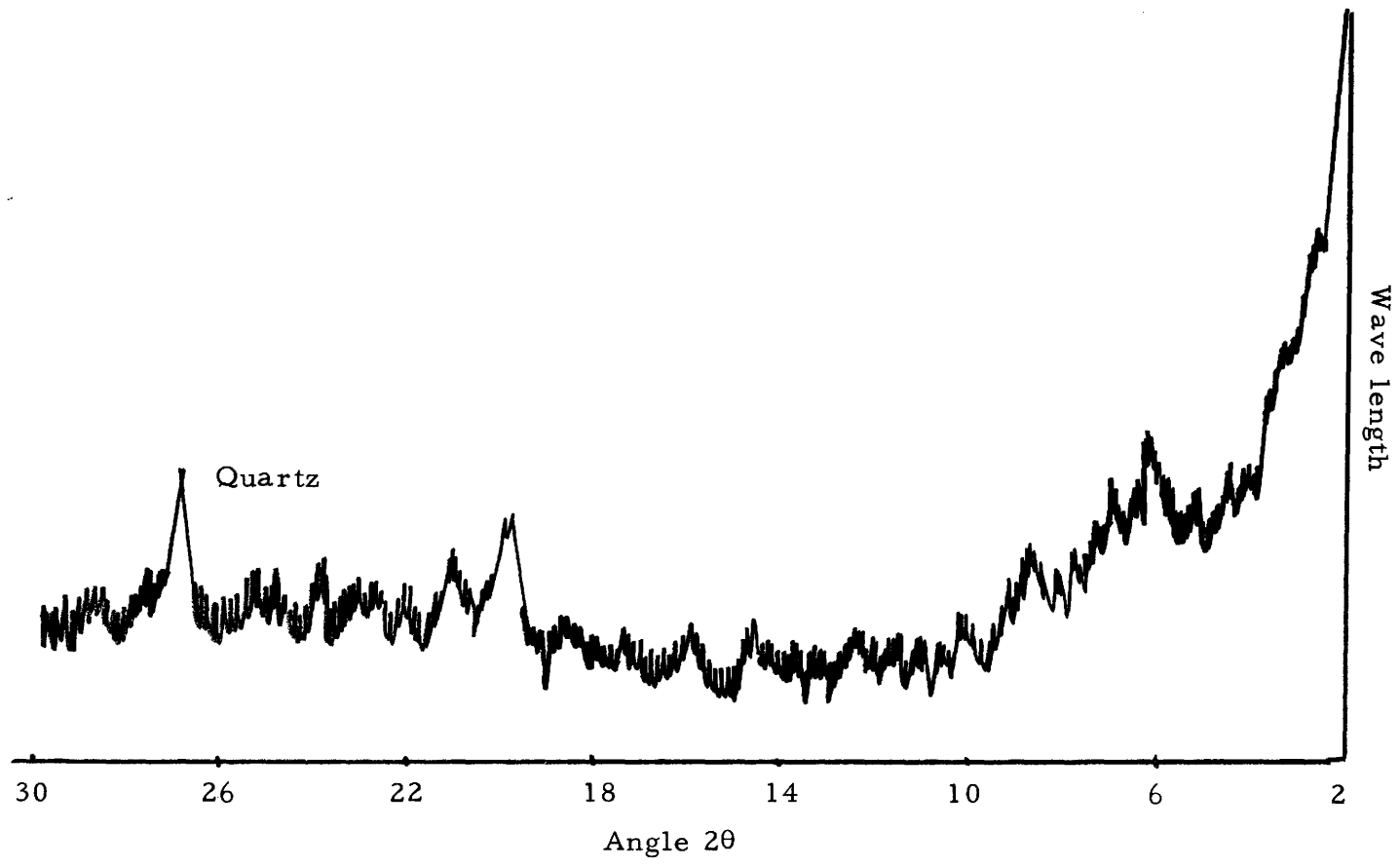


Figure 4. X-ray traces of 0-0.2 μ fraction of the sample suspended in distilled water

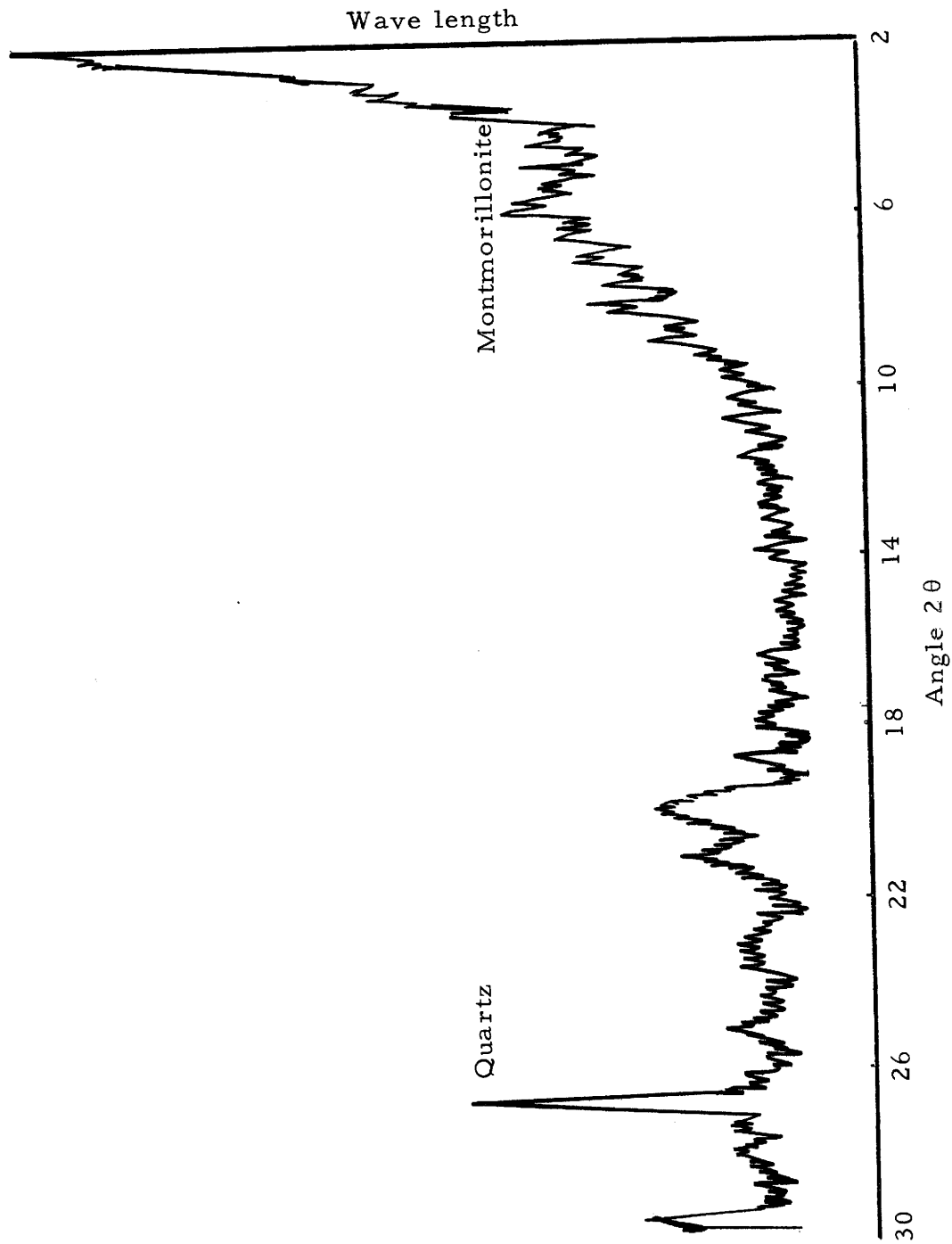


Figure 5. X-ray traces of 0.2-2 μ fraction of the sample suspended in distilled water

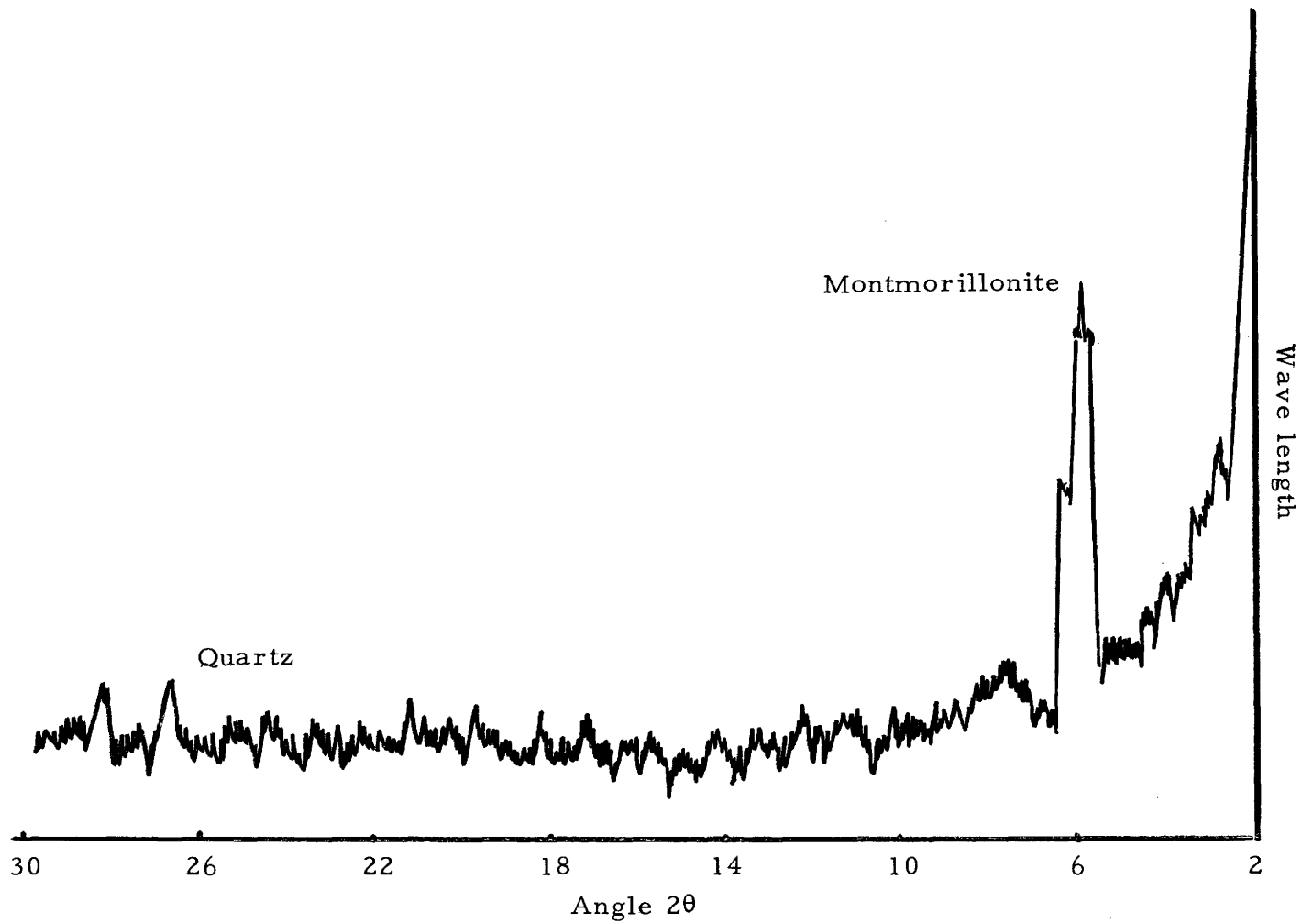


Figure 6. X-ray traces of 0-0.2 μ fraction of the sample treated by glycerol

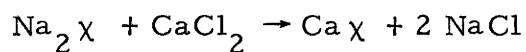
Sample preparation

The whole soil sample was saturated with sodium ion. This was accomplished by preparing suspension of the soil in a 3 N sodium chloride solution. The suspension was allowed to equilibrate for a period of 15 days, after which the supernatant liquid was removed. This treatment was repeated four times, using fresh NaCl solution, stirring, then allowing the suspension to settle until the supernatant liquid is clear; then it was discarded. To remove the excess salts, the sample was washed with distilled water until all traces of chloride in the filtrate disappeared. A test was performed in the laboratory to determine the exchangeable sodium percentage in the sample (Table 3).

The sodium-treated soil was divided into nine parts. The exchangeable sodium ion on eight of these parts was replaced partially on four samples and completely on the other four by the ions: potassium, hydrogen, calcium, and magnesium. In each case, a test was performed to determine the exchangeable sodium percentage in the sample. In the replacement of exchangeable Na^+ by another cation, the previous procedure of preparing suspensions of the sample and the chloride salt of the desired cation was used. In the case of replacing exchangeable Na^+ partially, the sample was oven-dried and weighed. Knowing the cation exchange capacity per 100 grams and the desired exchangeable Na^+ percentage, the amount of salt to be added was calculated according to the following reaction

Table 3. Exchangeable cation percentage on different samples

Sample	Exchangeable percentage				
	Na ⁺	K ⁺	H ⁺	Ca ⁺⁺	Mg ⁺⁺
1.	99.4				
2.	38.0	62.0			
3.	8.0	92.0			
4.	46.0			51.0	
5.				98.0	
6.	43.0				56.0
7.					100.0
8.	51.0		49.0		
9.			100.0		



where χ denotes the soil complex. In the case of replacing Na^+ with H^+ ion, the sample was treated with HCl until all CaCO_3 was removed. The sample was suspended thereafter in a solution of HCl of sufficient strength (3N) for complete saturation. One-half of the H^+ -treated sample was then titrated by NaOH solution to reach the desired exchangeable sodium percentage. It is realized, however, that such treatment of soil by HCl may decompose part of the clay fraction, liberating exchangeable aluminum in the system.

Engineering Properties

Atterberg limits (liquid and plastic limits) and unconfined shear tests were conducted for all nine samples, using distilled water as the pore fluid. In addition, tests were conducted using solutions of NaCl, CaCl_2 , MgCl_2 , KCl, and HCl as pore fluid. These latter tests, however, were performed only on the five samples that were completely saturated with Na^+ , Cl^{++} , Mg^{++} , K^+ , and H^+ , respectively. Concentrations of 0.1, 0.3, 0.5, 0.7, 1.0, and 1.5 N were used for each electrolyte.

Atterberg limits

1. Liquid limit. Tests have been conducted according to the standard method (A.S.T.M., 1944a). The distance which the cup drops

(1 cm) was properly set and checked at frequent times.

2. Plastic limit. Tests were conducted following standard method (A. S. T. M. , 1944b). A piece of brass rod one-eighth inch in diameter was used to compare the size of soil threads. Tests were conducted on duplicates.

Unconfined compression test

The unconfined compression test measures the compression strength of a cylinder of soil to which no lateral support is offered. The shear strength is taken as equal to one-half the compressive strength. Because no lateral pressure is employed, the unconfined compression test is used only for cohesive soils since a cohesionless soil will not form an unsupported cylinder. This test, however, is the simplest and quickest laboratory shear test.

Sample preparation for the compression test

It was important for this test to prepare the samples in such a way that they have void ratios, degree of saturation, and water contents duplicated as closely as possible. A sufficient amount of oven-dried soil was mixed thoroughly with a predetermined amount of distilled water or with the appropriate solution. Mixing of the soil was accomplished by hand mixing. The mixed soil was compacted in a cylindrical brass mold (2 inch inside diameter) in successive equal layers. The cylinder was then covered by a waterproof plastic and

allowed to come to equilibrium over 24 hours. The different soil samples were extruded using the machine in Figure 7. Samples were trimmed to a length between 5 and 5.5 inches. Each sample was measured carefully and weighed before the test. For all samples, tests were conducted in duplicate.

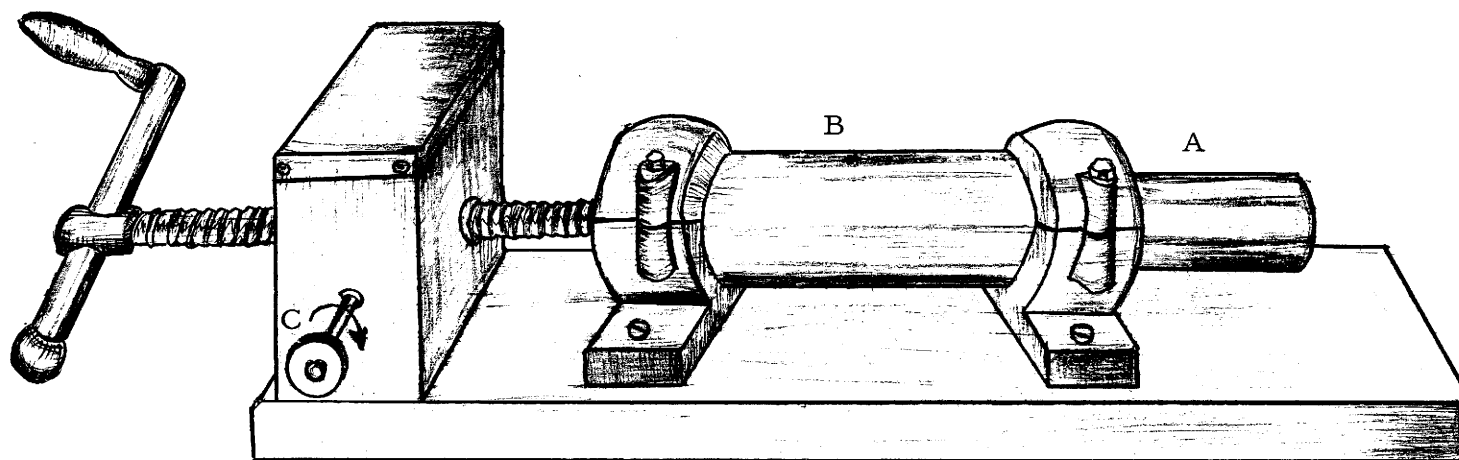
Apparatus

The apparatus used was of the controlled strain type; i. e., the strain was applied at a uniform rate. In the apparatus shown in Figure 8 the load is applied to the specimen by a variable speed motor drive. The magnitude of the load is measured by the double proving ring dial and the strain is indicated by the strain dial.

Testing of the sample

For each sample tested, the following procedure was adopted.

1. The sample was placed with its vertical axis as near the center of the loading plates as possible.
2. Extensometers were adjusted and initial readings of the proving ring dial, timer, and vertical deflection dial were recorded.
3. Compression was started, and readings were taken every minute until the stress-strain curve was past its peak (the proving ring dial starts to decrease).
4. The specimen was placed in a preweighed dish and weighed.
5. The water content of the entire remolded specimen was determined.



- A The extruded sample
- B Brass cylinder
- C Pulley to a driving motor

Figure 7. The extruding apparatus

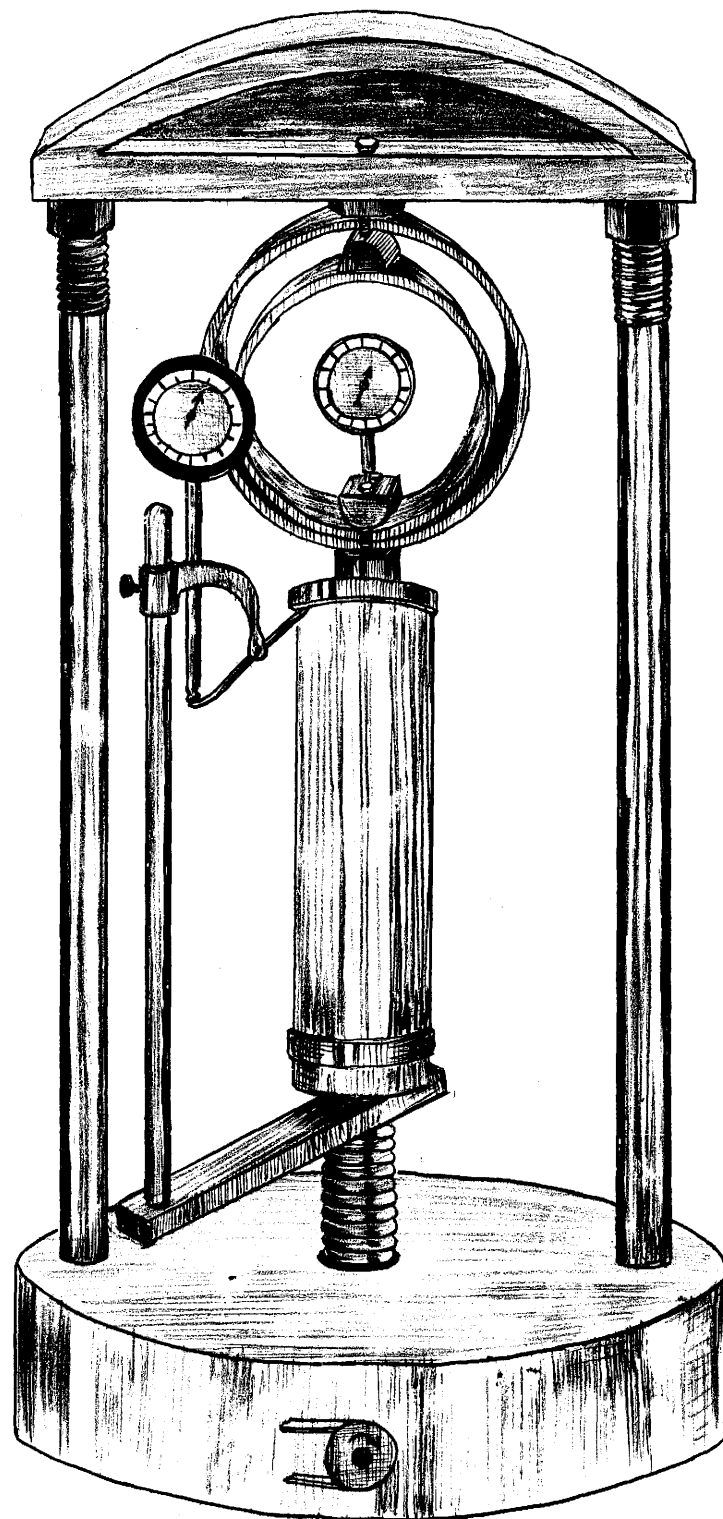


Figure 8. The unconfined compression test apparatus

Calculation of data

1. Strain. The strain was calculated from the equation

$$\epsilon = \frac{\Delta h}{h_0} \dots \dots \dots 12$$

where ϵ is the strain in inches/inch, Δh is the change of specimen length as read from the extensometer; i. e., $(h_0 - h)$, and h_0 is the initial length of the sample in inches.

2. Stresses. Since the sample thickens under compression, the area was corrected for the increase in diameter. The corrected area; i. e., the average cross-sectional area A was found from

$$A = \frac{A_0}{1 - \epsilon} \dots \dots \dots 13$$

in which A_0 is the initial area of the specimen. The vertical load P was calculated from the equation

$$P = \Delta k_{pr} \dots \dots \dots 14$$

where Δ is the proving ring dial movement and k_{pr} is the proving ring constant in pounds per inch. The shear stress τ was taken equal to one-half the compressive stress and can be found from the equation

$$\tau = \frac{P}{2A} \dots \dots \dots 15$$

Modulus of elasticity

To evaluate the modulus of elasticity, it is a common practice to

draw a straight line to average the data for the first part of the stress-strain diagram neglecting the curvature. The modulus of elasticity (Young's modulus) was taken as the slope of the straight line. This is expressed mathematically as (Higdon, et al., 1962, p. 45)

$$E = \frac{\sigma}{\epsilon} \dots \dots \dots 16$$

where E is the modulus of elasticity.

Preliminary Experiments

1. Soil water and compaction uniformity

Since the moisture content and the degree of compaction play a very important role in the compression test results, a preliminary experiment was conducted to find out the variation in initial specific weight and water content throughout the specimen. For a given soil, the compacted density varies with the degree of compaction.

A soil sample was compacted in the same manner and under the same conditions used throughout this investigation, in a lucite tube (2 inches inside diameter) which divided into six parts of known lengths (Figure 9). The divided cylinder was actually placed inside another cylinder before soil compaction to prevent any loss of water or soil. The sample was allowed to come to equilibrium for 24 hours, then was cut into six parts of known volume. Each part was weighed to determine its specific weight. The samples were dried at 110° C for 24 hours and reweighed to determine the water contents,

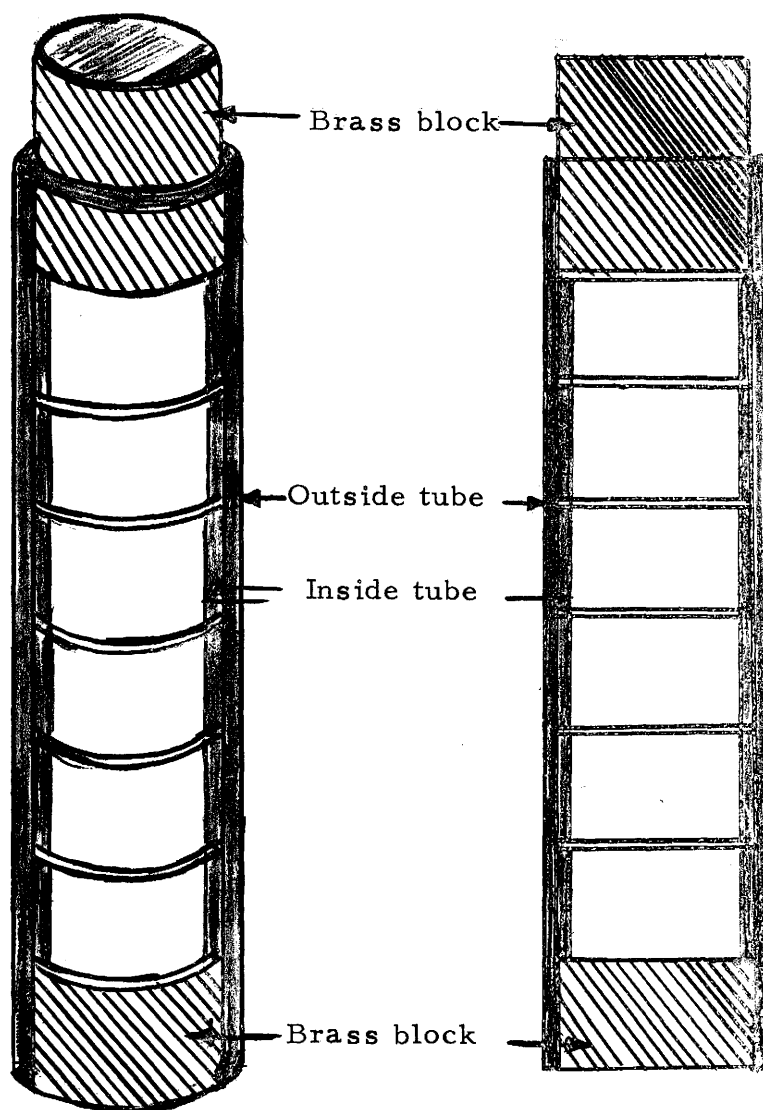


Figure 9. Diagram of the lucite tubes used to determine specific weight and water content throughout the length of the sample

Results. Figures 10 and 11 show that the compacted density varied by ± 0.4 pounds per cubic foot from that averaged over the whole specimen, and the water content varied by ± 0.15 percent from that of the whole sample. Such close specimen uniformity may not always be obtained for every sample. The results obtained from three different samples, however, were in the range of variation reported in Figures 10 and 11.

2. Comparison between unconfined compression test and triaxial shear test

Although the triaxial shear test is more reliable for research purposes and for other testing in which control of the boundary conditions is desired, the unconfined compression test was performed because of its simplicity. A preliminary experiment was conducted, however, to find out how well the two methods agree.

The apparatus used for performing a triaxial shear test is shown in Figure 12. It consists of a pressure-tight test chamber, an air pressure control valve, a pressure gage, and the necessary pipes and fittings. A cylindrical sample is encased in a thin rubber membrane to isolate the sample from the fluid in the test chamber. After the sample has been placed, fluid is introduced into the chamber under the desired pressure. Failure is induced by an additional thrust supplied by a hand wheel.

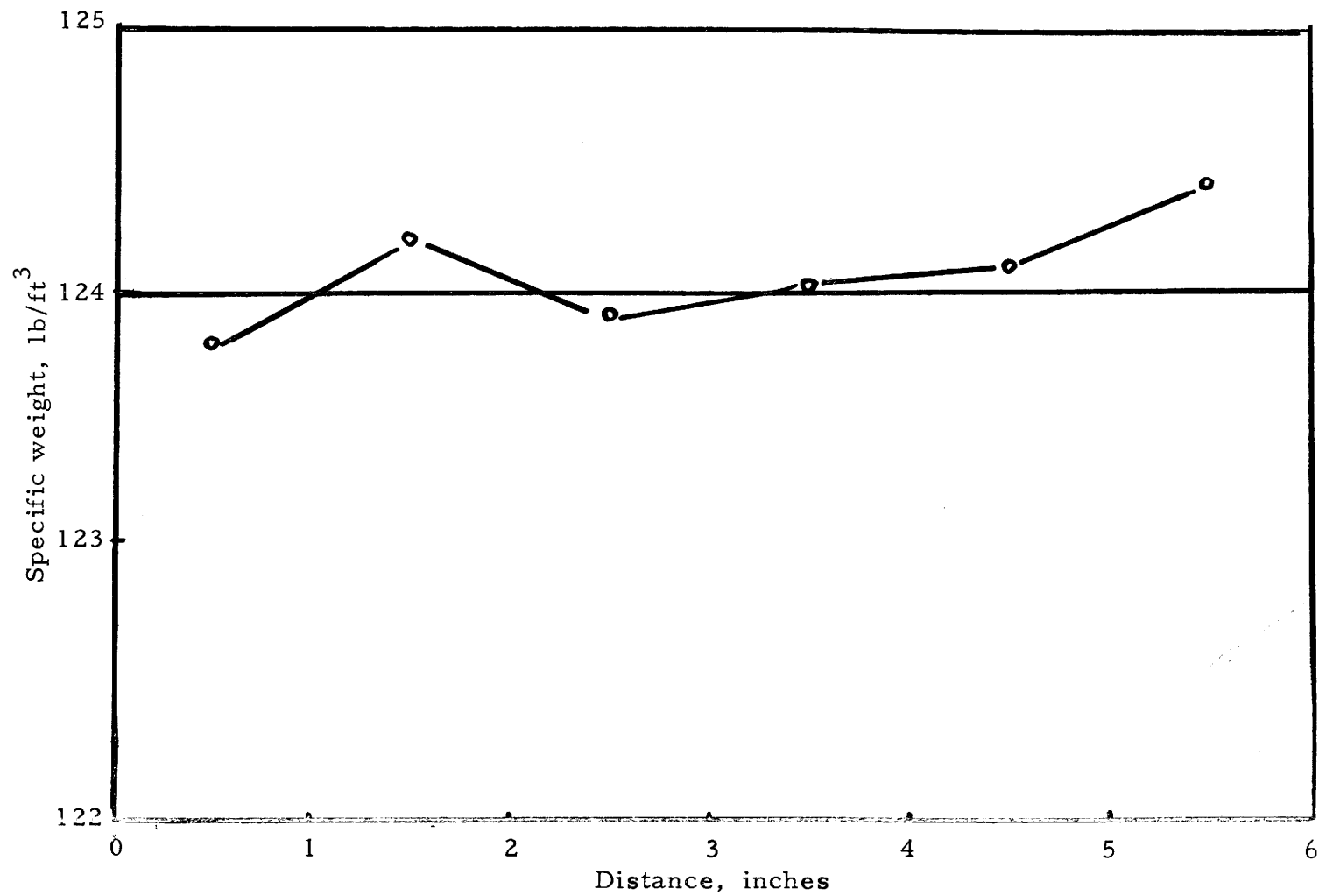


Figure 10. Specific weight throughout a Mg^{++} -saturated sample, determined every inch for a remolded column before the shear test

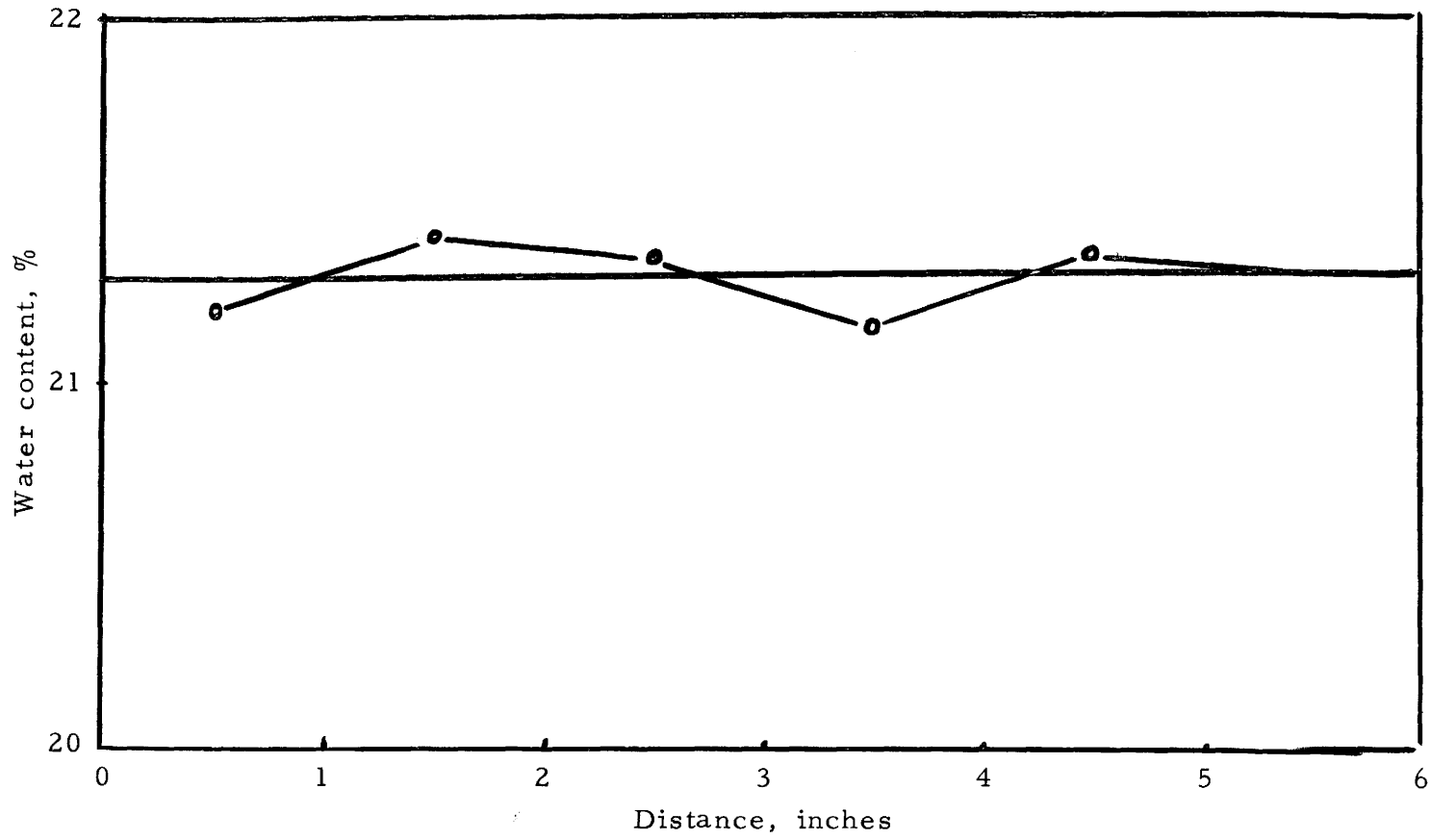


Figure 11. Water distribution through a remolded ($\text{Ca}^{++} + \text{Na}^{+}$)-saturated sample before the shear test

Results. Reasonably good agreement of results has been obtained from the unconfined compression and triaxial shear tests for a Na^+ -saturated sample using both distilled water and 0.1 N NaCl solution as pore fluid (Figures 13 and 14). The physical properties of the samples (compacted densities, water contents), the lateral pressures used, and the shear strength values obtained by both tests are listed in Table 4.

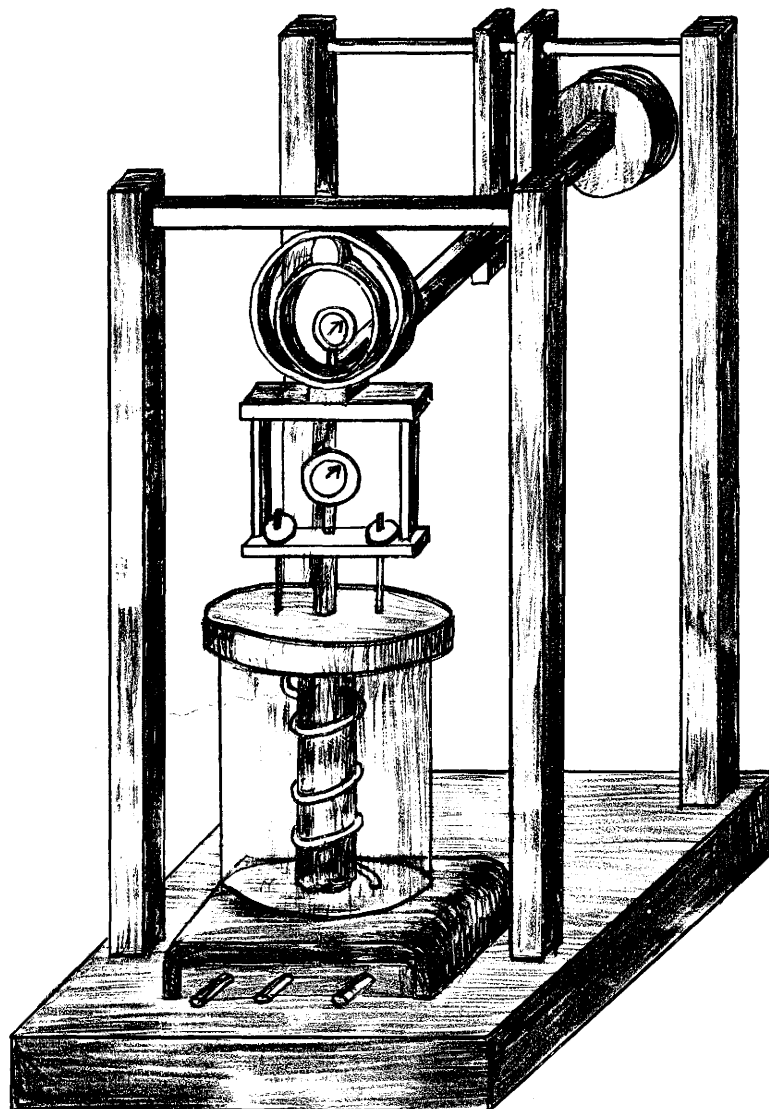


Figure 12. Triaxial shear test apparatus

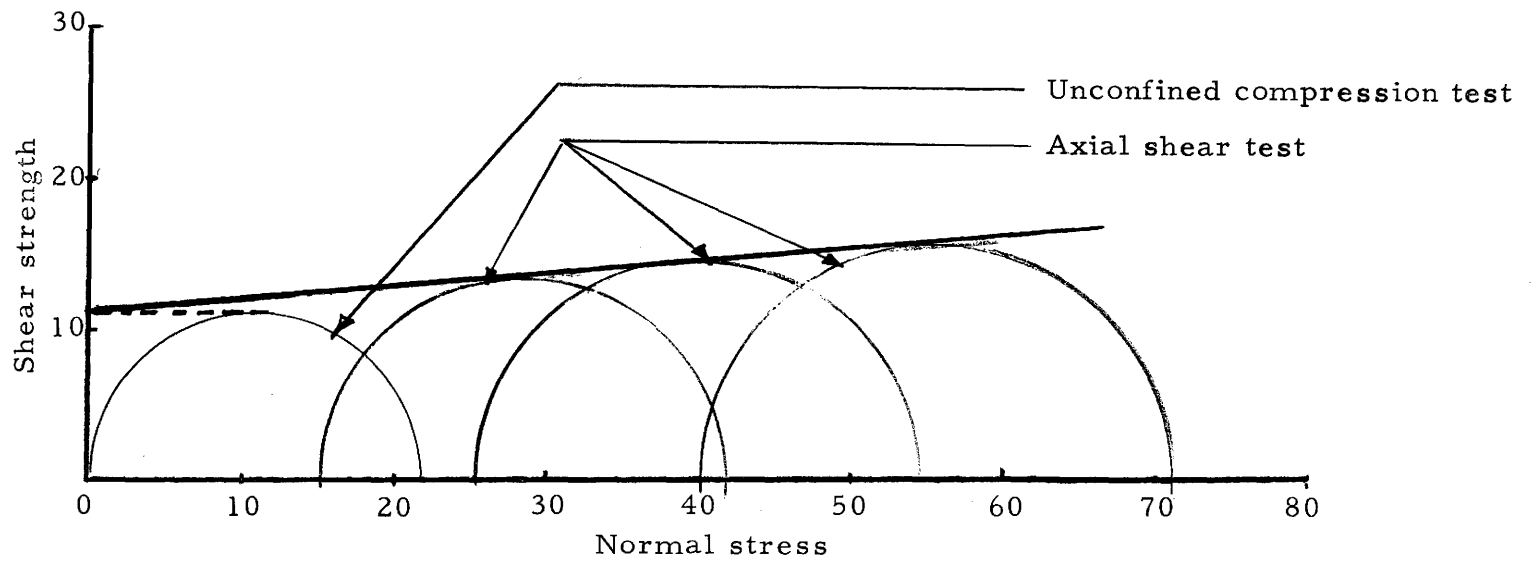


Figure 13. Strength envelope for Na^+ -saturated sample compacted with distilled water. A comparison between the results obtained from both triaxial shear test and unconfined compression test is shown

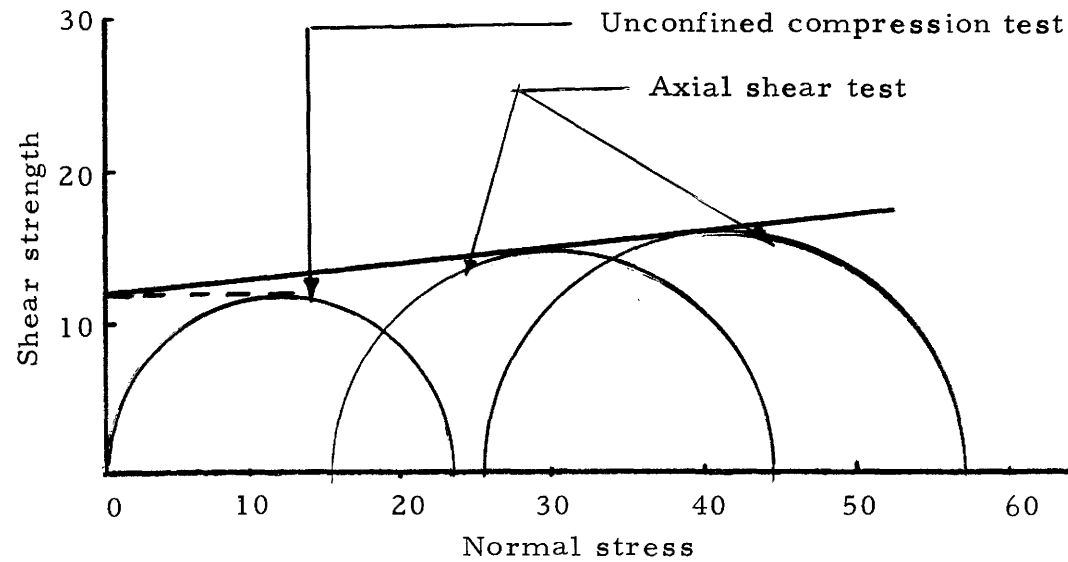


Figure 14. Strength envelope for Na^+ -saturated soil compacted with 0.1 N NaCl. A comparison between the results obtained from both triaxial shear test and unconfined compression test is shown

Table 4. A comparison between shear strength values obtained from triaxial and unconfined compression tests

Pore fluid	Triaxial test				Unconfined test		
	Compacted density	Water content	Lateral pressure	Shear strength	Compacted density	Water content	Shear strength
	lb/ft ³	%	psi	psi	lb/ft ³	%	psi
Distilled water	125.2	21.2	15.0	11.0	124.2	21.5	11.0
	124.8	21.6	25.0				
	125.0	21.3	40.0				
0.1 N. NaCl solution	126.0	21.3	15.0	11.0	125.3	21.3	11.5
	124.9	21.4	25.0				

EXPERIMENTAL RESULTS

Experimental results are presented under two headings: (1) Plastic properties, and (2) Strength properties. Plastic properties consist mainly of liquid and plastic limits data, whereas, strength properties are based on the results of the unconfined compression tests.

Plastic Properties

Liquid limit

Liquid limit values are plotted as a function of salt concentration in the pore fluid (Figures 15 to 19). An increase of the soluble salt concentration resulted in a decrease in liquid limit for all samples saturated with Na^+ , K^+ , Ca^{++} , Mg^{++} , and H^+ . A sharp decrease of liquid limit values is seen in the case of Na^+ - and K^+ -saturated samples (Figures 15 and 16), whereas a gradual decrease is observed for Ca^{++} -, Mg^{++} -, and H^+ -saturated samples (Figures 17 to 19).

As shown in Figure 20, an increase of the exchangeable sodium percentage increases the liquid limit values. It is clear that the liquid limit value dropped when exchangeable Na^+ was replaced by exchangeable K^+ , but increased passing from K^+ to Ca^{++} , Mg^{++} , and H^+ ; i. e., liquid limit is in the order $\text{Na}^+ > \text{H}^+ > \text{Mg}^{++} > \text{Ca}^{++} > \text{K}^+$.

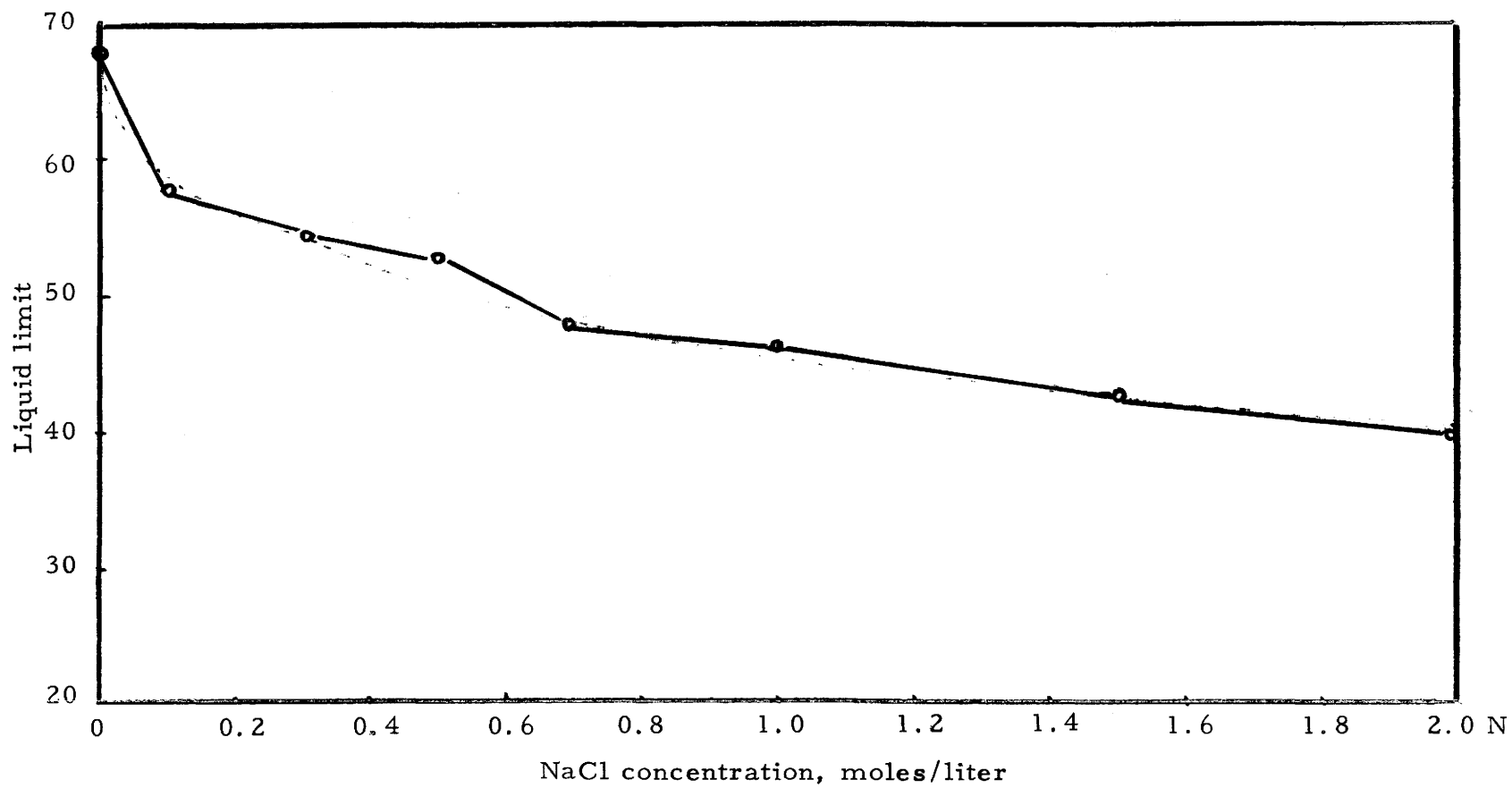


Figure 15. Liquid limit of Na^+ -saturated soil plotted as a function of NaCl concentration

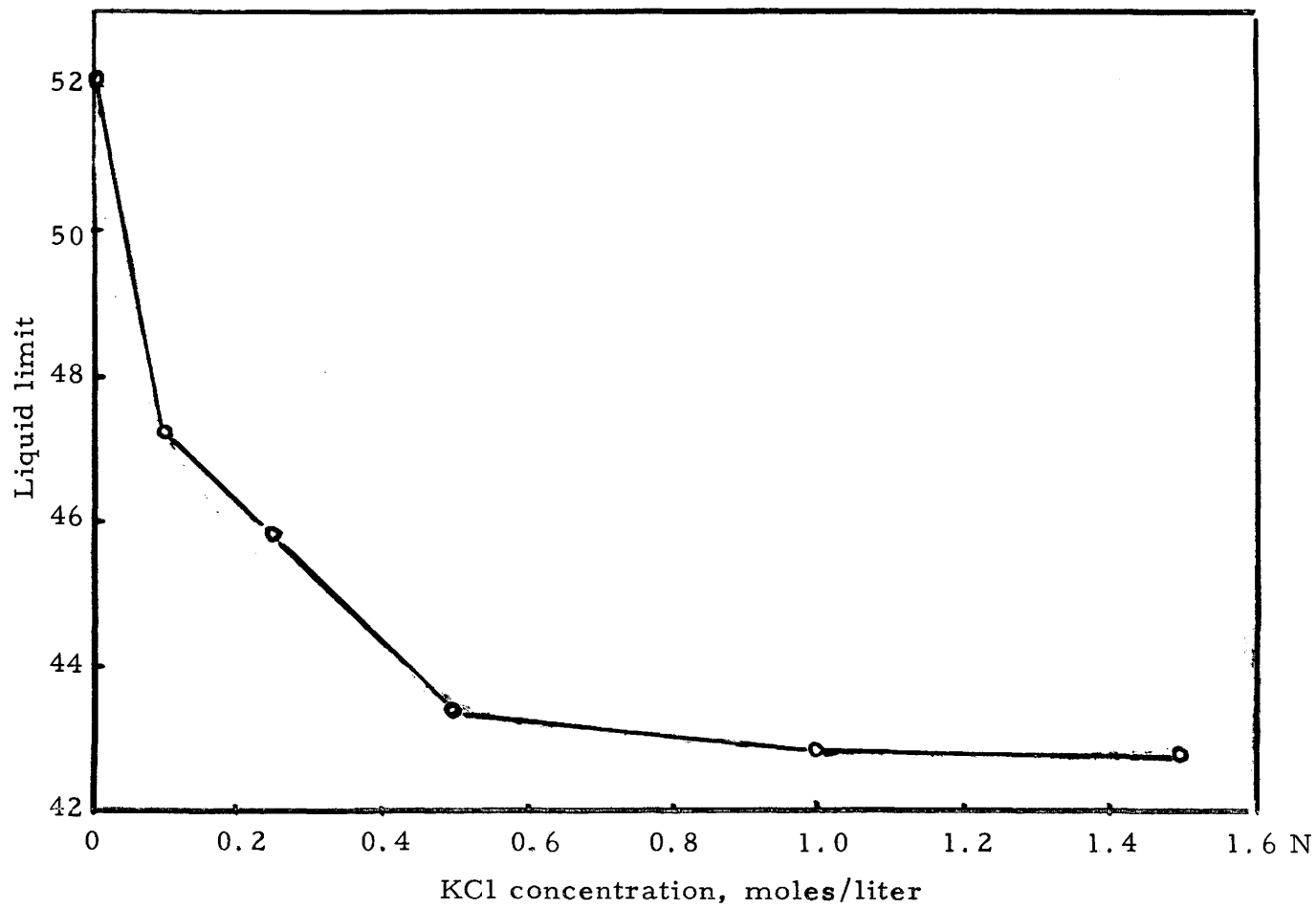


Figure 16. Liquid limit of K^+ -saturated soil plotted as a function of KCl concentration

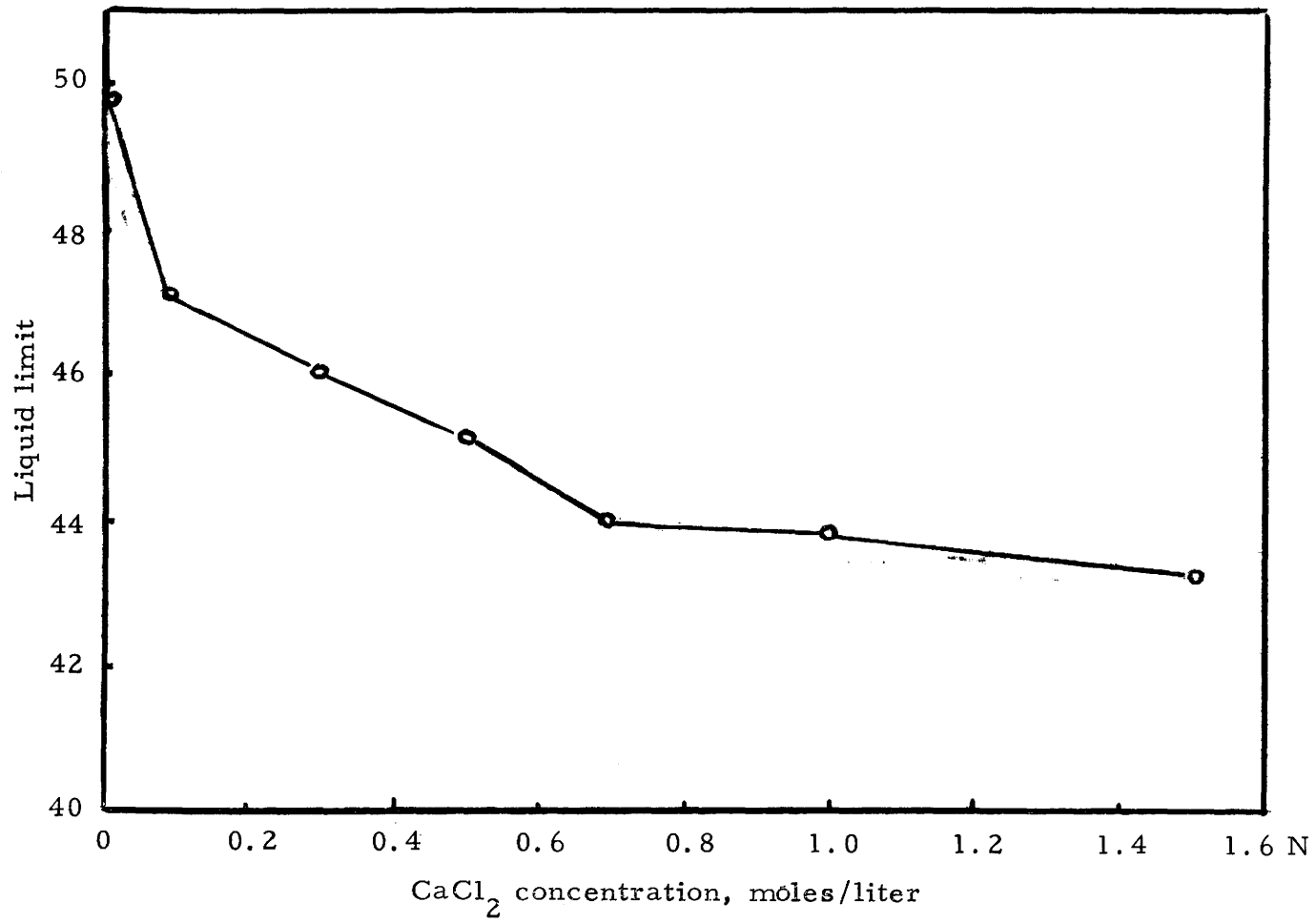


Figure 17. Liquid limit of Ca⁺⁺-saturated soil plotted as a function of CaCl₂ concentration

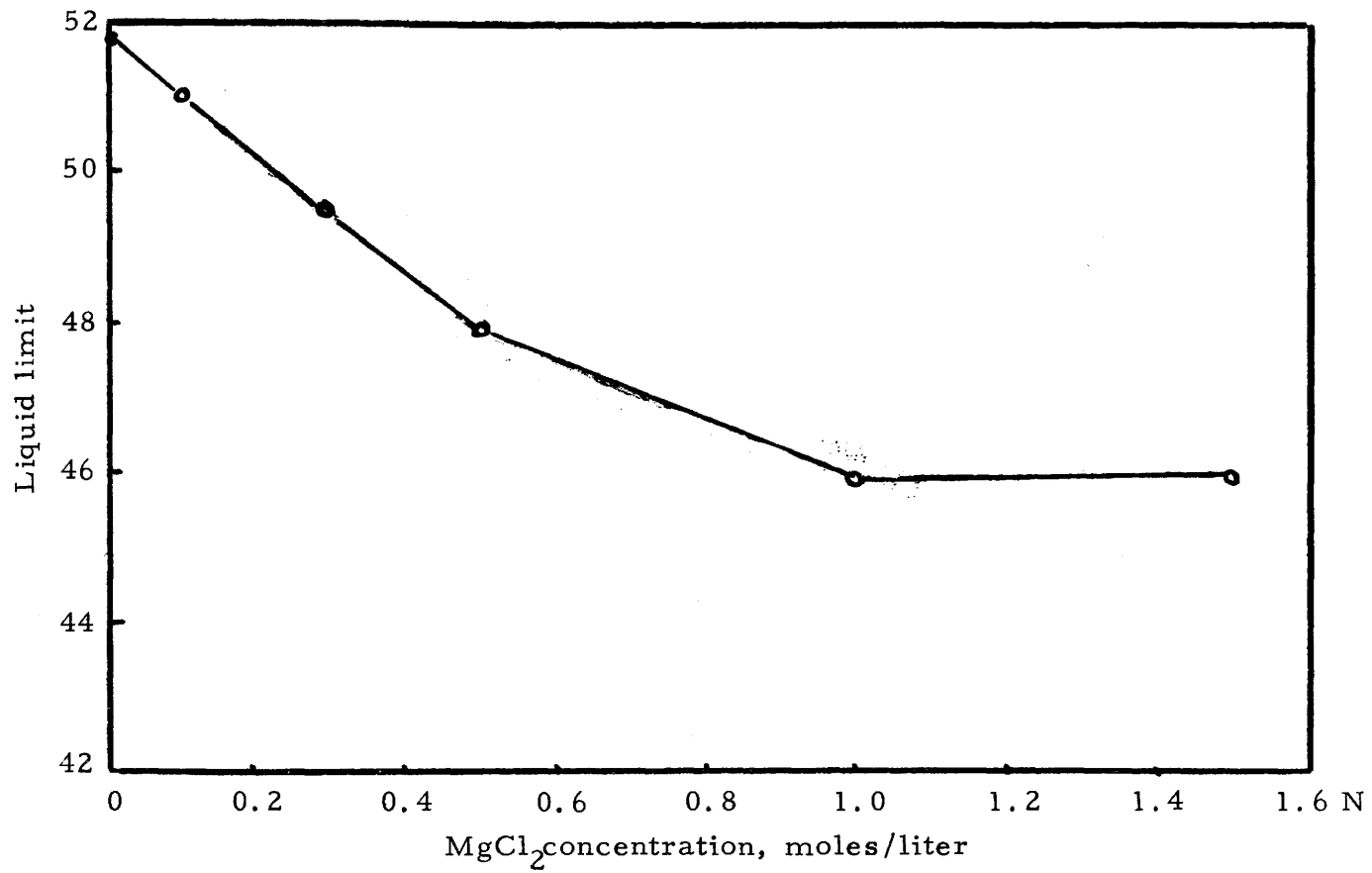


Figure 18. Liquid limit of Mg⁺⁺-saturated soil plotted as a function of MgCl₂ concentration

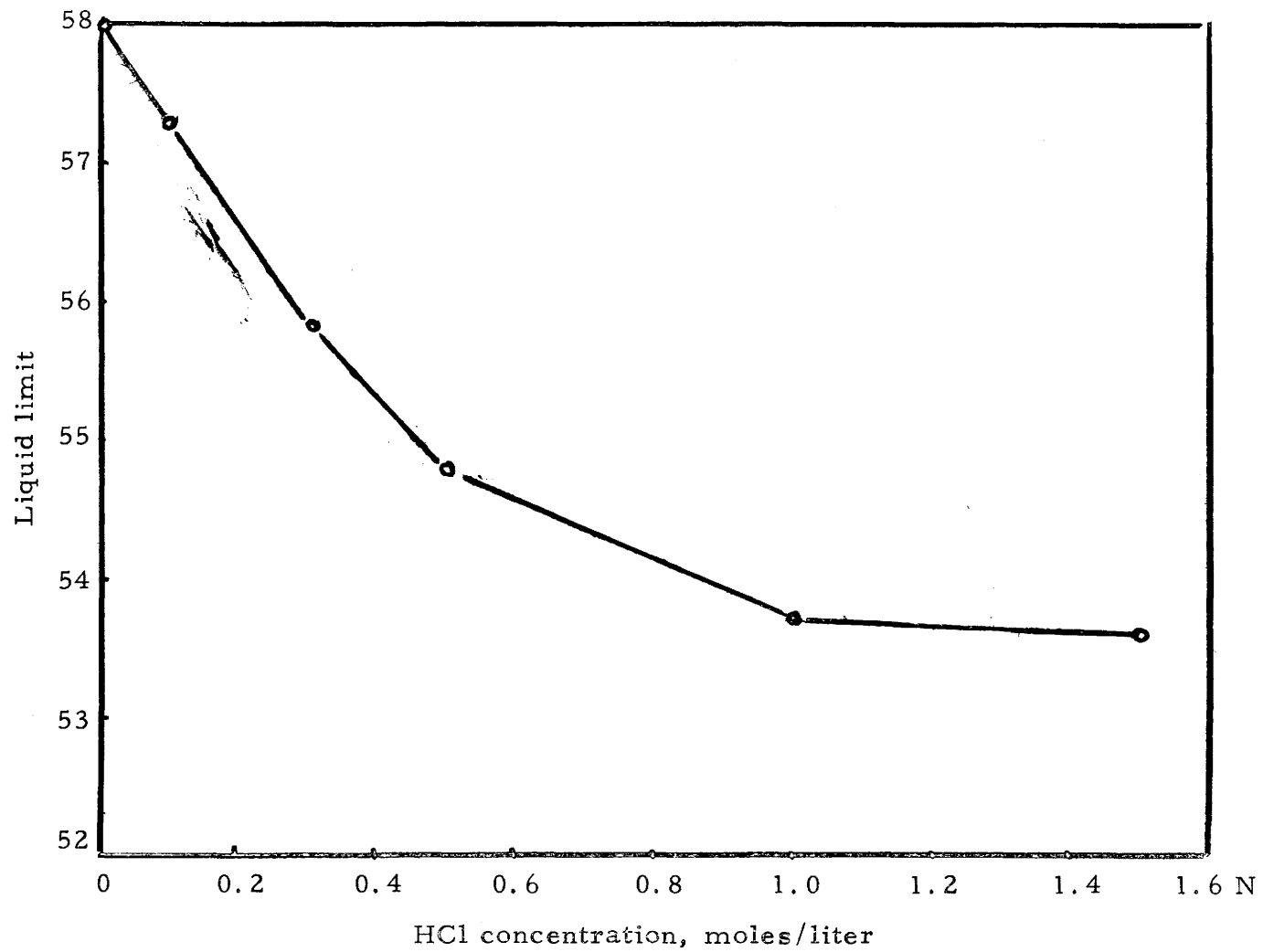


Figure 19. Liquid limit of H⁺-saturated sample plotted as a function of HCl concentration

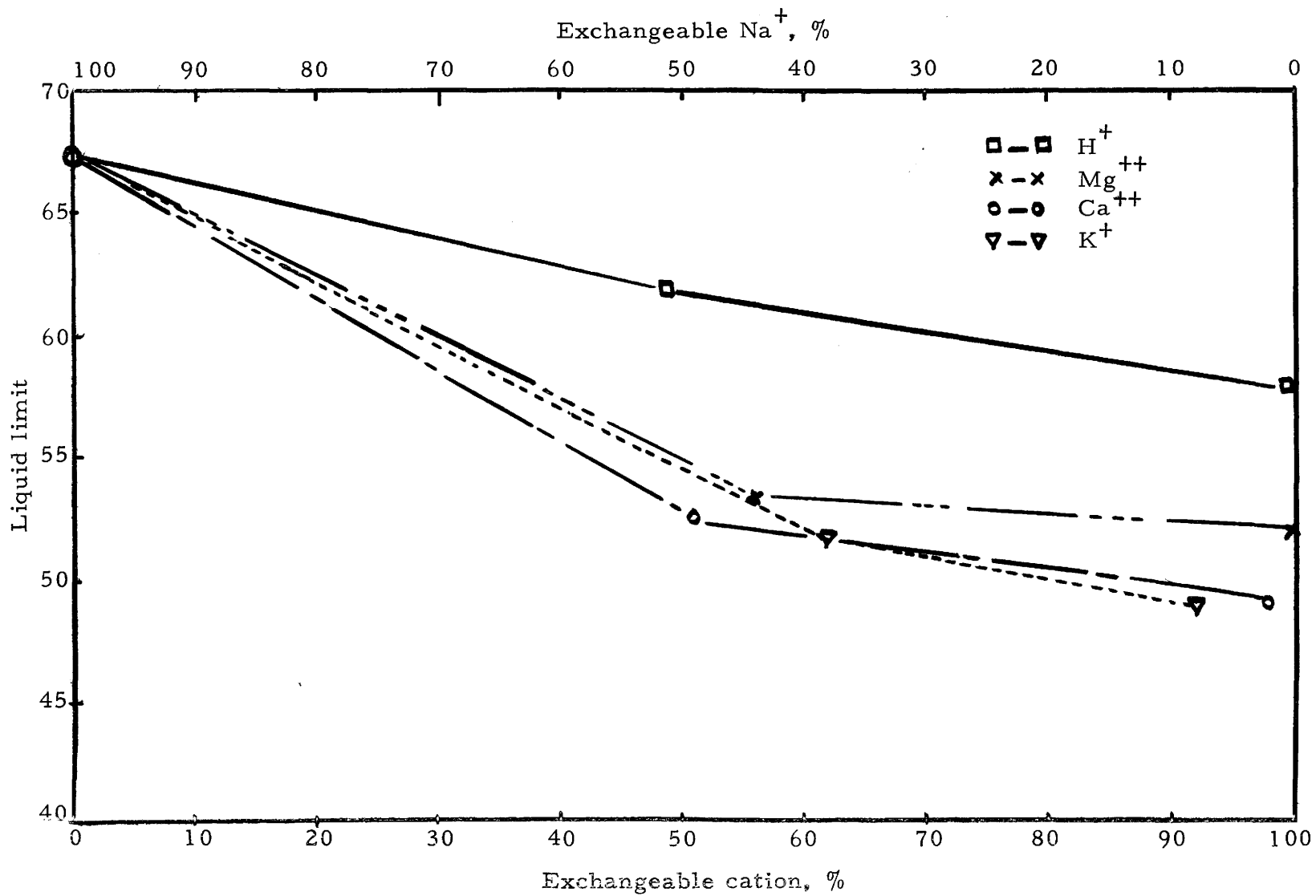


Figure 20. Liquid limit plotted as a function of exchangeable sodium percentage. The effect of different exchangeable cations as they replace Na⁺ is indicated

Plastic limit

Although liquid limit values showed a progressive decrease with increasing pore fluid concentration, plastic limit did not vary significantly (Table 5). Hence, plastic index (liquid limit - plastic limit) decreased with soluble salt concentration in the sample.

The effect of exchangeable cations on the plastic limit (Figure 21) shows in general that K^+ ion causes a lowering of plastic limit and a decrease in plasticity index. Mg^{++} and Ca^{++} ions produced a rather similar effect in lowering plastic limit, but not as much as K^+ ion. H^+ ion showed a peculiar effect on the soil sample. (The texture and structure of the HCl treated samples changed* in such a way that plastic limit could not be determined.)

Strength Properties

Stress-strain curves

Typical stress-strain curves (Figures 22 to 26) are shown for samples saturated with Na^+ , K^+ , Ca^{++} , Mg^{++} , and H^+ , respectively. Comparing stress-strain curves within the Na^+ or K^+ series of tests (Figures 22 and 23), it can be seen that the failure strain for samples compacted at high pore fluid concentration is relatively small, while for samples compacted with distilled water, failure was at higher strains.

*Reference is made to Figure 57.

Table 5. Plastic limit of soil as affected by the type of ion and soluble salt concentration

Soil sample	Plastic limit						
	Pore fluid concentration						
	Distilled water	0.1 N	0.3 N	0.5 N	0.7 N	1.0 N	1.5 N
Na ⁺ -saturated	33.8	33.6	34.0	33.8	33.5	33.9	34.0
K ⁺ -saturated	31.9	31.4	31.6	32.1	32.0	31.7	31.7
Ca ⁺⁺ -saturated	32.7	32.9	32.7	32.4	33.0	32.9	32.5
Mg ⁺⁺ -saturated	32.8	32.6	32.5	32.6	32.2	32.0	32.7
(Na ⁺ + K ⁺)-saturated	32.6						
(Na ⁺ + Ca ⁺⁺)-saturated	33.4						
(Na ⁺ + Mg ⁺⁺)-saturated	33.3						

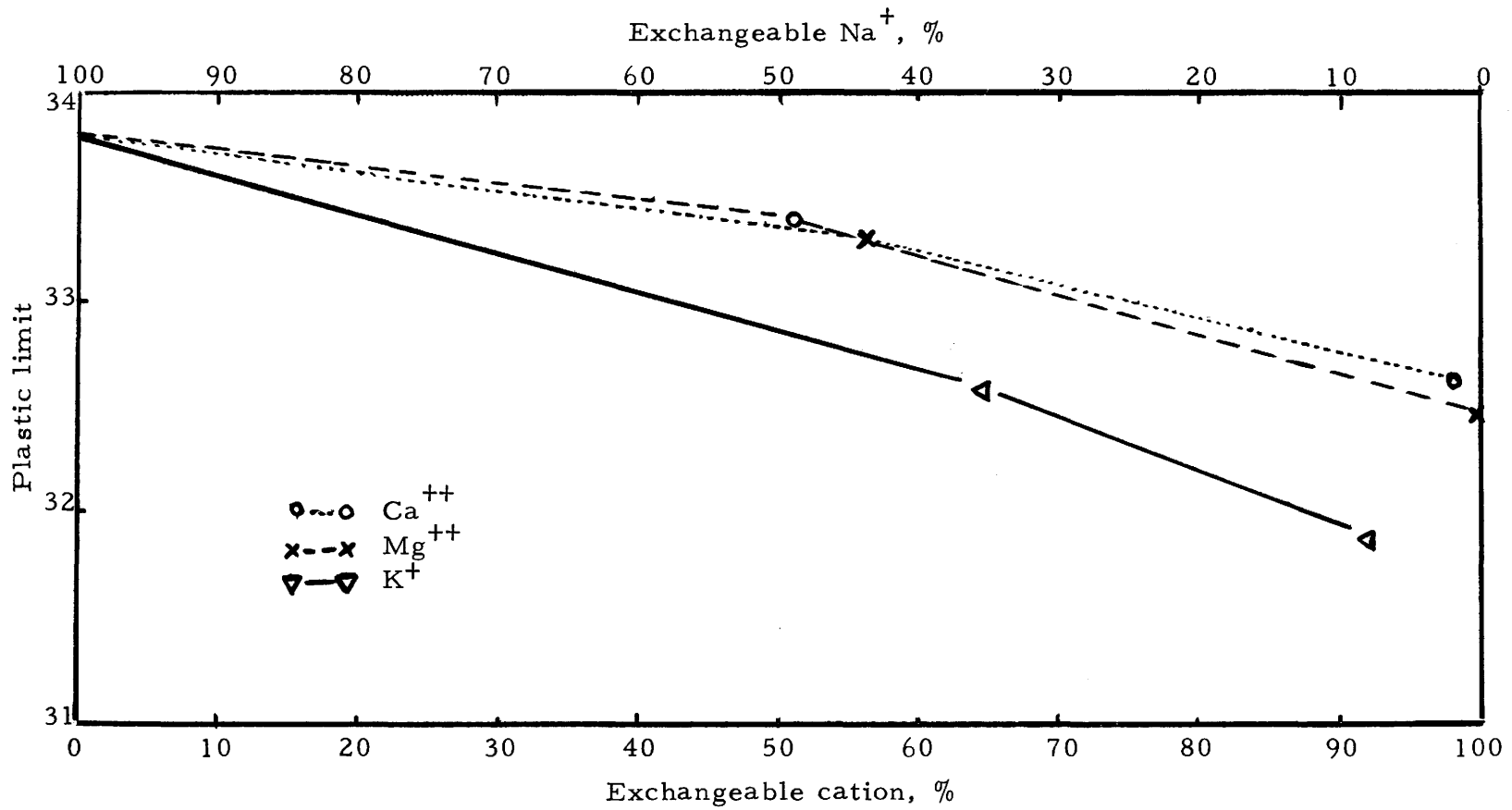


Figure 21. Plastic limit plotted as a function of exchangeable sodium percentage. The effect of different exchangeable ions as they replace Na⁺ is indicated

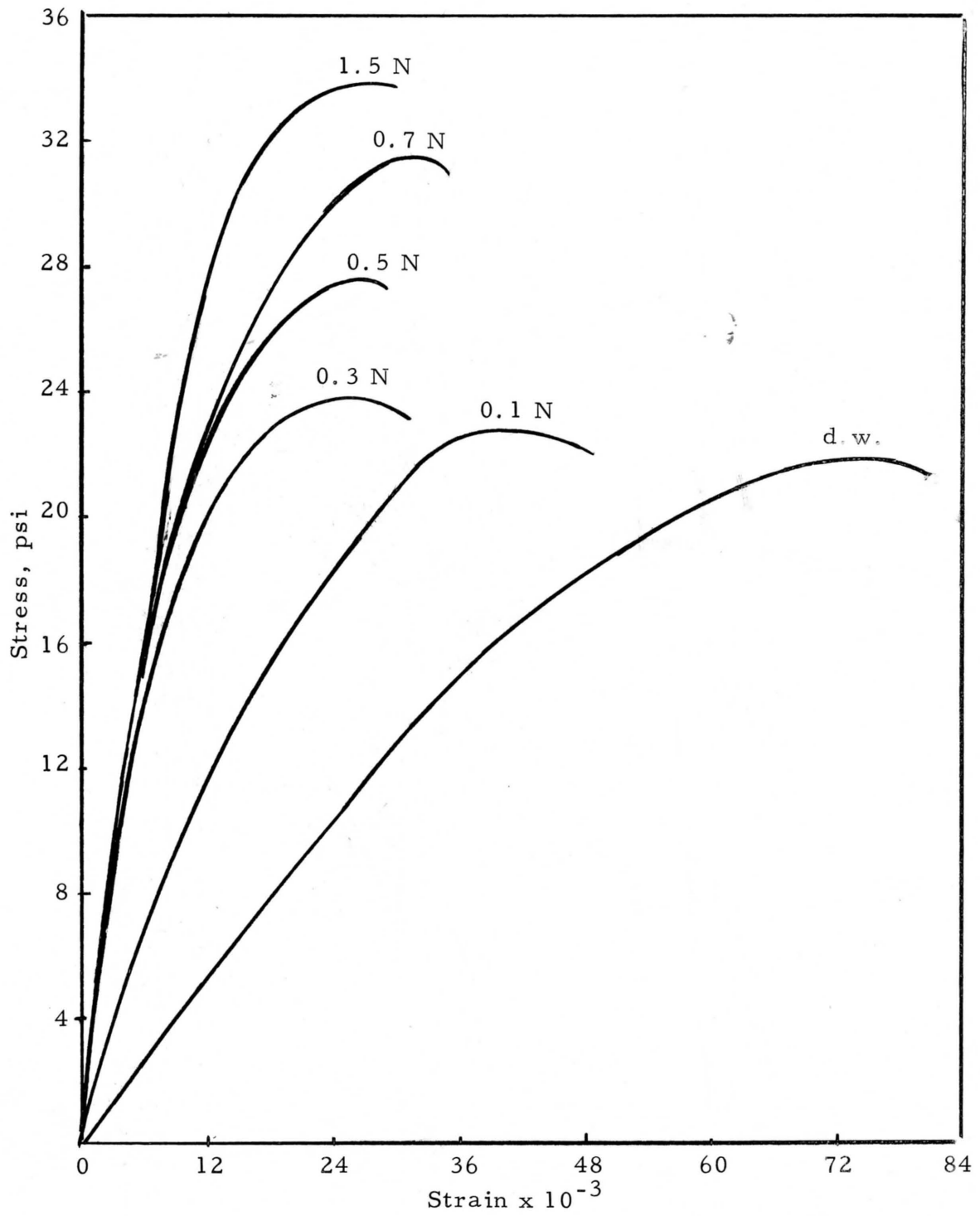


Figure 22. Stress-strain curves for Na⁺-saturated soil compacted with distilled water (d.w.) or different concentrations of NaCl solution as indicated on the plots

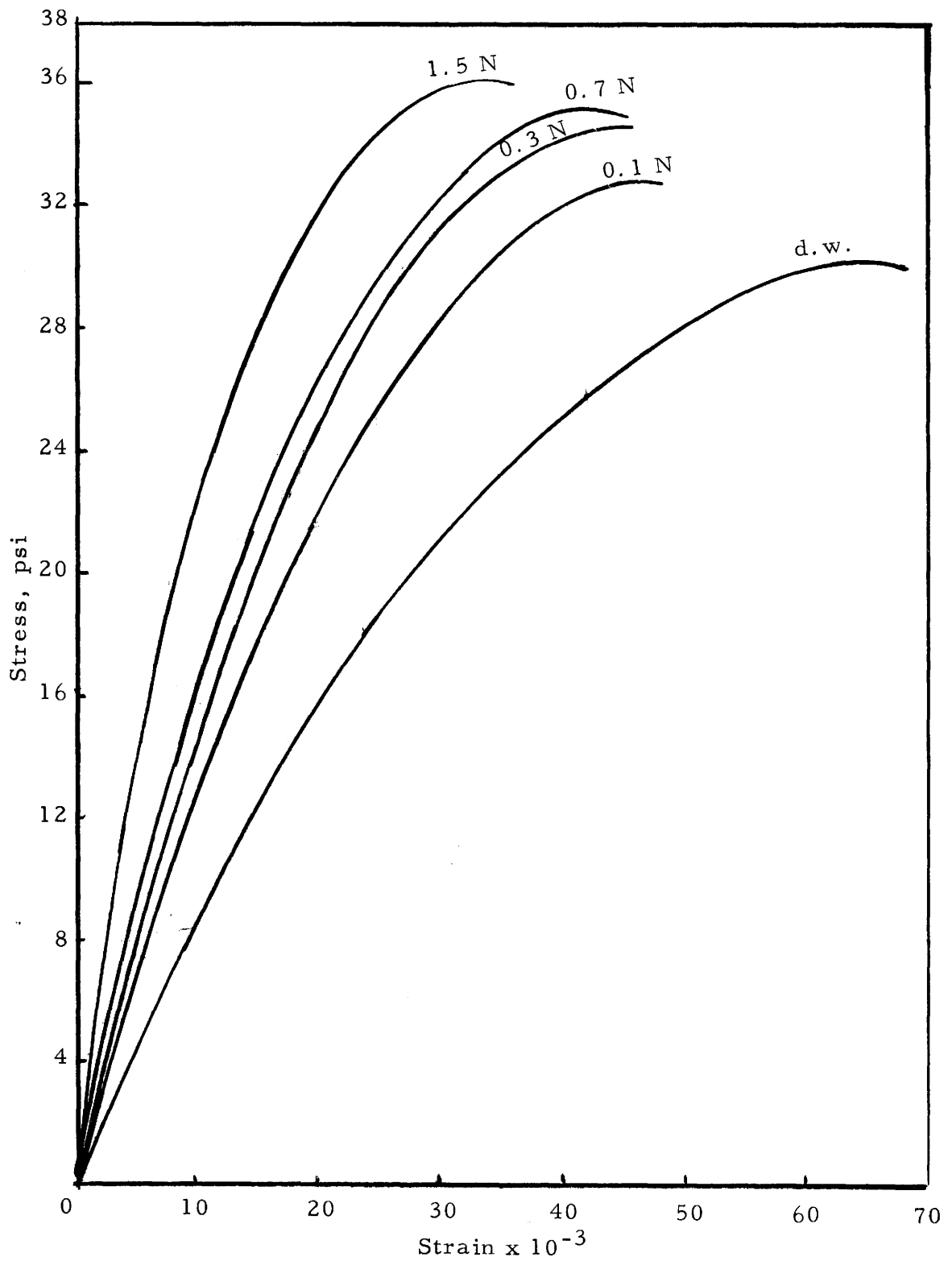


Figure 23. Stress-strain curves for K^+ -saturated soil, compacted with distilled water (d.w.) or different concentrations of KCl solutions as indicated on the plots

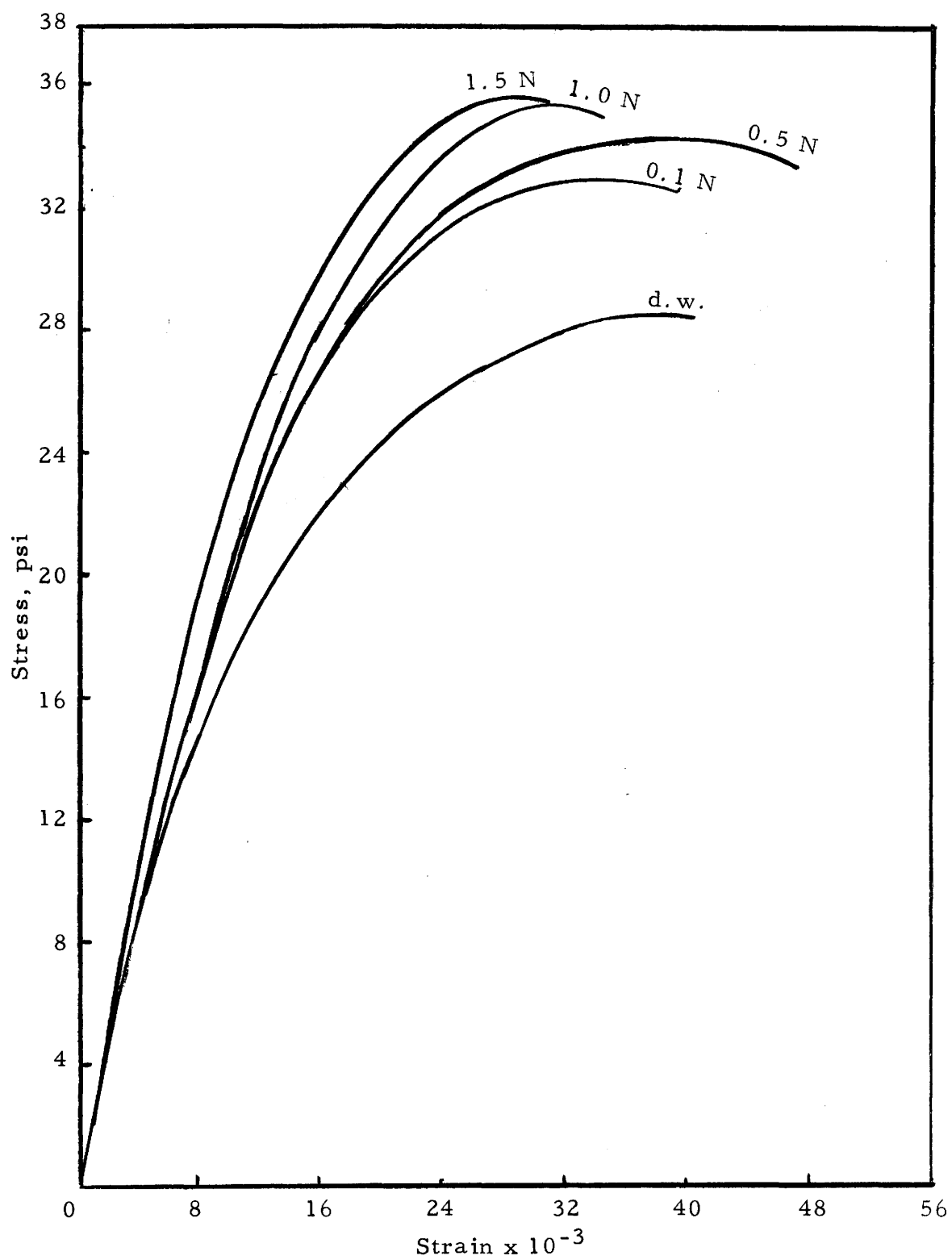


Figure 24. Stress-strain curves for Ca⁺⁺-saturated soil, compacted with distilled water (d.w.) or different solutions of CaCl₂ as indicated on the plots

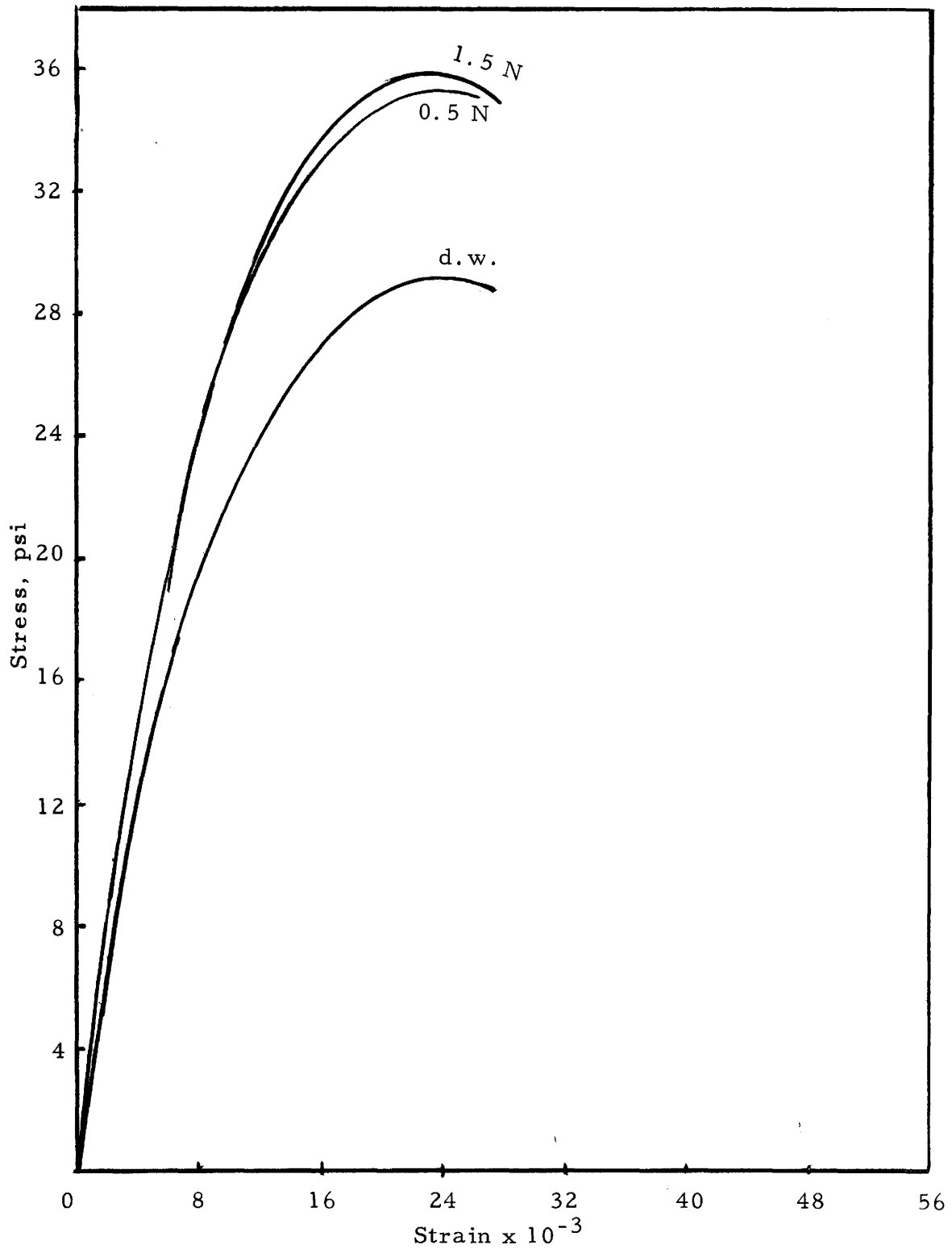


Figure 25. Stress-strain curves for Mg^{++} -saturated soil, compacted with distilled water (d.w.) or different concentrations of $MgCl_2$ solutions as indicated on the plots

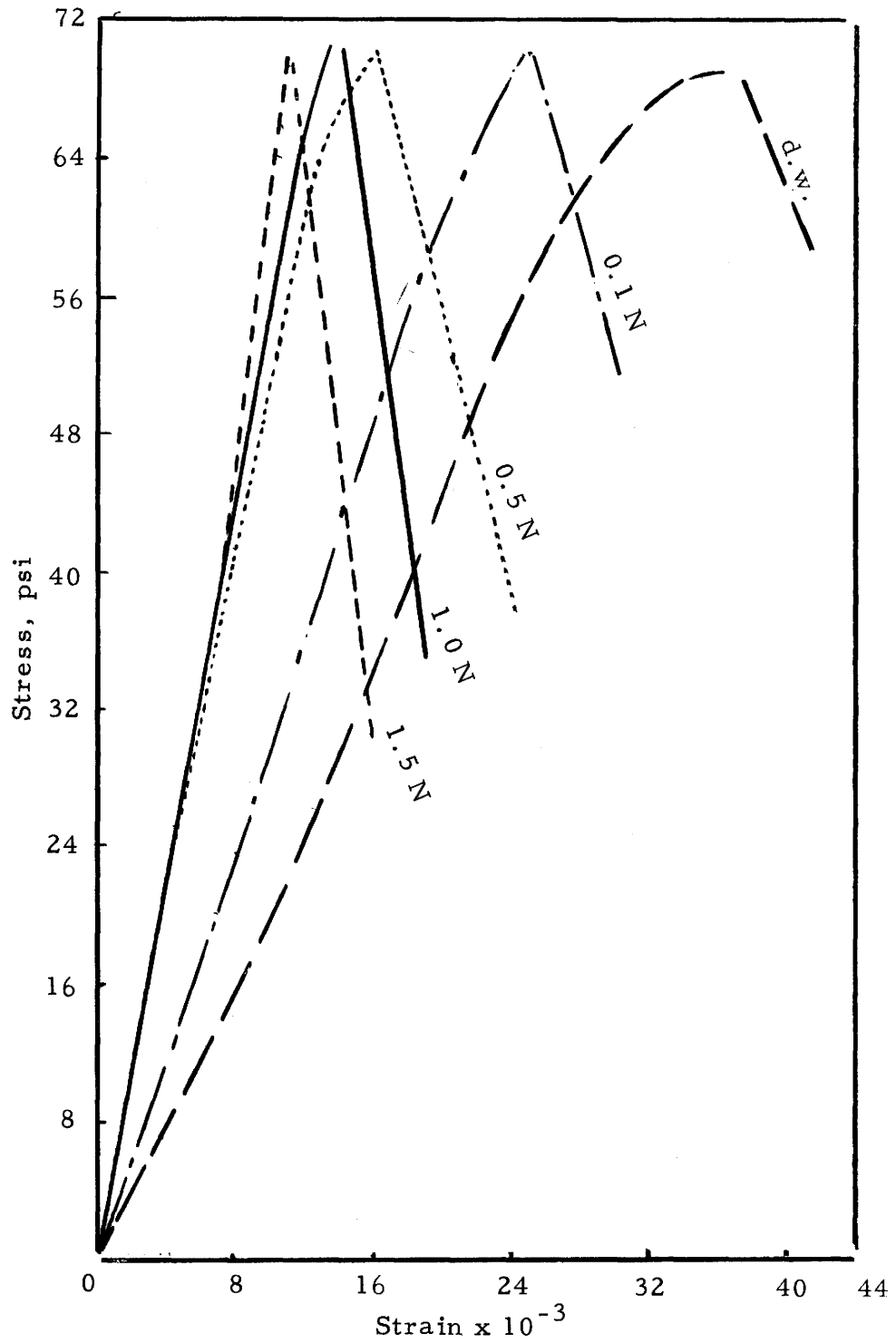


Figure 26. Stress-strain curves for H^+ -saturated soil compacted with distilled water (d.w.) or different concentrations of HCl solutions as indicated on the plots

For Mg^{++} - and H^+ -saturated samples (Figures 25 and 26), the stress-strain curves are almost straight lines (particularly with high salt concentration in the pore fluid) from zero to shear stresses approaching failure. The failure strains are exceptionally small for H^+ -saturated samples with HCl concentration in the pore fluid more than 0.5 N. These strains were about 1.0 to 1.6 percent (Figure 26). It is clear (Figure 27) that Ca^{++} -saturated soil approaches failure in a fashion which is intermediate between that of K^+ - and Mg^{++} -saturated soils. Furthermore, the degree of curvature in stress-strain relations appears to decrease in the order $Na^+ > K^+ > Ca^{++} > Mg^{++} > H^+$, while the strain at which failure occurs for samples compacted with distilled water follows the order $Na^+ > K^+ > Ca^{++} > H^+ > Mg^{++}$ (e.g., for Na^+ -saturated sample failure strain was 7.5 percent, whereas with Mg^{++} -saturated sample it was 2.4 percent).

Stress-strain curves are drawn in Figures 28, 29, 30, and 31 to show the relative effect of exchangeable K^+ , Ca^{++} , Mg^{++} , and H^+ percentage compared to Na^+ -saturated sample, with distilled water used as the pore liquid in all cases. It can be seen that failure strain decreases with decreasing exchangeable Na^+ percentage. The decrease is relatively sharp where Na^+ ion on the soil clay fraction was replaced with Mg^{++} , H^+ , or Ca^{++} ions, whereas it is gradual in the case exchangeable K^+ was used to replace exchangeable Na^+ . This is further illustrated in Figure 32 where failure strain is plotted against exchangeable sodium percentage for different exchangeable ions.

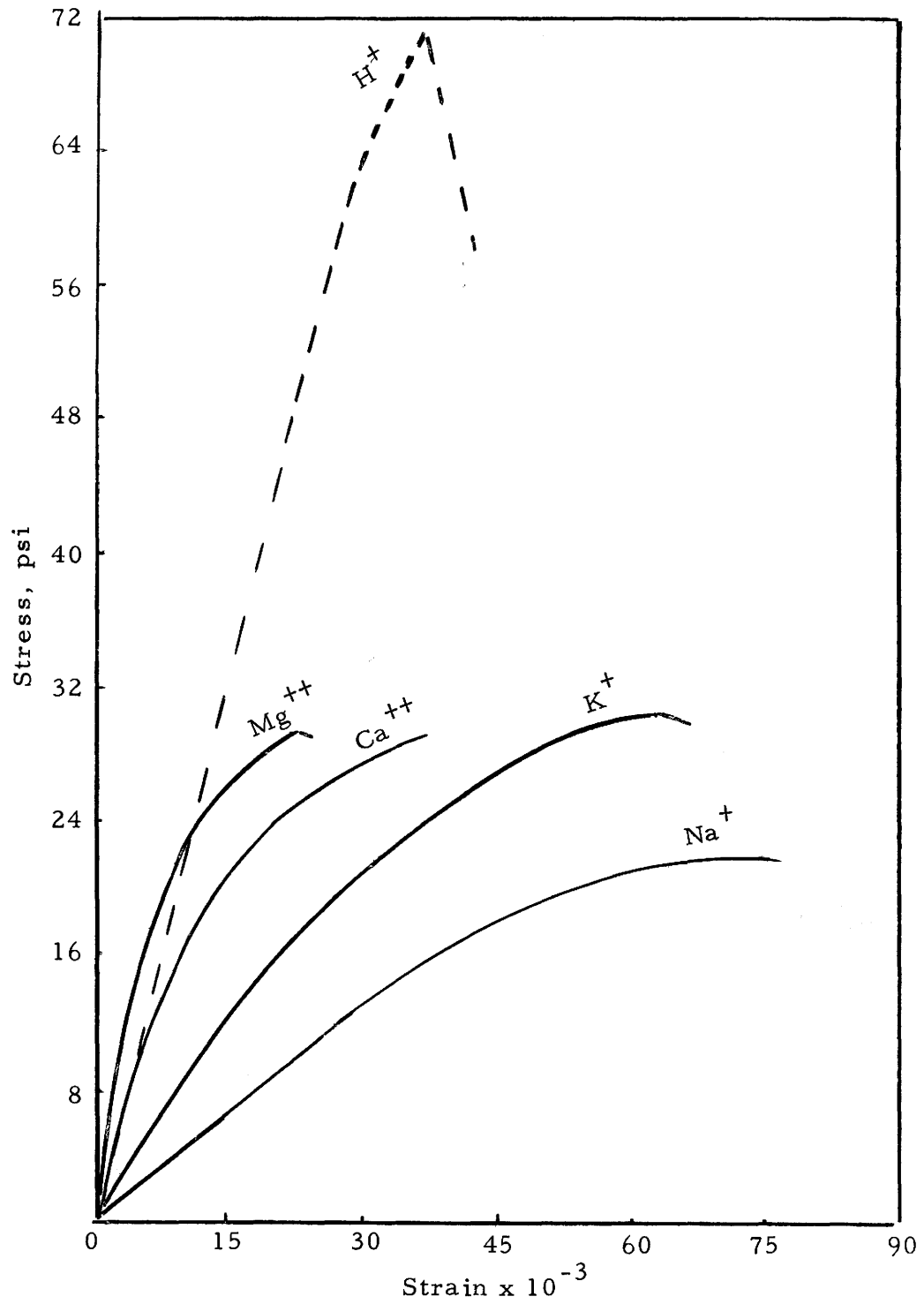


Figure 27. Stress-strain curves for different soil samples saturated with the denoted ions. Samples were compacted with distilled water as the pore fluid

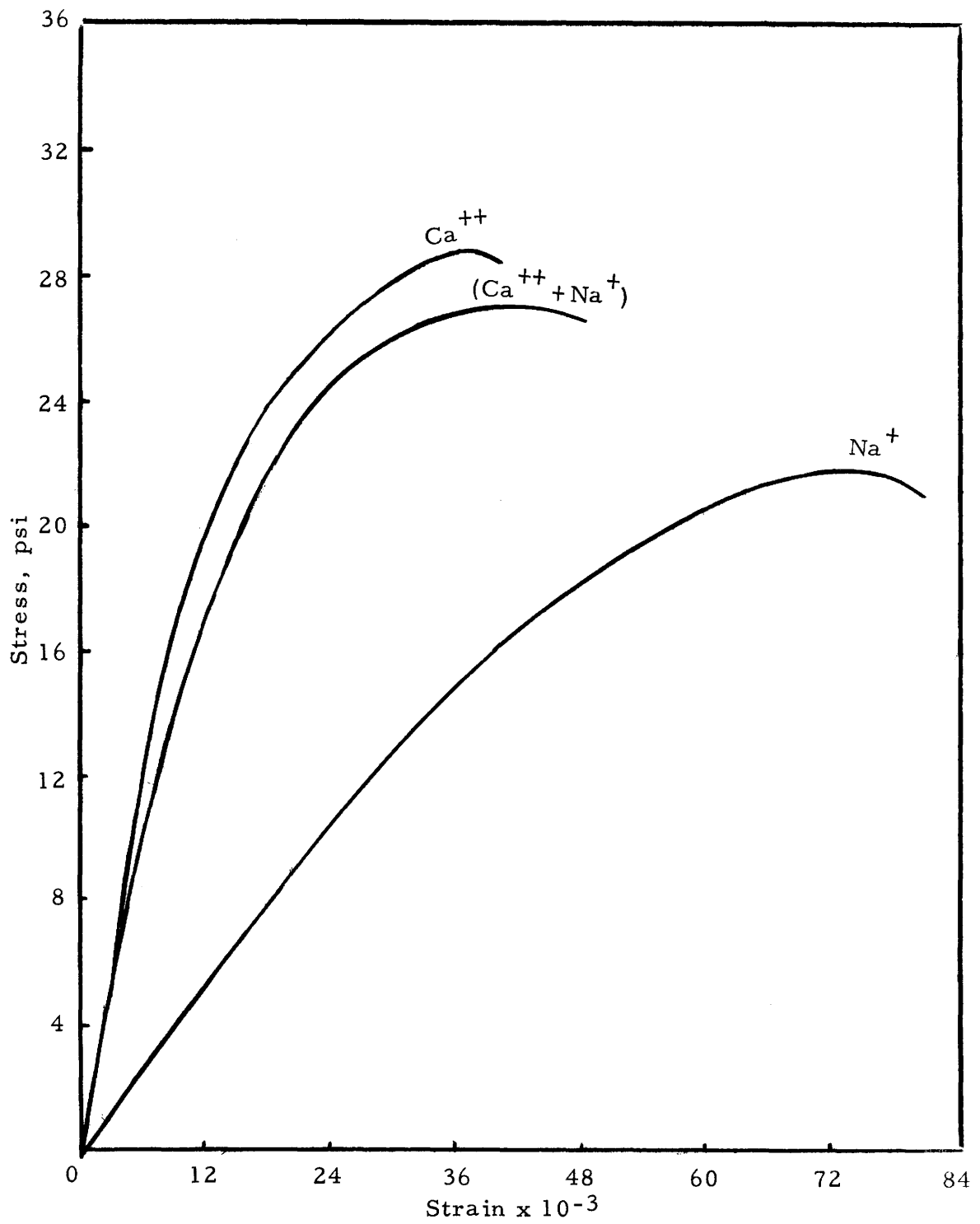


Figure 28. Stress-strain curves for Ca⁺⁺ - and (Ca⁺⁺ + Na⁺) - saturated soil compared with that of Na⁺ - saturated soil, all compacted with distilled water

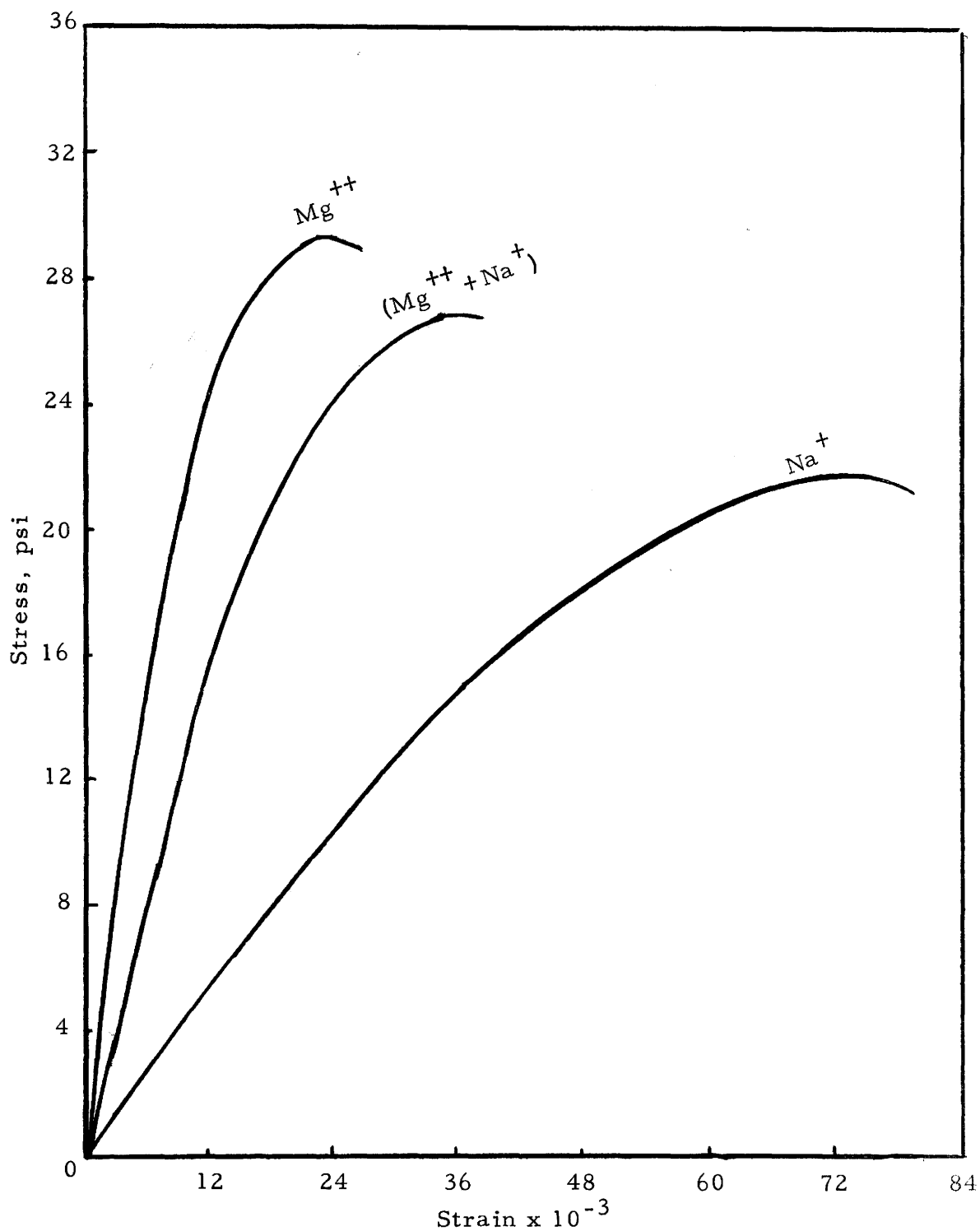


Figure 29. Stress-strain curves for Mg⁺⁺ - and (Mg⁺⁺ + Na⁺) - saturated soil compared with that of Na⁺ - saturated soil, all compacted with distilled water

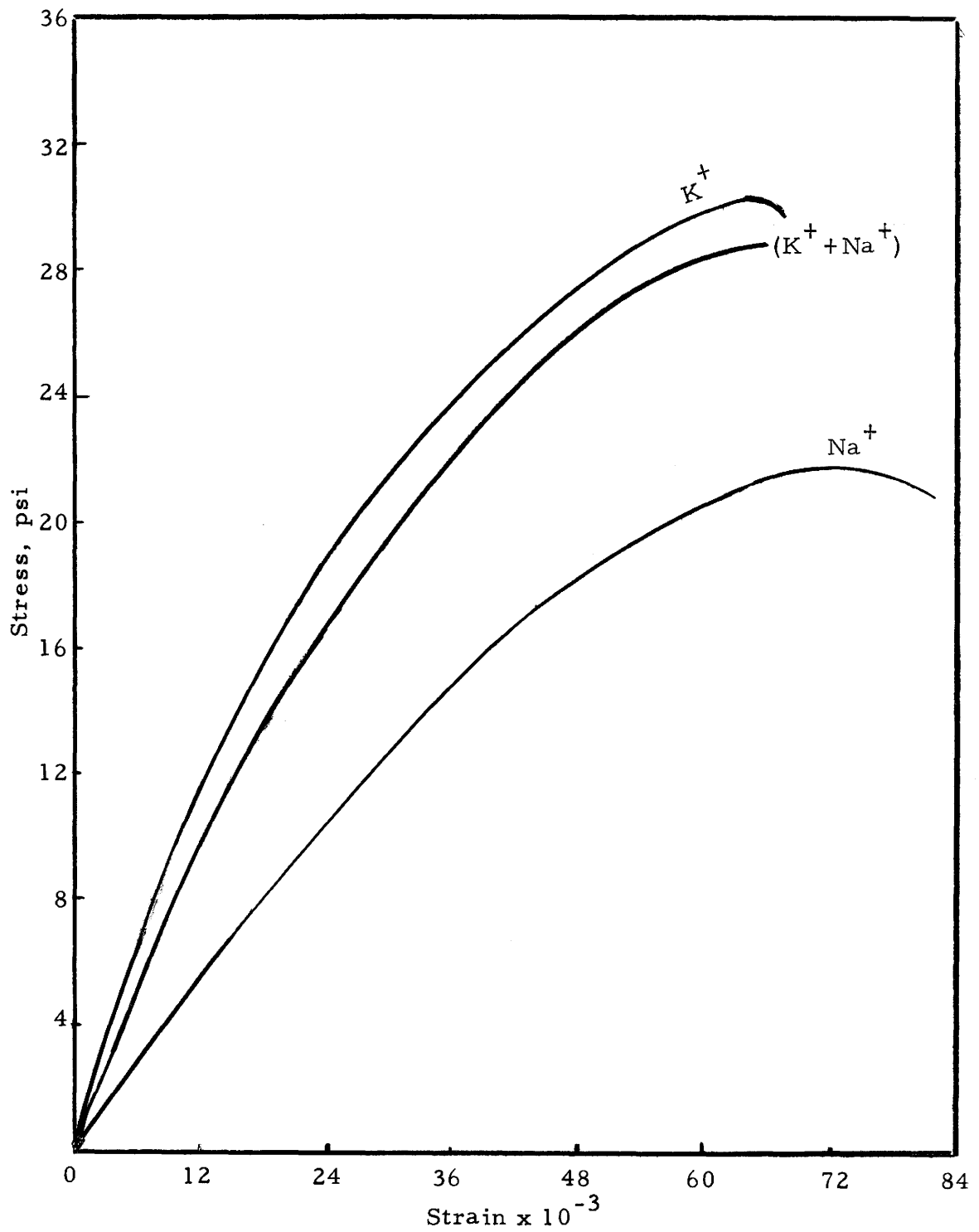


Figure 30. Stress-strain curves for K⁺ - and (K⁺ + Na⁺)-saturated soil compared with that of Na⁺-saturated soil, all compacted with distilled water

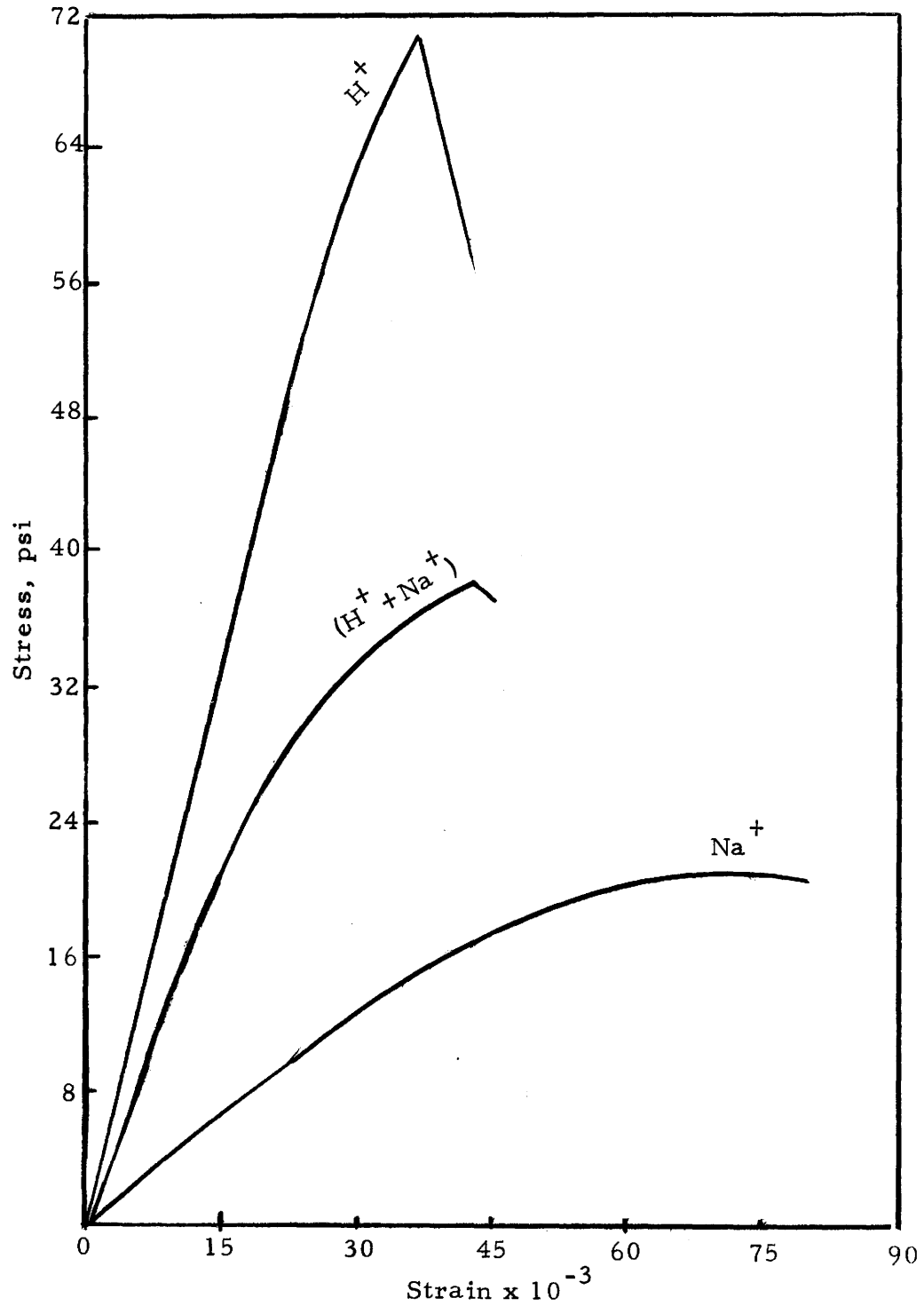


Figure 31. Stress-strain curves for H⁺- and (H⁺ + Na⁺)-saturated soil compared with that of Na⁺-saturated soil, all compacted with distilled water

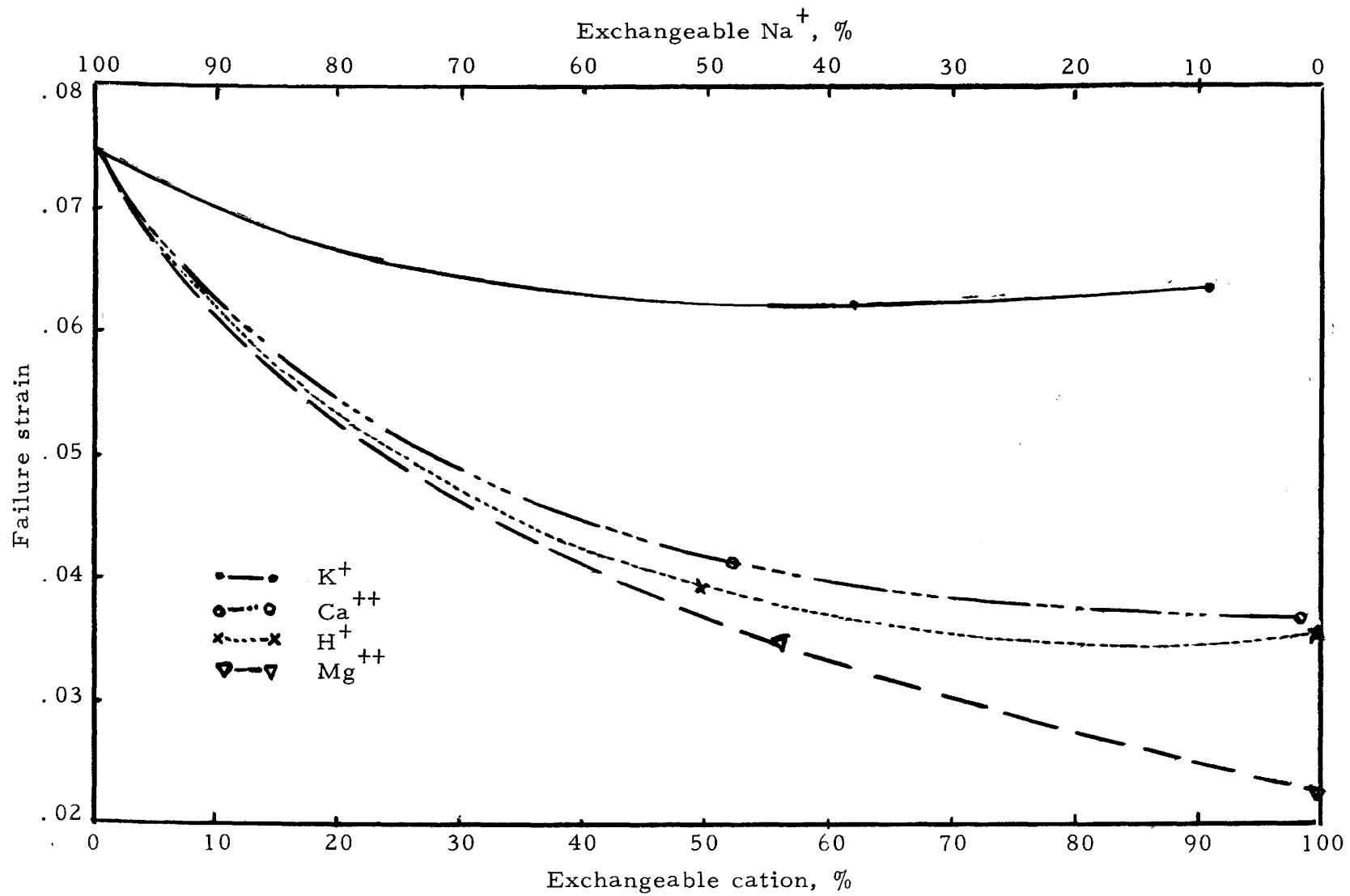


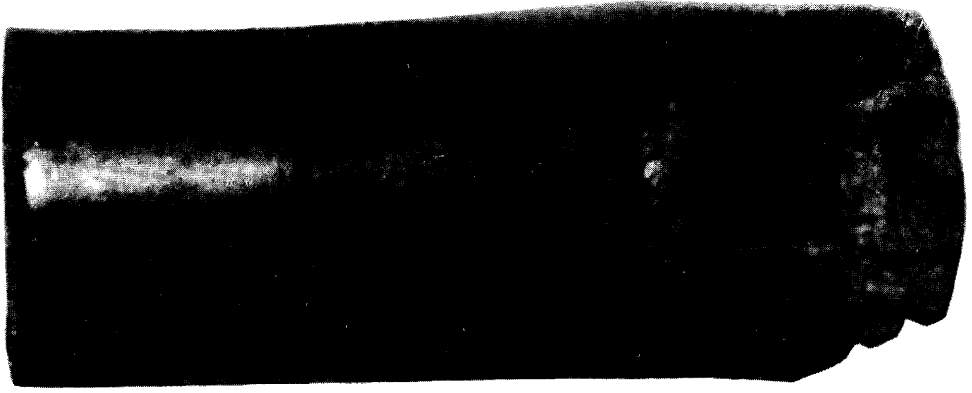
Figure 32. Failure strain plotted as a function of exchangeable sodium percentage. The effect of different exchangeable cations is indicated

A comparison between a H^+ -saturated sample and two Na^+ -saturated samples at failure is shown in Figure 33. It can be seen that the H^+ -saturated sample failed by cracking or splitting, whereas the Na^+ -saturated samples failed by flowing or bulging. Samples saturated with K^+ , Ca^{++} , and Mg^{++} showed intermediate behavior between that shown by Na^+ - and H^+ -saturated samples. More and deeper cracks developed in the order $H^+ > Mg^{++} > Ca^{++} > K^+ > Na^+$.

Modulus of elasticity

The values of Young's modulus E for different samples considered in the investigation varied from 800 to 11000 psi as shown in Table 6. The value of E was influenced by both the type of ion adsorbed on the clay fraction and the concentration of the pore fluid. Plots of E against salt concentration in the pore fluid are shown in Figures 34 to 38 for different samples with the specified ions. It is clear that for all samples the modulus of elasticity increases with soluble salt concentration in the pore water. However, E appears to be affected less by the pore liquid concentration in the cases of Mg^{++} - and Ca^{++} -saturated samples (divalent ions) than in the cases of Na^+ -, K^+ -, and H^+ -saturated samples (monovalent ions). A progressive increase of E with decreasing exchangeable sodium percentage is shown in Figure 39. The relationship in the case of an exchangeable Na^+ replaced by K^+ or H^+ is approximately a straight line. This relationship is not definite, however, because the number of the experimental

C



B



A



Table 6. Modulus of elasticity of soil as affected by the type of exchangeable ions and soluble salt concentration

Soil sample	Modulus of elasticity, psi						
	Distilled water	0.1 N	0.3 N	0.5 N	0.7 N	1.0 N	1.5 N
Na ⁺ -saturated	856	1820	3660	4000	4660	4660	4660
K ⁺ -saturated	1640	2400	2800	3100	3500	4000	4800
Ca ⁺⁺ -saturated	660	4000	4000	4000	4000	4000	5000
Mg ⁺⁺ -saturated	5000	5400	5900	6450	6450	6450	6450
H ⁺ -saturated	4400	6000	8400	10000	10200	10600	11000
(Na ⁺ + K ⁺)-saturated	1466						
(Na ⁺ + Ca ⁺⁺)-saturated	3000						
(Na ⁺ + Mg ⁺⁺)-saturated	2660						
(Na ⁺ + H ⁺)-saturated	2840						

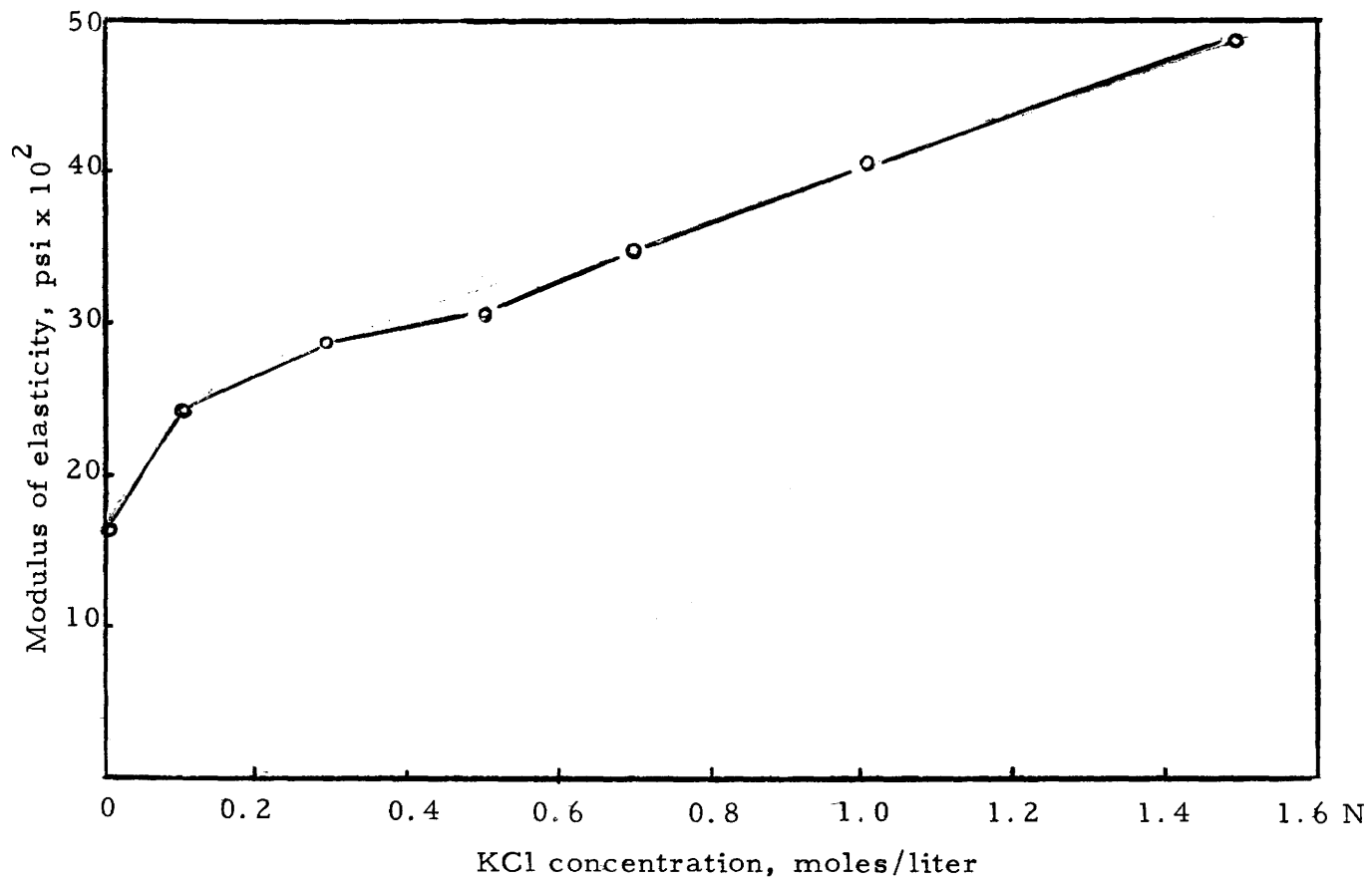


Figure 34. Modulus of elasticity of K^+ -saturated soil plotted as a function of KCl concentration

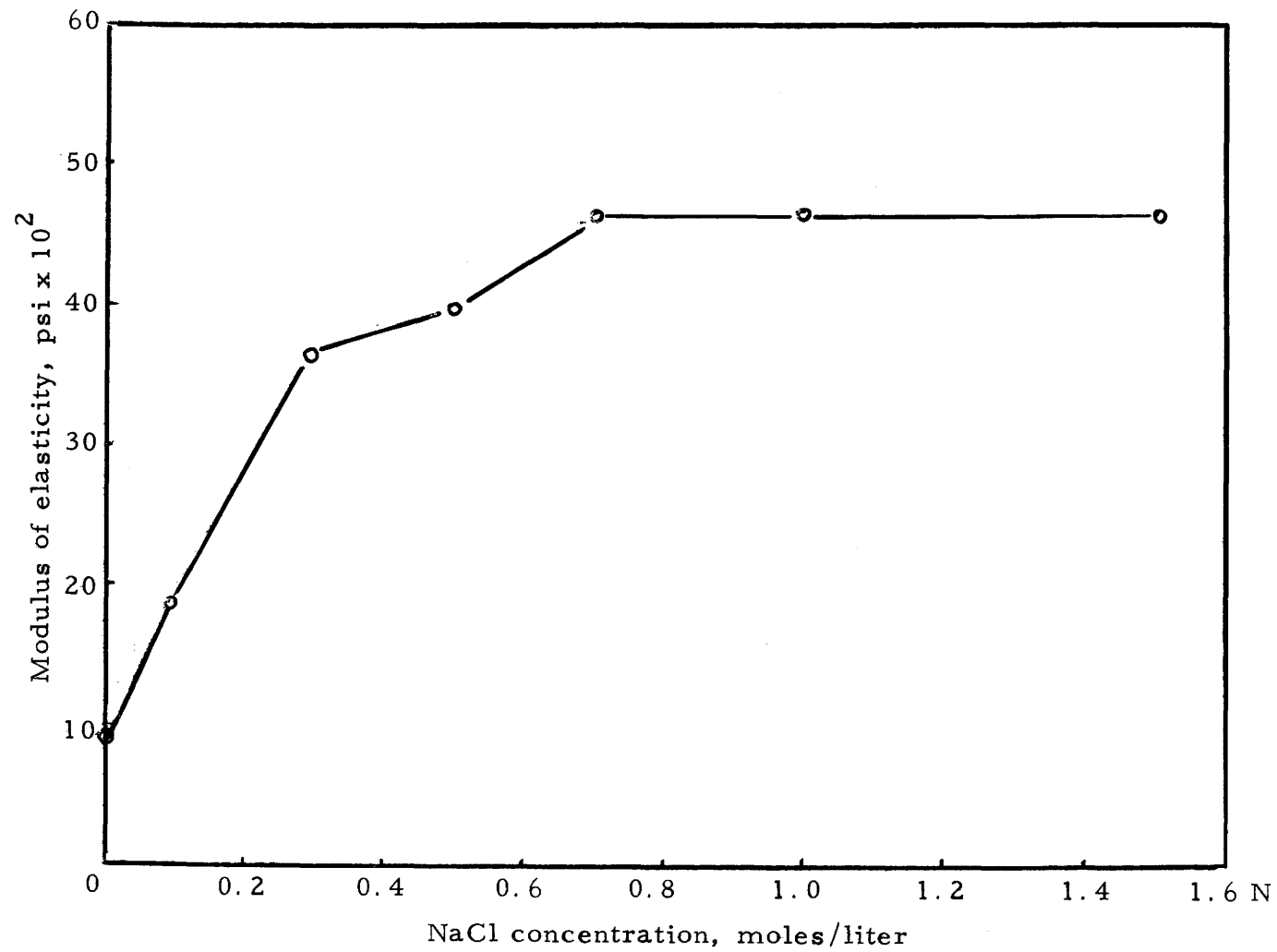


Figure 35. Modulus of elasticity of Na^+ -saturated soil plotted as a function of NaCl concentration

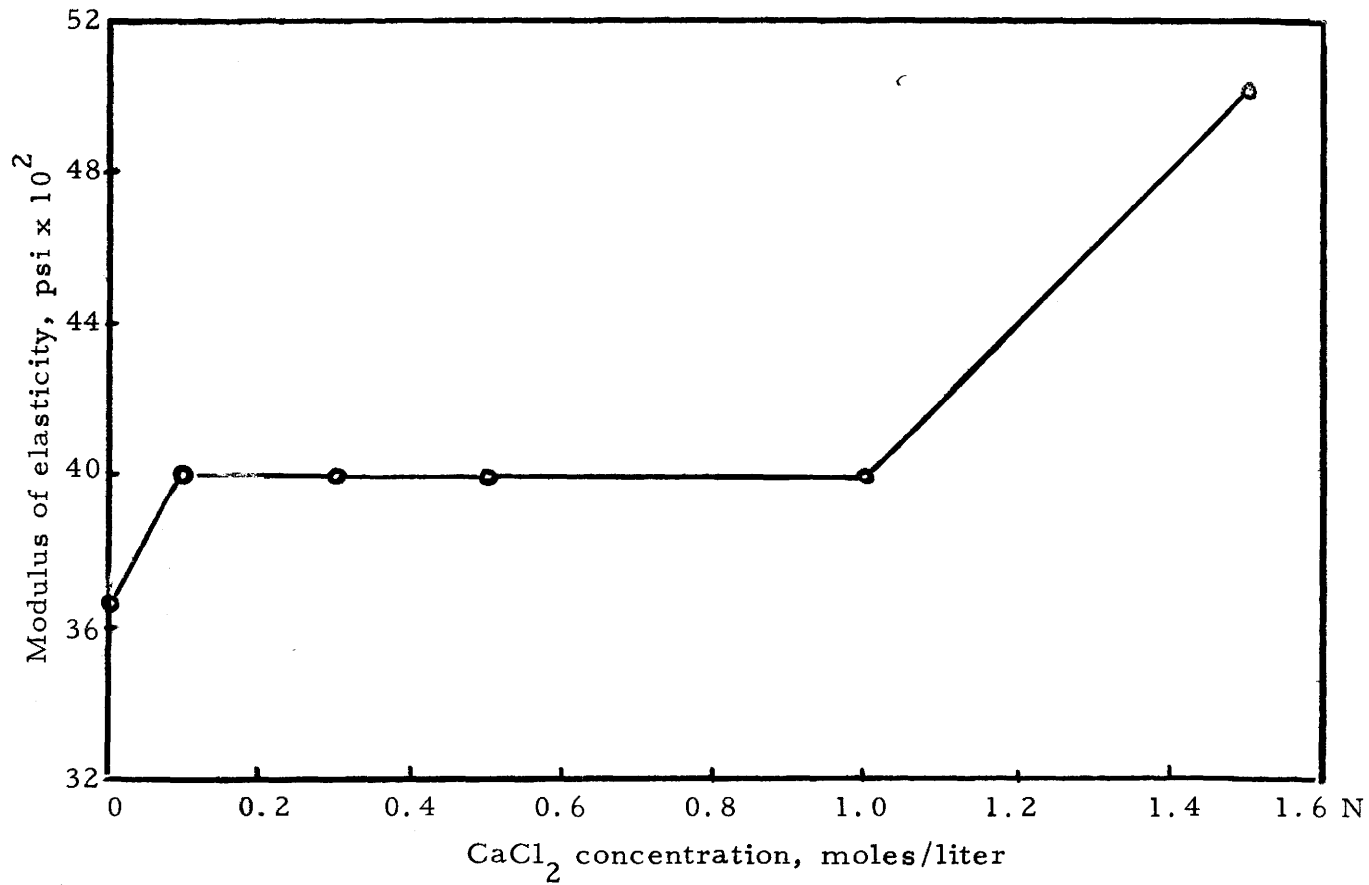


Figure 36. Modulus of elasticity of Ca⁺⁺-saturated soil plotted as a function of CaCl₂ concentration

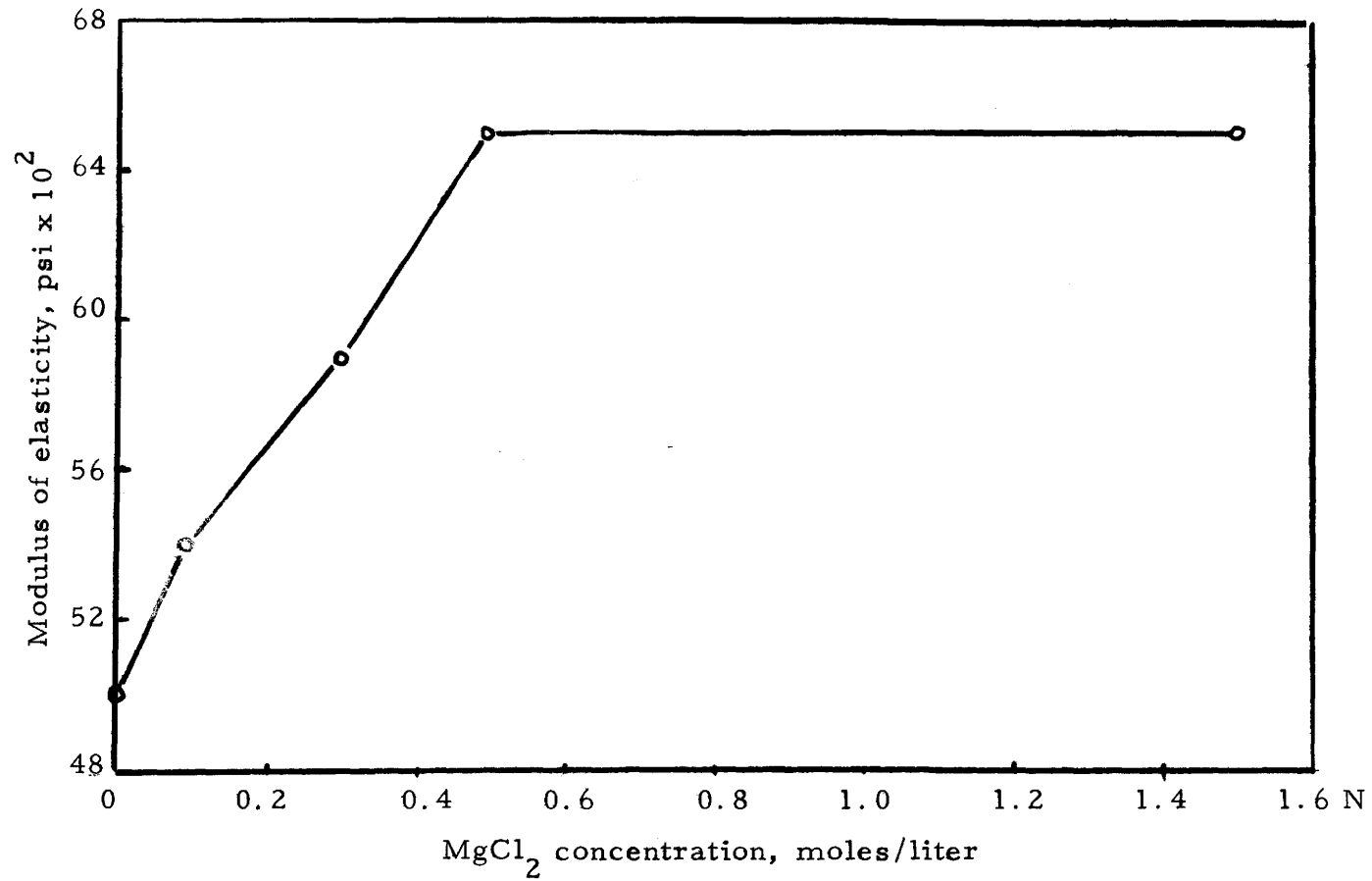


Figure 37. Modulus of elasticity of Mg⁺⁺-saturated soil plotted as a function of MgCl₂ concentration

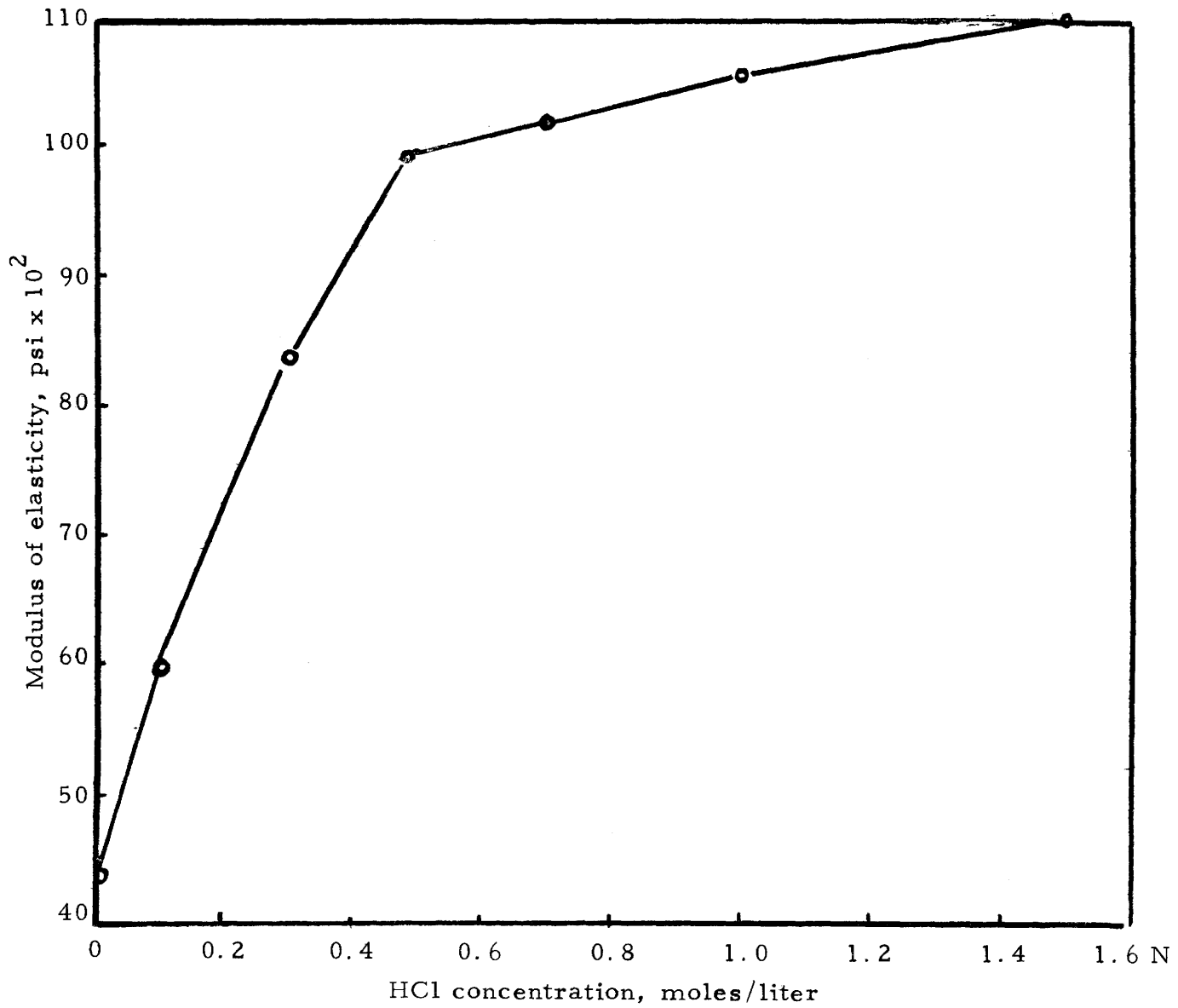


Figure 38. Modulus of elasticity of H^+ -saturated soil plotted as a function of HCl concentration

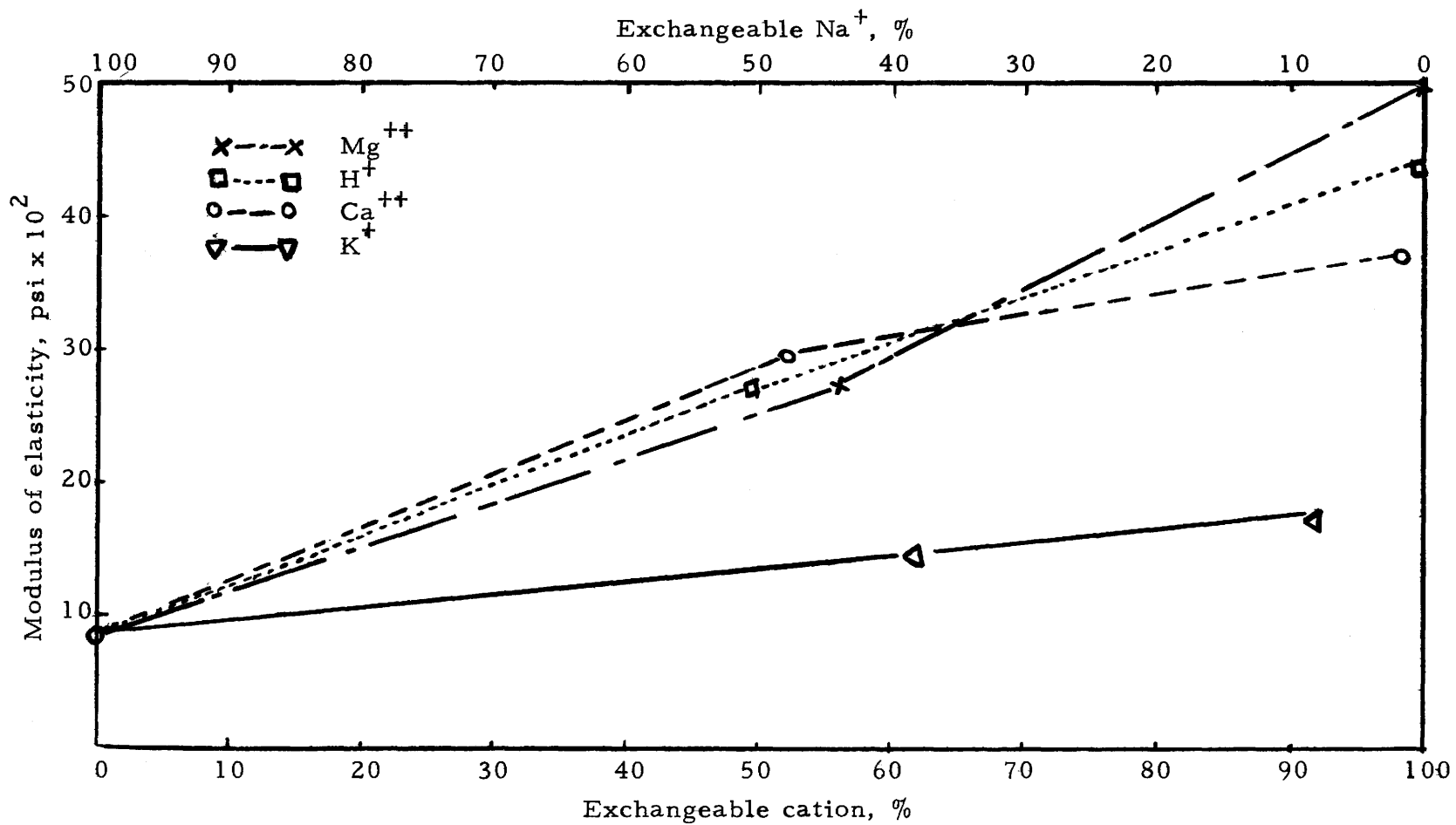


Figure 39. Modulus of elasticity of soil plotted as a function of exchangeable sodium percentage. The influence of different exchangeable cations as they replace Na⁺ is indicated

points on the curve is insufficient.

Shear strength

The shear strength of all samples (with the exception of H^+ -saturated specimen) was found to be a function of the salt concentration in the pore filling liquid (Figures 40, 41, 42, and 43). A relatively high increase in shear strength with increasing salt concentration was found in the case of Na^+ -saturated soil. In this case (Na^+ soils), the shear strength of the sample using 1.5 N sodium chloride was 154.5 percent of that when distilled water was used as pore fluid. However, the influence of the same range of concentration (1.5 N) on strength was less in the cases of Ca^{++} (124.4 percent of distilled water), Mg^{++} (122.3 percent of distilled water), and K^+ (119.3 percent of distilled water). In the case of H^+ -saturated sample, no increase in strength was detected for the different concentrations used (distilled water - 1.5 N HCl) as shown in Figure 25.

An increase in strength with decreasing exchangeable sodium percentage is definite (Figure 44). When distilled water was used as the pore fluid, a substitution of exchangeable Ca^{++} or Mg^{++} for exchangeable Na^+ increased the strength by 31.8 percent and 34.6 percent respectively, while a substitution of exchangeable K^+ for exchangeable Na^+ increased it 45.4 percent. The increase in strength for the H^+ -ion situation was out of proportion with other cations. This latter amounted to 223 percent.

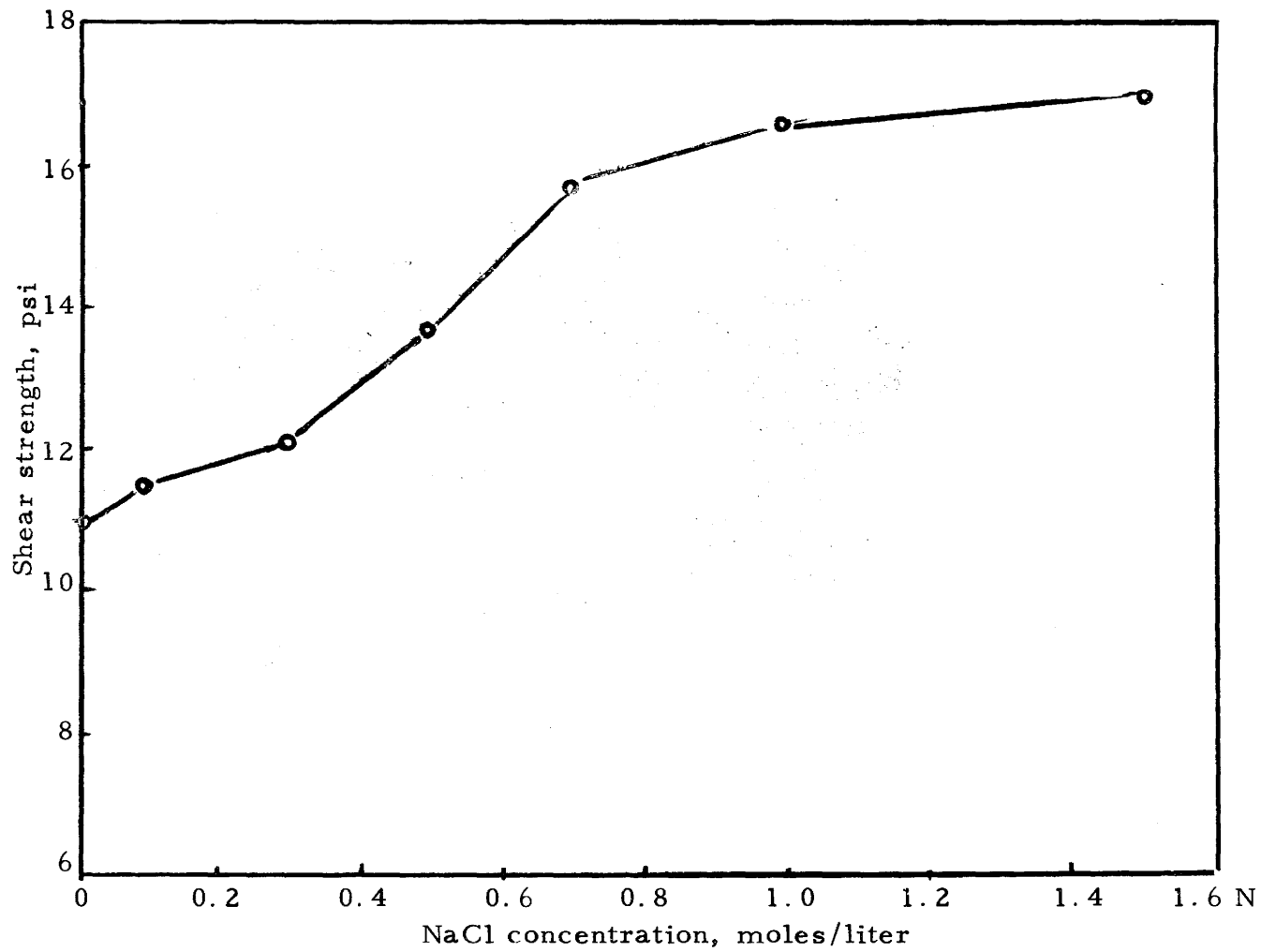


Figure 40. Shear strength of Na^+ -saturated soil plotted as a function of NaCl concentration

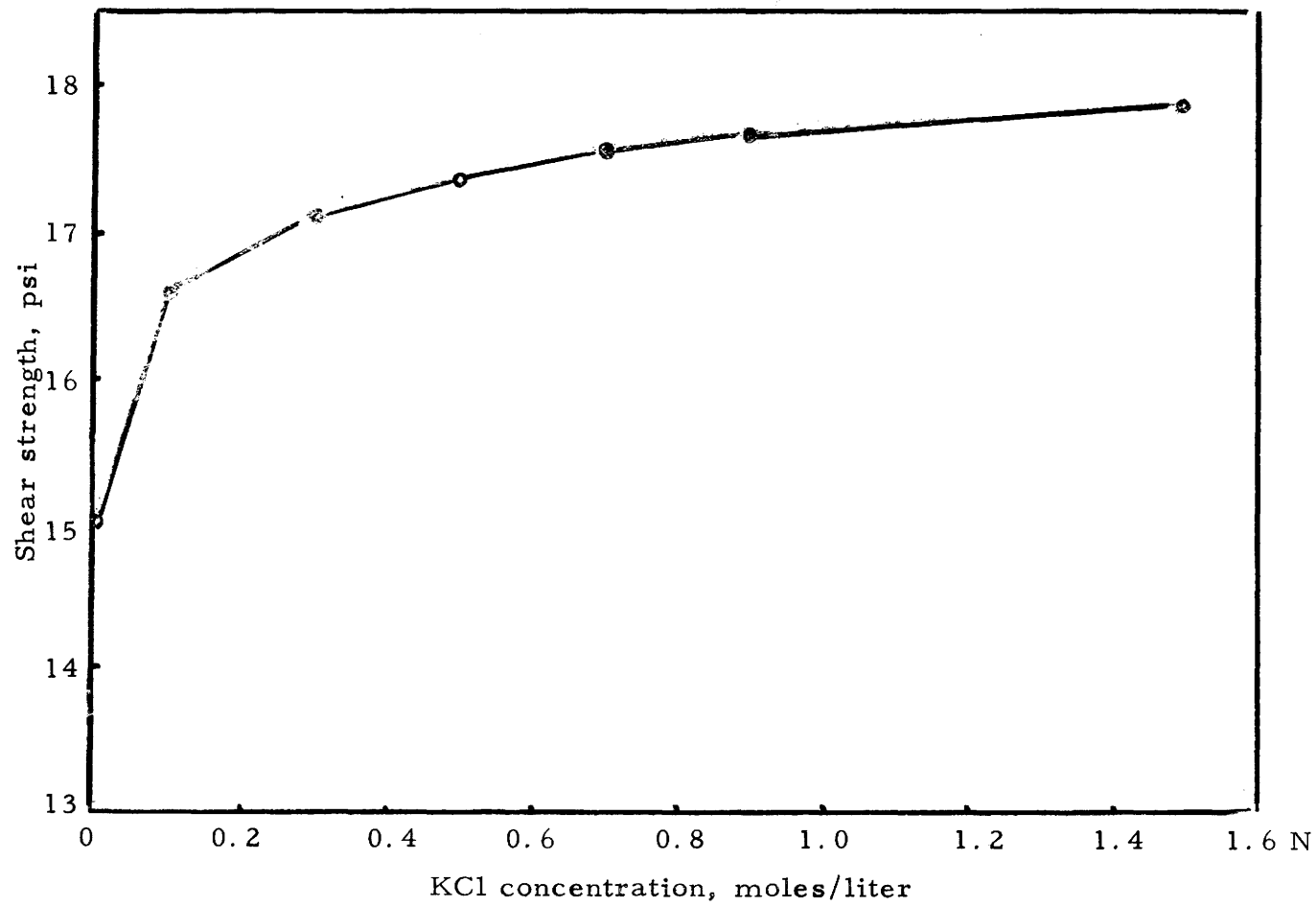


Figure 41. Shear strength of K^+ -saturated soil plotted as a function of KCl concentration

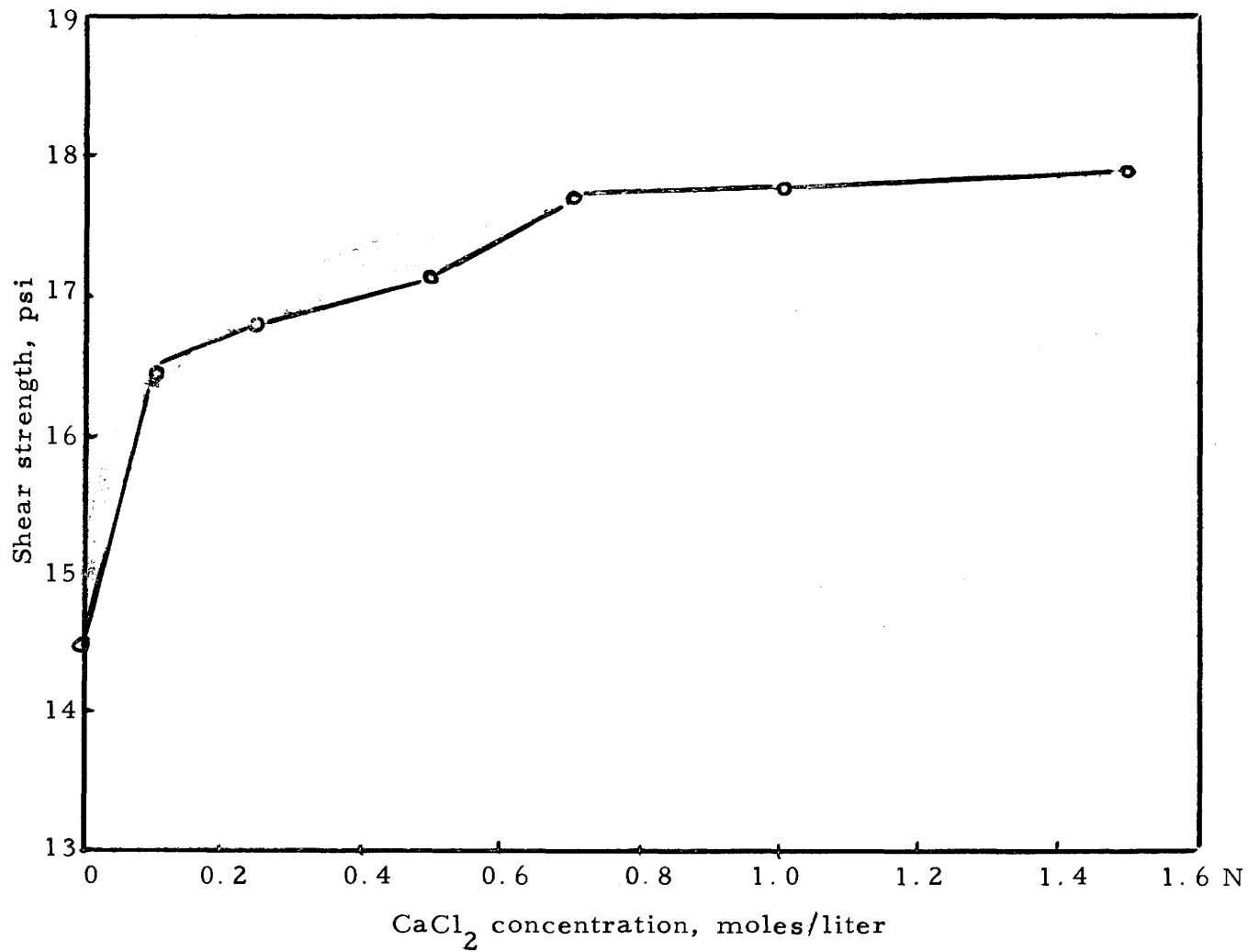


Figure 42. Shear strength of Ca⁺⁺-saturated soil plotted as a function of CaCl₂ concentration

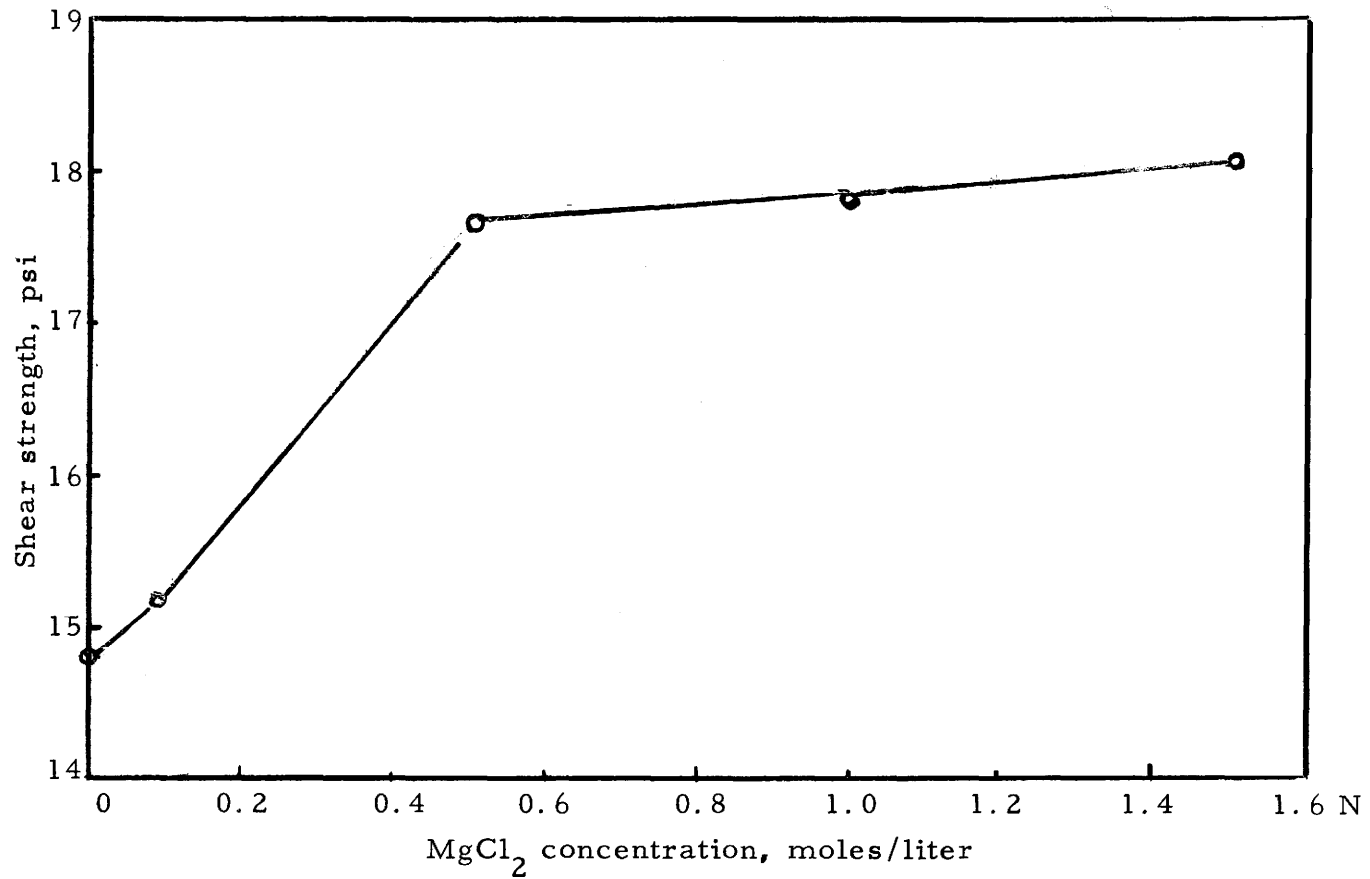


Figure 43. Shear strength of Mg⁺⁺-saturated soil plotted as a function of MgCl₂ concentration

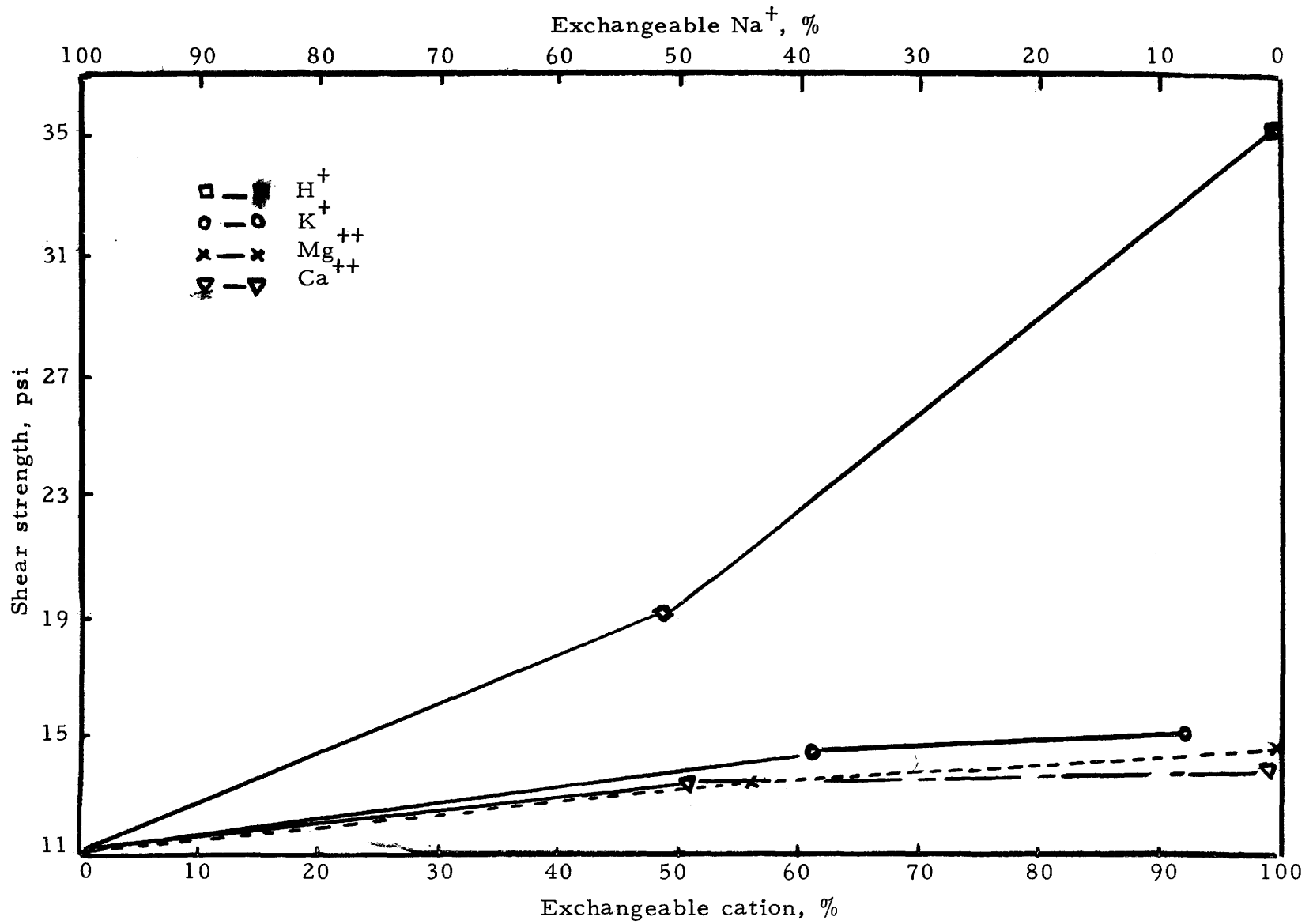


Figure 44. Shear strength of soil plotted as a function of exchangeable sodium percentage. The influence of different exchangeable cations as they replace Na⁺ is indicated

DISCUSSION

The following discussion will be restricted to the interpretation of results in terms of what has been reported in the current engineering literature. A general discussion, however, will be presented in the end of this chapter to discuss all the experimental results in terms of Stern's theory of electric double layer.

Plastic Properties

Atterberg limits

A simplified procedure for determining the liquid limit has been investigated by the Waterways Experiment Station (1949). The method is based on the results of 767 liquid limit tests, according to which it was concluded that the plot of blows against water content on a log-log scale is a straight line. Hence, the liquid limit W for the soil investigated could be found from one test using the equation

$$w = W_N \left(\frac{N}{25} \right)^{1.21}$$

in which W_N is the water content of the soil which closes the groove in N blows in the standard limit device.

It was found in this study that the slope of the curve (blows against water content on a log-log scale) is not constant; i. e., it changes with

both salt concentration in the pore fluid and the type of ion adsorbed on the clay surfaces (Figures 45 to 53). The slope tends to flatten, however, with decreasing exchangeable sodium percentage (Figures 50 to 53). It seems that the slope of the plot of blows against water content on a log-log scale is a function of the elastic properties of the soil which in turn change* with the type of ion adsorbed and soluble salt concentration in the sample.

The results presented in this dissertation showing that exchangeable ions have an influence upon liquid limit (Figure 20) are in agreement with Rosenquist's opinion (1957). However, Michaels (1959) showed that inconsistencies exist in the relationship between liquid limit and the nature of exchangeable ions in various clay minerals. He observed that with montmorillonite, and to a lesser degree with illite clay minerals, the liquid limit decreases as the order $\text{Li}^+ > \text{Na}^+ > \text{K}^+ > \text{Rb}^+ > \text{Cs}^+$. On the other hand, with the clay mineral of kaolinite, the order of liquid limit is reversed as $\text{Li}^+ < \text{Na}^+ < \text{K}^+ < \text{Rb}^+ < \text{Cs}^+$. Moum and Rosenquist (1961) found that in the case of montmorillonite clays the liquid limit dropped from Li^+ to Na^+ and from Na^+ to K^+ , but increased on passing from K^+ to Rb^+ and from Rb^+ to Cs^+ , whereas in kaolinite it varies in the manner $\text{Li}^+ > \text{Na}^+ > \text{K}^+ = \text{Rb}^+ = \text{Cs}^+$. To explain this controversy, they presented two curves (Figure 54)

* Referred to discussion on stress-strain curves and modulus of elasticity.

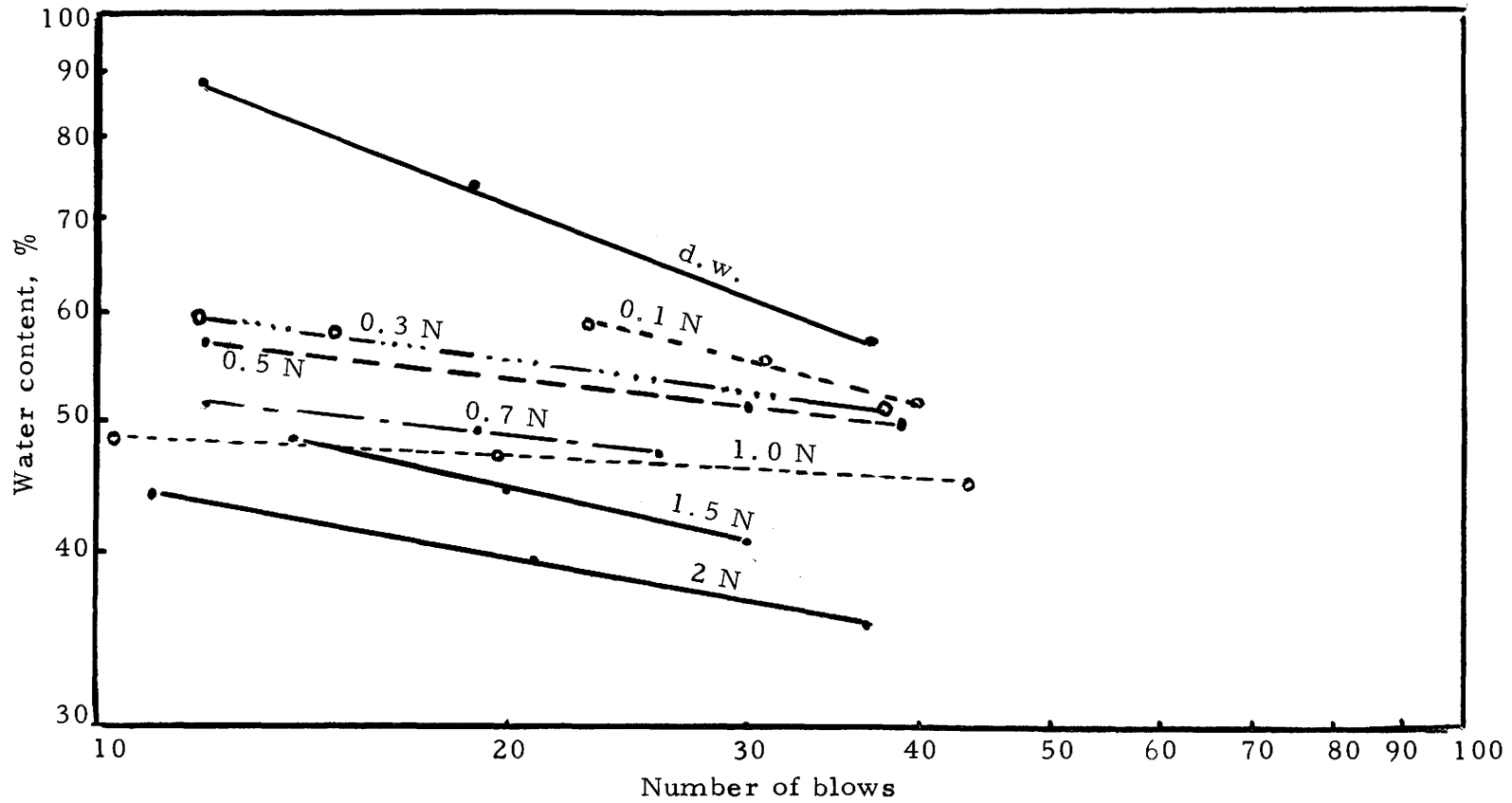


Figure 45. Percentage water content plotted as a function of the number of blows on log-log scale for Na^+ -saturated soil. Different concentrations of pore fluid are indicated

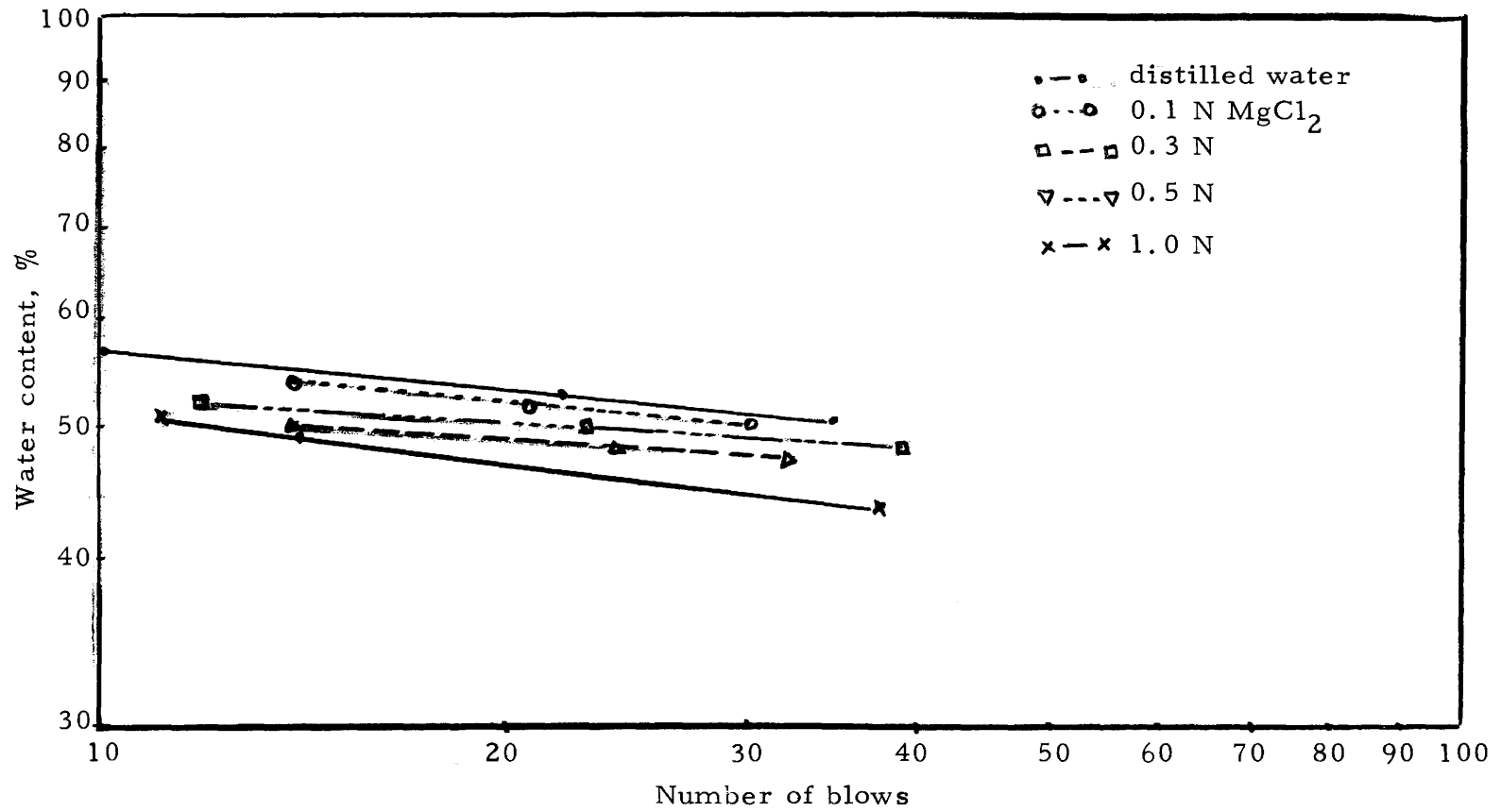


Figure 46. Percentage water content plotted as a function of the number of blows on log-log scale for Mg^{++} -saturated soil. Different concentrations of pore fluid are indicated

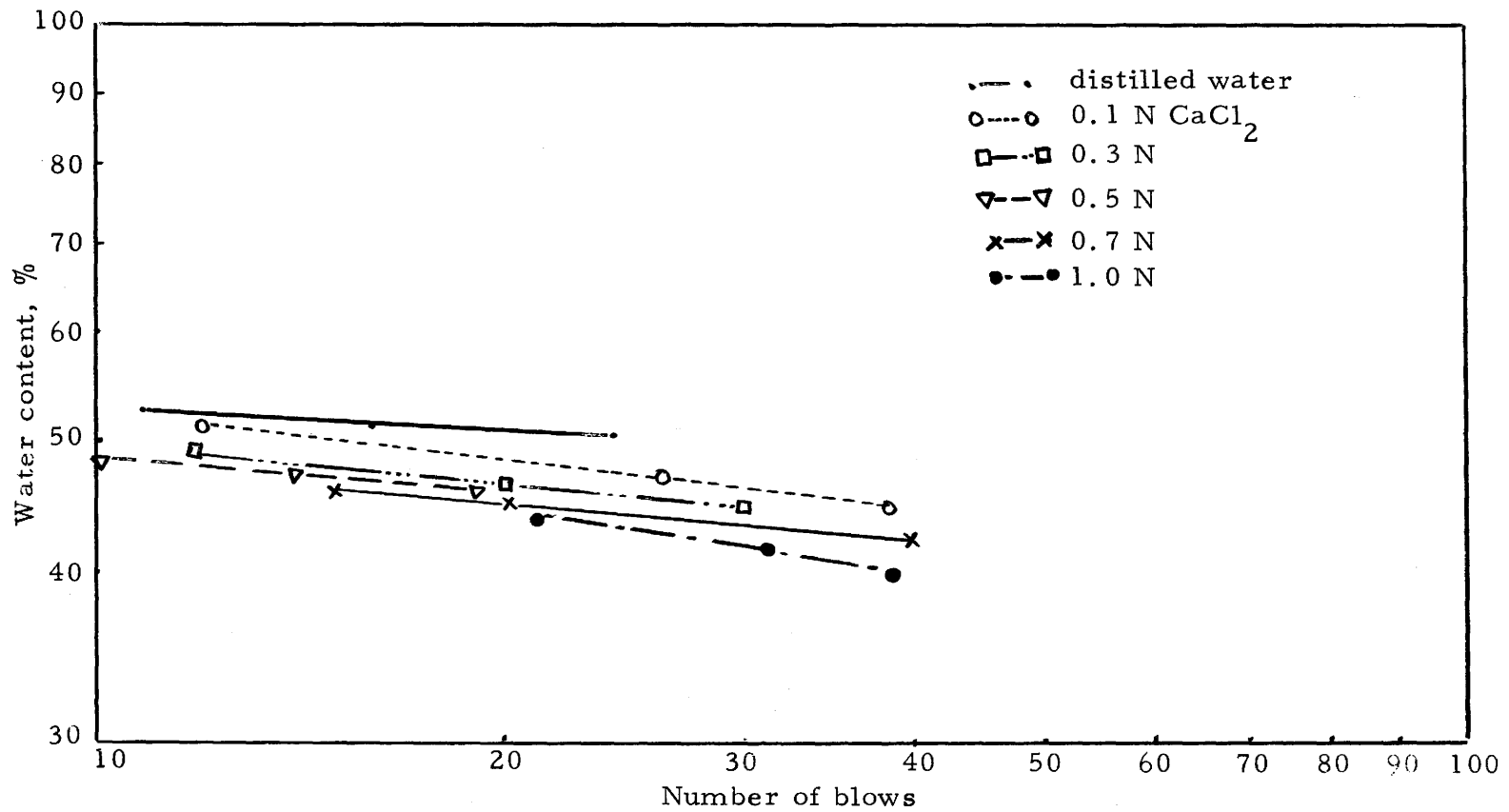


Figure 47. Water content plotted as a function of the number of blows on log-log scale for Ca^{++} -saturated soil. Different concentrations of pore fluid are indicated

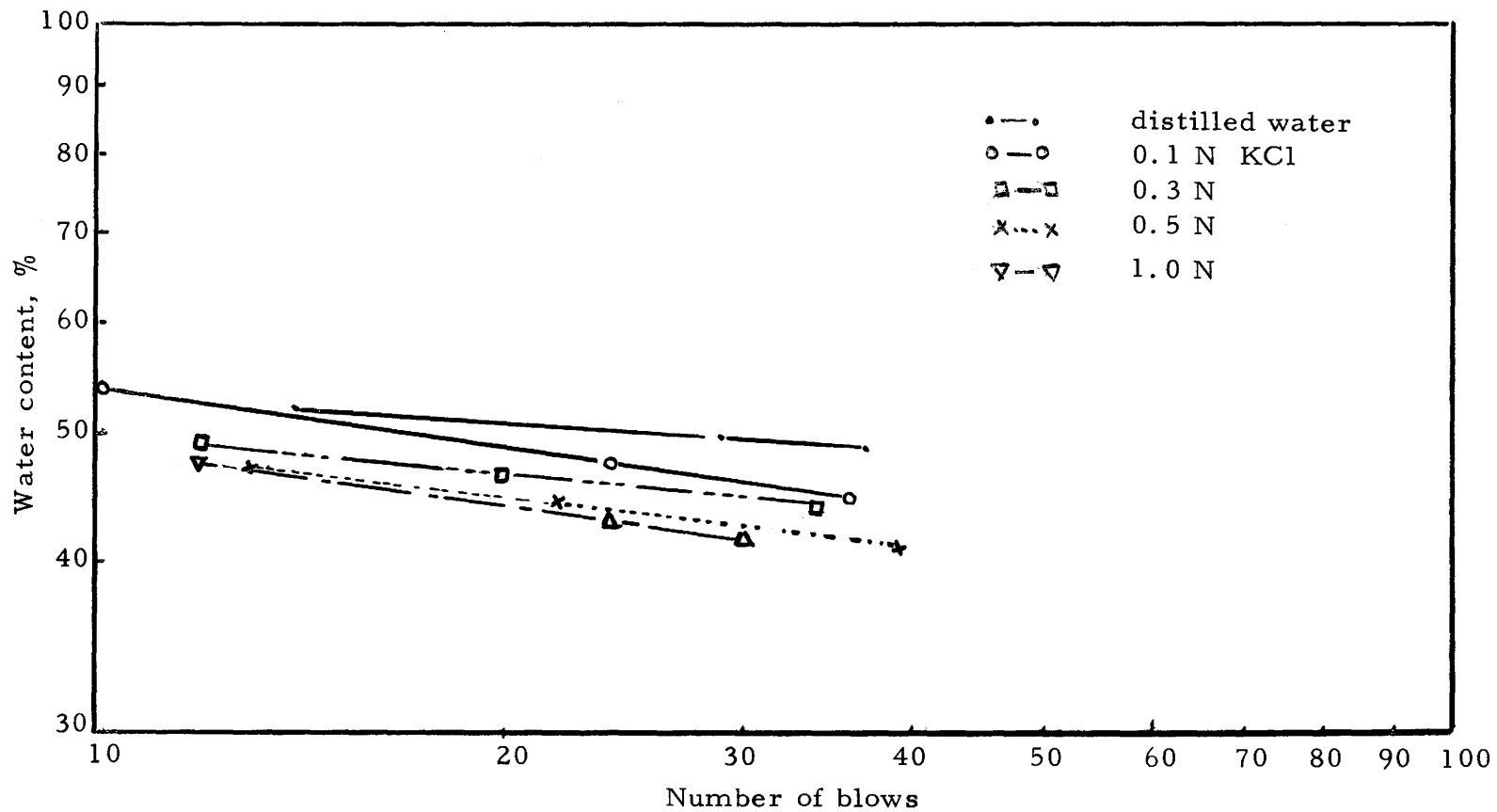


Figure 48. Percentage water content plotted as a function of the number of blows on log-log scale for K^+ -saturated soil. Different concentrations of pore fluid are indicated

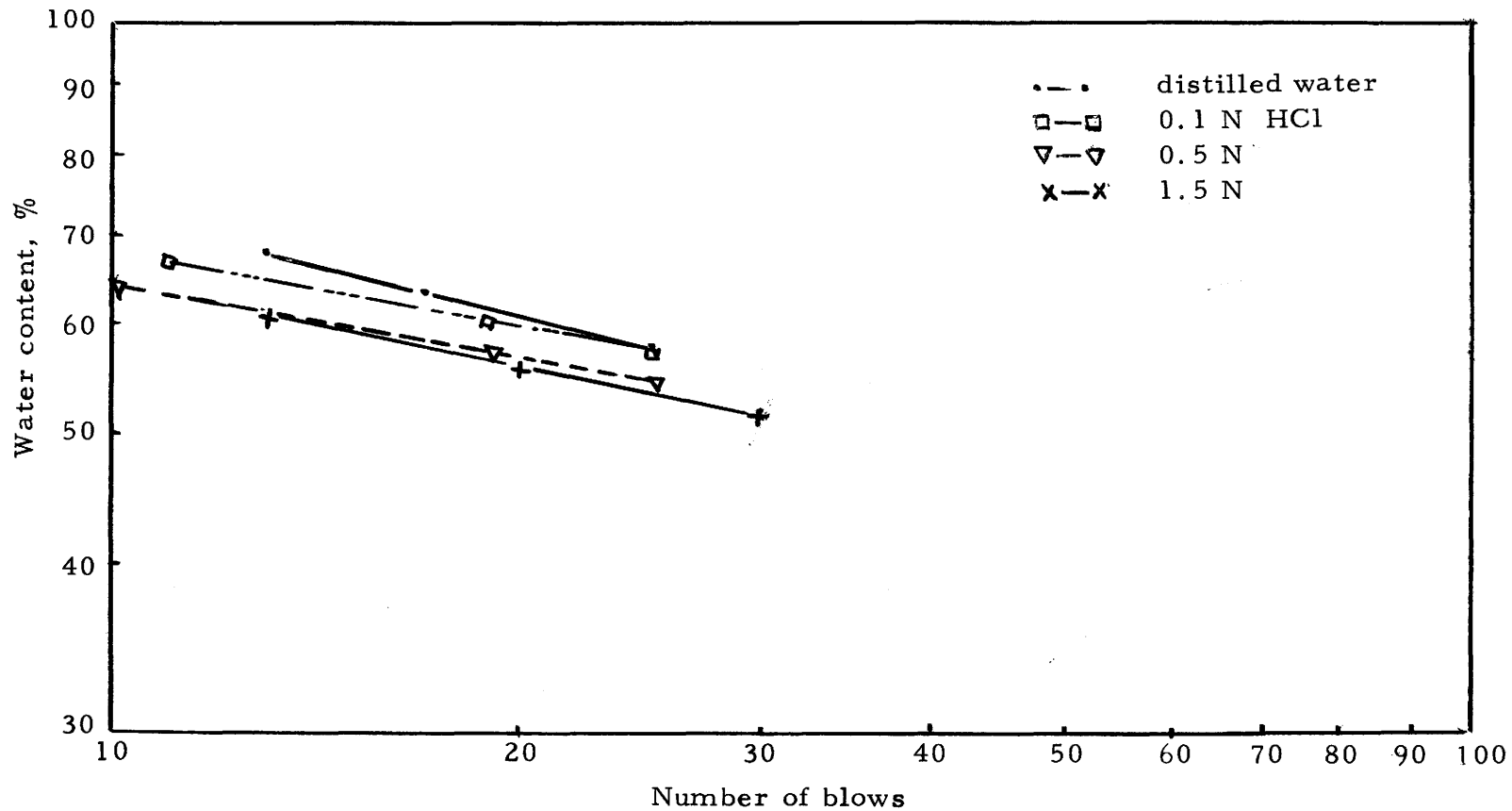


Figure 49. Percentage water content plotted as a function of the number of blows on log-log scale for H^+ -saturated soil. Different concentrations of pore fluid are indicated

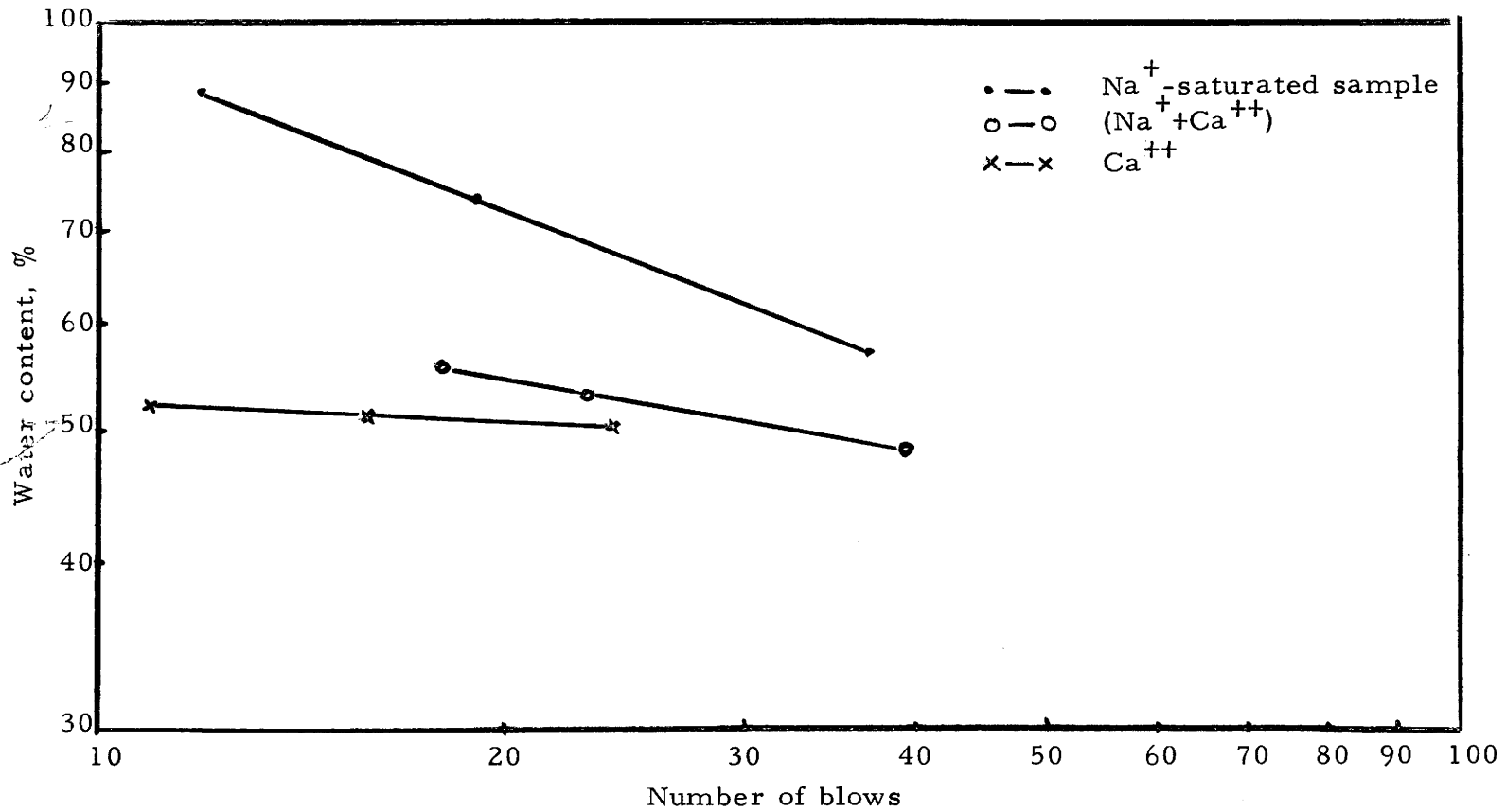


Figure 50. Percentage water content plotted as a function of the number of blows on log-log scale for Na⁺-saturated soil and for soils where exchangeable Na⁺ is replaced partially and completely with Ca⁺⁺

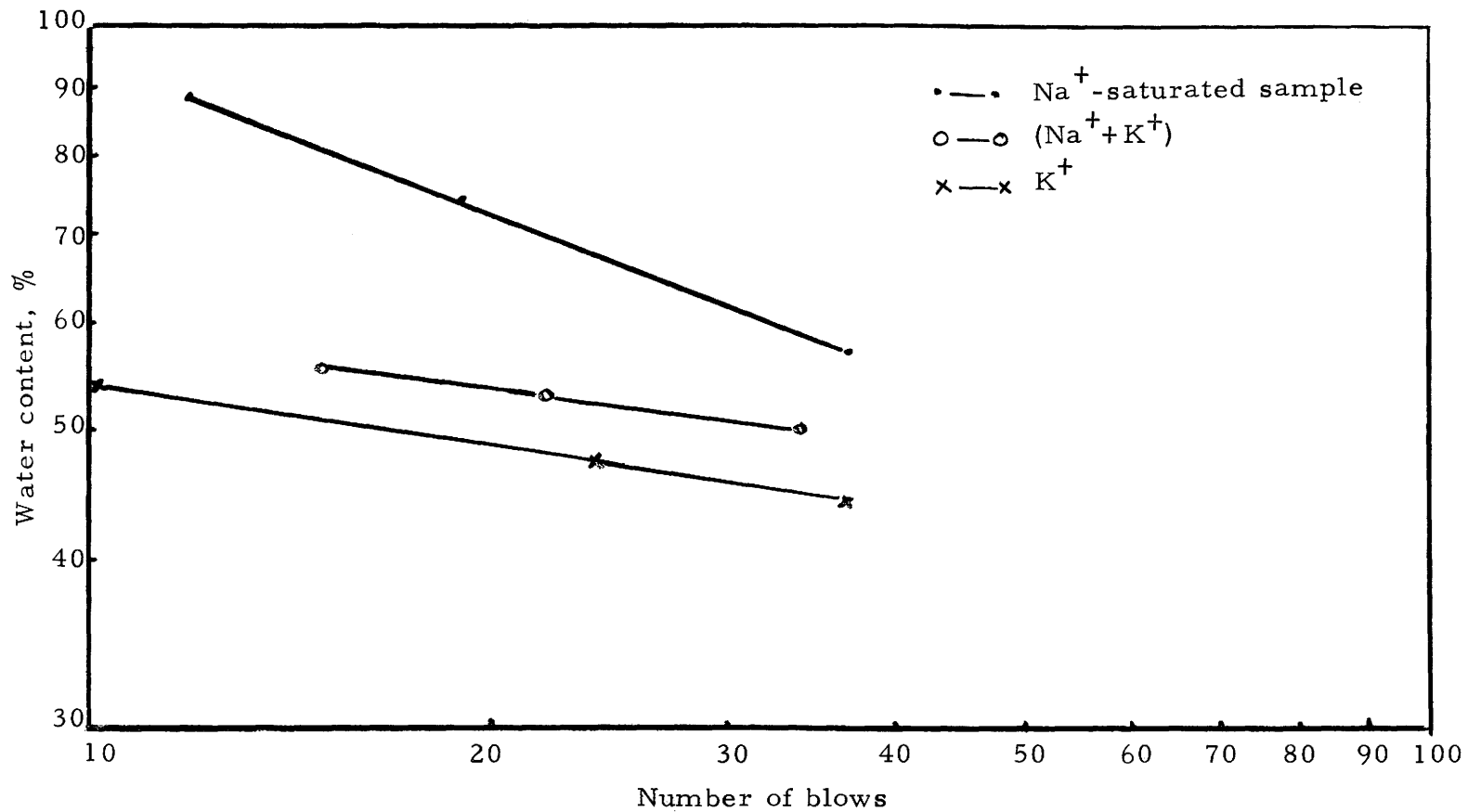


Figure 51. Percentage water content plotted as a function of the number of blows on log-log scale for Na⁺-saturated soil and for soils where exchangeable Na⁺ is replaced partially and completely with K⁺

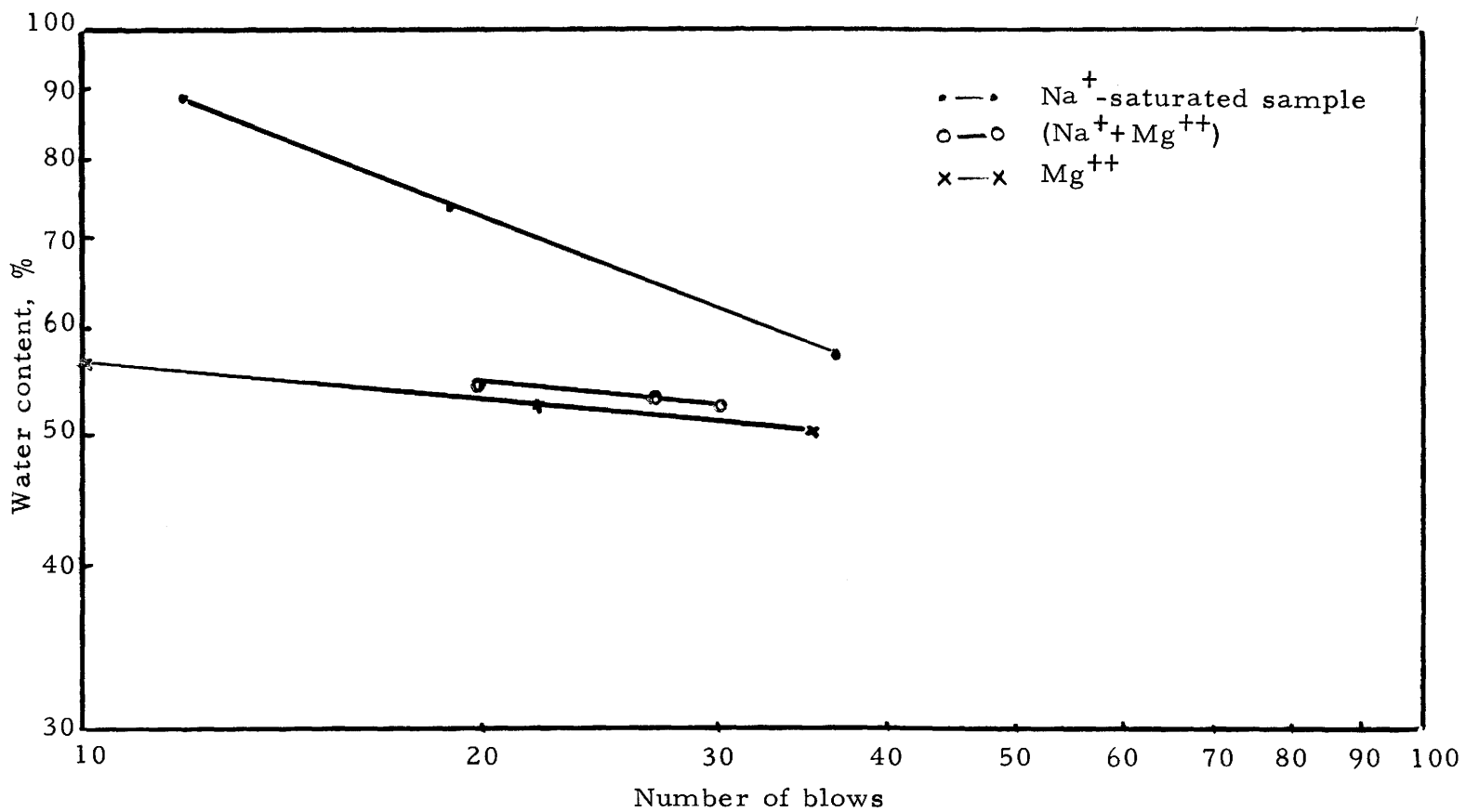


Figure 52. Percentage water content plotted as a function of the number of blows for Na⁺-saturated soil and for soils where exchangeable Na⁺ is replaced partially and completely with Mg⁺⁺

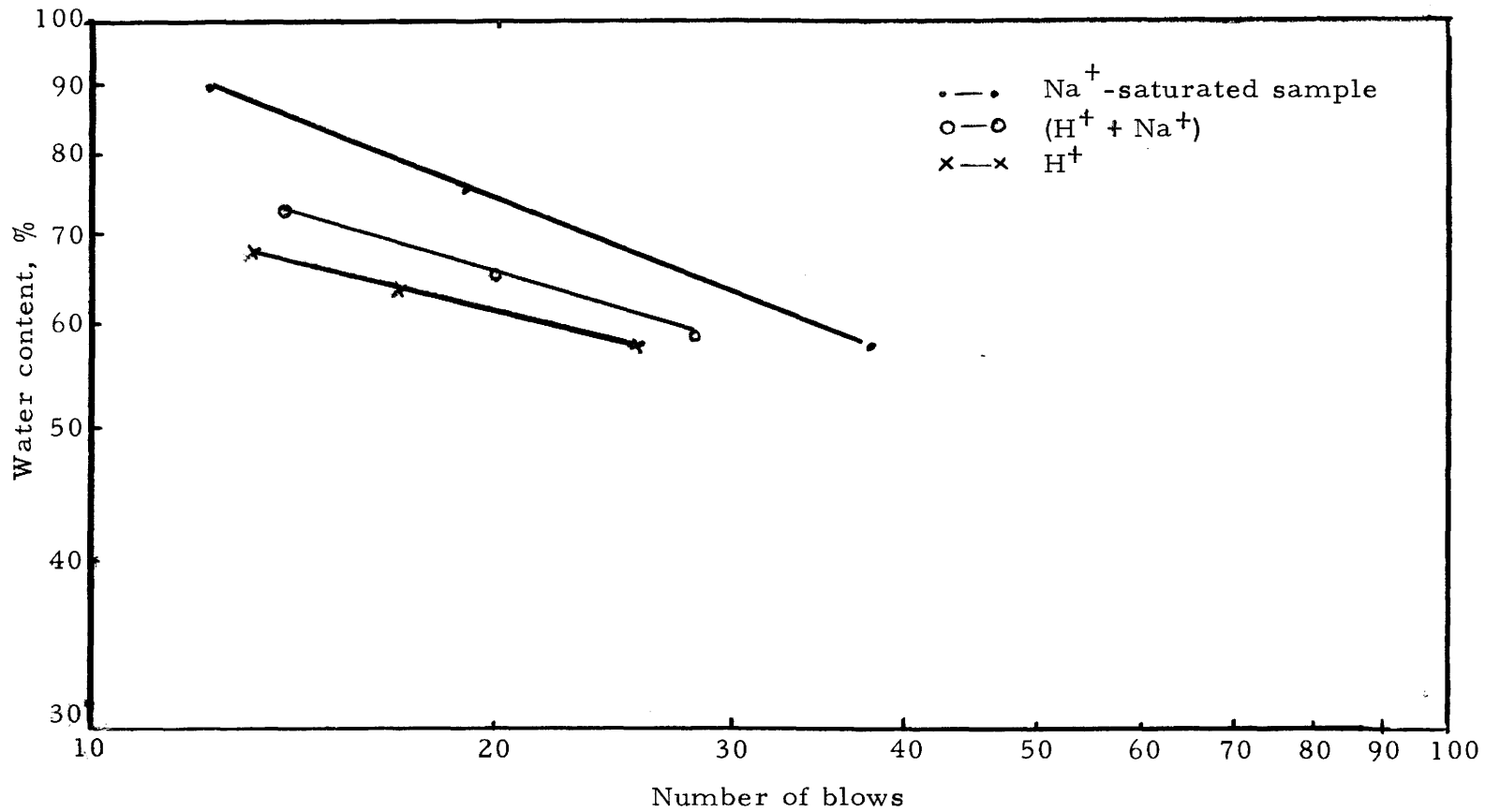


Figure 53. Percentage water content plotted as a function of the number of blows for Na⁺-saturated soil and for soils where exchangeable Na⁺ is replaced partially and completely by H⁺

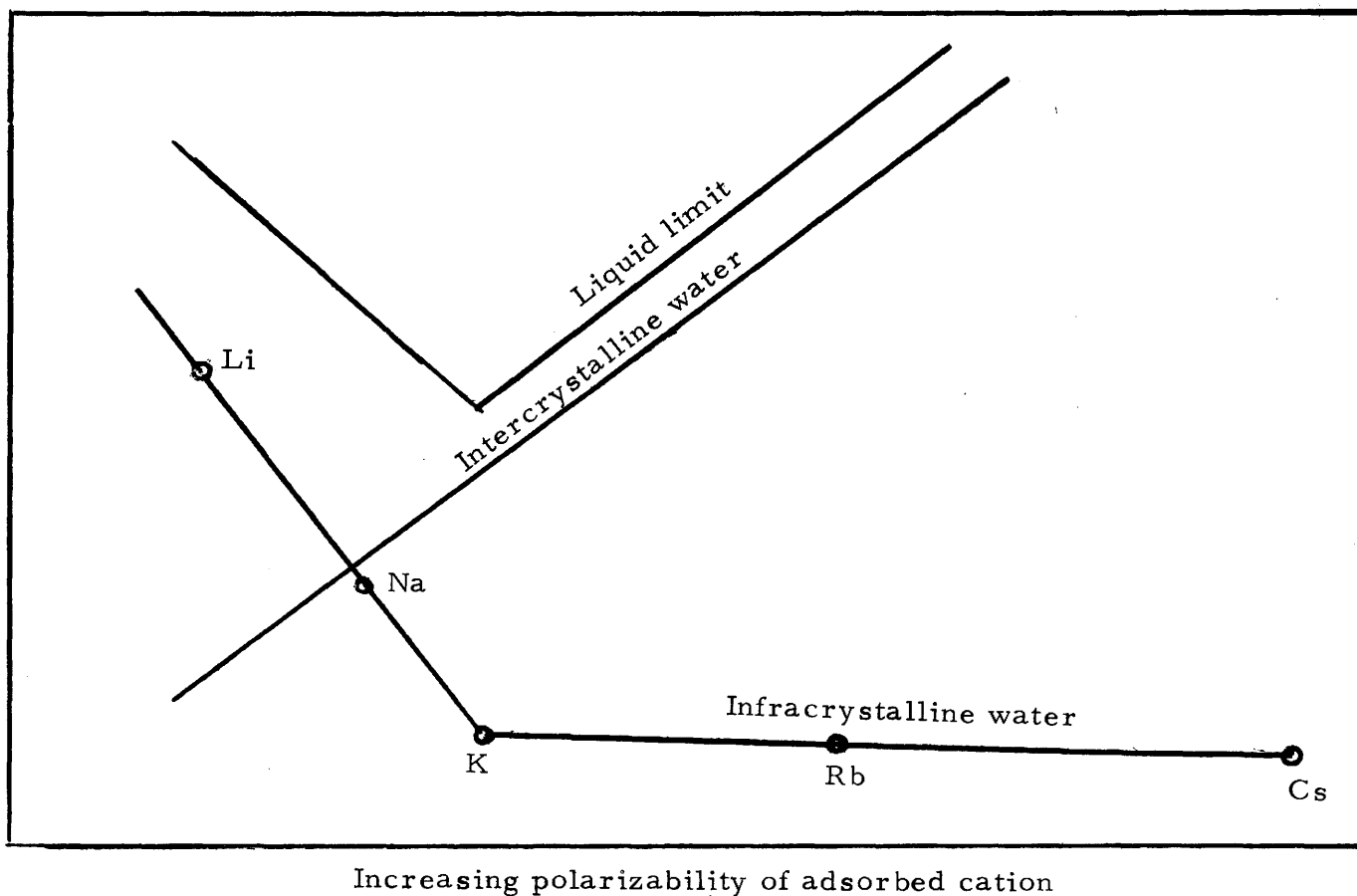


Figure 54. Water content as a function of polarizability of adsorbed cations (Redrawn from Moun and Rosenquist, 1961)

showing that the amount of intercrystalline water (water adsorbed in the inner structure of the crystal) at the liquid limit depends upon the bonding strength between the mineral, whereas the amount of intracrystalline water (water adsorbed between clay particles) is a function of the adsorption force. When these two curves are summed up, the liquid limit values are obtained. In the case of kaolinite clays, intracrystalline water is the only one involved, whereas both kinds of water exist in montmorillonite clays. This explanation, however, is still in contradiction with the results reported by Michaels (1959). It appears that the situation in clay minerals is rather complex, because the inner structure of the montmorillonite crystal is accessible not only to water molecules, but also to cations in the system. Thus, the different degree of hydration of these ions plays an important role in the liquid limit values. The results of this investigation are in agreement with those of Michaels (1959) and Moum and Rosenquist (1961), since it was found from x-ray analysis of the original soil sample that the dominant (about 85 percent) clay fraction is montmorillonite.

The effect of exchangeable ions (with the exception of H^+)* on Atterberg limits (liquid and plastic limits) may be explained on the basis of the differences in hydration of the clays. There are two types of hydration that influence Atterberg limits; namely, water of hydration

* Anomalous behavior of HCl treated soil is discussed separately under the subheading, "General discussion."

held between particles in aggregate formation, and the hydration hull around the individual particles. The K^+ -saturated soil requires a smaller amount of water to produce the plasticity effects than Na^+ -saturated soil (Baver, 1928). This is merely a difference in the hydration of particles as a function of the hydration of ions. In the case of Ca^{++} - and Mg^{++} -saturated samples, a certain amount of water is required to fill the pores in their aggregates, in addition to the water oriented on the particle surfaces. Therefore, a larger quantity of water is necessary to produce the same plasticity effect than in the case of K^+ -saturated soil.

The decrease of liquid limit values with increasing soluble salt concentration is in agreement with the results reported by Lambe (1957). Lambe found that for ten different soil samples the liquid limit was reduced considerably when 10 percent calcium acrylate was added. However, the reduction of liquid limit values in Lambe's experiment may be due to the effect of exchangeable calcium rather than to soluble salt concentration.

Strength Properties

Stress-strain curves

Schmertmann and Osterberg (1960) stated that, for cohesive soils, the cohesion strength component* develops to its maximum value at low

* Reference is made to the conventional concept of shear strength as made up of friction and cohesion terms ($s : \sigma \tan \phi + c$).

strains (about 1 percent), while the friction component requires a much greater strain to reach its maximum. In this regard, Casagrande and Hirschfeld (1960) suggested a hypothesis of double failure, namely, a yield strength at a very low strain which develops immediately following the straight line portion of the stress-strain curve, and an ultimate strength that develops at high strains. They suggested that the first failure which would account for low strains corresponds to a bond at the points of contact between particles, whereas the high strains at the ultimate strength would reflect the mobilization of internal friction.

Following the above reasoning, it seems that considerable strain is required to achieve maximum interference between particles and therefore maximum frictional strength. This would be true for a dispersed soil; e. g., Na^+ -saturated sample (Figure 22) where particles are packed closer together and one would expect particle interference. Since the cohesion is generally thought to be based on the electrical attraction forces between clay particles (Lambe, 1960b) and large strains would not be required to activate these forces, failure strain values decrease with decreasing exchangeable sodium percentage (Figure 32); i. e., increasing electrical attraction forces.

When a sample fails by cracking; e. g., H^+ -saturated sample (Figure 33), the inference may be drawn that the bonds within the sample have been destroyed rather completely. On the other hand, in a plastic type of failure (bulging or flowing); e. g., Na^+ -saturated samples

(Figure 33), the material remains intact and no cracks develop (Whitman, 1960). Since the bonds within a soil sample depend on the level of pore fluid concentration and the type of exchangeable cation, the fashion in which the sample fails is controlled by these factors.

Modulus of elasticity

The modulus of elasticity is a measure of the stiffness properties of a material; namely, its inherent capacity to resist elastic displacement under stress (Hetenyi, 1950). The range of elastic deformation (the particles move one with respect to the others, but they do not lose their bonds) is controlled by the force due to the interaction between particles, which in turn is affected by the interparticle forces in the system. In this study, an increase in attraction forces or a decrease in repulsive forces would increase the contact pressure between particles, which results in an increase of modulus of elasticity. Since the type of exchangeable ion and the salt concentration in the pore liquid affect the interparticle forces, it would be expected that the modulus of elasticity would be influenced by both factors.

Cooling and Skempton (1942) found that the modulus of elasticity increases with increasing strength. This is in accordance with the results obtained in this study with the exception of the H^+ -saturated sample (Figure 55). For London clay, Skempton and Henkel (1957) found that the increase of E with strength was approximately a straight line relationship. This was expressed as

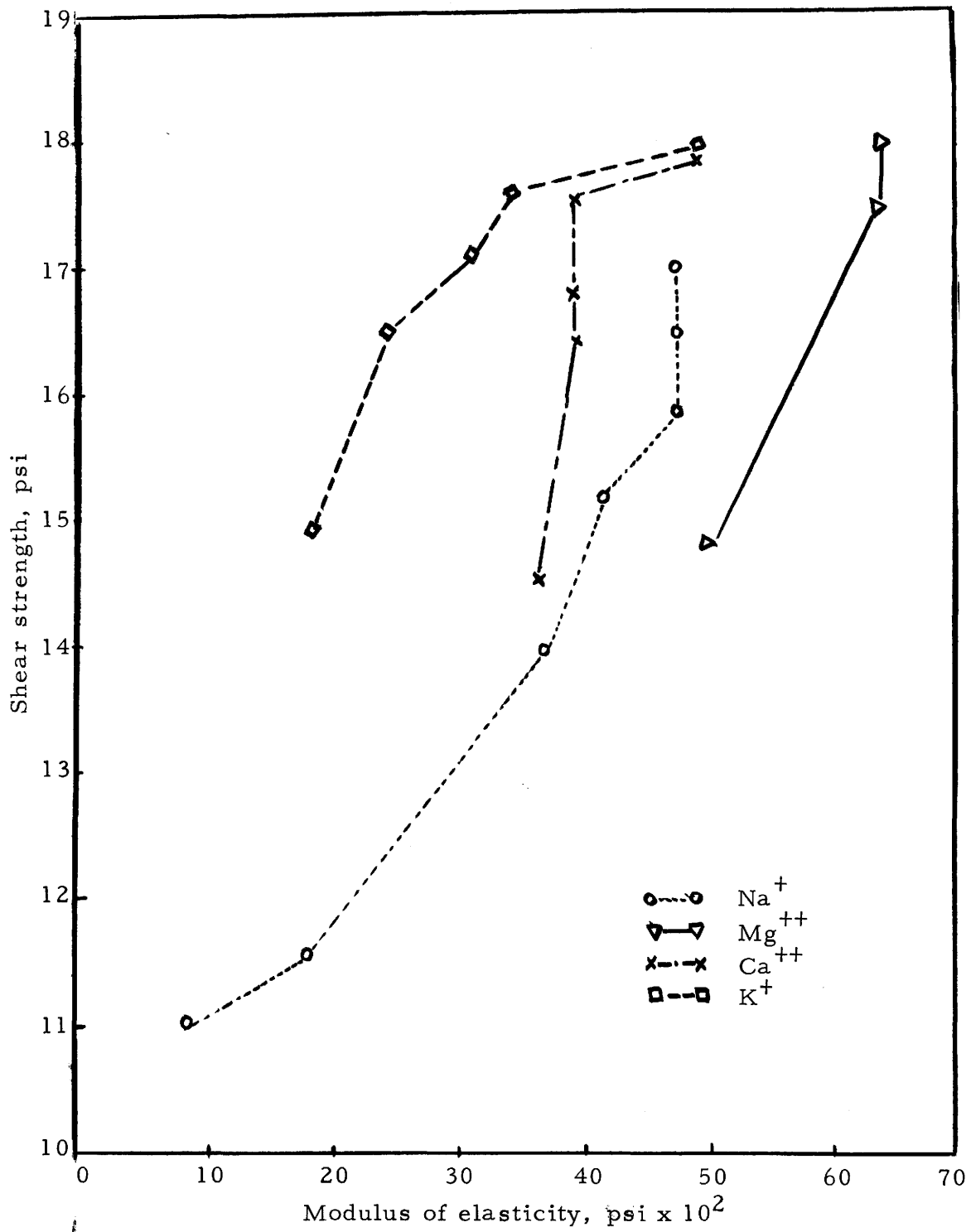


Figure 55. Shear strength of experimental soil samples plotted as a function of modulus of elasticity

$$E = 140 \tau$$

Such relationships as shown in Figure 55 are not linear and are affected by the type of adsorbed ion on the clay fraction.

Shear strength

Since shear strength in cohesive soils manifests the bonding of particles which is a function of the interparticle forces, one would expect that it is influenced by both the type of exchangeable ion and soluble salt concentration. Lambe (1960b) suggested that the shear strength in cohesive soils could be expressed in terms of the net interparticle forces (Equation 10). Furthermore, he used the Gouy-Chapman theory of double layer to present the effects of various environments in the soil-water system on shear strength. The Gouy-Chapman concepts predict that any change in the soil-water system that expands the double layer; i. e., increases the repulsive forces, reduces the shear strength of a clay. Addition of salt to soil-water system or exchanging cations from low to high valence or from hydrated to less hydrated suppresses the double layer (decreases repulsive forces). If this suppression is adequate, the clay particles will approach each other close enough to react to attraction forces (Van der Waals), thus causing flocculation.

Although the Gouy-Chapman theory as described above explains some of the observed results, e. g., increase of strength with addition of NaCl salt to the system or by replacing exchangeable Na^+ with

exchangeable Ca^{++} , there remain several factors that are not fully explained. For example, experimental results presented in this dissertation show that the K^+ -saturated sample in distilled water has higher strength than samples saturated with divalent cations, e. g., Ca^{++} and Mg^{++} (Figure 44). This is in contradiction to the Gouy-Chapman theory which infers that divalent cations are more strongly attracted toward the negative surface than monovalent ions, and therefore, form a thinner layer (more stable).

General discussion

The increase of strength, modulus of elasticity, and the decrease of liquid limit with increasing salt concentration in the pore fluid and decreasing exchangeable sodium percentage could be generally explained in terms of the Stern theory of double layer. Stern (Kruyt, 1952, p. 132) applied the Gouy-Chapman theory with the first layer of ions not immediately at the surface but at a distance δ away from it. Stern further considered the possibility of specific adsorption of ions and assumed that these ions are located in the plane δ . This layer of adsorbed ions is called the Stern layer. As mentioned in the previous discussion regarding the Gouy-Chapman theory, the alteration of electrolyte concentration in a soil-water system alters the attraction-repulsive balance between particles. Increased ion concentration in the external fluid causes a suppression of the double layer which increases the net attractive force between particles (Warkentin, et al., 1957). Likewise,

different exchangeable ions influence the specific adsorption within the Stern layer, while in turn influencing the double layer (Bolt, 1955). These factors, therefore, alter the degree of flocculation and hence soil aggregation.

Specific adsorption arises in soil-electrolyte systems when the crystalline silicate structure makes specific demands upon the ions; that is, certain clays are able to fix ions of appropriate sizes in a non-exchangeable form. An example of such a case is the potassium ion, which is of such size and coordination number that it can easily fit into the surface holes in the silica sheets. Upon drying, the adjacent clay sheets come close enough together for the potassium ion to fit tightly into the holes and it then becomes nonexchangeable. Dyal and Hendricks (1952) showed that drying a potassium saturated montmorillonite gave rise to a mineral with interstratified expanding and nonexpanding layers, and x-ray analysis confirmed this fact. It seems, therefore, that the clay fraction of the sample used in this study (montmorillonite) is converted into minerals of the illite group when saturated with potassium ion. The difference in strength between illite and montmorillonite, the writer thinks, is a result of the difference in swelling behavior which is related to repulsive force (swelling pressure is proportional to repulsive forces). In montmorillonite clay minerals, swelling manifests an interlayer swelling in addition to interparticle swelling, whereas in illite (less hydrated) only the interparticle swelling

exists. This argument is supported by the Atterberg limits, shown in Figures 20 and 21, where potassium-saturated samples showed the lowest limits among all other samples.

Despite the fact that the potassium ion increased the soil strength more than either calcium or magnesium ions (in distilled water-soil system), it is clear (Figure 27) that K^+ -saturated samples increased in strength progressively as the strain increased, in a fashion similar to that of the Na^+ -saturated sample, i. e., the shear strength develops at high strains. This behavior toward failure reflects the initially dispersed structure, whereas initially flocculated structures (Ca^{++} - and Mg^{++} -saturated samples) develop high strength at low strains.

The role of hydrogen in soils is rather complex since this ion seems to be preferentially adsorbed and held very tightly by silicate minerals. The hydrogen ion also tends to form chemical bonds with oxygen on the surface. Furthermore, it has been demonstrated (Marshall, 1949, p. 107) that hydrogen clays are in reality hydrogen-aluminum systems. Aluminum arises by the decomposition of parts of the clay lattice due to progressive attack by hydrogen ions. It was suggested (Marshall, 1949, p. 107) that the chief action is breakage of Al-O-Si linkages in clay minerals.

Experimental results showed a substantial increase in strength when exchangeable sodium was replaced by exchangeable hydrogen (Figure 44). Furthermore, the shear strength for H^+ samples was not

a function of electrolyte concentration. The anomalous behavior of the hydrogen ion, however, may be attributed to exchangeable aluminum which is released when treating soil with HCl. From the work of Mukherjee, et al. (1947), it is shown that the aluminum ion is present in large proportions in acid clays prepared by electro dialysis or leaching with dilute acids. Hence, it appears that the method used in this study to prepare H samples was rather severe (suspensions in 3 N. HCl), which resulted in a breakdown of a large part of the clay fraction, liberating exchangeable aluminum. This is confirmed by x-ray analysis (Figure 56, with comparison to Figure 6). Therefore, strength properties of HCl-treated samples are controlled by the aluminum ion. This ion, being trivalent, is strongly adsorbed on the clay colloid and would explain the high shear strength values of the HCl-treated samples. Furthermore, the breakdown of part of the clay fraction might result in the formation of aluminum colloids, which are characterized by their cementing action between soil particles. The increase of pore fluid concentration by addition of HCl apparently did not affect the strength value, because hydrogen ions are single charged and would be expected to cause little or insignificant suppression of the double layer compared with the exchangeable trivalent aluminum ions on the clay surfaces. Shear strength independence of pore liquid concentration could be also a result of the large effect of specific adsorption in the Stern layer compared to the change in the double layer characteristics, so that the effect of the latter could not be detected.

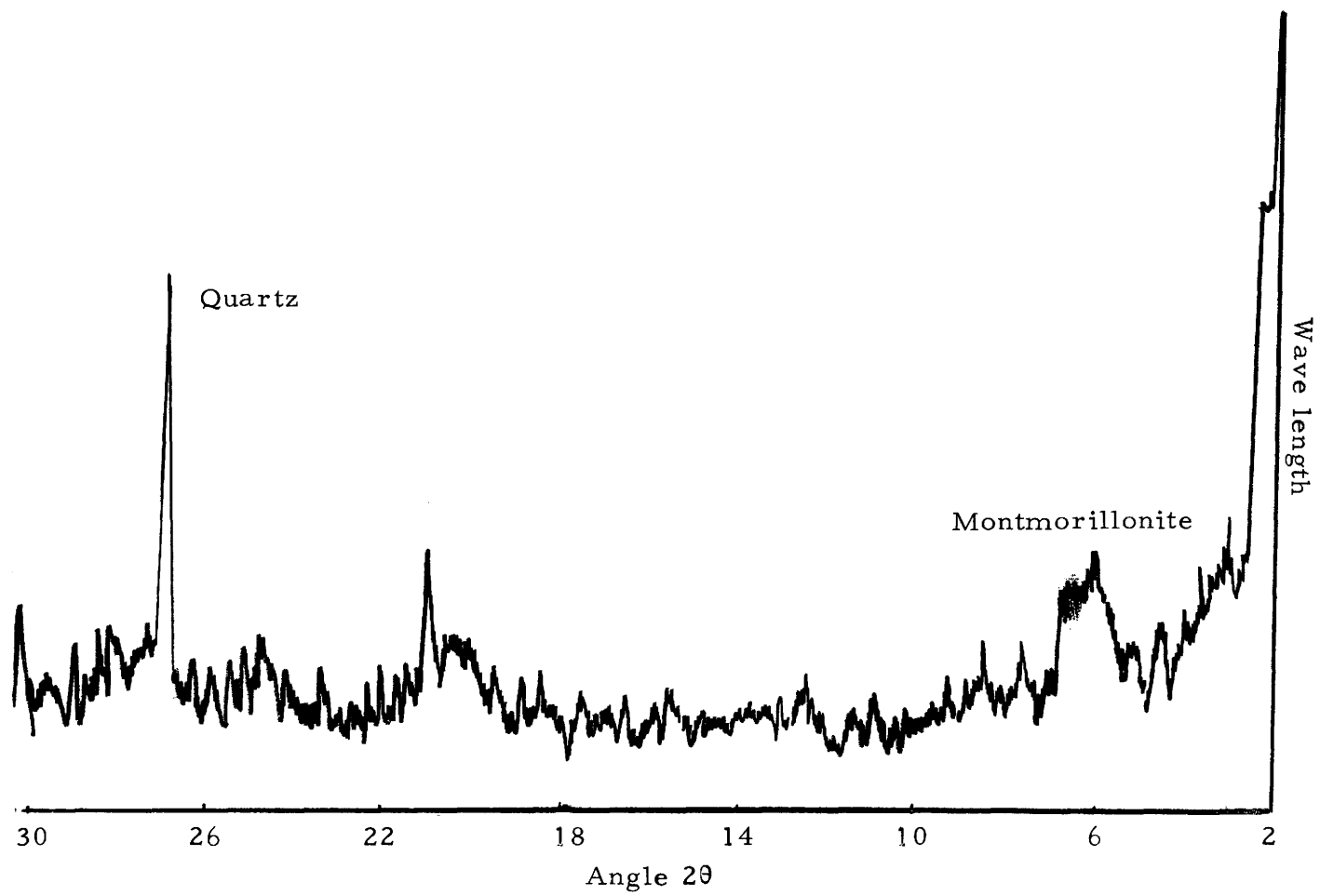


Figure 5'. X-ray traces of 0-0.2 μ fraction of the soil treated with HCl

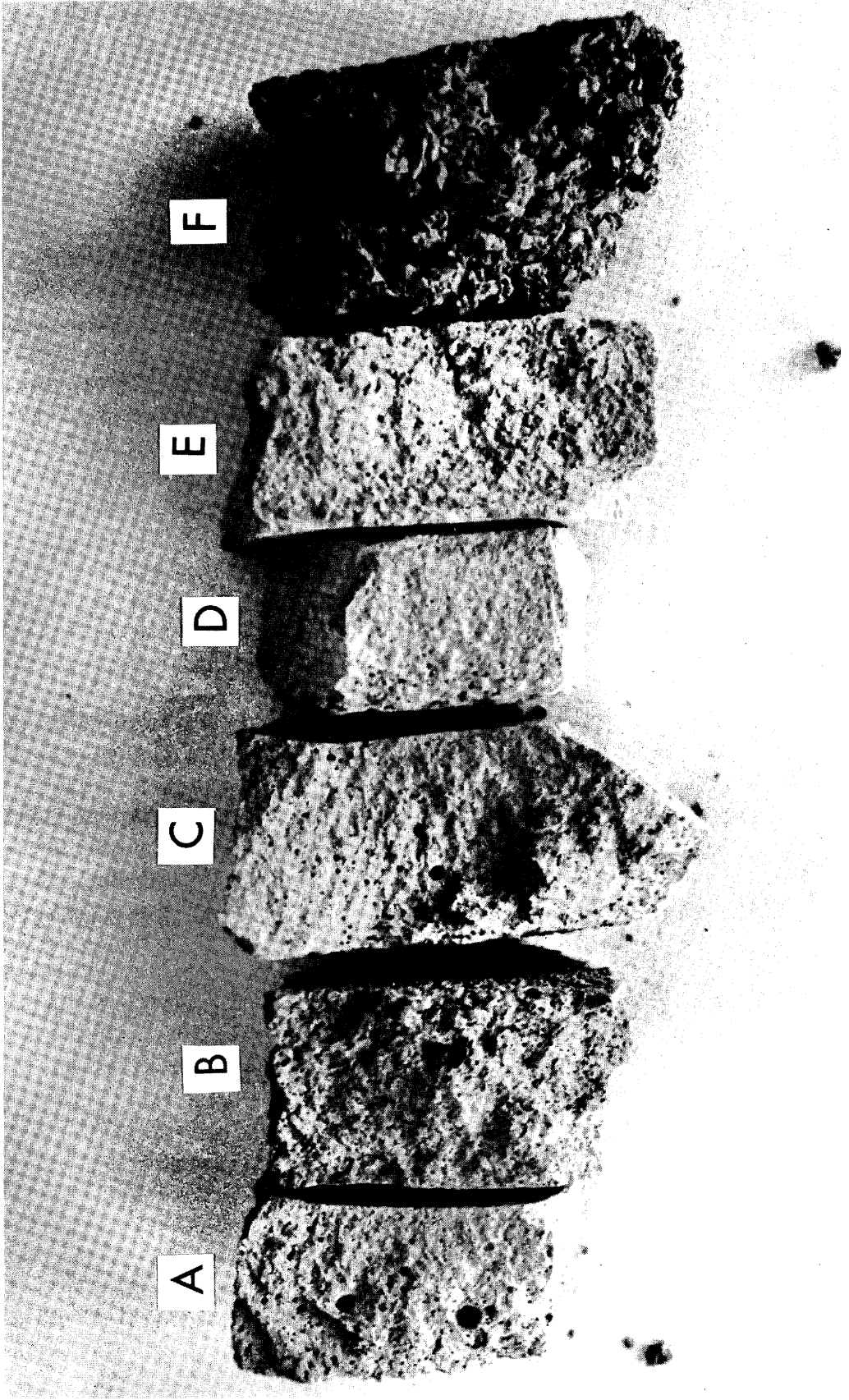
The plastic limit for HCl-treated samples could not be determined mainly because of the destruction of most of the clay fraction. This, in addition to the removal of CaCO_3 , resulted in a sample which is completely different in structure and texture than all other treated samples (Figure 57).

It should be emphasized, however, that the strength and plastic characteristics shown by HCl-treated soil cannot be attributed only to the effect of exchangeable hydrogen. A thorough study regarding acid-treated soils is needed, using methods of preparation which result in a high percentage of exchangeable hydrogen (Aldrich and Buchanan, 1958).

Reliability of experimental data

There are many sources of test errors that can affect the measured values of Atterberg limits and shearing strength. Weight measurements were of sufficiently precise nature that errors from such sources, if any, are felt to be negligible. With the methods* used in this study, remolded specimens were prepared with very small variations in initial density and water content. In all tests, initial compacted density (i. e., before test) was controlled to within ± 1.1 pounds per cubic foot (0.9 percent) and moisture content to within ± 0.35 percent. These densities and moisture contents are considered to provide results well within the precision of test measurements.

* See Experimental Methods and Procedure.



In order to detect errors due to uncontrolled conditions, duplicate samples were tested to ensure freedom from inconsistencies.

Evaluation of experimental results

The information presented in this dissertation gives a warning of dangerous conditions which may develop in a soil. The presence of large amounts of exchangeable sodium in a cohesive soil should be regarded as a danger signal. These results show also that the change in strength properties of an earth structure with time may be a result of a change in the percentage of the dominant adsorbed cations and/or the concentration of salts in the pore fluid.

The sloughing phenomenon which often occurs on the sloping sides of drain ditches may be explained as follows. The factor of safety for a cohesive soil which is statically stable will decrease if the dominant exchangeable ions (Ca^{++} , K^+ , and Mg^{++}) are replaced with Na^+ ions. The substitution of exchangeable sodium is a phenomenon which naturally occurs in the banks of open drains. Therefore, the strength of slopes decreases progressively with increasing exchangeable sodium percentage and failure may be brought about. The chance of failure increases when salt is removed from the slopes by percolating rain water or by irrigation with salt-free water. It should be realized, however, that there are other factors, such as moisture variation and mechanical rupture produced in soil strata through frost action, which contribute also to the slope's failure.

SUMMARY AND CONCLUSIONS

A study has been conducted to find out both the effect of different exchangeable cations and the concentration of ions in the soil pore fluid on strength and plastic properties of a sample of cohesive soil taken from the bank of an open drain ditch. Nine samples were prepared and tested for Atterberg limits and unconfined compression test. These samples were as follows:

- (a) Sodium-saturated sample (100 percent exchangeable Na^+)
- (b) Potassium-saturated sample (92 percent exchangeable K^+)
- (c) Calcium-saturated sample (98 percent exchangeable Ca^{++})
- (d) Magnesium-saturated sample (100 percent exchangeable Mg^{++})
- (e) HCl-treated sample resulting in a ($\text{H}^+ + \text{Al}^{+++}$)-saturated sample (100 percent exchangeable ($\text{H}^+ + \text{Al}^{+++}$))
- (f) Sodium and potassium-saturated sample (34 percent exchangeable Na^+ and 62 percent exchangeable K^+)
- (g) Sodium and calcium-saturated sample (46 percent Na^+ and 51 percent exchangeable Ca^{++})
- (h) Sodium and magnesium-saturated sample (43 percent exchangeable Na^+ and 56 percent exchangeable Mg^{++})
- (i) Sodium and ($\text{H}^+ + \text{Al}^{+++}$)-saturated sample (51 percent exchangeable Na^+ and 49 percent exchangeable ($\text{H}^+ + \text{Al}^{+++}$))

All samples were tested with distilled water in the pores (it should be realized, however, that some of the adsorbed ions dissociate in the pore water). In addition, the first five samples were tested using the chloride salt of the adsorbed ion as the pore-filling liquid. Hydrochloric acid was used as pore fluid for HCl-treated samples. Concentrations of electrolytes used were 0.1, 0.3, 0.5, 0.7, 1.0, and 1.5 normal.

For unconfined compression tests, remolded samples were prepared with very small variation in compacted density (± 0.9 percent) and water content (± 0.35 percent). Furthermore, duplicate samples were tested to ensure that all variations were within the allowed experimental errors.

The experimental results of this study may be summarized in the following points:

1. A general decrease of liquid limit and an increase in both shear strength and modulus of elasticity, with increasing salt concentration in the pore fluid, were observed for all samples, with the exception of HCl-treated samples for which shear strength was independent of HCl concentration in the pore water.
2. An increase in both shear strength and modulus of elasticity with decreasing exchangeable sodium percentage was definite.
3. The increase of soil shear strength caused by different exchangeable cations with distilled water in the soil pores was in the order

$\text{Na}^+ < \text{Ca}^{++} < \text{Mg}^{++} < \text{K}^+ < \text{HCl-treated soil}$, whereas the modulus of elasticity increased in order $\text{Na}^+ < \text{K}^+ < \text{Ca}^{++} < \text{HCl-treated soil} < \text{Mg}^{++}$.

4. A decrease in the liquid limit with decreasing exchangeable sodium was found.

5. The influence of the different exchangeable cations used on soil liquid limit was in the order $\text{Na}^+ > \text{HCl-treated sample} > \text{Mg}^{++} > \text{Ca}^{++} > \text{K}^+$.

From this study it could be concluded that:

1. The strength and plastic properties of cohesive soils are affected greatly by the type and concentration of adsorbed ions as well as by the ions concentration in the pore fluid.

2. The failure of the sloping sides of drain ditches may be attributed at least in part to increasing exchangeable sodium percentage.

3. For satisfactory and economical design of earth structures, a knowledge of chemical as well as physical properties of cohesive soils is essential. The boundary conditions which may change the soil physico-chemical properties with time should be considered.

4. Strength properties of HCl-treated soils are controlled by aluminum rather than hydrogen ions. Exchangeable aluminum was liberated as a result of the breakdown of the clay fraction in soils when treated with HCl.

5. The experimental results are not completely explained in terms

of the Gouy-Chapman theory of double layers. However, the Stern theory which considers the specific adsorption of ions on clay surfaces is a better alternative and does explain the results adequately.

Recommendations for Further Research

1. The study of the effect of organic matter, soil conditioners, and adsorbed organic ions (or molecules) on strength properties of soils is important in analyzing the stability of slopes.
2. An analytical study to relate soil compaction, structure, and strength properties of cohesive soils is needed before any attempt is made to develop numerical equations based on colloidal theory. Correction factors for noncolloidal particle concentration, externally applied forces, and degree of soil compaction would be necessary.
3. A detailed study regarding strength properties of acid soils is recommended for both its basic and applied importance. Methods for the preparation of hydrogen soils where exchangeable aluminum is reduced to a minimum should be investigated before relating the strength properties to exchangeable H^+ .

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