

DEPARTMENT OF SOILS INVESTIGATION

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A STUDY OF CHANGES IN PHYSICAL PROPERTIES OF PUTNAM SOIL INDUCED BY IONIC SUBSTITUTION

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SYNOPSIS

This paper presents the effect of chemical factors on the mechanical stability of a Putnam Clay soil. Through chemical treatment and the medium of base exchange this soil was altered to produce homoionic soils whose bases were (1) sodium, (2) potassium, (3) magnesium, (4) calcium and (5) aluminum. These modified soils along with the natural soil were tested, by standard procedures, to determine the following characteristics: (1) specific gravity, (2) size composition, (3) liquid and plastic limits, (4) shrinkage limit, (5) field and vacuum moisture equivalents, (6) optimum moisture content for compaction, (7) total and rate of consolidation, (8) permeability, and (9) shear tests by two methods. The results of these tests are given in six Tables and twelve figures and the relationship between the physico-chemical and the mechanical properties of the different soil systems are discussed. The paper gives 24 references.

A definite need exists in engineering practice to predict the physical and especially the mechanical properties of soils on the basis of fundamental concepts and easily performed tests. The work presented here was conceived to demonstrate on one soil, typical of northeastern Missouri, the important bearing of chemical factors on the engineering properties of soils. Also, it was planned to collect as many data of a quantitative character on this soil as was feasible, which in the future might be coordinated with data obtained on other soils in such a way as to furnish a solid foundation for a general theory concerning the chemical factors as determinants of the physical properties of cohesive soils. This study is primarily concerned with fundamental and theoretical concepts which can be derived from experimental data; however, practical engineering uses of the observed phenomena have already suggested themselves and certain specific applications have been anticipated by the practice.

It has long been realized that cohesive soils representing complex colloidal systems change their properties not only as a function of water content, but also as a function of temperature and pressure. The susceptibility of soils to changes caused by these conditions varies with the soil character. In view of the fact that the failure of the subgrade support often develops after the road has been constructed, the study of the susceptibility to change of the mechanical properties of soil is of utmost importance. It appears that a satisfactory understanding of soil behavior and the conditions which affect it must be based on comprehensive knowledge of both the physical and chemical factors involved.

In the past, engineers have described mechanical stability of soils in terms of friction and cohesion, neither one of these possessing a definite physical meaning. In order to give these terms definite meanings or to substitute for them factors which have such meanings, a series of tests must be performed on a con-

siderable number of physically and chemically well defined soils. Minor variations in soil character, such as can be easily brought about by ionic exchange, promise to be very valuable for this purpose.

Obviously, this paper can be concerned only with a portion of the general problem. It represents an investigation of the physical properties of a chemically well defined soil made so through controlled base exchange.

HISTORY AND AVAILABLE DATA

Many, though not all, of the physical characteristics of soil depend upon the quantity and kind of clay and colloid present. Several soil investigators have, therefore, given considerable attention to the physical and chemical properties of the soil colloids. However, only a limited amount of work has been done from an engineering point of view. So far, the engineer must glean information from the results obtained by the soil and colloid scientist and put this information into a form which is appropriate for his own purposes.

The concept of base exchange had its beginning in 1850 when J. T. Way (17)¹ found that soils are able to remove ammonia from solution. Later, in 1858, Eichorn (5) showed that natural double silicates, known as zeolites, have the power of exchanging one base for another.

Wiegner (18) (1912) showed that base exchange of silica gel has all the characteristics of an exchange adsorption process.

Turner (16), working with tropical soils, was able to evaluate the exchange capacities of both the organic matter and of the clay.

Hissink (7), de Sigmond (4), Kelley and Brown (8), and others have shown that physical properties of soils are de-

pendent upon the exchangeable bases of the colloids.

It has been shown by Bayer (1), Mattson (12), and others that the silica-sesquioxide ratio of the colloid and the nature of the adsorbed ions influence such physical properties as viscosity, plasticity, and volume changes of soil-water systems.

Winterkorn and his coworkers have made extensive studies of the cause and the mechanism of water adsorption by mineral surfaces, in order to lay a broad scientific foundation for the practice of soil stabilization. Among other facts, it was found that:

1. Mineral surfaces adsorb liquid films in accordance with the strength and character of the dipole of the component molecules (19).
2. The swelling of a soil system depends on the number and type of exchangeable cations per unit surface and on the geometrical character of this surface. Plane surfaces tend to hold thicker water films, though with less energy, than curved surfaces (2, 20).
3. The subgrade soil constants are functions of the adsorbed ions and are thus subject to control by chemical means (21, 22, 23).
4. The success of bituminous and other types of soil stabilization depends on the chemical factors of soil composition as well as on the type and amount of stabilizer used (24).

John D. Sullivan (14) has shown in his work relative to ceramics that plasticity, workability, and other properties of clays may be controlled by the use of proper exchange ions.

PREPARATION OF HOMOIONIC SOILS

The tests outlined in this report were conducted on a Putnam clay obtained one mile southwest of Moberly, Missouri, on

¹ Figures in parentheses refer to list of references at end.

TABLE 1

Size fraction	Predominant minerals	Refractive indices
(1) 2 mm. to 02 mm ..	About 80% oligoclase About 3% montmorillonite or beidellite Remainder. Muscovite, glaucophane, tourmaline, diopside and limonite	
(2) 02 mm to 5m.	About 30% of particles (albite) About 30% of particles About 20% of particles About 5% muscovite A little montmorillonite	1 516 to 1 536 1 536 (mean) 1 536 to 1 545
(3) 5 m. to 2 m	About 20% } About 40% } Chiefly feldspars About 30% } About 5% muscovite Small amount of beidellite and montorillonite Quartz rare.	1 516 { 1 527-1 536 1 536-1 545 1 545-1 553
(4) 2 m. to 500 mm	Mineral intermediate between feldspar and beidellite	
(5) 500 mm. to 200 mm.	Appreciable amount of intermediate mineral plus beidellite	
(6) 200 mm ..	Almost entirely beidellite	

the farm of Mr. Newman. Putnam subsoil is a gray-brown silty clay of glacial and loessial origin which has been developed by a podsolic type of weathering. The mineral composition of the different size fractions of this soil has been determined previously by Marshall (11) and is given in Table 1.

The total base exchange capacity of the soil was determined by means of the ammonium acetate method.² Tables 2 and 3 show the data obtained by means of the base exchange capacity tests.

Twelve hundred pounds of clay, part of which was used in another research project, was employed in making the different homoionic soil variations after the method used by Sullivan (14). One sixth of the total sample was left untreated, and is referred to as "natural" soil. The remaining quantity of soil was chemically treated to replace all exchangeable ions with hydrogen (H⁺). Five equal portions of "hydrogen" clay were chemically treated to produce homoionic soils whose

² Base exchange determinations were made by Dr. G. W. Eckert.

TABLE 2
BASE EXCHANGE CAPACITY OF PUTNAM SOIL
AS DETERMINED BY TWO TITRATION
METHODS

Method of titration	Base exchange capacity milliequivalents per 100 gms	
	Sample A	Sample B
Phenolphthalein indicator	29 32	29 32
Potentiometric titration	30 55	31 15
Average value	30 08	

TABLE 3
EXCHANGEABLE BASES CONTAINED IN THE
NATURAL PUTNAM SOIL

Exchange base	Milliequivalents per 100 gm soil	Method of determination
Hydrogen	12 0	Titration with ammonium acetate
Calcium	11 6	Calcium oxalate
Magnesium	4 5	Magnesium phosphate
Sodium	1 4	Magnesium uranyl acetate
Potassium	0 0	Sodiumcobaltinitrite
Total	29 5	

bases were sodium (Na^+), potassium (K^+), magnesium (Mg^{++}), calcium (Ca^{++}), and aluminum (Al^{+++}). The method of chemical treatment was as follows

After the clay had been obtained from the field, the total quantity of the sample was allowed to dry in air. When the clay was dried, it was crushed to pea-gravel size in a 6-in. capacity jaw crusher. One-seventh of the sample was further crushed to 1/16 in. size and was set aside as "natural" Putnam clay. The remaining portion was made into a slurry by mixing with distilled water in a 750-gal. capacity wooden tank equipped with a turbo mixer.

From the results of the determination of base exchange capacity, the amount of hydrochloric acid necessary to transform the raw clay into "hydrogen" clay was calculated.

A quantity of acid in excess of that needed for complete base exchange was added to give a resulting normality to the slurry of 0.05 N HCl.

The slurry was stirred for 15 min after the acid had been added and was then allowed to stand for 10 hr. At the end of this time, 200 gal. of distilled water were added, and the mixture was thoroughly stirred. Thereafter the mixture was allowed to stand over night and the clear supernatant solution decanted. The same procedure was applied successively until only traces of the chlorine ion could be detected after addition of silver nitrate.

At the completion of the washing, the water was removed as thoroughly as possible by decantation, and the slurry of hydrogen soil was divided into six equal parts, each of which was poured into large galvanized steel containers heavily lined with paraffin.

(An attempt was made to filter the slurry through an Oliver rotary vacuum filter equipped with a 3-ft diameter drum with a 1-ft face. However, the Putnam clay is so finely divided that a layer of soil

too thin to be removed from the face of the drum prevented the proper functioning of the filter).

A portion of the hydrogen clay was air dried and crushed to 1/16 in. size in roll crushers.

The five remaining samples were treated with hydroxides of the metallic ions which were previously enumerated. As the sodium and potassium hydroxides react readily with the soils, these two samples were air dried and crushed within a few days after the hydroxides had been added to the samples. However, as the calcium, magnesium, and aluminum hydroxides possess small solubility, these chemicals were prepared in colloidal form and were permitted to react with the soil during the three summer months to insure complete base exchange. At the opening of the school year, these samples were spread out to dry in air and then were crushed to 1/16 in. size.

TESTS PERFORMED ON THE NATURAL AND HOMOIONIC SOILS

The following characteristics of the different soil modifications were obtained by the standard testing procedures:³

1. Specific gravity by the pycnometer method.
2. Size composition by the Bouyoucos method as standardized by the U. S. Public Roads' Administration.
3. Subgrade soil constants (Public Roads' Administration), including:
 - a. Liquid limit
 - b. Plastic limit.

³ For description of tests and procedures see Casagrande, A., and Fadum, R. E., "Notes on Soil Testing for Engineering Purposes," Soil Mechanics Series No. 8 Harvard University, January, 1940. The subgrade soil constants were performed by the personnel of the Missouri State Highway Department. Most of the other test data were obtained by Wm. F. Lefevre, research assistant.

- c. Plastic index.
 - d. Shrinkage limit.
 - e. Field moisture equivalent.
 - f. Vacuum moisture equivalent (Bouyoucos).
4. Optimum moisture content for compaction and maximum dry density (Proctor).
 5. Total consolidation and rate of consolidation in Casagrande-modification of Terzaghi oedometer.
 6. Permeability by falling head permeameter and also from rate of consolidation.
 7. Shear resistance at different degrees of consolidation by plane-

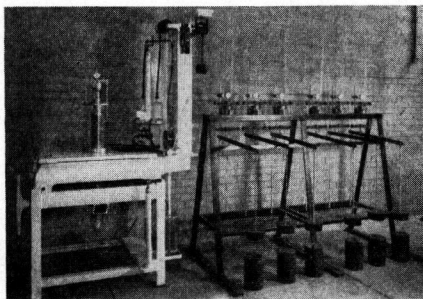


Figure 1. View of Cylindrical Compression Apparatus, and of Set of Casagrande-Type Consolidometers.

shear apparatus for cylindrical specimens.

8. Shear resistance of soils compacted at optimum moisture content (Proctor method) by means of cylindrical compression tests.

TEST RESULTS AND THEIR PHYSICAL MEANING

Table 4 and Figure 2 give the results under tests 1 to 4, inclusive. The results of the mechanical analysis are also given in Figure 3. It is interesting to note that there is an easy transition of part of the soil constituents from clay to the silt size and vice versa as a result of the exchange ions. This might be a result of incom-

plete dispersion occasioned by the action of certain exchange ions; or it might be caused by the difference in hydration of the clay particles possessing different ex-

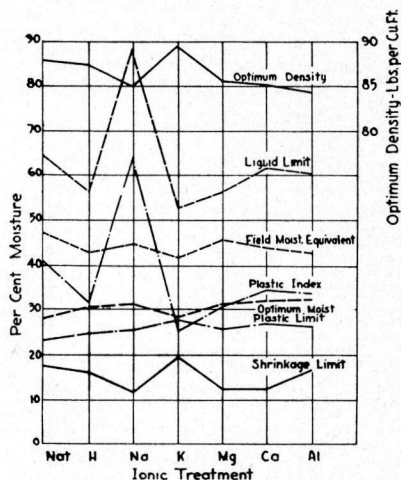


Figure 2. Properties of Putnam Clay as Functions of the Exchange Ions

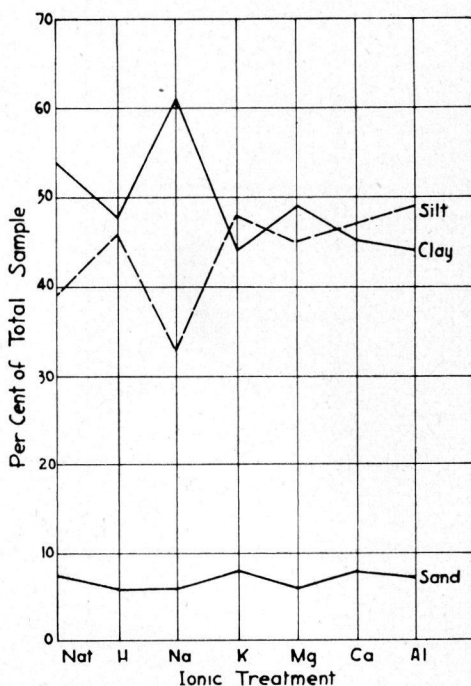


Figure 3. Variation in Apparent Size Composition as Function of the Exchange Ions.

TABLE 4
RESULTS OF SUBGRADE SOIL TESTS AND OF PROCTOR COMPACTION

Ion	Nat	H	Na	K	Mg	Ca	Al
TEST							
Lower liquid limit	64.5	56.4	88	52.8	56.3	61.9	60.2
Lower plastic limit	23.5	24.8	25.4	27.7	25.4	27.0	26.3
Plastic index	41.0	31.6	62.6	25.1	30.9	34.9	33.9
Volume change @ F M E.	56.0	52.4	68.6	40.1	68.9	63.8	51.7
Shrinkage limit	17.9	16.3	11.8	19.4	12.4	12.4	16.4
Shrinkage ratio	1.91	1.94	2.08	1.80	2.01	2.00	1.95
Field moisture equivalent	47.2	43.3	44.8	41.7	46.7	44.3	42.9
Specific gravity	2.708	2.663	2.661	2.675	2.645	2.680	2.721
Hygroscopic moisture	5.79	5.32	4.20	4.60	4.38	5.76	6.69
Vacuum moisture equivalent	57.1	54.3	Water logged	53.7	58.7	58.9	55.1
MECHANICAL ANALYSIS							
Per cent passing No. 40	100	100	100	100	100	100	100
Per cent passing No. 60	99.6	99.8	99.4	100	99.8	99.8	99.8
Per cent passing No. 200	98.1	98.7	97.5	98.5	98.6	98.5	98.3
Silt (05- 005 mm)	39.0	46.0	33.0	48.0	45.0	47.0	49.0
Clay (005 mm)	54.0	48.0	61.0	44.0	49.0	45.0	44.0
Colloids (001 mm)	33.0	26.0	48.0	21.0	25.0	22.0	23.0
PROCTOR COMPACTION							
Max dry weight, lbs per cu. ft.	88.0	87.4	85	89.6	85.6	85.2	84.4
Moisture at optimum density, %	28.6	30.9	31.3	28.4	31.2	32.3	32.4
Voids ratio	.915	.895	.951	.855	.924	.961	1.01

change ions. This latter factor is demonstrated by the following computation:

Assume a spherical soil particle possessing a radius of 0.001 mm. and a specific gravity of 2.65, experiments performed by Bayer and Winterkorn (2) have shown that:

1 gm. of K—Putnam clay may bind 0.50 gm. H₂O

1 gm. of Na—Putnam clay may bind 4.02 gm. H₂O

therefore

1 cc of K—Putnam clay may bind 1.33 cc H₂O

1 cc. of Na—Putnam clay may bind 10.65 cc. H₂O

This hydration makes the effective radius of the (0.001 mm. radius) solid particle

for K—Putnam clay = 0.00133 mm.
and its apparent specific gravity = 1.71;

for Na—Putnam clay = 0.00226 mm.
and its apparent specific gravity = 1.14;

Using Stokes' formula for standard conditions the following rates of fall are obtained for the different clay particles:

(1) Non-hydrated particle:
v = 0.036 cm. per sec.

(2) K-clay particle:
v = 0.027 cm. per sec

(3) Na-clay particle:
v = 0.015 cm. per sec

Using these rates of fall and calculating therefrom particle sizes, disregarding the hydration of the particles, one obtains the following apparent diameters:

(1) Non-hydrated particle: 0.0020 mm.

(2) K-clay particle : 0.0017 mm.

(3) Na-clay particle : 0.0013 mm.

In view of these and other deviations from the theoretical conditions which are encountered in the hydrometer analysis,

the data obtained by means of the latter can only be considered as indicative of both the size and the surface activity of the soil components

Another indicator of the amount of internal soil surface in a system which is a function of the particle size and of the activity of this surface is the liquid limit. By definition the liquid limit is that amount of water expressed in per cent per dry weight of soil which must be added to a soil in order that the water layers most distant from the soil particle surface acquire the properties of free water.

data on semi-logarithmic graph paper gives almost straight lines for the different homoionic soils. The penetration data obtained at the optimum moisture content of the different homoionic soils are well grouped together, indicating a condition of equal shear resistance for the soils at the optimum moisture content. The data for the plastic limit are much more scattered as regards the penetration of the needle. From a comparison of the methods for obtaining the plastic limit and the optimum moisture content for compaction, the procedure for the latter appears to be better defined and to be more

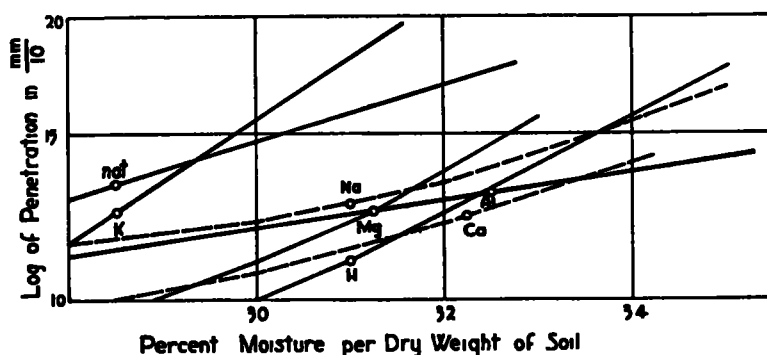


Figure 4. Relationship between Logarithm of Penetration and Moisture Content in Proctor Compaction. (Encircled Points Denote Optimum Moisture Contents.)

Soils at the liquid limit can be considered as of equal, though small, shear resistance. The shear resistance of a cohesive soil has been found to be approximately a logarithmic function of the moisture content.

The optimum moisture content for compaction can also be considered as representing a stage of equal shear resistance, the latter being defined by the impact necessary to mold the sample. The results of tests made on the penetration of a needle into samples compacted by the Proctor method at different moisture contents are given in Figure 4. A logarithmic relationship between depth of penetration and moisture content of the soil is again evident. The plotting of these

nearly a measure for such a property as shear resistance than the rolling procedure used for the plastic limit. This view is strengthened by the fact that extrapolation of the moisture per cent-log penetration curve to a penetration of about 1,000 gives the liquid limits for the different homoionic soils in the proper relative positions.

Further work along this line of thought might lead to a definition of the optimum moisture content as a point of equal shear resistance which similarly to the Casagrande procedure for the liquid limit, may be obtained by interpolation between the results of two trials. Such a method would be of considerable practical value.

It is significant in this connection that

the optimum moisture contents for the homoionic soils, while lying close to the plastic limits, have the same trend as the liquid limits.

Some time ago the Public Roads' Administration reported on a "Study of Sand Clay Materials for Base Course Construction" (3). In this investigation the resistance to extrusion of soils compacted by the Proctor procedure was determined by means of the Hubbard-Field apparatus. Out of five samples which possessed essentially identical size composition curves but varied in their plastic index, the Hubbard-Field stability at maximum density fell rather closely together for the three samples of intermediate plastic indices (3,025; 3,195; 2,790 lb.; for P.I.'s 5; 9; and 13, respectively). The non-plastic soil gave 1,450 lb. while the very plastic soil (P.I.=18) gave 1,975 lb. It is quite reasonable that the soil with P.I.=0 should fall out of line. The deviation of the very plastic soil is probably due to the fact that the extrusion apparatus of Hubbard-Field causes types of deformation and plastic flow different in a very plastic material from those in a less plastic system. Therefore, the data of the Public Roads' Administration appear to corroborate the hypothesis that at and immediately after compaction of a cohesive soil the state of optimum moisture content and maximum density represents a state of equal shear resistance. However, this shear resistance is liable to change with time as a function of the physico-chemical nature of the internal soil surfaces.

In summarization, the soil data given in Table 4 and in the accompanying figures demonstrate that:

1. The consistency constants are functions of the exchange ions.
2. The curve for the optimum moisture content for compaction is, in general, close to and somewhat higher than that for the plastic limit. The potassium clay, how-

ever, is an exception in so far as it possesses the highest plastic limit and the lowest moisture content for optimum compaction.

3. The sodium soil shows the greatest difference in moisture content of all between the shrinking limit and liquid limit, while the potassium clay shows the smallest.
4. There appears to exist a correlation between the shrinkage limit and the dry weight at optimum compaction. The higher dispersity which increased the lubrication requirements for the sodium clay and made for a high liquid limit, also resulted in a greater density when the soil was dried out entirely; however, for the different ionic soil modifications the curve of the moisture contents needed for lubrication in the Proctor compaction goes more or less parallel to that for the liquid limit.

CONSOLIDATION

The consolidation tests were for determining relative volume changes as well as the time-rate of volume decrease, as a function of the ions, that a laterally confined specimen undergoes when subjected to axial loads.

The consolidation tests were made in Casagrande type consolidometers, Figure 1. Sufficient water was added to a quantity of soil to raise its moisture content to the vicinity of the liquid limit. The sample was then allowed to stand over night to adsorb the water.

The sample was put into the consolidometer after the Ames dials had been set in place at a height predetermined by means of a gage block, accurately machined to 1 in. Thus, the thickness of the sample could be measured at any time.

Four increments of load of 1,000, 2,000, 4,000, and 8,000 lb. per sq. ft.

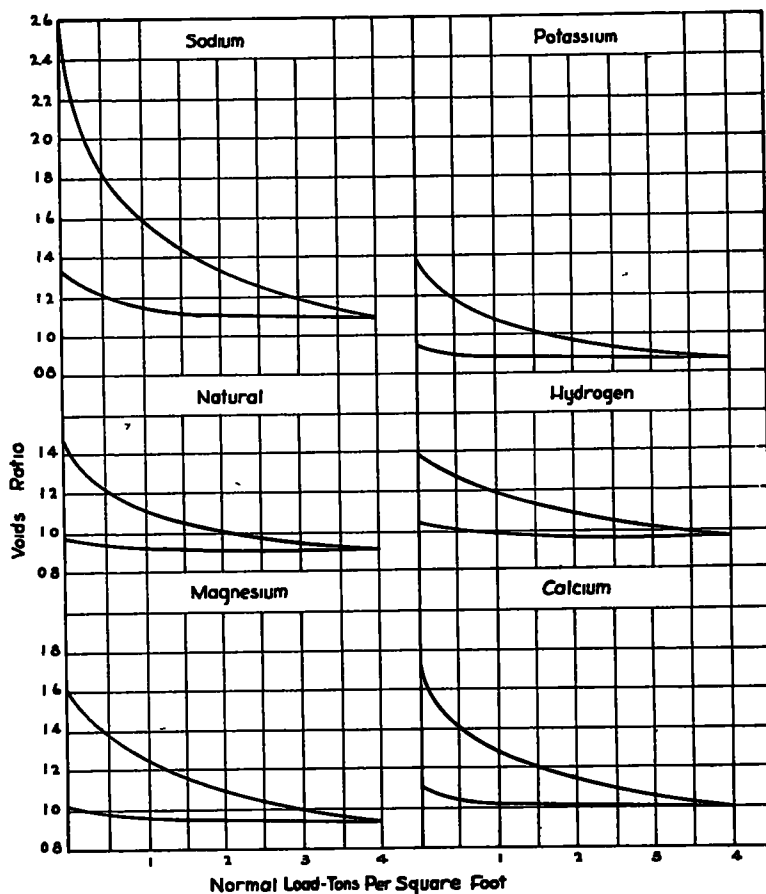


Figure 5. Load-Voids Ratio Relationship of Homoionic Putnam Soils

were applied to the samples. Each increment was allowed to act until the sample showed little or no consolidation in a 24-hr. period. The experimentally obtained load-voids ratio relationships are given in Figure 5.

Terzaghi (15) has developed the theory of the consolidometer on the basis of the analogy of the phenomena involved with those encountered in the cooling of a plate.

If

e = voids ratio,

p = vertical pressure,

$a^1 = \frac{de}{dp}$ = coefficient of compressibility,

k = coefficient of permeability,

g = density of water,

$K = \frac{k(1+e)}{g \cdot a^1}$ then the

fundamental equation for the consolidation process is

$$\frac{Kd^2u}{dz^2} = \frac{du}{dt}$$

Wherein:

u = excess hydrostatic pressure in soil

z = distance of a point from the surface of the sample

t = time

Integration of this equation between the

desired limits and manipulation of the resulting formula give.

$$c = \frac{0.848 H_0^2}{t_{90}}$$

Where

$$c = \frac{K}{(1+e)^2} = \text{coefficient of consolidation};$$

t_{90} = time in seconds at which 90% of the theoretical consolidation is attained;

H = one-half of the total height of the specimen and

$$H_0 = \frac{H}{1+e} = \text{one-half of the reduced height of the specimen}$$

If the value for c is known from the settlement analysis, then the coefficient of permeability can be determined from:

$$k = ga'c(1+e) \text{ or}$$

$$k_0 = gra'c, \text{ where}$$

g = density of water

a' = coefficient of compressibility (corrected to correspond with 100% theoretical consolidation, according to M. I. T. fitting scheme)

c = coefficient of consolidation, and

k_0 = reduced coefficient of permeability.

THE COEFFICIENT OF CONSOLIDATION

The time-consolidation curves for the different homoionic soils are given in Figure 6. From these the coefficient of compressibility, c , was obtained by the

use of the equation: $c = \frac{0.848H_0^2}{t_{90}}$ in

which the factors are defined as before. The values for t_{90} were obtained by employing the M. I. T fitting scheme. The c -values for the different ions and the different pressure ranges are given in Table 5. This table also contains the average e -values which were used in making Figure 7. This graph giving the

c -values as a function of the voids ratio for the different ions is expressive of the total influence of the exchange ions on the consolidation process. This influence is a double one, first on the amount of water which can be held under the different compression loads (see de/dp curves in later test) and on the viscosity of the water which is being expelled under the different hydrostatic excess pressures in the system. The coefficient of consolidation is the smallest for the Na-modification and largest for the K-modification. The apparent lack of trend of the c - e curves suggests that a closer study of the meaning of the c -value is in order. The coefficient of consolidation has been in-

troduced as the quotient $\frac{k}{a'(1+e)}$, k and a' being the coefficient of permeability and compressibility respectively. In a structureless soil where every particle is completely and uniformly surrounded by an adsorbed water film the ease with which part of the water film may be sheared off (compressibility) and the viscosity of the expelled water (permeability) should follow the same function. The experimental data indicate strongly that this function is a linear one, i.e., that for structureless cohesive clays c or $\frac{k}{a'}$ is not

a function of the voids ratio. Deviations of the experimental data from a constant value are probably due to structural influences, in so far as they are not caused by inherent limitations of present methods of test procedure and evaluation.

THE COEFFICIENT OF COMPRESSIBILITY

The coefficient of compressibility has been defined as $a' = \frac{de}{dp}$. Figure 5 shows the experimentally determined voids ratios as functions of the consolidation pressures. Table 5 shows the coefficients of consolidation as functions of the exchange cations and of the consolidation range.

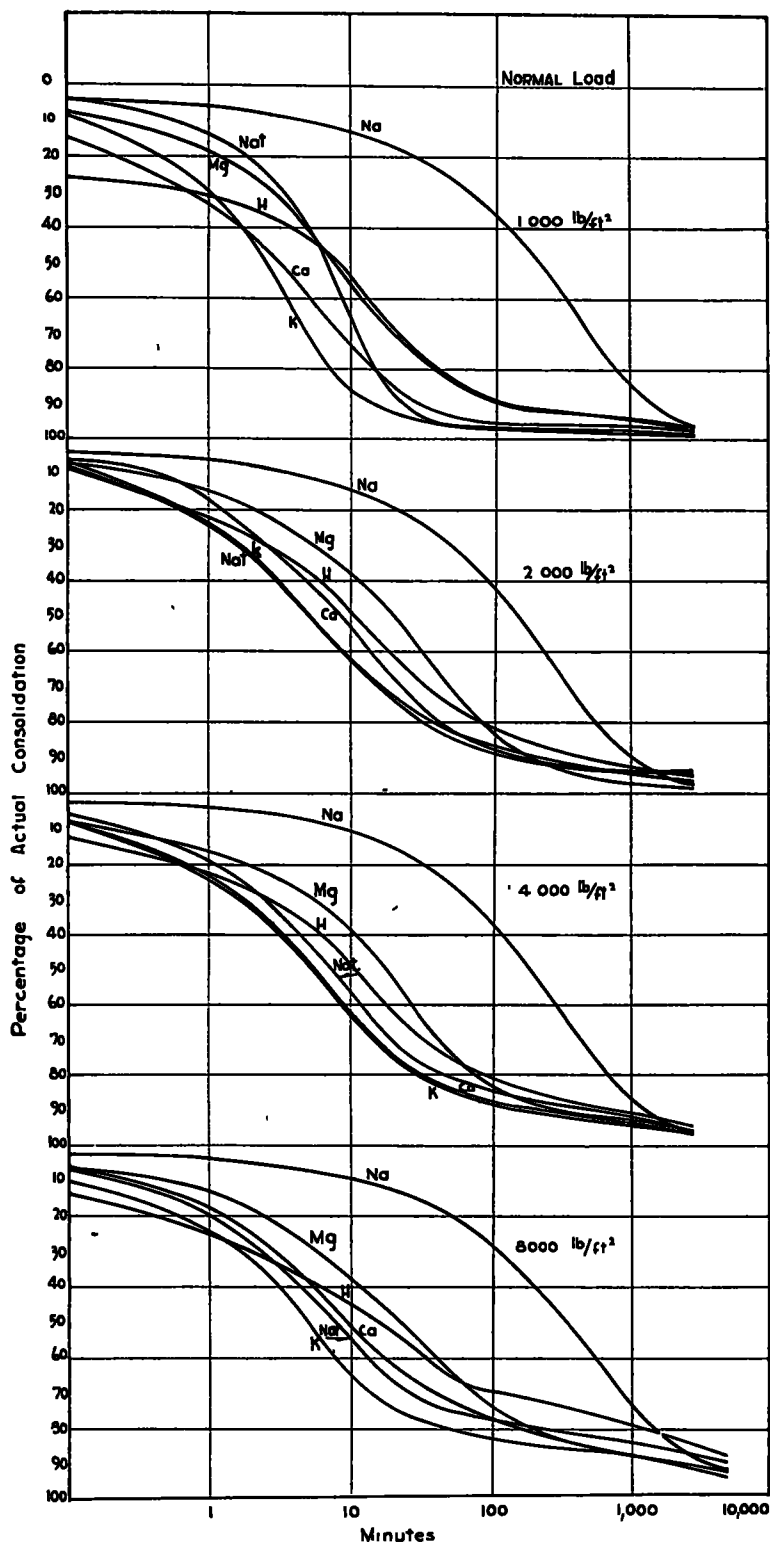


Figure 6. Influence of Exchange Ions on Consolidation Rates

The voids ratio at the liquid limit has been chosen as representative for zero consolidation pressure.

Figure 7 shows the coefficient of compressibility for the different homoionic soils as a function of their voids ratios. Since the compressibility indicates the intensity of the forces holding the water

set of curves. The great difference between the effect of the Na- and K-ions is impressive.

THE COEFFICIENT OF PERMEABILITY

From consolidation data

The coefficient of permeability k can be calculated from the equation $k=ga'c$

TABLE 5
PHYSICAL DATA DERIVED FROM CONSOLIDATION TEST

P-range in Kg per cm ²	IONS	H	Na	K	Mg	Ca	Nat	
0-1	t90	2,306	49,700	614	1,140	870	1,450	seconds
	C	0 74	0 0172	2 97	1 615	1 57	0 901	×10-4
	K ₀	0 925	0 1162	6.92	4 15	3 03	2 72	×10-8
	a'	1 25	6 75	2 33	2 57	1 93	3 02	×10-4
	C av.	1 33	2 16	1 28	1 48	1 57	1 37	.
	C ₁ av.*	1 40	2 10	1 30	1 45	1 53	1 48	.
1-2	t90	1,815	34,560	614	3,375	1,009	615	seconds
	C	0 94	0 0247	2 97	.545	1 350	2 125	×10-4
	K ₀	0 751	0 0556	2 58	60	1 50	2 29	×10-8
	a'	0 80	2 25	0 87	1 10	1 11	1 08	×10-4
	C av.	1 22	1 71	1 12	1 30	1 34	1 17	.
2-4	t90	1,880	36,700	740	3,850	1,220	1,430	seconds
	C	0 91	0 0233	2 46	0 478	1 12	0 913	×10-4
	K ₀	0 473	0 0305	1 30	0 433	79	0 53	×10-8
	a'	0 52	1 31	0 53	0 75	70	0 58	×10-4
	C av	1 13	1 46	1 02	1 17	1 21	1 05	..
4-8	t90	960	53,000	735	3,370	1,220	1,380	seconds
	C	1 78	0 0161	2 48	0 546	1 12	0 947	×10-4
	K ₀	0 605	0 0100	0 70	0 208	0 39	0 23	×10-8
	a'	0 34	0 62	0 28	0 38	0 35	0 24	×10-4
	C av.	1 02	1 21	0 91	1 02	1 08	0 95	.

*C₁av is average C if the voids ratio at the liquid limit is taken as that representative for stress condition $P=0$. This C₁av is used only for the calculation of the $\frac{de}{dp}$ data

films, this figure gives a picture of the water-fixing ability of the Putnam soil as a function of the exchange ions. At a voids ratio of 1.1, for example, the intensity of the water-holding forces decreases in the series: Na>H>Mg=Ca>K=Nat. At a voids ratio of 1.5, the series is: Na>Ca>Mg>Nat>K. For any e-value the intensity of the water holding forces may be obtained from the

(1+e) which has been given above. For purposes of comparing the effect of the different ions on the permeability it is more appropriate to use the reduced coefficient of permeability which is defined by

$$k_0 = \frac{k}{(1+e)} = ga'c$$

The k_0 values for the different pressure ranges are given in Table 5. Figure

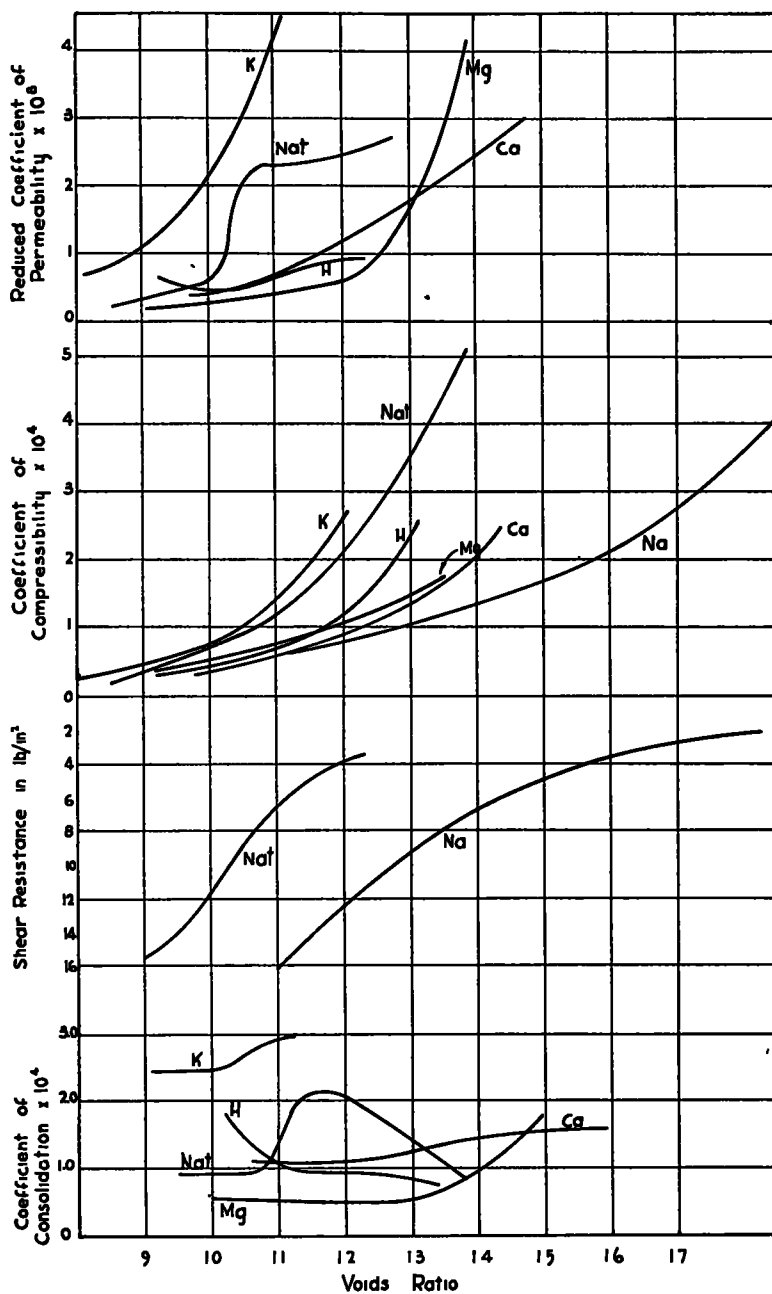


Figure 7. Mechanical Properties of Putnam Soil as Functions of Exchange Ions and of Voids Ratio

7 gives the reduced coefficients of permeability for the different homoionic soils as a function of the voids ratio. Abrupt changes in the direction of the curves indicate different thicknesses of water layers the viscosities of which are appreciably influenced by the exchange ions present. Again, the greatest influence is exerted by the Na-ion, the smallest by the K-ion.

From falling head permeameter tests

The coefficients of permeability were also determined by permeameter tests.

than when made on the compression branch. This phenomenon might be due, at least to some extent, to a greater disturbance of the dipole linkages between the particles in expansion than occurs in compression. Obviously, in compression only the weakly held water dipoles are expelled, while in expansion the whole dipole chain may be disturbed resulting in a lowered viscosity of the water.

Figure 8 gives essentially the same data as Figure 4, however, in different form. They show the change in volume

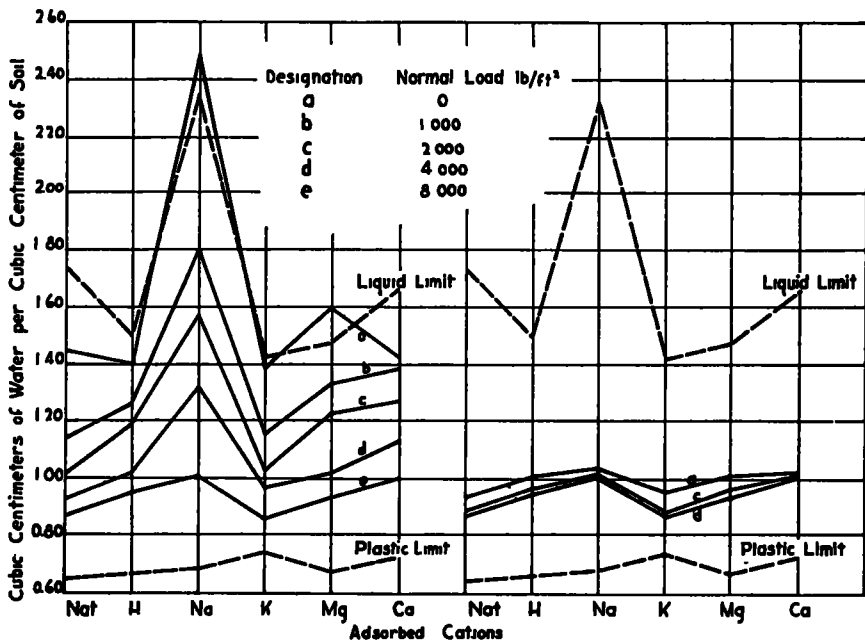


Figure 8. Water Holding Capacity of Putnam Soil as a Function of the Exchange Ions and of the Normal Pressure

The same general picture of the influence of the exchange ions obtained, although there are the expected variations of the actual experimental data from those calculated on the basis of the Terzaghi theory of consolidation. Permeability constants for the same voids ratios of a specific soil tended to be larger when the experiment was made on the expansion branch of the consolidation-expansion test

of water held per cubic centimeter of soil as a function of the consolidating loads and of the ions. Sodium soil suffers a large decrease in volume of water as the loads are applied. The volume of water in the natural soil decreases considerably under the first load increment, but, even as the loads become large, the volume change is relatively small. Calcium soil has the least amount of total volume

change of all the homoionic soils in both consolidation and expansion. This indicates that the water films around the calcium soil particles are compact and are subject to only moderate reduction by increase in load. The small increases of moisture content during the reduction of pressure indicate that the calcium has but small attraction for additional water. In the case of potassium, the water film is also compact and slightly reduced by pressure; however, when the load is entirely removed, a large volume increase results which is due either to a strong attraction of the potassium ion for water or to an increased springiness of the soil system. Sodium soil, if unconfined, will attract a large amount of water. However, this attraction is not sufficiently forceful to produce a marked volume increase when consolidation loads are removed.

As has been indicated in the foregoing discussion, the effects of the ions on the engineering properties of soils may be due not only to the change in water affinity by the ions, but also to changes of the elastic properties and of the aggregation of the solid soil constituents. As far as organic matter is concerned, such an effect has been demonstrated on a loess pampaneo (23).

SHEAR TESTS

The stability characteristics of the homoionic soils were investigated by means of plane and triaxial shear tests. For the plane-shear tests six samples of each soil mixed with water to bring them to the lower liquid limit were placed in Casagrande type consolidometers, and consolidated each under a different vertical load. When the sample had ceased to consolidate further under load, it was transferred directly from the consolidometer into the plane shear device. The shear specimen was subjected to the same normal load under which the sample was consolidated. Since the sample had expanded slightly while it was being trans-

ferred to the shear device, the normal load was applied until no further consolidation was noted prior to shearing.

Shear load increments were applied at 1-min. intervals until failure of the specimen.

Moisture contents of the samples were determined at the completion of the shear test and voids ratios of the consolidated samples were calculated. Table 6 gives the data for the shear resistance recorded for the respective axial consolidation loads, and the corresponding moisture contents.

Figure 7 contains the plotted shear resistance-voids ratio relationships obtained by plane shear tests on natural and sodium clays. At a particular voids ratio the shear values differ widely for the homoionic soils. The data used in this figure have been obtained by McDaniel and are believed to be of somewhat greater accuracy than those obtained by Lefevre. However, they are in close range with those obtained by the latter. Following the same line of reasoning as was applied in the discussion of the *c-e* function, it might be supposed that the compressibility of a water saturated cohesive soil is closely correlated with the shear resistance of the soil if compression means the expulsion of water molecules which are in the attraction field of the soil particles. The simplest such function would be:

$\text{Shear resistance} \times \text{Coefficient of Compressibility} = \text{constant}$ for a specific soil over most of its *e*-range. If this product is formed using the data of McDaniel the values for the sodium soil at voids ratios of 1.8, 1.6, 1.4 and 1.2 compare as 7.6, 6.65, 7.8 and 7.45 respectively; those for the natural soil for the voids ratios 1.2, 1.1, and 1.0 as 4.95, 4.95 and 4.75 respectively.

Relative to these changes in shear values, attention might be drawn to the findings of Kogler and Scheidig (9) indicating a general inverse relation between the angle of friction and the plastic index.

According to this generalization, the angle of friction of natural Putnam clay can be increased from 19 deg. represented by a plastic index of 41 to an angle of 23 deg. represented by a plastic index of 25 through treatment with potassium The sodium ions changing the plastic index to

62 should reduce the angle of friction to about 12 deg Figure 9 made up of experimental data obtained by Fefevre are in general agreement with those calculated by means of the Kogler-Scheidig relationship.

In recent years the triaxial shear test

TABLE 6
PLANE SHEAR DATA

Sample	Shear load lb per sq in	Normal load lb per sq. in	Moisture content
Nat 1 .	4 00	7 32	43 7
	7 34	14 26	41 5
	12 10	28 14	37 2
	17 60	42 02	35 0
	20 40	55 90	34 2
	27 80	69 78	34 6
Nat 2	4 00	7 32	45 0
	7 34	14 26	40 4
	13 60	28 14	38 8
	15 00	42 02	35 7
	14 85	55 90	34 1
	23 10	69 78	33 0
K	5 37	7 32	40 0
	6 65	14 26	37 7
	14 85	28 14	35 0
	20 40	42 02	32 8
	25 90	55 90	31 6
	31 30	69 78	30 2
Na	4 00	7 32	59.8
	7 34	14 26	56 4
	9 49	28 14	47 6
	12 20	42 02	42 0
	15 00	55 90	40.8
	17 60	69 78	40 1

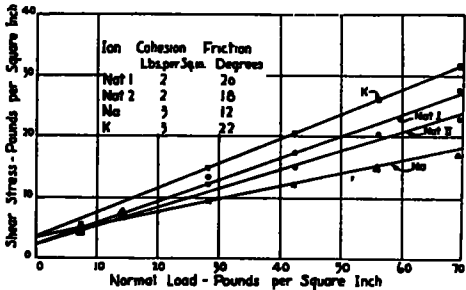


Figure 9. Influence of the Exchange Ions on the Angle of Friction

or cylindrical compression test has gained prominence in engineering soil testing. In this method, a specimen is compressed as in an ordinary compression test, except that it is surrounded by a liquid under a definite pressure. The axial pressures leading to failure of the specimen at the different lateral pressures are determined and the data are analyzed by means of Mohr's theory to obtain the cohesive and frictional properties of the soil system under consideration. Also,

deformations of the specimens as functions of the normal and lateral pressures are usually determined and evaluated as far as possible.

In the present investigation a series of triaxial tests was performed on samples of homoionic soils which had been compacted to maximum density at optimum moisture content in order that the density and moisture conditions of the samples

in. vertical load intensities with different confining pressures. The same figure shows the percentage of moisture of the specimens as a function of the ions.

In general, the deformation is large for those ionic soils which possess high optimum moisture contents. The deformation data show that stability of cohesive soils is functionally connected with the thickness and structure of the moisture films surrounding the soil particles

Figure 10 also gives penetration data in tenths of millimeters obtained by means of the cone penetration method. In gen-

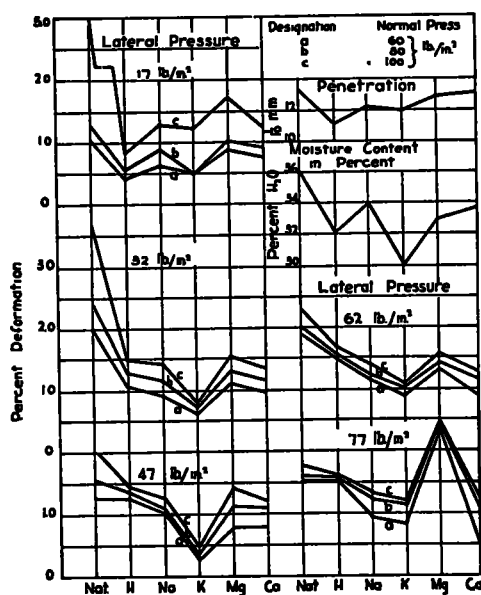


Figure 10. Deformation in Cylindrical Compression Test as a Function of the Exchange Ions.

would be on a comparable basis for analysis of the test results and for correlation with the water film characteristics. Compaction was accomplished by use of a standard Proctor hammer dropped from a height of one foot eleven times on each of four layers of soil in a mold 2.8 in. in diameter and 8 in. high. The samples were kept in a moist room to prevent loss of moisture and were tested the same day on which they were compacted.

Figure 10 shows the data on unit deformation of the homoionic soils in triaxial compression for 60, 80, and 100 lb. per sq.

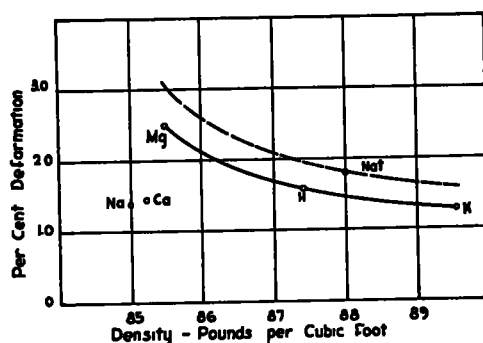


Figure 11. Influence of Exchange Ions on Stability. (Lateral Pressure 77 lb./in.², Axial Pressure 100 lb./in.²)

eral, the soils with a low optimum moisture content have low penetration values. The trends of the curves for penetration and unit deformation are in correlation, that is, the higher the penetration, the greater the deformation.

Figure 11 is a plot of unit deformation of the homoionic and natural soils against the density (dry weight per cubic foot) of these soils. The values of the deformations were taken arbitrarily from the triaxial shear tests in which the samples were subjected to a lateral pressure of 77 lb. per sq. in. and an axial stress intensity of 100 lb. per sq. in.

For a single soil, it can be expected that deformation will decrease with increase in density along some curve as shown by the dotted line in Figure 11.

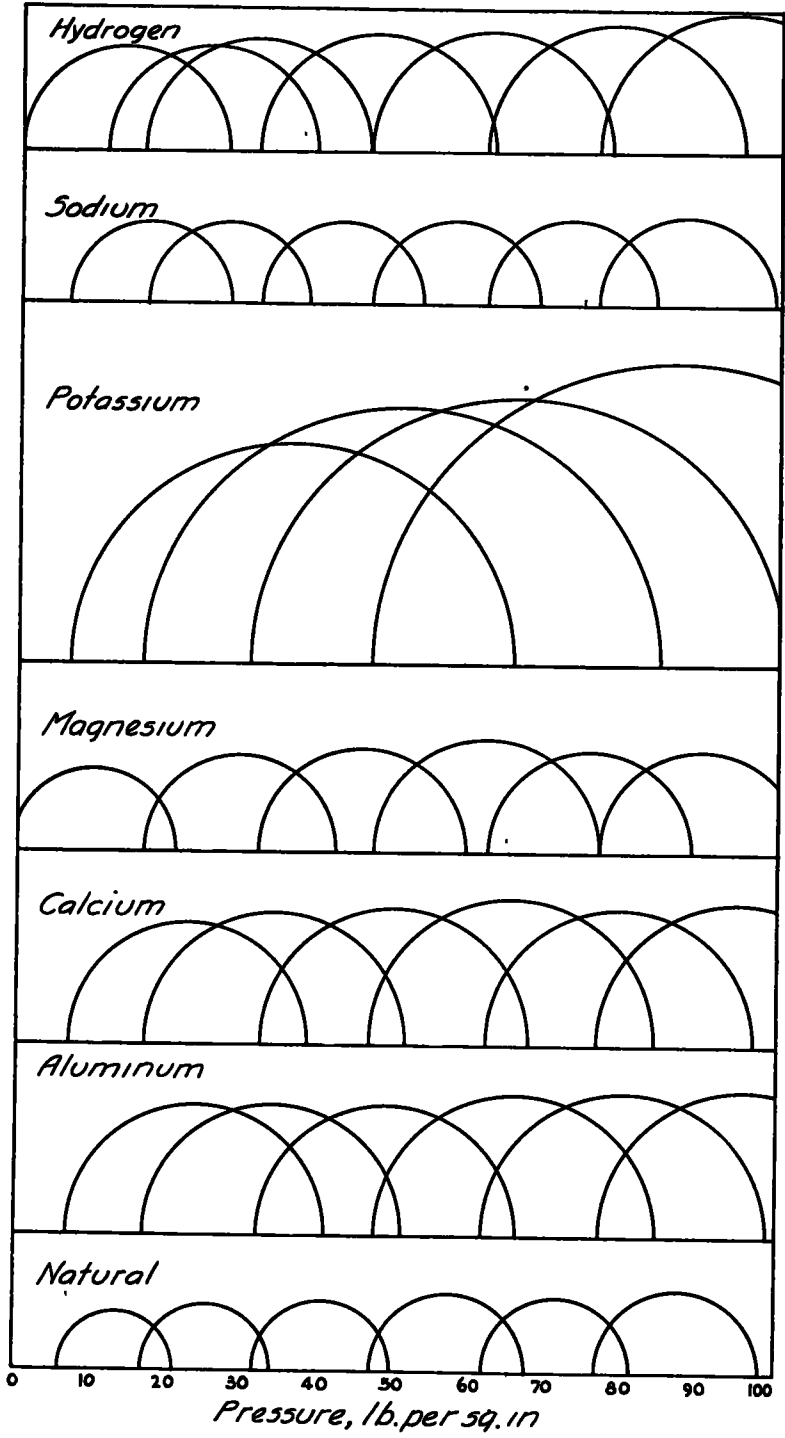


Figure 12. Mohr-Circles for Homioionic Putnam Soils

Using this curve as a base, it can be seen that all the homoionic soils are (at optimum Proctor compaction) more stable than the natural clay. Although the sodium and calcium soils have relatively low optimum densities, their inherent stability is markedly greater than that of the natural soil.

A curve through the plotted points of magnesium, hydrogen, and potassium soils show an increase in stability with increase in density as would be expected in this general relationship. It could also be expected, then, that these soils would exhibit similar stability properties. This fact is borne out in the work by Gibbs (6). Curves plotted from penetration-moisture content data for magnesium, hydrogen, and potassium are close to each other, indicating that these three soils possess similar stability properties, while curves for the other soils lie in different ranges.

From consideration of the permeability characteristics of the Putnam soils it is obvious that the data obtained in the triaxial tests do not represent equilibrium conditions. The low permeability and the size of the specimen did not permit the dissipation of the hydrostatic excess pressure in the system caused by the rather suddenly applied lateral and axial pressures. For this reason all the triaxial data obtained in this study are only of an indicative character. The fact that equilibrium conditions were not reached in the test showed itself quite definitely in the analysis of the data by Mohr's method resulting in envelopes of very small or no slope. The results of this analysis are shown in Figure 12. This figure may serve to emphasize the difficulty of making and evaluating triaxial tests on cohesive soils

CONCLUSIONS

The work which is presented in this paper has been performed to indicate some of the connections between the

physico-chemical and the mechanical properties of soil systems. Although many types of soils must be treated and tested in a similar manner before final conclusions can be reached and definite laws established, certain noteworthy trends have been demonstrated and may be summarized as follows:

1. The engineering properties of a soil can be radically altered through change of the adsorbed ions
2. The physical properties of a cohesive soil are dependent upon the structure and character of the water films surrounding the solid soil constituents.
3. The water film structure is determined by the intensity and the extension in space of the force fields about the adsorbed ions and their interrelation with the force fields of the solid soil particles.
4. Variations in the shear and consolidation behavior of homoionic soils containing intermediate amounts of moisture can be pictured as due to changes, caused by the ions, in the viscosity of the adsorbed water films
5. The Atterberg tests furnish evidence of the water film structure which can be used in predicting trends of other engineering properties of soils

It is hoped that the results of this investigation may represent a definite though small advance toward the final goal of predicting the engineering properties of soils on the basis of a few fundamental tests and concepts.

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