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Bulletin 108

SOIL AND SOIL-AGGREGATE STABILIZATION

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1955

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2101 Constitution Avenue Washington 25, D. C.

Dedicated to

CHESTER A. HOGENTOGLER

1887-1949

Chester A. Hogentogler

Pioneer in the experimental observation of the qualities and behavior of natural soils as foundations and surfaces of roadways;

Leader in the formulation of testing methods for determination of the mechanical properties of soils;

Developer of the first practical system of classification of soils according to their relative suitability for use in highway construction; and

Preeminently successful in his long and patient effort to disseminate among road builders everywhere a correct understanding of the character and uses of soils in highway construction, as revealed by research.

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Bulletin 108

Soil and Soil-Aggregate Stabilization

A SYMPOSIUM

PRESENTED AT THE
Thirty-Fourth Annual Meeting
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1955

Washington, D. C.

Department of Soils

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Chief, Soils Section, Bureau of Public Roads

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F. L. D. Wooltorton, Acting Planning Engineer, c/o P. W. D. Head Office, P. O. Box No. 662 Nairobi, Kenya Colony, East Africa

Preface

THE purpose of this bulletin is to portray the present state of the science of soil stabilization, that body of principles which must guide the responsible engineer in the design and execution of actual projects and also in his decisions with respect to the potential merits of methods suggested by advances in other sciences.

The position of the responsible engineer is similar to that of a modern surgeon, who utilizes and organizes the assistance of the physiologist, the pharmacologist, the anesthesist, and others of his staff, who is open to their suggestions but who carries the responsibility and must make the decisions. These decisions are properly based on his own science and his practical experience. The responsible executive engineer must do likewise.

The domain of the science of soil stabilization has been outlined on several previous occasions. Ancillary to this science are the sciences of geology, pedology, climatology, microbiology, macro- and micromeritics (including granulometry), chemistry and physics, colloid science as related to surface phenomena, and the presently developing science of construction materials. Without the tremendous aid of these sciences there could not exist a science of soil stabilization. However, a mere summation of all these sciences would be at once much more and much less than the science of soil stabilization.

The latter, the same as every other science, became organized when the body of specific knowledge in its particular field had become so large that it could not be retained by even the most prodigious memory and when the need for practitioners of the art could no longer be filled by the traditional master-apprentice method. Science, then, represents economy in the retention and transmission of knowledge. A good example of the working of this economy is the invitation extended last century by the House of Commons to MacAdam and Telford to present the principles of their road construction methods in order that the supervisors of other road districts should be enabled to equal the performance of these two eminent engineers.

The great economy in retention and transmission achieved by the organization of a field of knowledge into a science does not eliminate the need for a teacher and for practical experience before responsible mastership can be attained. Rather, it is the practice which gives the breath of life to the science that otherwise would remain a pure and sterile exercise of the mind.

The emphasis in this symposium is on scientific facts and principles that can and have been applied and on factors and phenomena that must be taken into account in the engineering application of soil stabilization methods already developed or which will be developed in the future. In the center of the picture is the scientific engineer, and the committee organizing this symposium feels very gratified indeed about the prominent and responsible scientist-engineers that could be induced to contribute to the symposium. That three of these are from Africa, reflects the important activities on that awakening continent.

Yet, with this emphasis on the responsible scientific engineer, it is not forgotten that much of the knowledge needed for his work must be furnished by representatives of other sciences and disciplines, and grateful ear is lent to the geologist, the pedologist, the soil physicist, the colloid scientist, and the microbiologist who have so kindly responded to the call to contribute to this symposium and to our science. Unfortunately, within the time-space frame of this symposium it was impossible to have representatives of all our sister and ancillary sciences or even to have those represented show all that they can do and already have done for us. Neither was it possible to have our own science presented in every important detail or even in every important phase. However, it is hoped that what could be achieved gives a true though necessarily limited picture of our science at work and that details which had to be omitted are sufficiently covered by the references in the respective sections and also by the recorded discussions.

The members of the committee sponsoring this symposium and all those that contributed to its realization would feel richly rewarded if its contents should prove to be of help to the engineer, who wants to be informed on the scientific principles of soil stabilization, and to the scientist in adjoining and ancillary fields who would like to know in what areas and phases he could most effectively contribute to our science and to the main purpose of our science. This purpose is to provide mankind with the knowledge and tools for building more and better roads, airports, houses, and other structures urgently needed in many parts of the world.

Acknowledgment

The Chairman of the Committee on Physico-Chemical Phenomena in Soils is grateful for the excellent cooperation enjoyed by him in the organization and presentation of this Symposium on the Science of Soil Stabilization.

First of all, thanks are due to the contributors of papers who took time out from their busy professional lives to prepare for us the excellent presentations which we just had the privilege to experience. Certainly, we all feel that these papers gave us not only a better understanding of those methods of soil stabilization that already are important tools of the highway engineer—granular-soil stabilization, soil-cement, soil-bitumen, and soil-resin—but have also shown the way to future developments. Thanks are due, also, to the members of the Committee on Physico-Chemical Phenomena in Soils for critically perusing the contributions and for suggesting improvements and amendments that were kindly accepted by the contributors.

Great appreciation is expressed to Fred Burggraf, director of the Highway Research Board; Frank R. Olmstead, Chairman of the Department of Soils; and to the other officers and members of the Board for their sincere interest and effective help in making this symposium a reality and a success.

Organization and presentation of this symposium would have been impossible without the keen interest shown in this undertaking by W. Mack Angas, vice admiral U. S. Navy, ret., chairman of the Department of Civil Engineering, School of Engineering, Princeton University. The Department of Civil Engineering of Princeton University defrayed the expenses for secretarial help, for reproduction of the papers and illustrations needed for advance circulation, for the redrafting of figures that had been submitted in foreign languages and with foreign measures, and for a number of other items incidental to this type of undertaking. For this generous help grateful acknowledgment is expressed to Admiral Angas and to J. C. Elgin, dean of the School of Engineering, and Harold W. Dodds, president of Princeton University.

—HANS F. WINTERKORN, Chairman,
Committee on Physico-Chemical
Phenomena in Soils

Contents

The Science of Soil Stabilization

HANS F. WINTERKORN, Director,
Soil Physics Laboratory, Princeton University

This paper defines the science of soil stabilization and views soil systems from many different angles in order to lay the proper foundation for an understanding of the different soils encountered by the engineer, of the desirable and undesirable properties of these soils, and of the possible supplementation of these properties in order to change the soils into construction materials. This introductory paper is limited to the pointing out of important fundamentals and also to the presentation of certain approaches and data that are not normally found in soil-stabilization literature.

● THE science of soil stabilization is that body of principles that explains and guides the practice of soil stabilization. Soil stabilization, in its widest meaning, comprises every physical, physico-chemical, and chemical method employed to make a soil serve better its intended engineering purpose. In its specific meaning, as commonly understood in highway and airport engineering, soil stabilization is the name given to those methods of construction in which soils are treated to provide base courses, and occasionally surface courses, which can carry the applied traffic loads under all normal conditions of moisture and traffic for an economic service life of the paved area. The paved areas may be roads, airport aprons and runways, parking and loading places, feeding courts or other surface structures of comparable stability and durability requirements.

The major established uses of soil stabilization are: (1) lifting a country or region out of the mud or out of the sand for better economic development, now especially important for under-developed areas; (2) providing bases and surfaces for secondary and farm-to-market roads, where good primary roads are already in existence; (3) providing bases for high-type pavements where high-type rock and crushed gravel normally employed for such bases are not economically available; (4) for city and suburban streets where the noise-absorbing and elastic properties of certain stabilized soil systems possess definite advantages over other construction materials; and (5) for military and other emergencies where an area must be made trafficable within a short period of time.

Soil stabilization involves: (1) diagnosis of the resistance properties of a given soil and required supplementation of these properties for the intended use in physical, physico-chemical, or chemical terms; (2) translation of the supplementary requirements into available materials and processes, and decision on use of specific method (or choice of method) on the basis of economy, practical feasibility, or special (military or other emergency) considerations; (3) construction, consisting normally of comminution, mixing with stabilizing material, and densification; and (4) economic considerations relating to the total cost composed of cost of materials, construction, and maintenance for the service life of the structure.

There exists a tendency on the part of laboratory workers to overlook the importance of Items (3) and (4). This may lead to a dangerous self-deception and may even impede the proper development of the science of soil stabilization. This science is not a pure but an applied one, and the actual processes of application must not only be considered but must be analyzed scientifically in order that the most rational and most effective, i.e., the most economical, method of construction be used in each particular case. In many cases, the use of a chemical construction aid may increase only slightly the cost of stabilizing materials but decrease greatly the cost of construction by facilitating the mixing and compaction process.

SOILS AND THEIR PROPERTIES

The term "soil" covers a large variety of materials existing under widely differing conditions. For a thorough understanding of soils and their properties it is well to look at them from different angles. A first step is to list definitions of the term soil as employed by different disciplines that deal intensively with the material covered by this term.

DEFINITIONS OF THE TERM "SOIL"

Highway Engineering

Soil consists of disintegrated rock and organic matter found on the surface of the earth, the particles of which may range in diameter from less than 0.0001 inch to a few inches and in which the fines are a product of natural, weathering forces. Soil may or may not contain organic matter. Engineering soils include bank gravel, bank sand, blow and dune sand, agricultural soils ranging from those of predominatly sandy texture to colloidal clays, and mixtures and combinations of these (1).

The different size fractions of the mineral portions of soils are named and defined in Table 1.

Textural classification of soil is based on gradation. The scheme of classification and naming is shown by Figure 1.

Geòlogy

Soil is the superficial unconsolidated mantle of disintegrated and decomposed rock material, which when acted upon by organic agencies, and mixed with varying amounts of organic matter, may furnish conditions necessary for the growth of plants. In its broadest sense the term "soil" has been used to include all the mantle of rock decay (2).

Pedology

Soil is the climatically conditioned petrologic and biogenic transformation product of the outermost layer of the solid earth crust (3); it is a natural body, differentiated into horizons varying in type and amounts of mineral and organic constituents, usually unconsolidated and of varying depths (4); soil is a unique creation that differs from the parent material below in morphology, physical properties and biologic characteristics; and the soil mantle of the earth may be termed "the pedosphere" alongside the atmosphere, the lithosphere, and the hydrosphere (5). The pedologic soil is a dynamic system subject to temperature, moisture, and biologic cycles and developing in a certain genetic direction under the influence of climate. The rate of this development is influenced by parent material, vegetation, and human activity (6).

With respect to scientific content and general usefulness, the pedologic soil concept is the most important. It has resulted in a natural system of soil classification and soil mapping which, though originally qualitative, can be easily supplemented with semi-quantitative and quantitative engineering information. General pedological soil maps exist for practically every part of the world. For many localities, soil maps are available on a scale as low as one inch to the mile.

TABLE 1

Size Fractions of Mineral Portions of Soils

Name	In Inches			In Millimeters			By Sieve Number
Gravel	3.0	to	0.08	76.2	to	2.0	3-in to No 10
Sand	0.08	to	0.002	2.0	to	0.05	No 10 to No 270
Silt	0.002	to	0.0002	0.05	to	0.005	
Clay			<0.0002			<0.005	
Colloids			<0.00004			<0.001	

Pedological soil types can be recognized easily from air photographs (7) and certain conclusions with respect to soil stabilization can be drawn solely from recognition of the pedological soil type of a certain area. The principal soil areas of the world are shown in Figure 2. The climatic and vegetational soil types of the United States are shown in Figure 3. The general and specific application of pedology to engineering is treated in detail by Wooltorton in the third paper of this symposium.

SOIL AS A POLYDISPERSE SYSTEM

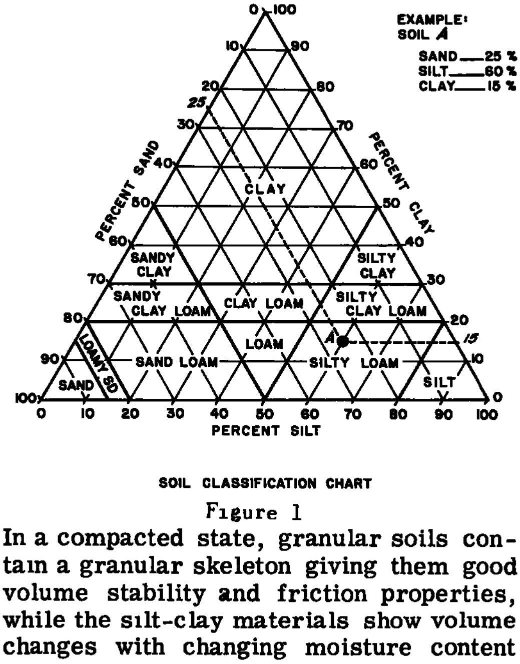
Soil is a polydisperse system composed of (1) solid inorganic and organic particles, (2) an aqueous phase carrying matter in solution (and sometimes in dispersion), and (3) a gaseous phase of varying composition. The gaseous phase is functionally related to biologic activity. The aqueous and the gaseous phases are usually considered together as pore space or porosity. The porosity varies in amount and in dimensional and form characteristics from soil to soil, from layer to layer, and in the surface layers from season to season.

Characteristics of the Solid Phase

Soils as polydisperse systems may contain particles ranging from atomic size (10^{-8} cm) to gravel and stone size. Since the soil information of greatest interest is found mostly in the international pedologic literature it is indicated to use the accepted international terms and definitions in presenting granulometric compositions.

Designation	Diameter in mm
Stones	>20
Gravel	20 - 2
Coarse sand	2 - 0.2
Fine sand	0.2 - 0.002
Silt	0.02 - 0.002
Clay	<0.002

Materials larger than 0.02 mm. are usually called granular; those smaller than 0.02 are called silt-clay materials. Soils containing more than 65 percent of coarse material are called granular soils; those containing more than 35 percent of silt and clay are called silt-clay materials.



SOIL CLASSIFICATION CHART

Figure 1

In a compacted state, granular soils contain a granular skeleton giving them good volume stability and friction properties, while the silt-clay materials show volume changes with changing moisture content and possess low angles of internal friction.

Soils are given textural names in accordance with the identification chart in Table 2; this was taken from Reference 8, and also represents international pedologic usage:

TABLE 2

Mechanical Composition of the Principal Soil Classes

Soil Class	Limits in the proportions of the Soil Separates sand	silt	clay
	%	%	%
Sand	80 - 100	0 - 20	0 - 20
Sandy loam	50 - 80	0 - 50	0 - 20
Loam	30 - 50	30 - 50	0 - 20
Silty loam	0 - 50	50 - 100	0 - 20
Sandy clay loam	50 - 80	0 - 30	20 - 30
Clay loam	20 - 50	20 - 50	20 - 30
Silty clay loam	0 - 30	50 - 80	20 - 30
Sandy clay	55 - 70	0 - 15	30 - 45
Silty clay	0 - 15	55 - 70	30 - 45
Clay	0 - 55	0 - 55	30 -100

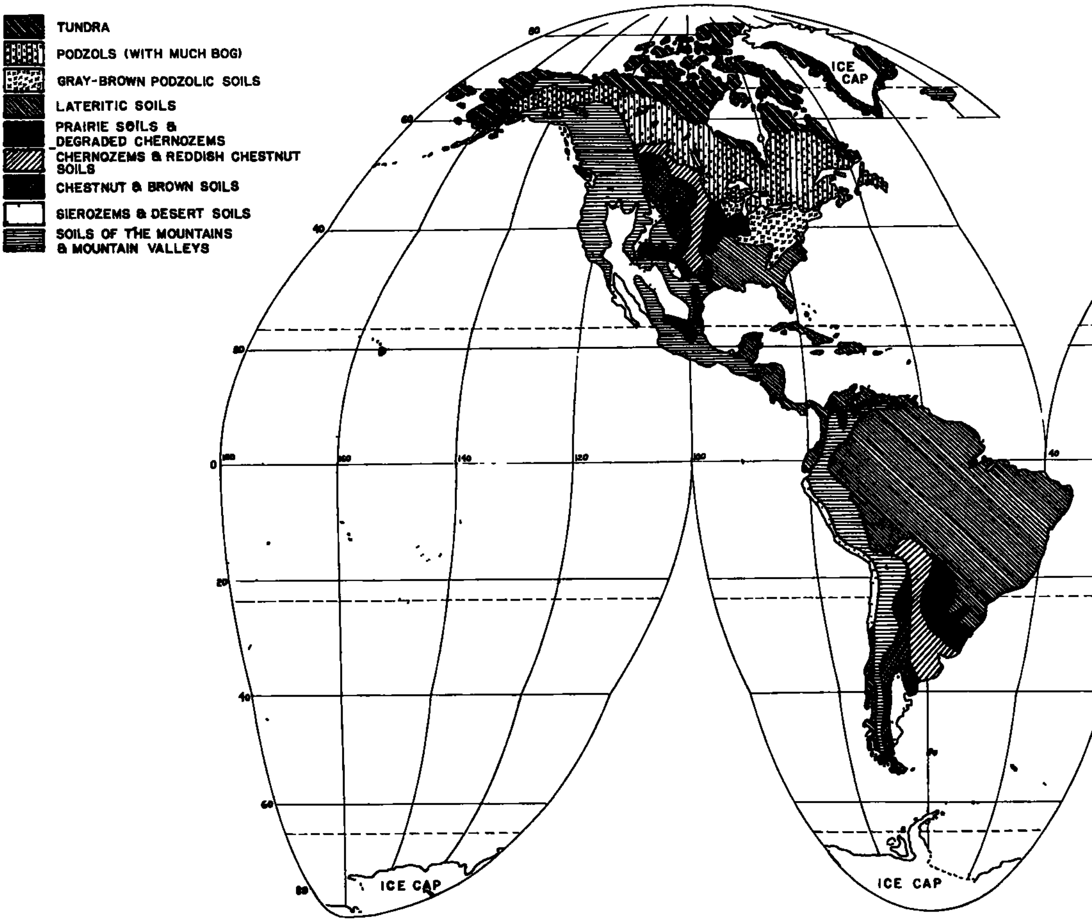
Combining the silt and clay fractions as silt-clay materials, we may establish three major physical groups of mineral soils.

Designation	Percentage silt-clay
Granular soils	0 - 20
Cohesive-granular soils	20 - 35
Cohesive-nongranular soils	35 - 100

During the early days of agricultural chemistry it was quite common to make gross chemical analyses of soils and soil materials. However, the analytical data did not indicate the availability as plant food of the determined elements, nor were they of diagnostic significance with respect to other soil properties of practical agricultural value. This type of analysis was succeeded by determination of plant-food values, on one hand, and of the granulometry and mineralogy of the solid soil constituents on the other hand. The recent introduction of nuclear meters for determinations in situ of soil moisture and soil density, and the possibility of further development for other purposes of this type of methodology has given real scientific and practical value to the "obsolete" data on elementary soil composition. Most of the best data on elementary soil composition are found in the older rather than in the newer soil literature.

Considering the great variability of the chemical composition of the parent materials from which soils are formed, an equally great variability in soil composition may be expected. Fortunately, the active soil-genetic factors reduce this variability by something akin to an analytical separation on a gigantic scale. This separation brings about: (1) breakdown of complicated pyrochemical minerals into simpler hydrochemical compounds; (2) complete or partial removal of soluble reaction products or their concentration in specific layers of the soil profile; and (3) dislocation of colloidal and clay-sized particles and their concentration in specific layers of the soil profile.

The most general chemical soil classification was made by Marbut (9) by dividing soils into two general classes: (1) Pedalfers, soils of humid climates (pre-


TUNDRA
PODZOLS (WITH MUCH BOG)
GRAY-BROWN PODZOLIC SOILS
LATERITIC SOILS
PRAIRIE SOILS & DEGRADED CHERNOZEMS
CHERNOZEMS & REDDISH CHESTNUT SOILS
CHESTNUT & BROWN SOILS
SIEROZEMS & DESERT SOILS
SOILS OF THE MOUNTAINS & MOUNTAIN VALLEYS
ICE CAP
ICE CAP
ICE CAP
80
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20
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20
40
60
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180
160
140
120
100
40

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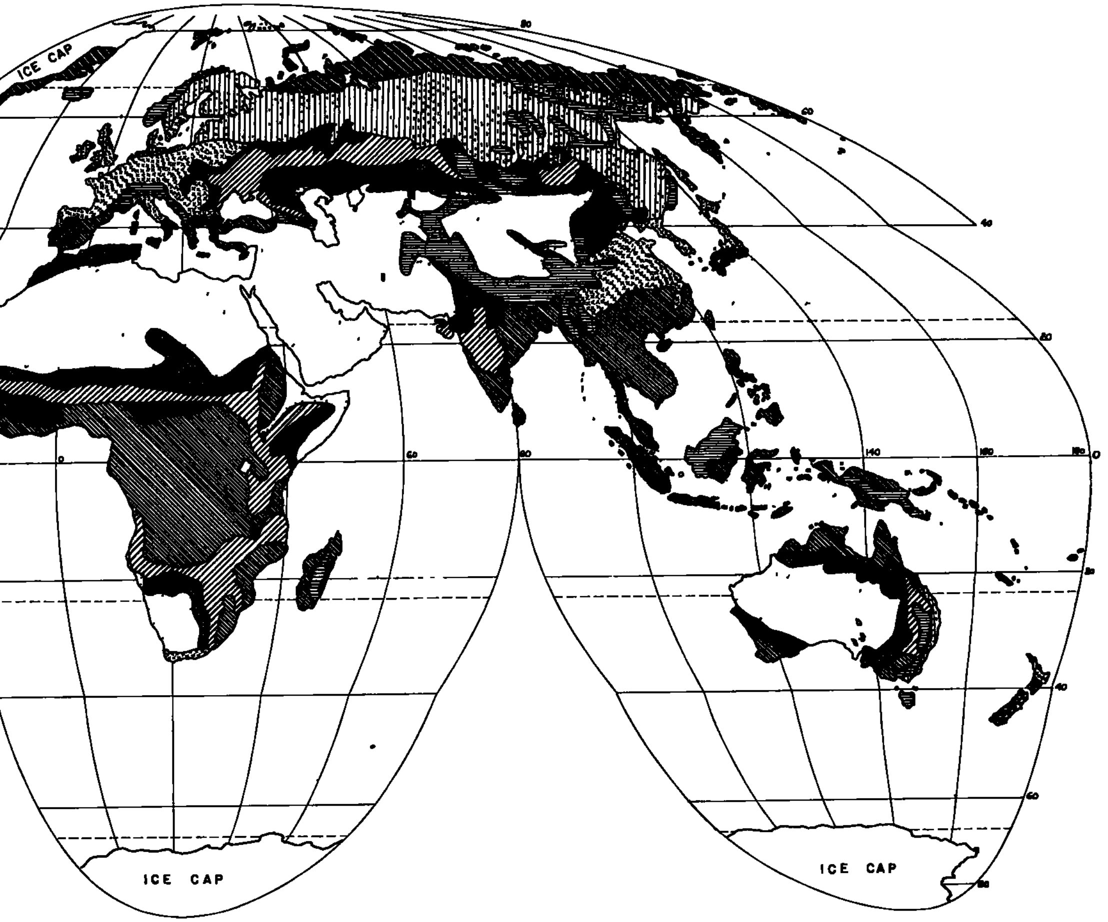
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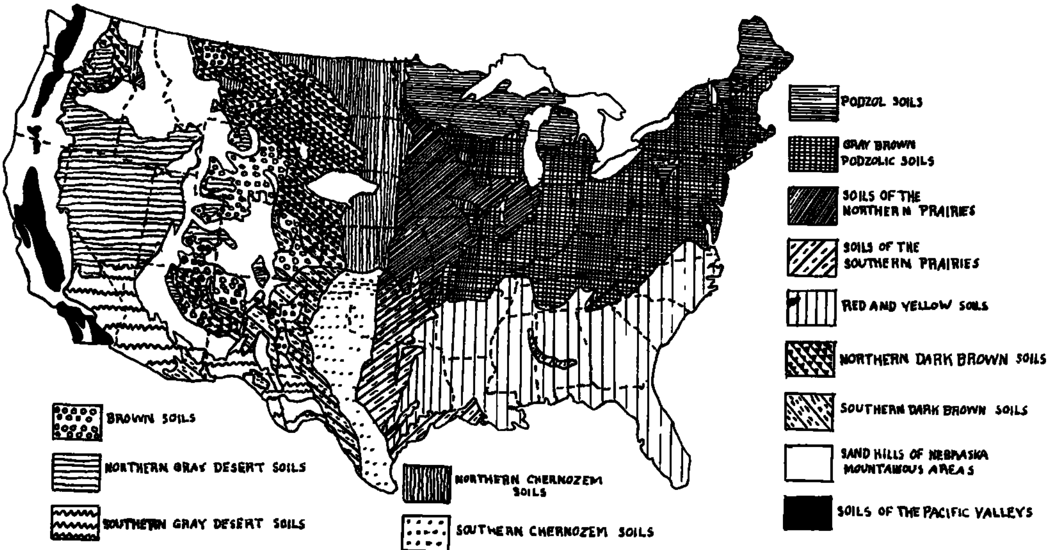
cipitation $>$ evaporation, therefore, water percolation and elutriation) possessing a relative concentration of iron and aluminum compounds in one of the profile horizons and (2) Pedocals, soils of semiarid and arid climates (precipitation $\leqq$ evaporation) possessing a $CaCO_3$ concentration zone within or on the surface of the soil profile.

The general shift from the chemical composition of parent rock to that of the A and B horizons of soils and the difference in chemical composition of the A and B horizons is shown in Table 3. This table presents analytical data reported by Clarke (10) as oxides but herein transposed into elementary composition.

for peat and bog soils. It is usually below 20 percent, but a heavy clay soil may contain over 10 percent of water and still may be dry and dusty to the eye and to the touch.

Soil organic matter is a mixture of many different compounds, the more important of which are carbohydrates, protiens, fats and resins, and waxes. The absolute amounts and relative proportions of these compounds vary with the climate and with macrobiologic and microbiologic activity in the soil. The organic matter is concentrated in the surface layers and decreases with increasing depth in the soil. It ranges from less than 1 percent in inorganic soils and sands to almost 100 percent of the solid matter in peat bogs.



Climatic and vegetational soil groups of the United States (After Marbut 1935)

Figure 3

The elementary composition given in Table 3 refers only to the solid soil mineral matter and does not contain the contribution made by the variable soil moisture and the soil organic matter. Soil moisture provides additional O and H atoms, soil organic matter mainly C, N, O, H, S, and P atoms. Soil moisture is usually expressed as a weight percentage of the solid soil matter dried at 110 C. in accordance with the formula:

Percentage moisture

$$= \frac{\text{wet weight-dry weight}}{\text{dry weight}} \times 100$$

Soil moisture may vary from less than 1 percent for dry sand to over 100 percent

In the semiarid Chernozem soils, the organic matter may run up to 10 percent of the weight of the dry soil. The average carbon content of humus is 56 percent. The average carbon-nitrogen ratio is 10 to 1. A value of 10 percent of organic matter, therefore, corresponds to 5.6 percent of carbon and 0.56 percent of nitrogen. Lateritic soils of the wet tropics, despite their often brilliant red and yellow inorganic appearance, have been found to contain up to 2 percent and more of organic matter.

The total amount of carbon in a soil is the sum of that contributed by the organic matter and that present in the profile as inorganic carbonates, especially those of

C_a and M_g. Horizons enriched in C_a and M_g are typical of Pedocal soils.

The words sand, silt, and clay have been introduced and defined earlier as designating certain size fractions of the ultimate particulate components of soils. It is of interest to see what conclusions with respect to soil chemical composition may be drawn from recognition or determination of the granulometry of a soil.

TABLE 3

Average Elementary Composition of the Earth Crust and of the A and B Horizons of Soils

Element	Earth crust (percent)	Soil	
		A Horizon (percent)	B Horizon (percent)
Al	7 13	5 30	6 24
Fe	4 38	2 35	4 05
Ca	3 52	74	71
Mg	2 35	36	51
K	2 54	1 43	1 71
Na	2 52	61	61
Mn	08	09	09
Ti	47	54	54
P	13	06	08
S	13	05	05
Si	28 60	35 70	35 70
O a)	48 18	52 78	49 73

a) Oxygen percentage does not include contribution from free water content

Sand (2.0 - 0.02 mm.) is predominatly quartzic and silicic in humid climates but may be any kind of mineral in dry climates. The white sands of New Mexico are gypsum sands; coral and shell beach sands may consist almost completely of calcium carbonate; the black sand of Yellowstone Park and some of the blue and purple beaches of the Pacific Islands are obsidianite sands as are some of the Alaskan and Aleutian beach sands (11).

TABLE 4

Elementary Composition of Silt Components

Minerals	Formula	Percentages Elementary Composition						
		O	Si	Al	K	Na	H	F
Feldspars								
Orthoclase	$KAlSi_3O_8$	41 5	33 0	10 4	15 0	0	0	0
Albite	$NaAlSi_3O_8$	44 3	35 0	11 1	0	9 5	0	0
Muscovite	$(OH, F)_2KAl_2(Si_3AlO_{10})$	44 0	21 0	20 2	9 8	0	0 24	4 7
Quartz	SiO_2	53 2	46 8	0	0	0	0	0

Silt (0.02 to 0.002 mm.) particles resemble quite closely the composition of the parent rock with feldspar, muscovite, and quartz usually well represented. Representative formulas and compositional data are given in Table 4. We may conclude that in the silt fraction the silicon content ranges between 21 percent and 47 percent and the oxygen content between 42 percent and 53 percent.

The older soil literature contains many excellent data on chemical composition of the clay fraction (0.002 mm.) of different soils and soil categories. Many physical properties of clay systems could be related to ratio $Si0_2/Al_20_3 + Fe_20_3$ found in chemical analysis of the clay, and it became a widespread practice to characterize natural clays by this ratio. In laterite soils this ratio falls below two; in other soils it is above two; it is two in the clay mineral kaolinite. Typical ratios (12) are given in Table 5.

During the last 25 years it has become possible to extract and recognize more or less pure crystallized minerals from the clay fraction. This development is treated in an excellent manner by Hauser in the fourth of this symposium. However, in view of the expected further development of nuclear meters that probe into atomic and nuclear compositions, it was considered worthwhile to present in Table 6 the elementary chemical composition of the principal clay minerals, as calculated from the oxide data given in (13).

The data on chemical composition of soil constituents are summarized in Table 7.

Characteristics of the liquid phase

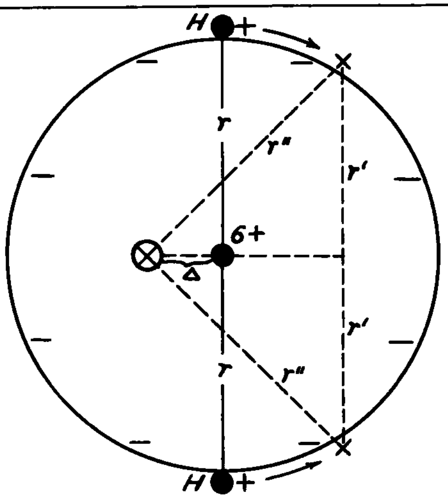
The chemical formula for water is H_2O, which represents a composition of 88.8 percent (by weight) of Oxygen and 11.2 percent Hydrogen. The H_2O molecules, however, because of their electric structure (Figure 4) are associated as well as dissociated as a function of temperature. Quantitative values are given below:

Temperature (°C)	Concentration of H/and OH^- ions in pure water
0	2.8×10^{-8}
8	7.8×10^{-8}
25	1.0×10^{-7}
34	1.45×10^{-7}
50	2.3×10^{-7}

Association of Water Molecules as a Function of Temperature.
From Raman spectra (14)

Temperature (°C)	H_2O	$(H_2O)_2$	$(H_2O)_3$
Ice	0	41	59
Water at 0	19	58	23
Water at 38	29	50	21
Water at 98	36	51	13

Although the molecular size of a compound is indeterminate in the solid state



TRIANGULAR SHAPE OF THE H_2O-MOLECULE AS A RESULT OF THE POLARIZABILITY OF THE OXYGEN ION [1]

1. Potential Energy when H-ions and oxygen nucleus fall on a straight line. Repulsion of positive parts:

$$\frac{+2 \times 6e_0^2}{r} + \frac{e_0^2}{2r}$$

e_0 = charge of one electron

Attraction between electron Shell and H-ions:

$$: \frac{-2 \times 8e_0^2}{r}$$

$$= 4.77\ 10^{-10} \text{ electrostatic units.}$$

2. Potential energy when H-ions are moved as indicated by arrows.

$$\text{Repulsion: } \frac{2 \times 6e^2}{r''} + \frac{e^2}{r'}$$

Attraction: Same as above.

At small dislocations r" increases at greater rate than r' decreases. Equilibrium = lowest potential energy if H+ positions are at an angle between 100° and 110°.

[1] After Born, Heisenberg and Hund, Zsch. f. Physik 23, 388; 26, 196 (1924) 31, 81; 32, 1 (1925); Eucken, Arnold "Lerhbuch der Chemischen Physik," Leipzig, 929, (1930).

Figure 4

Tammann (15) and Bridgman have shown the existence of five different types of solid water that are functions of pressure and temperature conditions. Four of these types have corresponding liquid phases (see Figure 5).

The temperatures and pressures in the triple points of the one-component H_2O-system are given below:

Phases in Equilibrium	Temperature °C	Pressure (kg/cm²)
Water-Ice I-Ice III	-22	2050
Ice II-Ice-I-Ice III	-34.7	2170
Water-Ice V-Ice III	-17.0	3530
Water-Ice V-Ice VI	- 0.16	6380

The melting point of water decreases with increasing pressure to a minimum at -22 C. for 2,050 kg. per sq. cm; subsequently it increases, passing the 0 C. mark around 6,300 kg. per sq. cm. and passing through 45 C. at about 12,000 kg. per sq. cm.

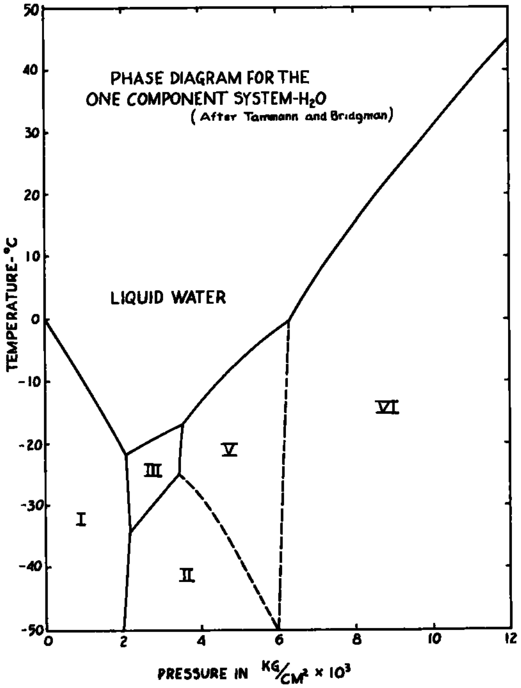


Figure 5

These data have been presented because: (1) The peculiar behavior of water is due to the polarity of its molecule; that is, its electric and geometric properties. (2) The large adsorption forces exerted on water molecules by the surfaces of

TABLE 5

Relationship Between Silica-Sesquioxide Ratio and Base Exchange Capacity

Clay	Base Exchange capacity ME/100 gm	Source	Ratio Sio_2/R_2O_3
Cecil	13	An Alabama clay loam	1.3
Susquehanna	47	Well-oxidized Alabama soil	2.3
Putnam	65	Heavy Missouri silt-loam	3.2
Wabash	78	Missouri alluvial clay	3.2
Lufkin	82	Black Belt soil from Alabama	3.8
Montmorillonite	95	From Wyoming Bentonite	5.0

Note: The base exchange capacity represents the milliequivalents of cations adsorbed on the surface of 100 grams of colloid and exchangeable for other cations that may be introduced into the system.

solid soil particles act similar to externally applied pressures; that is, they may liquify solid water or solidify liquid water. (3) Adsorption forces of the order of magnitude that solidify water at 45 C. (12,000 kg. per sq. cm.) are not at all uncommon on the surfaces of soil particles. (4) All physical properties (mechanical, electrical, optical, acoustical) of cohesive soils are functionally connected with the behavior of the water substance in strong pressure, electrical and magnetic fields such as exist on the surfaces of the particulate matter in soils.

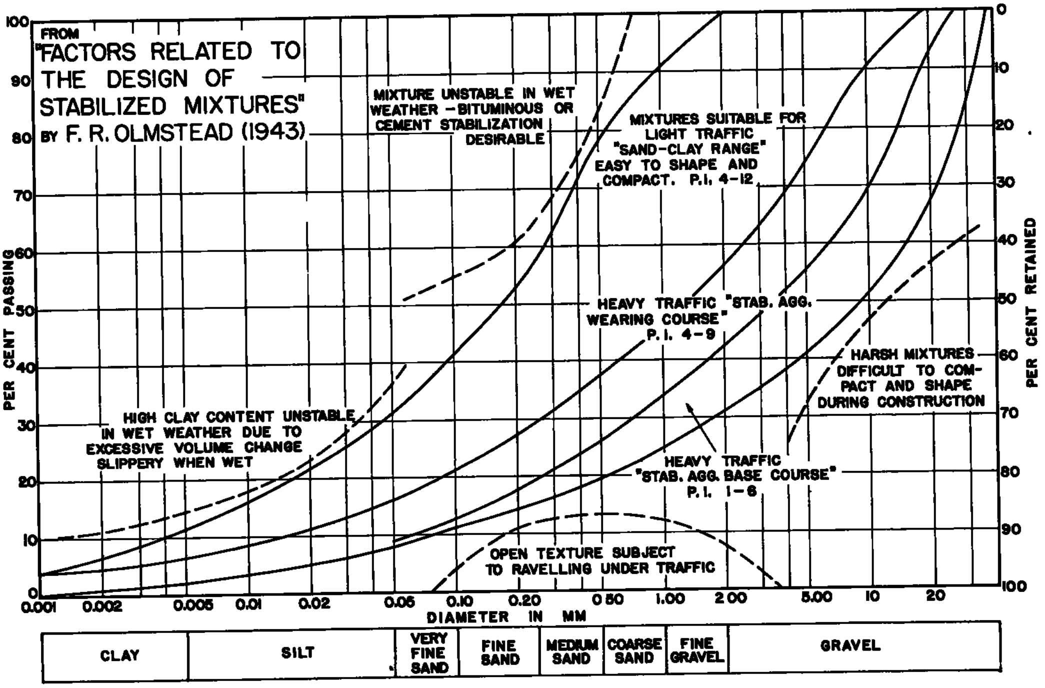


Figure 6

Water in soil is never pure but holds materials in solution and dispersion. Dissolved materials are mainly salts and acids. In saline soils the solutions may actually be saturated with different salts. In humid climates the solute concentration is relatively low. The water layers next to the solid particles are under high adsorption pressures, which may be larger than 25,000 kg. per sq. cm. The water may be in a solid condition at temperatures above 50 C. The adsorption forces decrease exponentially to about 50 kg. per sq. cm. at the so-called hygroscopic moisture content, and hence more slowly to zero for the water content at which the soil-water system behaves essentially as a liquid (liquid limit). The course from hygroscopic moisture content to the liquid limit is illustrated by the following data (13).

Condition	Adsorption pressure in kg. per sq. cm.
Hygroscopicity	50
Permanent wilting point	12.5
Wilting point (dead water)	6.25
Vacuum moisture equivalent	0.55

The hygroscopicity of a soil increases with increasing clay content and with increasing SiO_2/R_2O_3 ratio of the clay. At the interphase between the surface of the solid soil components and the water film an electric potential is established, which gives rise to electrokinetic phenomena in soils. The magnitude of this potential and the thickness of the interphase or electric double layer is a function of the surface-chemical composition of the solid, of the amount of water present, and the type and amount of ions carried in solution (16).

TABLE 6

Percentage of Elements in Common Clay Minerals

Mineral Element	Kaolinite	Halloysite	Montmorillonite	Beidellite	Nontronite	Mica-like Minerals
Si	20.6-25.3	18.9-21.7	22.6-24.2	21.4-22.3	14.7-22.5	23.7-24.5
Al	15.9-21.3	17.9-20.8	10.6-14.4	6.4-14.7	0.21-12	11.5-17.4
Fe	0.22-1.46	0-0.29	0.15-1.02	0.58-13.5	11.0-29 8	0-4.6
Mg	0-0.6	0-18	1.26-3.96	0.12-1.8	0.06-2.4	1.2-2.7
Ca	0.021-1.1	0.06-0.57	0.6-2.6	0.35-2.0	0.43-3.2	0-0.42
K	0-1.25	0.25	0.17-0.5	0.08	0.08-0.32	5.1-5.7
Na	0-0.89	0.074-1.48	0.22-0.59	0.07-0.74	0-0.15	0.07-0.37
Ti	0-0.86	-	-	0.48	0-0.06	0.3
H[a]	1.2-1.6	1.5-2.6	1.9-2.63	1.9-2.5	0.56-1.44	0.71-0.78
O[a] av.	53.79	56.62	57.18	56.06	50.45	50.22
H_2O contents included in H and O above	11.0-14.3	13.4-23.7	17.1-23.7	17.3-22.6	5.1-13.0	6.4-7.0

Note [a] This includes the contribution from hydratation and other strongly absorbed water which does not make the clay wet.

The water content at which a soil passes from an essentially solid to a plastic condition is called the plastic limit (17); that at which the soil passes into a liquid condition is called the liquid limit. These limits, like the hygroscopic water content, increase with increasing clay content and with increasing SiO_2/R_2O_3 ratio of the clay and are, in addition, functions of the type and amount of exchange ions on the clay and of the ions in the aqueous phase.

The mechanical resistance of cohesive soils increases with decreasing water content. Physical properties that increase with increasing water content are: permeability to water, dielectric constant, heat conductivity, and sound velocity. Properties that rise to a maximum and subsequently fall with increasing water content are: electric conductivity, electro-osmotic water transmission (16), thermo-osmotic water transmission (18, 19), and ease of compaction or density achieved with a certain compactive effort.

Properties of soil-air

The portion of the soil porosity not filled with water represents the soil-air. Soil-air is in constant exchange with the atmosphere and its composition reflects that of the atmosphere except for the concentration of those components that are used up or produced by microbiological activity in the soil. Such substances are mainly oxygen, which is used up, and carbon dioxide, which is produced. Atmospheric air contains 20.939 ± 0.004 volume

TABLE 7

Average Elementary Composition of the Different Size Fractions of Soils

Dominant Elements	Percentage of dominant elements in Clay	in Silt	in Sand[a] (average comp. humid climate)
O	50 - 57	41 - 53	41 - 53
Si	15 - 25	21 - 47	21 - 47
Al	02 - 21	0 - 20	0 - 20
Fe	0 - 30	0 - 5	0 - 5

Note [a] For special sands (desert, coral, shell, and other beach sands) the chemical composition must be individually ascertained.

percent of oxygen and 0.031 ± 0.0016 volume percent of CO_2. According to Ebermeyer (20) the proportion of CO_2 in the soil atmosphere is a function of all the physical and chemical factors that condition soil activity and productivity. Typical data found by him for the CO_2 content at 70 cm. depth in volume percent of the soil atmosphere are given below:

Humus-free, fine granular quartz sand	0.340
Humus-free, coarse granular quartz sand	0.340
Humus-free, fine granular lime sand	0.548
Humus-free, heavy, clayey loam	0.726
Humus-free, loess loam	0.745
Humus-and lime-containing loam	1.314
Humus-containing sandy loam garden soil	1.638
Humus-rich, fertilized, loamy sand soil	1.876
Moor soil	6.649
Garden soil, very rich in humus and lime	6.880
Fertilized with stable manure	7.020

The oxygen content of soil-air decreases as a function of the increase of the CO_2 content, since the CO_2 is derived from the Oxygen. More detailed data on soil-air can be found in the pertinent chapters of References 13 and 13a.

It is often desirable to know the actual volume of solids in a given volume of natural soil. Representative values for different soils are given below:

Soil Type	Pore space %	Volume of solid particles %
Moor soil	84.0	16.0
Sand soil	39.4	60.6
Loess loam	45.1	54.9
Clay soil	52.7	47.3

Natural soils always possess air spaces even if allowed to take in all the water they can. Of course, after long-time flooding this air space may be rather small. The air capacity necessary for the normal growth of different plants is quoted in (13) (p. 281).

TABLE 8

COLLAMERITICS

The Science of Composition and Properties of Non-Metallic Construction Materials

based on

Properties of the aggregates	Properties of Cementing Agents	Combination of Aggregates and Cementing Agents.
A Physical	A Inorganic	A Design of
I Granulometry Laws of Arrangement and Packing as a function of 1) Size and gradation 2) Shape factors a) spherical b) cubic c) plate-like d) needles and fibers	I Simple: Gypsum and lime Plasters. II Complex: Sorel and hydraulic cements. B Organic I Bituminous: asphalts, pitches, tars.	I Mortars with inorganic or organic cements. II Concretes: Portland Cement-, bituminous-resinous-, clay-, etc. III Plastics: Powder-, paper-and fiber-filled.
II Mechanical 1) Strength and toughness 2) Abrasion resistance	II Natural and synthetic Resins, elastomers and related substances.	
B Physico-Chemical and Chemical	III Gums and Glues	
I Reactivity and Bonding with cementing materials		
II Reactivity with deleterious substances		

Plant	Air Capacity %
Sweet grasses	6 - 10
Wheat	10 - 15
Oats	10 - 15
Barley	15 - 20
Sugar beets	15 - 20

THE SOIL IN SITU

Soils are creations of climatic forces and may be considered attuned to them. These forces derive from daily and seasonal temperature variations, from fluctuations in moisture content, from the annual swell and sink of the biologic potential, and from any other periodic phenomenon that affects the surface layer of the earth. As a result of these factors, soils in situ are not mixtures of their components but are natural organized systems, as was shown in the section on soil genesis.

These systems continue to be exposed to the forces that formed them and their properties are in a continuous state of flux. As a result of this situation, soils in situ share many essential properties with "living" systems and may almost be considered as living even if we disregard the large microflora and microfauna dispersed in soils that render soils actual living systems. Important consequences of this fact are discussed by Wooltorton in his excellent and comprehensive contribution to this symposium (paper No. 3 on Engineering Pedology and Soil Stabilization).

Soil Aggregations

The primary particles of a soil are very rarely encountered as independent individual constituents but are cemented together into secondary aggregations and crumbs by means of inorganic or organic binders (21). The stronger secondary particles even persist in disturbed and molded specimens, which results in relatively large angles of friction of clay soils in the molded wet or in coherent air-dry condition. The aggregation increases the size of the soil pores which results in a greater permeability for water and air (in accordance with Poiseuille's law) than the soil would have in single-grain structure even at the same porosity value.

Soil Dynamics

The periodic daily and seasonal warming and cooling of the earth surface result in a temperature wave penetrating into the soil. The physical picture is as follows: (1) the normal daily and yearly temperature fluctuations on the surface of the earth can be expressed as sinusoidal waves; (2) because of the heat conductivity of the earth, these surface waves are transmitted into the interior; however, the amplitudes (differences in temperature) are dampened because of the heat capacity of the earth substance; and (3) because of the time necessary for heat transmission, the maximum of the dampened temperature waves become increasingly retarded with increasing depth of penetration until at the depth of one wave length, the retardation equals the time period. For the same reason, the time of highest temperature at the soil surface is the time of lowest temperature at a depth of one-half wave length.

These relationships can be expressed mathematically as follows: The speed of propagation of the temperature wave is:

where

$$V = \frac{\lambda}{T}$$

Since $\lambda = 2\sqrt{\frac{\pi\,kT}{\rho\,c}}$

$$V = 2\sqrt{\frac{\pi\,k}{c\,\rho\,T}}$$

The speed of progression of the temperature wave is therefore inversely proportional to the square root of the period, while the wave length is directly proportional to the square root of the period. In other words, a long period has a long wave with a small speed of propagation, while a short period has a short wave with a great speed of propagation.

Since the square root of 365 is about 19, the length of the daily temperature wave is only $\frac{1}{19}$ of that of the yearly wave.

The maximum amplitude θ_x at a distance x from the surface is the following function of the maximum amplitude θ on that surface.

$$\theta_x = \theta e^{\frac{-2\pi x}{\lambda}}$$

If $x = \lambda$, then $\theta_{\lambda} = \theta e^{-2\pi} = \theta \frac{1}{535}$

If $x = \frac{\lambda}{2}$, then $\theta_{\lambda/2} = \theta e^{-\pi} = \theta \frac{1}{23}$

The damping effect is usually expressed by the ratio of the amplitudes at a distance of one-half wavelength. The damping ratio is therefore 23, which means that an amplitude of 23 deg. on the surface is reduced to 1 deg. at a distance of one-half wavelength.

Employing average values in the cgs. system for physical soil constants, namely:

$k = 0.004$
$\rho = 1.6$
$c = 0.4$

we obtain for the length of the yearly wave:

= 1,560 cm.

and for the length of the daily wave:

= 82.4 cm.

L. A. J. Quetelet (1796-1874) who made the first reliable soil-temperature determinations gave 17 m. as the length of the yearly wave and 1 m. as the length of the daily wave. He also noted that, because of heat convection, the actual amplitudes at the lower depths were somewhat greater than calculated. The magnitude of the amplitudes at the lower depths is illustrated by the following data by Quetelet:

Depth m.	Observed m.	Calculated m.
0.19	13.28	13.28
0.37	11.35	10.28
1.95	7.6	.9
3.9	4.5	-
7.8	1.4	.6

For actual soil conditions, heat produced by microbial decomposition of soil organic matter must be taken into account in addition to heat convection as modifying elements of the straight heat-conduction concept. The predominance of heat conduction in the soil-temperature picture is sufficiently obvious, however, to make its basic theory an important tool in the hands of the highway and airport engineer.

Moisture moves in and out of the soil profile and up or down in the profile as a result of meteorological and soil physical factors. As has been seen previously, the dominant direction and pattern of moisture movement determines the climatic soil type encountered.

The following potentials (22) are available for water movement in soils: (1) gravity potential acting on the water mass; (2) potential due to hydration energy of the ions, related to the heat of wetting, which can be determined directly or calculated from vapor absorption experiments; (3) potential due to the osmotic energy of ions held in a sort of Donnan equilibrium at the internal surface of the system; (4) the so-called capillary potential due to the surface tension of water; (5) the thermo-osmotic potential due mainly to the change in water affinity of the internal soil surface with change in temperature (19); and (6) vapor-pressure differences causing diffusion in the vapor phase (This phenomenon is of little practical importance in temperate climates but appears to be important in arid and monsoon climates).

Although the order of magnitude of these potentials can be approximately calculated from theoretical considerations and with the help of experimental data, little is known concerning the resistance to flow under these potentials. Except perhaps for the pure capillary flow under a pure capillary potential, flow under the named potentials is primarily of a film and not of a capillary character. Since practically nothing is known concerning the structure and shear resistance of such films, much theoretical and experimental work needs to be done in this field (23).

MECHANICAL RESISTANCE OF SOIL SYSTEMS

Assemblies of atoms, ions, and simple molecules may be in the solid, liquid, or gaseous state depending upon the relative magnitude of the mutual attractive forces and the kinetic dispersive forces. The latter increase with increasing temperature. At low temperatures the attractive forces prevail and the material is in the solid state; at intermediate temperatures the attractive and dispersive forces are more or less balanced resulting in the liquid state, and at high temperatures the dispersive forces prevail bringing the system to the gaseous state.

Typical for pure compounds is their definite melting point; the only difference between the solid and the liquid at the melting point is the lesser density or the larger volume per unit mass of the liquid. This means that in the melting process, the heat of fusion introduced into the system has performed the work of pushing apart the constituent atoms or molecules; in other words, holes have been introduced into the system.

The much lesser resistance to mechanical forces of the liquid as compared with that of the solid at the melting point must then be due essentially to the presence of these holes. This concept is strengthened by the fact that for most atoms and simple compounds the volumes of a certain mass of material under atmospheric pressure at the densest state (V_o), the melting point (V_m), and the boiling point (V_b) are related as follows:

$$V_o : V_m : V_b = 1 : 1.21 : 1.42$$

Also, within the liquid state, the internal friction (η) of non-associated liquids has been found by Batschinski (24) to obey the following equation:

$$\eta = \frac{c}{V - V_s}, \text{ in which}$$

c = constant

V = volume of the liquid

V_s = volume of the unexpanded solid.

There exists an intrinsic similarity between assemblies of particles of atomic and molecular size, on one hand, and of macroparticles of gravel and sand sizes on the other hand (25), which, of course, also holds for intermediate sizes. The main lesson to be learned from this similarity is the great importance of volume relationships with respect to mechanical resistance properties, of granular assemblies.

Previously, soils have been subdivided into the following three groups:

Designation	Percentage Silt + Clay
1. granular	0 - 20
2. cohesive granular	20 - 35
3. cohesive	35 - 100

Granular Materials

The mechanical resistance of soil systems is normally expressed as shear resistance S. For granular soils the shear resistance is usually expressed by the equation:

$$S = \sigma_n \tan \phi, \text{ in which}$$

σ_n = normal stress on the shear plane

$\tan \phi$ = coefficient of friction

The coefficient of friction is not a constant for a certain material but is a function of the voids ratio and also of σ_n. As a matter of fact, the shear phenomenon itself is a function of the voids ratio. At low voids ratios the shear forces must: (1) provide spaces into which the particles can move, i.e., the system must be expanded, and (2) move particles into these spaces.

At high voids ratios, there are sufficient spaces into which the particles may move; as a matter of fact, some of the excess spaces may even collapse, leading to a lower voids ratio and a greater resistance of the system.

At an intermediate (critical) voids ratio, the number of voids present is sufficient to accomodate the moved granules, the system suffering neither expansion nor densification.

In analogy with atomic and molecular solids we can consider a granular system as being essentially in the solid state as long as its voids ratio is below the critical and provided that it is confined within a certain volume. At the critical voids ratio the system passes, as through a melting point, from an essentially solid to a potentially liquid state.

In the solid as well as in the liquid state, the mechanical resistance is a function of the voids ratio as well as of form factors. For a particular material, the coefficient of friction can be expected to obey a relationship of the following type:

$$\tan \phi = \frac{k}{V - V_s}, \text{ in which}$$

k = constant

V = bulk volume of the particle assembly

V_s = effective volume of the solid particles.

The most-striking example of the influence of density or packing on resistance properties is the difference between the hardness of diamond and graphite.

The direct analogy between atomic and molecular assemblies, on one hand, and macromeritic assemblies on the other, holds not only for uniformly sized particulate components such as chemical elements and compounds and corresponding uniformly sized glass beads, beach sands, or similar materials, but also for mixtures of materials of different sizes, and thus for the graded materials so important in road construction.

The principles of granulometry and the extent to which they can be economically applied to construction materials have been studied most thoroughly for portland-cement concrete, (26,27,28). From these studies much can be learned for analogous soil systems which so far have been treated more or less on an empirical basis (29). A most important new application of granulometric principles is described in the paper by Pimentel dos Santos, which if the fifth paper of this symposium.

For particles of nonuniform and nonspherical size the densest packing is determined experimentally. In these experiments samples of the material are densified in a container of known volume either by rodding or by vibration. From the weight of the material required to fill the container and from its specific gravity, the absolute volume of the solid materials is calculated. Its subtraction from the total volume of the container gives the volume of voids.

In mixtures of materials of two size ranges such as gravel and sand the smaller size components will interfere with the close packing of those of larger size. Consequently, the absolute volume of the coarse aggregate contained in a unit volume of a mixture of coarse and fine aggregate is always smaller than the absolute volume of the coarse aggregate that can be packed by itself into a certain space. If the latter is called b_0 and if b is the volume of the coarse aggregate in the densified mixture of coarse and fine aggregate then the fraction $\frac{b}{b_0}$ is always smaller than one. Its actual magnitude is a function of the size range of the coarse aggregate and of the fineness modulus of the sand (30). This means that the coarse aggregate is bedded in the finer material. This bedding makes the mixture more workable and allows a modification of the properties of the entire system by modifying the bedding material. The latter may be done by introducing air bubbles or particles possessing special elastromeric, plastic, or other desired properties.

Since the resistance properties of granular materials are dependent solely on friction and since the frictional resistance is directly proportional to the normal pressure on the shearing plane, the resistance properties are very low in the surface layers of granular masses where the normal pressure on the shear plane is small. Therefore, such systems require cementation for their stabilization. The particles can be cemented together by any one of the large number of inorganic and organic cements or by combinations of inorganic cements with organic cements or with waterproofing agents. If the granular system contains both coarse and fine aggregates the cemented system is called a concrete. If the largest granules are of sand size, then the cemented system will be akin to a mortar. Such systems have been called collameritic systems, (colla = glue; and meros = particle) and include portland cement concrete, bituminous concrete, clay concrete (granular stabilized soil), water-proofed clay concrete and sand-clay as well as water-proofed sand clay (see Table 8). Since the cementing substance is the most active and often also the most expensive component, it is advisable to use as small a proportion of it as possible. This means, that the cementing substance should be used primarily for cementing and not for filling purposes. More economical fillers are usually available.

The type of cementing substance which can be used for the purpose of glueing coarse aggregate and sand particles together depends primarily on the prevailing climate. While under American conditions, mainly portland cement, flyash-lime combinations, and bituminous materials have been employed, in drier climates, gypsum plasters (31) sorel- , and other water susceptible cements can be used successfully. With proper waterproofing the latter cements are suitable for soil stabilization even in moist climates (32).

So far we have dealt with systems composed mainly of granular soils having less than 20 percent of silt-clay material. In the range of granular cohesive materials which contain up to 35 percent of silt and clay, we may have, at the higher silt and clay contents, well-graded material that corresponds to the established requirements of granular stabilization. With lesser contents in silts and clay, there is a lack of natural binder which must be made good by addition of binder soil or of cementing and waterproofing substances.

It is of considerable interest that binders composed of both silt-clay materials and of organic or inorganic cements have definite, and not only economical, advantages over using the pure cementing material. The work of Silbergh (33) had indicated the existance of an optimum clay content (depending on the activity of the clay) in the stabilization of sandy soil by various inorganic and organic stabilizers.

The paper by Reinhold in this bulletin shows for stabilization of a sandy soil with portland cement an optimum clay content of about 25 percent, an excellent construction material being obtained with 25 percent of clay and 10 percent of portland cement.

Silt-Clay Materials

Silt-clay constituents in excess of about 35 percent keep the granular particles from close contact and interlocking, with marked lowering of the friction of the system and with the loss of the skeleton effect. Such systems often possess great cohesive strength in dry condition but lose the strength in the presence of moisture which they absorb avidly under swelling.

There is, therefore, a dividing line between non-swelling and swelling soils which occurs at silt-clay contents of about 35 percent. In the non-swelling soils, even at the highest moisture content, the volume of the swelling constituents does not exceed the pore space left by the mutually contacting larger grains. In the swelling soils the coarse particles are isolated in a matrix of finer particles. The actual limiting proportion of swelling components depends upon the granulometry of the system and on the water affinity of the silt-clay material.

TABLE 9

MECHANICAL RESISTANCE PROPERTIES OF TWO SOILS AND EIGHT OF THEIR HOMOIONIC MODIFICATIONS AS DERIVED FROM TENSILE AND COMPRESSION TESTS ON AIR-DRY SPECIMENS AND FROM CONSISTENCY MEASUREMENTS IN THE MOIST STATE

		Strength •					
Soil	Exchange cations	Tensile (psi)	Compress. (psi)	Internal Pressure (psi)	Angle Ø (deg)	Plasticity Index	Angle Ø (from P. I.) (deg)
Cecil	H	61	73	369	5.17	34	20.7
	Na	52	86	132	14.23	30	21.8
	K	63	110	148	15.72	38	19.7
	Mg	70	102	76	10.72	34	20.7
	Ca	51	125	86	24.83	34	20.7
	Ba	75	121	198	13.53	36	20.2
	Al	72	147	141	20.20	31	21.5
	Fe	60	142	103	24.00	36	20.2
	Natural	65	124	137	18.18	37	20.0
Hagers-town	H	105	445	133	38.17	21	24.0
	Na	135	342	233	25.79	40	19.2
	K	88	341	119	36.15	25	23.0
	Mg	110	575	136	42.85	27	20.5
	Ca	182	357	381	18.97	23	23.5
	Ba	153	341	278	22.33	21	24.0
	Al	93	368	124	36.65	16	25.4
	Fe	98	302	145	30.67	16	25.4
	Natural	160	312	349	18.78	26	22.8
Condition		Air dried				Moist to wet	

THE MECHANICAL RESISTANCE PROPERTIES OF SILT-CLAY MATERIALS

The shear resistance of cohesive soils is usually expressed by the equation:

$$S = \sigma \tan \phi + C$$

where S = shear resistance,

σ = normal stress on the shear plane,

$\tan \phi$ = coefficient of internal friction and

C = Cohesion

Actually, friction and cohesion in such systems are not independent of each other but are both functions of the density, the moisture content, and of the manner in which the particular moisture content has been attained. The friction portion of the shear resistance may be expected to follow the same type of volume law that has been previously described for the internal friction of liquids and of granular materials. The first step in understanding soil cohesion is to consider it as a result of the attraction forces acting from the mineral surfaces through the water films and of geometric factors that have the functions and therefore the properties of an angle of friction. Accordingly, one may write:

Cohesion = internal pressure x tan ϕ.

For soils at very low moisture content this internal pressure is theoretically of the order of magnitude of 10^6 pounds per square inch. It falls logarithmically to zero at a moisture content at which the soil flows under its own weight, i.e., at which the soil behaves as a liquid (liquid limit). Because of secondary aggregation and nonideal arrangements of the cohesion-giving water films, the actual strength of cohesive soils is much smaller than the theoretical. It has been found (34) that even monocrystals of pure minerals such as sodium chloride give tensile strength values that are two to three orders of magnitude smaller than those calculated from theory. It is, therefore, not surprising that experimentally determined strength values for carefully dried clay soil specimens and the related internal pressures are systematically lower than 1,000 psi. and usually lie between 100 and 400 psi.

In Table 9 actual data are presented for compressive and tensile strength of specimens of nine ionic modifications of two different clay soils. The specimens were molded in the plastic range and carefully air dried (35). From the experimental data, angles of internal pressures were derived using the Mohr circle method. The Table also contains angles of friction derived for the same soils from the plasticity index according to Kögler and Scheidig (36). It is of interest that the angles of friction derived for the different ionic modifications of the same soil vary less for the plastic state for which Kögler's formula was assumed to hold, than they do for the solid state for which the values had been obtained from compressive and tension data by use of the Mohr circle method.

Soils can be considered as (1) solids if their moisture content is below the shrinkage limit and if they have been dried to this point from higher moisture contents; as (2) being in the plastic state if their moisture content falls between the plastic and liquid limit; and (3) as liquids if their moisture content is above the liquid limit.

The location of these limits depends on the clay and colloid content of the soils, on the type and the activity of the clay fraction, and on the granulometry of the entire soil including its clay fraction. The greater the tendency to structure formation inherent in the clay the higher is its shrinkage limit. Also, the presence of ions furthering structure formation increases the shrinkage limit, while the presence of dispersing ions decreases the shrinkage limit.

The plastic state may be considered to begin as soon as the active surfaces in the soil system are covered with films of water that are sufficiently thick to be continuous and to have lubricating properties. Therefore, other factors being equal, the larger the amount of internal surface, i.e., the larger the clay content the higher the plastic limit.

However, dispersed clays may require thinner moisture film for lubrication than aggregated clay particles. For this reason the plastic limit sometimes falls with increasing dispersion of the clay fraction. The liquid limit represents theoretically a moisture volume which permits the independently acting primary or secondary soil particles to rotate freely. In fact, the possibility of free rotation of the constituent particles is considered as the most important index of liquidity. Therefore, the larger the proportion of plate-shaped particles and the larger the ratio of the volume of the rotation ellipsoids to the actual volume of the particles, the higher will be the liquid limits.

An increase in the dispersion of a clay may, on one hand, decrease the plastic limit and, on the other hand, increase the liquid limit. The presence of silt and sand fractions may modify these conslusions because of their interference and interaction with the clay particles. It has been found quite often that increasing amounts of clay were inactivated in part by increasing amounts of silt particles on the surface of which they were strongly adsorbed (37).

Finally, there is a great influence of the granulometry. The latter is treated by Pimentel dos Santos. The scientific study of the resistance properties of silt-clay-water systems falls within the realm of the science of rheology, and there exists a great need for more fundamental studies following the concepts of Eyring (38), Mack (39), Ruiz (40), Nijboer (41), and others (42).

Stabilization of Silt-Clay Materials

The crucial difficulty involved in the stabilization of cohesive soils that do not contain granular skeletons is the fact that the seat of the resistance providing cohesion is also the seat of the water affinity. The cohesion increases with decrease in water content. Theoretically, the simplest method of stabilizing such materials is to (1) deprive the component particles of their water affinity and (2) to cement these particles together.

This exactly is achieved by the oldest method of soil stabilization, namely the burning of clay soils for the making of pathways, which has been used extensively by the Australian aborigines (43). However, the practical difficulties involved in this method are obvious; despite much recent experimentation with large road burners the most-promising use of this method is the manufacture of coarse ceramic aggregate and its subsequent use in granular soil stabilization.

The economics of this problem have been well explored in Argentina where the process was concluded to be feasible under certain conditions of absence of natural granular materials. The relationship between pedologic soil type and effectiveness of thermal stabilization has been studied especially in Russia (44).

Another use of this principle is to replace the water attractive inorganic exchangeable cations by means of water repellent organic cations and to cement the soil constituents, that have now lost their water affinity, together by means of an organic cement. This can be done using aniline-furfural and similar resins (45).

Another method is to create strongly water resistant secondary soil aggregates which are then cemented together by means of inorganic binders such as Portland cement, flyash-lime combinations, gypsum plasters, sorel cements, etc.

A less-positive but still effective method, if properly used, is to stabilize the moisture content of a soil system by preventing the intake of excess water. This water-proofing may be achieved by various materials such as asphaltic and pyrogenous bitumens, natural and artificial resins, fats, waxes, etc.

The scientific factors involved in waterproofing of cohesive soils have been studied in considerable detail (46, 47). Since it is usually desirable that the stabilized system contain a certain minimum volume of stabilizer, combinations of highly active materials with less active ones are often indicated as well as combinations of inorganic and organic stabilizers.

Finally, systems may be synthesized by low-temperature polymerization in which the actual soil constituents are held as particulate inclusions in a fibrous network or felt. The fibers may or may not be attached to the clay particles by ionic, resonance or other forces. However, this method requires a rather large amount of water which brings the system to the sticky point and ofter makes mixing extremely difficult. Also the excess water is likely to soften the subgrade and rob it of its supporting power. In addition, a rather large amount of expensive stabilizer (usually above 10 percent by weight) is required. Nevertheless, this is an interesting development and has certain uses in fields other than road construction.

The scientific principles involved in the stabilization of cohesive soils, including the microbial problems, have been treated quite thoroughly in the literature (48) and need not be discussed here in greater detail. In all these methods, except perhaps the thermal one, it is important that, at the time of treatment, the soil has sufficient moisture to satisfy its hydration requirements and that the particles are surrounded by continuous moisture films without excess or free water, since the latter may lead to undesirable stickiness of the system. Table 10 shows the relationship between soil stabilization and its supporting sciences. Figure 6 is a reproduction of a chart published in 1943 by Olmstead (49) in a paper entitled "Factors Related to the Design of Stabilized Mixtures." The chart exposes most clearly the connection between soil texture and stabilization requirements.

SCIENTIFIC FACTORS INVOLVED IN THE CONSTRUCTION OF STABILIZED COURSES

The science of soil stabilization covers not only compositional, structural and resistance properties of stabilized systems but also the problems involved in their manufacture in the field. Constructionwise, soil stabilization involves the three unit processes: (1) comminution, (2) mixing, and (3) compaction.

In the case of purely granular soils the comminution factor is, of course, absent. The farmer who has a similar problem of comminution when he prepares the proper seed bed for his crop has learned that (1) for cohesive soils the plow is a most effective instrument and that (2) there exists a moisture range in which comminution to the desired particle size is most easy to achieve. The latter is within the plastic range but below the point where the water is already sufficiently free to stick to the plow metal (sticky point).

In regions where it is normally dry during the construction season it is easy to reach the most advantageous moisture content by addition of water. A real problem exists when soils have natural moisture contents above the sticky point; this may result in very costly delays of weeks and even months. These delays can often be shortened or even avoided by the use of "shortening agents." Among the latter aniline-furfural combinations have proved themselves as most versatile and most effective. While some of our modern comminution machinery has been developed with sufficient power to break up even dry, hard-caked soils to sizes sufficient for subsequent stabilizing treatment, it is better science and better practice to bring the soil first to that degree of moistness at which it is most easily comminuted to particles of the proper size.

Energy of Comminution

At the present time, the energy of comminution cannot be calculated exactly from theory alone; however, knowledge, even if only approximate, of this energy is of great economic as well as scientific importance. For this reason, an approach to the scientific solution of this problem is given in the following. The amount of energy required for comminution depends, among other factors, upon the ultimate particle size required and upon the extent to which granules of this size are already preformed in the natural soil. Accordingly, it should require relatively little energy to comminute soils to granules of gravel and sand sizes but very considerably more to break a clay soil down to the ultimate clay particle units.

There exist three general approaches for the calculation of comminution energy. In each approach certain assumptions must be made because of lack of pertinent sci-

TABLE 10

The Science of Soil Stabilization rests on the three pillars of

Pedology	Physical and Chemical Sciences	Body of Knowledge on Traditional Construction Materials
The science of physical and chemical soil characteristics as functions of: 1) parent material 2) climate 3) topography 4) organisms 5) time	These allow judgement of soil deficiencies and indicate supplementation in physical and chemical terms that are translated into economically available materials, such as 1) inorganic cements 2) organic cements 3) waterproofing agents with or without improvement of granulometry of the soil.	A stabilized soil, being a construction material, must resemble traditional construction materials in its essential resistance properties and structure. Especially important are granulometric considerations. Most nonmetallic construction materials and most stabilized soil systems fall in the category of collameritic systems.

entific data. (1) The work performed in creating two unit surfaces equals one half of the product of tensile strength and the distance over which the tensile force must be moved. (2) The work of comminution equals the maximum strain energy that can be stored by the body under consideration under the condition of strain application. Comminution is into "natural" structural units. (3) The work of comminution equals the surface energy of the new surfaces created.

The required assumptions are for (1) the effective action radius of the cohesive forces, for (2) the modulus of elasticity, and for (3) the surface energy.

We shall employ Approach 2 and utilize Approach 3 to calculate the dimensions of easily produced natural soil units.

Using an average tensile strength value of 200 psi. for a dry clay soil (strongest state) and a probable modulus of elasticity value of 92,600 psi. (from German dynamic measurements), we may calculate a failure strain of:

$$\epsilon = \frac{200}{92,600} = 0.00216 \text{ inch per inch.}$$

Multiplying this strain by half of the failure stress, we obtain:

$$\begin{aligned} 0.00216 \times 100 &= 0.216 \text{ in.-lb. per cu. in.} \\ &= 0.018 \text{ ft.-lb. per cu. in.} \\ &= 0.018 \times 1.356 \times 10^7 \text{ ergs per cu. in.} \\ &= 224 \times 10^3 \text{ ergs per cu. in.} \\ &= 39 \text{ joules per cu. ft.} \\ &= 1.45 \times 10^{-6} \text{ horsepower-hours per cu. ft.} \end{aligned}$$

The last value represents the maximum strain energy that can be stored per cubic foot of the soil if employed as a bar in tension. If employed in pure bending, the value falls to a third of that given, and if used as a cantilever, to a ninth.

Similar calculations may be made for soils in the plastic range using data published by Professor Tschebotarioff (50). These data are given in the next table.

Soils	Plastic Limit %	Liquid Limit %	H_2O Content %	Tensile Strength Kg/cm²	Strain at failure %
Bentonite	53	540	101	0.21	3.4
Kaolinite	32	70	37.6	0.03	0.2
Illite	26	60	31.5	0.40	0.8
Clay "Pr"	23	30	13.8	0.08	0.6

From these data, the maximum strain energy that can be stored in pure tension can be calculated approximately as the product of half of the failure stress and the failure strain. Accordingly, we obtain:

Elastic Energy Stored per Unit Volume up to Failure

Soil	cmkg/cm³	mkg/cm³	mkg/in³	mkg/ft³	joule/ft³	Hp-hr per ft³
Bentonite	0.00357	35.7×10^{-6}	58.5×10^{-5}	101×10^{-2}	9.9	3.69×10^{-6}
Kaolinite	0.00009	0.9×10^{-6}	1.47×10^{-5}	2.54×10^{-2}	0.25	0.093×10^{-6}
Illite	0.0016	16.0×10^{-6}	26.2×10^{-5}	45.3×10^{-2}	4.45	1.64×10^{-6}
Clay "Pr"	0.00032	3.2×10^{-6}	5.24×10^{-5}	9.05×10^{-2}	0.89	0.33×10^{-6}

Size of natural breakage units

Assuming that the entire stored strain energy is used for the creation of new surfaces without actual breaking down of primary soil particles but having the new surface going through the film water between the particles, and assuming that adsorption forces on the surfaces of the solid particles raise the surface energy of water to about 150 ergs per sq. cm., we obtain a surface energy of 6.45 x 150 ergs = 968 ergs per sq. in. Dividing 39×10^7 ergs per sq. cu. ft. by 968 ergs per sq. in. gives

$$4.03 \times 10^5 \text{ sq. in. per cu. ft.}$$

The surface in sq. in. per cu. ft. is 6 x 144 = 864 sq. in. for a cube of 1 foot side length and 6 x 1728 = 10,368 sq. in. if subdivided into cubes of 1 inch side length.

The general formula

$$\text{Surface in sq. in.} = \frac{10,368}{\text{side length in inches}}$$

For a surface of 4.03×10^5 sq. ft. we obtain a side length of:

$$\begin{aligned} \frac{10,368}{4.03 \times 10^5} &= 0.026 \text{ inch.} \\ &= 0.066 \text{ cm.} \\ &= 0.66 \text{ mm.} \end{aligned}$$

This value indicates that the resulting particles are of the order of magnitude of a medium sand. This reasonable result indicates that the extrapolated assumptions made in its calculation are also reasonable. This fact is important not only for estimating power requirements in soil comminution but also with respect to soil response to dynamic loading by blasting and forced mechanical and sonic vibrations.

The calculations just presented are, of course, only a first approach. However, this approach should be followed up and should finally lead to a more rational design of comminution equipment than is possible at the present time.

Mixing Step

The problem of the energy and power requirements for adequate mixing of stabilizers with the soil material in its normal moist condition has as yet not been properly studied. While some extrapolation can be made from the experience in mixing problems of the chemical industry, the special properties of the soil mixtures involved in stabilization indicate the need for specific studies with such materials.

Densification Step

Because of the very intimate relationship between the density of a stabilized soil system and its resistance properties, compaction is probably the most important single feature in soil stabilization. In studies performed for the Civil Aeronautics Administration it has been shown that the difference between normal and modified Proctor compaction in certain stabilized soils was equivalent to the effect of approximately 2 percent of portland cement. This means that systems stabilized at the higher density with 2 percent less portland cement were equal in resistance to those stabilized at the more of Proctor density with 2 percent more of portland cement.

However, for this to be the case, the higher density had to be reached with a moisture content sufficient to satisfy the hydration needs of both the soil and the portland cement.

In view of the large amount of compaction tests that have been performed in soil laboratories under well standardized conditions, it would seem that enough knowledge is available for judging the work and power requirements in compaction. Actually the wall effect of the small molds that are used in the laboratory has never been definitely evaluated, and rational extrapolation from laboratory tests to field conditions has as yet not been accomplished.

In addition, the molding compaction actually used in the field differs often quite essentially from the compaction by means of the impact of the falling hammer employed in the laboratory test. These and other compaction problems should be investigated on the basis of fundamental theoretical considerations.

Economic Considerations

Soil stabilization represents an engineering science and as such must include economics. Economic considerations are not restricted to the cost of materials which represents only a portion of the total cost but also must cover the costs of construction and of maintenance during the service life of the structure. It would be impossible to make here a complete analysis of all the important economic factors, but it can be pointed out that often a slight increase in cost of materials caused by the use of "construction aids" can be balanced by a lesser construction cost due to easier mixing and compaction of the treated soil. If such "construction aids" also improve the final quality of the system, so much the better.

The costs of traditional cementing and water proofing materials are relatively stable within certain regions, and it is not too difficult to decide which of several has economic preference in a particular location.

However, the relative costs vary for different areas. In one region portland cement may be more economical than bitumen as is the case in Portuguese East Africa, while in other regions, bitumen may be more economical than portland cement. The same holds true with respect to lime, flyashes, and other materials.

The economics are not as obvious in the case of chemicals that have not been used previously for soil stabilization purposes since the unit price of chemicals usually decreases with increasing industrial production. Therefore, it would be unreasonable to base economic conclusions solely on present prices of certain chemicals. However, it is very much to the purpose to point out certain fundamental economic facts.

It is obviously an economic waste to employ an expensive cementing material not only as a cement but also as a filler. As it is an economic waste to have one man build an entire automobile by himself, the same way it is uneconomical to have one chemical alone remedy all the various deficiencies of a soil. It is usually more economical to employ several mono- or bi-functional substances instead of one alone that is polyfunctional.

Even in the case of beach sand stabilization for military purposes, where normal economics are out of the picture, it is certainly advisable not to use the more expensive resinous substance to do the entire job of cementing the sand grains together, but to dilute the synthetic resin by means of pitch, asphalt, or other organic or even inorganic cementing materials which can be easily employed in the process (11).

Also, if certain elastomeric properties are desired, these can be obtained by incorporation of elastomeric powders or dispersions. This possibility resides in the fact that the large bearing systems created in soil stabilization are quasi- or practically isotropic.

There is, of course, no such thing as a completely and truly isotropic system in nature since even the atoms and the nuclei are nonisotropic. Practical isotropicity depends upon the size of the largest component of the system with respect to the size of the area of stress application. If the latter is much larger than the coarsest components of the resisting material then we have practical isotropicity.

In portland-cement concrete we allow, for instance, sizes of the large aggregate up to a third of the smallest dimension of the structural system.

From the foregoing considerations it can be seen easily that it is a sound economical principle to employ several lower cost mono- or bi-functional components to achieve certain combined properties than to employ an expensive polymer in which several functions have been embodied.

CONCLUSIONS

It has been intended in this introductory paper to expose those soil properties that are important for scientific soil stabilization, to indicate how these properties must be supplemented or modified in order to produce construction materials, and to point out material and economic facts of importance in actual construction.

If we accept Poincaré's definition that the essence of a science is to give the same name to different things, then it is evident that this condition is fulfilled in the case of scientific soil stabilization. The methods of stabilizing granular soils have been included in the collameritic system which embraces a large number of traditional construction materials as well as an important group of stabilized soils. At the same time it has been demonstrated how physico-chemical, chemical and physical principles are important in the proper stabilization of soils especially of those that do not possess a granular skeleton. Also, basic principles with reference to the economy of producing stabilized compositions have been pointed out.

This being an introductory paper, it was intended to indicate but not to anticipate the information which is presented in the subsequent papers on more specific subjects. This permitted concentration and emphasis on the principles involved. At the same time certain basic considerations and data relative to the complex system "Soil" could be presented which are not usually found in engineering literature.

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Inheritance Factor in Origin of Clay Minerals in Soil

FRANKLYN B. VAN HOUTEN
Department of Geology, Princeton University

Introductory Remarks by the Chairman: Soils are products of both inheritance and environmental factors. In most soils one or the other set of factors predominates. While this is well known and accepted by all soil scientists, individual workers tend to overemphasize the importance of either the geologic inheritance or the pedologic environment.

Van Houten shows in the following paper how important the inheritance factor is for large American soil areas. This is of great practical significance, since most soil-stabilization standards employed throughout the world are of American origin. If American soils, especially those on which stabilization methods have been developed, were unaffected by inheritance and were products solely of environmental factors, then the standard procedures developed on them could be applied without change to the same types of environment in other areas of the world. If, however, these American soils and those to which the standard procedures are to be applied are greatly influenced by inheritance, then it is obvious that methods proven in America may be unsatisfactory for other regions. This is the reason why it is so important to rely on scientific principles if one faces problems in virgin areas.

Van Houten is especially interested in the character of the clay fraction and he arrives at the significant conclusion that clay minerals of soils developed on sedimentary rock commonly are inherited from the parent material.

● ONE of the most-significant properties of a soil is its colloidal clay fraction composed essentially of crystalline particles of the mineral groups, kaolinite, montmorillonite, and illite. Few soil-clays consist of but a single clay mineral. Instead, most contain a complex mixture of several, although one commonly predominates.

During the first decade of study of the colloidal clay fraction of soils, from 1930 to 1940, the clay minerals present were almost universally thought to be primary products of weathering, synthesized in the soil, which reflect a particular environmental control. As a consequence, it was held that primary kaolinite, the more stable of the clay minerals, predominates in soils of warm, humid regions, whereas primary montmorillonite and illite, each less stable and containing more silica than kaolinite, prevail in soils of cooler and drier areas.

The clay minerals in soils developed on igneous and high grade metamorphic rocks are, of course, primary or authigenic, for these parent materials contain no clay minerals. Significantly, however, more than three fourths of the soil in the United States rests on clay-bearing sediments and sedimentary rocks whose clay minerals could be inherited by overlying soils.

Just as sand grains of a sedimentary rock may persist in a derived soil, so clay minerals of the colloidal fraction of the rock may also be inherited. Yet the concept of primary clay minerals in soils so dominated the pioneer decade that the possibility of inherited clay minerals in soils was never explored. Apparently as a result of this disinterest, few soil studies include an analysis of the clay minerals in the rock from which the soil was derived.

With increased interest in the clay-mineral content of both soils and sedimentary rocks during the past 15 years, soil scientists have been led to a new awareness of the geologic materials on which soils develop, while geologists are beginning to appreciate the soil scientist's contribution to knowledge about products and processes of weathering. Studies of the weathering sequence in soils and sediments by Jackson and his colleagues (1948) and by Jeffries and his students (1953) are notable examples. In fact, much of our basic information about the clay minerals has come from soil studies.

An outstanding summary of present knowledge about clay mineralogy is the work of a geologist (Grim, 1953), while a discussion of current problems concerning the mineralogy, genesis, and occurrence of clays and laterites was sponsored by the American Institute of Mining and Metallurgical Engineers (1952).

It should also be noted that some soil scientists have recognized that in accepting Glinka's emphasis of the role of climate in soil formation there followed "a stampede to the Russian point of view. Geology was forsaken for climatology. Recognition of the importance of the new factor was at first accompanied, as it so often is, by neglect of the old" (Lutz and Chandler, 1946, p. 60). One of the more-balanced views of the origin of soils, a consideration of all significant factors, has been developed by Jenny (1941). The soil scientist's need for knowledge of the geologist's classification of rocks in order to better understand the materials out of which soils are made has also been pointed out (Whiteside, 1954). 'This sort of profitable exchange has just begun, however.

In the present paper one phase of the interrelated studies of geology and soil science will be examined briefly. It is the role of the inheritance factor in the origin of clay minerals in soil. Examples will be cited and the consequences of the proposition will be discussed.

During the past 15 years a number of soil analyses have revealed that some soils have inherited their clay minerals from parent sedimentary rock, and that the inherited clay mineral commonly has been somewhat altered by soil-forming processes. As a result of this alteration, illite in soils generally has a lower K_2O content and SiO_2-to-Al_2O_3 ratio than the illite in many sedimentary rocks. The pertinent studies are cited to illustrate the extent to which the inheritance factor has already been demonstrated.

In central New York illite-rich soil showing no measurable differences in clay-mineral content throughout the profile, was formed on the Devonian Enfield shale whose clay fraction is predominantly illite. Thus, it seems reasonable to conclude that the soil-clay was inherited from the parent Enfield shale (Martin, 1954).

Reddish Prairie soils of Oklahoma were analysed "to determine the influence of parent material and soil weathering processes upon the clay mineralogy of these mature soils" (Wilkinson and Gray, 1954). The study revealed that "the clay mineralogy of the soils is the result of these minerals being present in the parent material," Pennsylvanian shale and Permian red beds.

Similarly, the abundance of kaolinite in the Gosport soil of Iowa has been attributed to its kaolinite-rich parent Pennsylvanian shale (Peterson, 1946), and montmorillonite in the White Store soil of North Carolina was interpreted as being derived from the montmorillonite-rich Triassic shale on which the soil has formed (Ross and Hendricks, 1945). Kaolinite-rich Norfolk soils developed on Cenozoic deposits of the Atlantic Coastal Plain have a clay mineral composition like that of the parent material, suggesting that "it is perhaps reasonable to assume that the greater part of the clay content now present in these soils existed in the soil parent material at the time it was laid down" (Holmes, Hearn, and Byers, 1938, p. 27).

According to Fiskel and McCaleb (1954) the differences in the clay mineral content of some soils formed on Florida Costal Plain sediments may be attributed to different kinds of parent materials; and Kelly, Dore and Page (1941) concluded that parent alluvial deposits probably were the source of most of the clay in a number of illite-rich Solonetz and Black Alkali soils they analyzed.

Most of the studies that recognize the role of the inheritance factor are based on analyses of soils developed on Pleistocene glacial deposits. Illite in the weathered zone or gumbotil of Illinoian till in Indiana was interpreted to have been inherited largely from the fresh till (Gravenor, 1954). In addition, Gravenor suggested that illite in the till was inherited mainly from the underlying Paleozoic rocks. Analysis of illite-bearing Miami and related soils (Bidwell and Page, 1951) indicated that these soils may have derived much of their clay from their parent material, illite-rich Wisconsin glacial drift.

Similarly, Martin and Russell (1952) suggested that abundant illite in four southern New York soils (Barth, Mardin, Volusia and Chippewa) was inherited directly from disintegrated Paleozoic shale debris in the parent Wisconsin till. McCaleb (1954) showed that Gray-Brown and Brown Podzolic soils in New York which are developed on illite-bearing glacial till contain abundant illite in all horizons of all profiles.

Despite this available information, many soil scientists have not recognized inheritance as a significant factor determining the clay-mineral composition of soils derived from sedimentary rocks. To a geologist familiar with the role of relic minerals in the regolith, on the other hand, these examples are impressive evidence of a neglected inheritance factor that may play an important role in the origin of clay minerals in many soils.

In order to test the validity of the inheritance factor, and at the same time to provide more-extensive information about the relationship of clay minerals in soils and their parent sedimentary rocks, 43 samples of sedimentary rocks and the A horizon of the derived soils from many soil areas were analyzed for the predominant clay minerals present (Van Houten, 1953).

In 35 of the 43 pairs of samples the predominant clay mineral group in the soil was found to be the same as that in the parent rock. Of the eight soils whose clay-mineral content differs markedly from that of the parent rock, one is a kaolinite-rich soil developed on chert. The other seven soils, also containing abundant kaolinite, were formed on limestone, dolomite, and calcareous shale rich in illite or a K-bentonite type of clay mineral.

The data assembled here support the conclusion that the clay minerals in many soils developed on sedimentary rocks have been inherited from the parent material. Many such soils contain an abundance of relic illite. Although long and intense weathering alters its composition, an illite-type mineral occurs abundantly in the regolith under a wide range of environmental conditions.

The effect of the inheritance factor is not limited to illite-bearing profiles, however. Some kaolinite-rich soils in southeastern United States have inherited at least part of their clay minerals from the parent Coastal Plain deposits, just as some of the montmorillonite in soils of the western Great Plains and the Black Belt of the Gulf Coastal Plain undoubtedly has been inherited from montmorillonite-rich parent rock. The data also reveal that, whereas illite is more common in soils than generally supposed, montmorillonite apparently is the least common of the important soil-clay minerals.

The prevalence of montmorillonite-rich soils in the western United States is commonly cited as evidence that the montmorillonite is a primary product of weathering in a dry climate. An alternative interpretation, presented here for consideration, is that these montmorillonite-bearing soils have inherited much of their clay fraction from parent material containing altered volcanic ash.

Not only are most of the analyzed montmorillonite-rich soils derived from Mesozoic and Cenozoic deposits containing partially altered volcanic debris, but most of the montmorillonite in rocks is restricted to Mesozoic and Cenozoic deposits whose volcanic ash has been diagenetically altered to montmorillonite. Significantly, much of the mantle of sedimentary rock in western United States is Cenozoic deposits containing volcanic debris. Hence, it seems reasonable to postulate that much of the montmorillonite in western soils was inherited from the altered volcanic ash in the parent material.

The fact that illite can be inherited in soils developed on sedimentary rocks gives added significance to the geologist's discovery that illite is the most-common clay mineral in the extensive mantle of sedimentary rocks. It predominates in the analyzed samples of ancient shales, underclays, limestones, dolomites, and slates; it is the principal clay mineral in Pleistocene till and loess; it is also abundant in recent marine sediments and in the alluvium of large rivers. On the basis of these observations, one can reasonably predict that in general most soils developed on sedimentary parent material will contain important amounts of illite that has been altered somewhat by the soil-forming processes.

Clearly, the inheritance factor must be considered in any clay-mineral study of soils developed on sedimentary rocks. Nevertheless, emphasis of this factor is not intended to diminish the importance of movement or fractionation of colloidal material through the profile, or to suggest that clay minerals inherited from parent rock cannot be altered by appropriate weathering conditions. Moreover, it should be noted that a soil containing some inherited clay may also have primary clay minerals produced by weathering of other less-stable minerals of the parent rock.

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Engineering Pedology and Soil Stabilization

F. L. D. WOOLTORTON, Planning Engineer
Public Works Department of Nairobi, Kenya Colony, East Africa

Introductory Remarks by the Chairman: The chairman became acquainted with Wooltorton's work during the early thirties when the colonel wrestled with problems of road construction in tropical regions as executive engineer in Shwebo, Burma. At that time, American and Western soil engineering did not appreciate or even recognize the extreme phenomena occurring in the formation of tropical soils and in roads built on and of such soils.

Wooltorton learned the hard way about the nonapplicability to his problems of many of our "standard" tests and procedures and about the limitations of soil mechanics as commonly conceived. His search for understanding as a basis of rational design methods led him to the use of pedological information and methodology. The latter, however, were often in a form alien to the civil engineering practicioner. To bring them into such form required many years of intensive studies and of hard practical work. How well Wooltorton has succeeded is evident from his recent book on "The Scientific Basis of Road Design," and in his contribution to this symposium.

While other engineers have been employing pedological tools for better and more-economical road construction, this use was in the main restricted to pedological soil-identification procedures and to the supplementation of the information contained in pedological soil maps with more quantitative engineering data. Wooltorton's contribution goes much farther than this. He realized that the dynamic factors that formed a soil and gave it its pedological characteristics continue to act, though in a modified form, in the soil-subgrade-base-pavement system and determine to a large extent the quality and the durability of this system. As the result mainly of his efforts, a new engineering discipline, "engineering pedology" was born. What this discipline is and what it can do for scientific soil stabilization is ably presented in the paper which follows.

● THERE is some confusion over what is meant by the word "pedology."[1] The appropriate root is the Greek word 'pedo' meaning 'ground'. Pedology is described by one of the greatest teachers Jacob S. Joffe (1) as "a branch of soil science of the science of soils. Its scope is the study of the phenomena presented to us by the soil body in its natural position." By "natural position" is meant that particular position in which it is desired to study the phenomena presented. In pure pedology, this position would be the undisturbed insitu position. In road engineering it would be either the undisturbed (as in cuttings) or some variation of the disturbed state (as in embankments).

Engineering pedology thus becomes the science of investigating soil engineering problems, making use of all natural laws known to be operative under the particular and specific conditions prevailing.

As, in its wider sense, soil material includes rock and any other soil-forming material, engineering pedology may be considered to embrace the study of all problems pertaining to any structural unit in any type of road pavement. It is true that cement, bitumen, and other chemical additives of various kinds cannot be included amongst the soil-forming materials, but the laws governing the behavior of a road unit into which they may have been incorporated are, in general, the same as those pertaining to units formed entirely of soil. Such natural laws as those governing ionic exchange, dipole exchange, adhesion, cohesion, energy conditions as illustrated by changes in moisture content, friction and permeability etc; or, in other words, those natural laws governing stability and changes in stability are applicable, in varying degrees, to all such materials.

It is not intended to suggest that engineering pedology supplants road soil mechanics, for it does not. In the overall picture the two are complementary.

Engineering pedology supplies the scientific background to road soil mechanics, acts as a guide and control in the general

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[1] The word is pĕdology and not pēdology or paedology used for the study of children.

application or soil mechanics, and supplies a medium for a type of basic research and of testing generally outside the scope of orthodox soil mechanics.

CONCEPT OF ENGINEERING PEDOLOGY

It is impossible to say who was the first researcher to use pedology as an aid in solving road problems. The approach, however, is not a new one. It is known that Olaf Stokstad of the Michigan Highway Department made use of pedology in engineering studies of highway problems as related to soils as early as 1924;[2] that its use was advocated during the Montana National Bituminous Conference, 1938; that L.D. Hicks of the North Carolina State Highway and Public Works Commission used it in the early forties, if not before; that a large number of states have made use of its methods; that Hans F. Winterkorn, of Princeton University, has constantly made use of this approach since the early 30's; and that it featured materially in a report by Harold Allen, of the Bureau of Public Roads, in 1945. During 1934 Wooltorton was forced to adopt the pedological approach for investigating a problem of soil movements, because the soil mechanics of those days was unable to assist him towards a solution.

The conception of the expression "engineering pedology" can, it is believed, be accredited to Winterkorn, who first used the term early in 1944. It is likely that the science, as a science, first took form and evolved its philosophy during discussions at Princeton later in the same year, at which time a program of basic research necessary for its development was outlined.

It is only right to record here that such an approach to the solution of soil-engineering problems has been persistently advocated in the past by many soil scientists, and that one of the first engineers to encourage its use was the greatly experienced and far-sighted American road engineer the late C. A. Hogentogler of the Bureau of Public Roads.

NECESSITY FOR ENGINEERING PEDOLOGY

Objection may be taken to the use of the expression "engineering pedology" with reference to road design on the grounds that a satisfactory one exists in that of "soil mechanics."

The author wishes to make it clear that his views on design are founded on his experience of conditions prevailing in tropical countries, where the design-outlook must frequently differ from that tenable in more-temperate regions. And because there cannot be one science of design applicable to tropical climates and another applicable only to temperate climates, he believes there should be only one science, applicable universally, in which design under any climatic condition implies a shift from the consideration of one set of variables to another in the overall scale of variables controlling design as an all-embracing science.

Consider for one moment the trouble sometimes caused by roads in a temperate region as a result of one of those occasional long droughts. Under some tropical conditions desiccation is a yearly occurrance.

Consider also what may happen to some lateritic soils on being worked when the soil may change from one which is non-plastic to one with a plastic index of 100 with an accompanying change in an F. M. E. of 60 to one of 250.

The author has recently been informed[3] that the swelling pressure of a red Nigerian laterized soil, containing considerably more montmorillonite than kaolinite, has been measured at 11. 5 tons per sq. ft.

Such phenomena are not commonly encountered under temperate climates, but the possibility of their occurrance should be provided for in the science of design and testing.

And yet, orthodox design recommendations, excellent as they are for use in temperate regions, are based on the presence of the moderately active clay mineral illite and the general absence of desiccation, despite, in the latter instance, the warning given to the dangers attendant on desiccation by H. Allen (1a) in 1945.

In terms of testing, orthodox recommendations do not always appear to be satisfactory in the tropics as regards: (1) applicability of base course recommendations for grading and plastic index; (2) reliability of C.B.R. values (determined according to standard specifications); it would seem that they are as likely to be too low as too high; and (3) specifications for the ex-

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[2] In 1926 Charles Kellogg made a detailed soil survey of roads in and on behalf of Michigan and included a comprehensive report on the engineering significance of the soils encountered.

[3] By Alexandre Ceresa, consulting engineer, Brussels

pansion test which do not take cognizance of the possibility of soil desiccation.

Additionally, there is no known scientific background to soil mechanics which will indicate the type of chemical additive (including cement, bitumen, etc.) likely to be most successful in the stabilization of a given soil—which is also a chemical.

For such reasons there is, it is considered, a clear case for a design science which will at once indicate which of the many variables require special consideration and how any specific problem may be most economically solved. In other words, there is a necessity for a scientific background to design based on soil properties. Without such a background successful soil stabilization would appear to be fraught with difficulties and uncertainties.

This requirement is fulfilled by pedology, which, to quote G. W. Robinson (2), "is the study of the soil from the standpoint and by the methods of pure science." It embraces "the study of the physical and chemical properties of a complicated colloidal system" by which soil engineering properties are determined.

These properties are the summation of those of the soil's primary and secondary minerals and the soil's organic matter as modified by the soil's air, containing impurities; the soil's moisture, containing substances in colloidal and true solution; and by the soil's artificial or natural structure.

With the exception of those of the primary minerals, dependent on the nature of the parent material, the properties are functions of the parent material and the climate, the movement of moisture within the soil profile, and the maximum and minimum values of the moisture content at the level under consideration.

The climatic and other controls mean that the overall properties will vary with the great soil group to which the soil belongs and with the horizon from which any sample may be obtained. This has been recognized, in particular, by the State of Michigan.

With such a philosophy, the emphasis is more on the properties suggested by the great soil group to which any sample belongs, which indicate the nature of the desirable testing, than on a standardized system of testing expected to indicate these properties.

It is realized that all the phenomena discussed in this paper are known and have all been investigated. In fact, much of the information utilized in the statements made has been obtained from U. S. research publications. It is also, however, known that the only specifications widely publicized are those really only applicable to the more-temperate climates; and that no instance can be brought to mind where any country outside Europe and the United States has not experienced an extensive crop of road failures following the introduction of modern methods of road design.

Roads sincerely designed and as conscientiously constructed have failed.

There are a number of reasons for such failures not obvious from the soil mechanics approach to design:

1. Those resulting from variations from the general conditions of design as some climatic condition not envisaged under the orthodox design recommendations.

Looked at from a slightly different angle, road soil mechanics is a habit which has been tailored to fit a specific body or set of conditions. When the body differs appreciably from the specific body, the habit no longer comfortably fits and some modification becomes necessary.

Road soil mechanics has been developed for use in connection with the so-called average or leached podsolic soils of the United States and Western Europe, where certain conditions represented by aspects of moisture content variations, the nature of predominant clay minerals, the base exchange capacity and the kind of replaceable bases present, tend to follow a similar pattern and to imply the presence of a constant in the various design correlationships. The importance of these conditions existing as an apriori to the applicability of orthodox design recommendations cannot be ignored. If this pattern does not pertain then additional variables are introduced and design procedure may logically have to be modified.

Designers, working on new ground outside the United States and Western Europe, tend to be unconscious, though not necessarily unaware, of all the various phenomena which may, in general, be active under any given set of conditions; they forget the physical implications of the empirical tests and, consequently, fail to analyze how far these implications satisfy the requirements of the possible variables.

Such phenomena, arising out of climatic conditions differing from those normally associated with "average" soils, are considered responsible for occasional failures in temperate regions and for more frequent failures in the tropics.

2. Those resulting from insufficient attention being given to incremental stability, or micro-design.

Micro-design concerns the relative stability of various combinations of adjoining ions, molecules, ultramicrons, micelles or particles etc. in both two phase and, when water may be present, in three phase systems.

Micro-design possibly plays a more important part in road design than in any other branch of structural design and, unfortunately, is an aspect of design generally scamped by the limitations of normal design procedure. In soil stabilization designs, it is an aspect which increases in importance as the properties of the soil material diverge from those of average soils.

Soil science and the chemical sciences supply the chemical and surface-chemical properties of the ingredients of any soil material and those of any additive. From these data the physical chemist can estimate, by the application of the principles of surface chemistry, such properties as adhesion and cohesion in any mixture of the two.

The properties of the constituents of various mixtures enable the different combinations to be graded, in a predictive way, in their probable relative order of suitability for any road structural unit. In particular, they make it possible to eliminate certain combinations as positively unsuitable. This, in itself, is a great time saver by avoiding unproductive experiments.

Additionally, for any given soil material, surface-chemistry predicts the type of properties required by an additive to ensure permanent stability and will indicate, when its applications are more developed, which one of the various types of additives will give the best and most permanent results.

This aspect of research and design which is in the process of being developed in several parts of the world is leading up to an electro-chemical theory of road design. The necessity for engineering pedology or for pedological engineering, as it will henceforth be referred to, is the necessity for a philosophy, a theory of the strength of road-making materials, and a theory of design which can be applied to any type of material under any climatic condition. A philosophy which will avoid, with reasonable constructional care, the possibility of any failure: a philosophy which will enable a road pavement to be designed and built with the same ease and certitude of economy and safety applicable to any building in structural steel or reinforced concrete.

It would be incorrect to suggest that all these aims have been achieved. It would, however, be fair to claim that during the short period of 10 years, during which the science of engineering pedology has been under development, much of its required theory has been outlined and great inroads have been made towards an understanding of certain aspects of this theory.

The purpose of this paper is to present the author's outlook on the design approach with a view to eliminate the repetition of certain types of road failure believed to be of a climatic origin; the excuse is the unprecedented contribution to the good of mankind expressed by American goodwill in aid to undeveloped countries.

It is impossible to review all aspects of the design approach; some only, representing those of more-common application, will be considered. Further information is obtainable from "The Scientific Basis of Road Design" (3) and, doubtless, from other sections of this symposium.

PEDOLOGICAL ENGINEERING AS APPLIED TO SHALLOW FOUNDATIONS

Philosophy of Pedological Engineering

The philosophy of pedological engineering does not admit that the properties of soils can always be adequately determined by a fixed routine of testing in the same way as is applicable to other engineering materials as concrete or steelwork. On a national basis, where there is not necessarily a great variation in soil material, this may perhaps be possible, but pedology knows no boundaries, its scope is global and it embraces all kinds of soil.

Soils representative of the great soil groups are, to the pedologist, each a different material with differing characteristics which do not necessarily respond to a standardized and simplified system of testing. Like the human body, with which soils have been so frequently likened, the actions and reactions of such soils are not subject to precise prediction from a knowledge of a few of their characteristics but must be treated more as those of individuals, each subject to divergent environmental influences.

In many respects the representatives may be considered as complex chemical substances. The closer one approaches the tropics and the more intense the weathering process, the more-obvious this comparison becomes. It is in such regions that one is most likely to find extremes of character or extensive horizons of secondary silicates, oxides, and hydroxides.

Each of these soils, formed and found under differing climatic and drainage conditions, varies in some way or ways from the others. They vary in their genesis, in their structure, and in their physical and chemical properties where all are in a great measure functions of their environment—their external and internal climate.

It is the summation of these characteristics and properties which is so important in all problems associated with shallow foundations.

Pedological engineering bases its investigations on such data and tends to treat each problem as initially one of soil chemistry and soil physics. It is in this respect that the pedological engineering approach differs so much from that of soil mechanics.

The following examples illustrate what the pedological engineer has in mind by these climatic, structural and chemical factors:

1. Under temperate climates, where rain falls intermittently throughout the year, the moisture content of the undisturbed subsoil does not fluctuate appreciably from the soil's plastic limit. This tends to imply the presence of a constant in the design formulas and the possibility of appreciable differential moisture changes, with their attendant volume changes, is ignored and ignored with impunity until one of those cyclical droughts occurs. There is then trouble for the engineer.

Under other climates, as the hot monsoon climate, this virtual constancy of the subsoil moisture content is seldom met. The permanent ground-water table is very low, though a perched water table may exist, during certain months, at a high level. The result is that the moisture content within an embankment may vary during the year, from some high value to some very low value, comparable more with the shrinkage limit than with the plastic limit. In other words, a moisture-content variable, which cannot be ignored, has now been introduced as an adjunct to design considerations.

(The thought occurs: Is this the real explanation for the more-common occurrence of embankments under tropical than under temperate climates rather than the greater likelihood of flooding?)

2. Though yellow adobe, underlying black adobe soil in California, and yellow kyatti, underlying black cotton soil in Burma are, on the basis of normal testing procedure, very unsuitable soils for a subgrade, yet they can sometimes be successfully used as such or even as a subbase course, provided they are not disturbed and are covered immediately to prevent desiccation, i. e. provided their natural structure and their natural resistance to moisture and volume change are not damaged.

3. The pedological engineer knows that if he has to stabilize a highly active electronegative clay colloid, as organic-matter-free bentonite, it is unwise to try and use an additive which is also electronegative unless, perhaps, the addive is a complex ionizable salt whose anion contains a strong water-repellant dipole. The simplest solution is to use an electro-positive stabilizer. If he must use an electronegative additive, then he must first change the sign of the colloid to be stabilized by the addition of a chemical reagent. Thus an electronegative bentonitic clay soil is not likely to be satisfactorily stabilized by an electronegative bitumen, though it is likely it can be if the sign of the colloid is first changed by the addition of lime—a fact long ago established by Texas Highway Department engineers. Bitumen, on the other hand, would be expected to be a good stabilizer for pure organic-matter-free laterite soils in the same way as it is known to be a good adhesive for crushed limestone.

Some organic· matter, oxides and hydroxides, which may themselves be stabilizers, may form coatings around the clay micelles and change the sign of the colloid. Such adventitious matter must, in consequence, be recognized.

Qualifications of a Pedological Engineer

The most-essential asset of a pedological engineer is to know how to interpret data and especially empirical data; to realize fully the conditions under which the data are applicable; to visualize clearly what this data implies; and to know how to interpret the data, which may not be engineering data, into engineering terms.

As examples may be quoted:

The practical and theoretical conditions under which the C.B.R. and similar data are likely to give most reliable information.

The implication of the P.I. data in physical terms with an understandable meaning; and what is meant by saying the P.I. should not exceed a certain value for some specific purpose as, for example, in base course design.

How to interpret and convert the research of soil scientists or, say, the physical-chemist on adhesives into engineering terms and apply these results to further the advance of soil engineering.

The second essential is to know the life history of the individual soil from its embryological stage to that of maturity; to know its anatomy and its circulatory system; to know the various compounds of which it is composed; and to know the properties of those substances. This is not so great a task as might be imagined once the fundamentals of soil genetics have been assimilated.

The important factors here are soil chemistry (especially surface chemistry), soil structure, what controls the movement of moisture, and the equilibrium value of the moisture moved.

The third essential is to know what he is aiming at, what the requirements for a particular design must be and how to effect those requirements.

For design involving embankments and cuttings these are, in general, to determine or to be able to estimate the maximum and minimum moisture contents for the most-vulnerable layer directly below the subgrade, the shear resistance or bearing value of this layer at its maximum moisture content, and the magnitude of the maximum possible distortion of the subgrade under differential moisture changes.

For the simplest types of base course, featuring only mechanical stabilization, the requirements are the avoidance of any overall volume change and a sufficiency of mechanical strength to be provided by the coarse fraction.

For stabilized units, involving the incorporation of chemical admixtures, the problem is initially one of chemical design to be proven later in volume change, strength, and durability by testing.

In each instance the maximum possible moisture content is important. In consequence, the laws promoting moisture flow and controlling moisture equilibrium are of importance. Chemistry tends to eliminate the trial-and-error methods associated with road soil mechanics and indicates the kind of stabilizer most likely to give the best results.

THE GREAT SOIL GROUPS

Weathering of Soil-Forming Material

Soil is formed by the physical and chemical weathering of such soil-forming parent material as solid rock, boulders, gravel, sand loess, and alluvium.

By physical weathering is meant the disintegrating effects of various physical forces as those produced by frost action, temperature changes, differential swelling (as within parent rock material on the decomposition of some of its mineral constituents), the erosive action of winds, glaciers, rivers and seas, and the volume change effects of roots, lichens, etc.

Chemical weathering, or decomposition, includes the processes of solution, hydration, dehydration, carbonation, decarbonation, oxidation and reduction in which microörganic activity may play an important role. Of these processes, hydrolysis is generally by far the most-important factor.

Chemical weathering is sustained by moisture within the soil and is accelerated by increase in the mean annual temperature, i.e., it can be many times more rapid under a tropical than under a temperate climate for comparable conditions of rainfall.

Moisture is introduced into the soil profile by rain or by flooding, when it tends to percolate and flow down the profile under gravity and capillary forces and, possibly, under film and vapor flow. The more permeable the soil and the greater the rainfall—provided it is not in the form of heavy tropical downpours of high intensity but of short duration when comparatively little will (under reasonably good surface drainage conditions) enter the profile—the deeper will the moisture penetrate.

In addition to this downward movement there is an often equally important upward flow from the permanent water table or from some temporary perched water table.

In both instances there appears to be some interchangeability between the various phases in which the moisture flows.

Moisture, other than that actually in the vapor state, moving in either direction contains chemical substances in solution which may accelerate hydrolysis.

Rainwater contains impurities of an acidic nature dissolved from the atmosphere. On entering the soil profile, its acidity and carbon dioxide content may be increased on passing through any layer or humus, or partly decomposed organic matter. As this moisture, which may now also contain humic acids, flows down the profile of a young soil, it dissolves out some of the more-soluble salts (as the chlorides and sulphates of the alkali metals) to remove them in soil drainage or to deposit them, with any humic residues in suspension, reversibly or irreversibly, in an horizon at the limits of downward movement.

As weathering proceeds and the profile matures, the tendency (especially under a temperate climate) is for these salts to be wholly removed. With the removal of the more-soluble salts from the upper horizons, the neutralizing effects which these salts may have on the acidity of the soil moisture decreases. With the usually accompanying increased acidity of the surface layer containing humic substances, the downward flowing soil moisture tends to become more acid. The less-soluble salts, as the sulphates and carbonates of calcium, are then attacked, mobilized, translocated and deposited at some lower depth to be gradually removed by drainage as the soil ages.

Finally, the clay particles themselves are subjected to hydrolysis or decomposition forces in which the complex silicates tend to be broken down into sesquioxides and silica. Under suitable conditions promoting high acidity, or low pH value, the sesquioxides may also be translocated. If the reaction is alkaline, the silica and not the sesquioxides may be removed.

In contrast to the acidic nature of the downward-moving moisture stream, the upward-moving stream will tend to be of an alkaline reaction, where the pH generally will be the greater the younger the soil. The effect of this flow, which only occurs during fine weather, will be to return the more-soluble and incidentally more-alkaline products of hydrolysis, as sodium and calcium hydroxide, temporarily to a higher zone. The effect of this upward movement of an alkaline solution containing hydroxides is a tendency towards a destruction of any soil structure and a dispersion of the clay fraction. If the solution be weakly acid, instead of alkaline, there will be a tendency to dissolve iron and aluminum salts from the lower layers, to translocate them in an upward direction, and as evaporation proceeds, to deposit them irreversibly in an horizon of accumulation.

In addition to the more-mechanical effects of leaching and deposition discussed, there is another aspect of soil weathering which is of importance.

The soil fraction most susceptible to weathering effects is the clay fraction composed of small particles, or micelles, of a crystalline mineral nature. The surfaces of these particles contain unsatisfied electrical charges which are satisfied by relatively loosely held ions. These ions may be cations or anions, depending on the sign of the charges, but for most clay minerals they are preponderatingly cations. The ions so adsorbed are partly dissociated when the soil is in an aqueous solution and are available for exchange by ions with a greater affinity for the mineral surface than is possessed by such ions.

In general terms, the more-common clay minerals may be considered as hydrated alumino-silicates in which the nucleous, or the mass of the micelle, may be considered to be a rather complex electronegatively charged silicate acting as an anion. Around the surface of this nucleous are attached, by electrolines of force, electropositive cations of the earth bases (as in a young cohesive soil) of sodium, potassium, calcium, and magnesium.

These mineral units, though in equilibrium with their aqueous phase for the prevailing pH conditions, are still chemically active. If the pH changes, then the mineralogical character of the micelle changes. It thus happens that the clay mineral is but seldom one of the known pure clay minerals but is more generally an impure mineral occupying some intermediate position in the scale of pure minerals. Likewise, the surface-adsorbed bases are subject to change.

Of the adsorbed bases, one is usually in predominance and imparts to the clay fraction the properties attributable to a clay fraction composed entirely of clay saturated with that base, though there are examples where the properties of the fraction are controlled by more than one base.

Young clay fractions are generally saturated either with sodium or calcium, depending to some extent on the nature of the parent material.

As the soil solution containing acids and dissolved bases moves through the soil profile, it reacts on the surfaces of the clay material, by base exchange, and the tendency is to change, as weathering proceeds and the pH decreases, the surface adsorbed bases progressively from, say, sodium to calcium to hydrogen (sometimes more accurately given as aluminum) with an accompanying change in physical as well as in chemical properties.

As soon as hydrogen-ions begin to replace cations of the earth bases, the clay is said to be unsaturated. This can only occur after all free calcium carbonate has been leached out of the soil profile.

When all the adsorbed earth bases are replaced by hydrogen, the clay is desaturated and acid hydrolysis is most active.

Some heavily weathered soils, as the lateritic soils of the humid tropics, are amphoteric. Their makeup is such that below a certain pH value, known as the isoelectric point, they act as electropositive colloids and dissociate mainly anions, while for pH values above the isoelectric point they act as electronegative colloids and dissociate mainly cations. This characteristic is of importance when seeking a stabilizer for such soils.

The most-mature soils of the humid tropics contain an horizon of sesquioxide material, or pure laterites. This material, though of a mineral nature, is not colloidal but forms a solid. It acts as if electropositively charged in much the same way as does limestone.

Soil Weathering or Soil-Forming Processes

The overall effect of any particular process will depend upon the relative importance to be given to the two movements of soil solution, i. e., the downward- and the upward-moving solutions.

This, of course, is a function of the climate—the amount of rainfall and the lengths of fine periods together with the temperature. The various processes have been classified as follows:

1. Salinization. Occurs in semiarid and arid regions under a neutral to slightly alkaline weathering process active in soils supporting little vegetation. The downward flow of moisture is of minor importance, and weathering is controlled more by the upward flow carrying sodium salts, calcium, and, sometimes magnesium sulphate and chloride. Evaporation causes an accumulation of these salts at the top of the coarse capillary fringe over the water table. This accumulation layer may be on the surface or fairly deep down the profile, depending on the level of the permanent or the temporary perched water table.

If sodium carbonate or bicarbonate is present, there will be replaceable sodium among the soil bases, and the clay will tend to be dispersed when wet and to be of a single grain type of structure when dry. With appreciable replaceable calcium or magnesium and free salts other than sodium carbonate, the soil may be slightly aggregated and faintly permeable when dry. The little rain which permeates the profile will carry down colloidal clay particles and form a clay-pan horizon under such conditions. Dissolved organic matter residues and calcium salts tend to convert this clay pan to a hardpan by a strong cementation action.

Because of limited rainfall and the impermeable nature of the wet profile, these soils are badly drained. If abundant calcium sulphate and chloride are present, drainage is much better.

2. Alkalization. A weathering process whereby the salts of those soils formed under a process of salinization are removed consequent on improved drainage conditions. As soon as replaceable sodium becomes the predominant replaceable base and the free salts are removed, the clay particles hydrolize with the liberation of sodium hydroxide, which causes the clay to disperse and become impermeable in the presence of water (by swelling effects). There is then a tendency at the beginning of the rains for clay migration and the formation of a clay pan. The degree of per-

meability may vary during the year. Thus certain salts translocated upwards during the dry season may lead to some aggregation.

3. Dealkalization. With a further improvement in the drainage under which alkalization is operative, the replaceable sodium is removed. If free calcium carbonate is available in the profile, the clay becomes saturated with calcium and possibly subject to the weathering process of calcification. If no calcium is present, the soil will be degraded to a hydrogen soil.

4. Gleization. A process productive, in poorly drained areas, of peat-type soils. Such soils are formed from parent materials which are almost impervious or are normally inundated. If alternatively wet and dry, which is probably more common in the tropics, iron compounds are reduced, causing calcium magnesium and manganese compounds to become soluble. A calcium saturation of about 50 percent is necessary for microbes to effect iron reduction.[4] In sandy soils the process is somewhat analogous to podsolization and results in the formation of ground-water podsols in temperate regions, characterized by the presence of a hardpan of organic matter or ironstone above low water level and of ground-water laterites in tropical areas.

5. Laterization. Occurs in the humid tropics under an alkaline changing to acid weathering process or in an area of alternating wet and dry seasons under alternating acid and alkaline leaching conditions. The process infers the decomposition of ferro-alumino-silicate clay minerals and the removal of silica under alkaline weathering. Iron and aluminum remain in the profile, and the accumulation may increase under acid leaching. It thus involves some podzolization. It occurs under forest cover, usually in the hills and more frequently over a suitable rock parent material.

Kaolinitic clays are sometimes found overlying the cemented horizon containing dehydrated sesquioxides of iron and aluminum and generally known to engineers as lateritic soils, the layer above the sesquioxide material may contain montmorillonite.

Under such tropical conditions, the organic matter is rapidly decomposed and plays no part in any cementation process.

6. Calcification. The weathering of a soil profile still containing free calcium carbonate. It may occur in both temperate and tropical climates of moderate effective rainfall. The free calcium carbonate maintains the colloidal clay in a base-saturated condition and imparts an aggregated granular structure to the soil, hindering the eluviation of clay material and the formation of any pan. Weathering occurs in a slightly alkaline to slightly acid medium and causes the translocation of carbonates to a lower level. The humus is mild and easily decomposed.

Soils formed under such a process are classified as pedocals, i. e., they contain an horizon of carbonates. Representatives are differentiated on the basis of color, organic matter content and the extent and amount of carbonate accumulation (i. e., to a large extent on the effective rainfall).

7. Podsolization. After the removal of free carbonates, the clay complex tends to become unsaturated, and the soil solution increases in acidity. Podsolization occurs in humid tropical or temperate areas and takes place in an acid medium. An acid organic-matter surface layer is formed. The clay fraction is desaturated, dispersed, and hydrolized, resulting in the translocation of clay material, ferro-aluminum, and iron compounds to be deposited at lower and different levels.

In the absence of calcium, the surface organic matter does not support a high bacterial population and its decomposition is thus very slow. Part of the decomposed organic matter dissolves in a favorable medium and combines with iron to form ferrous humates. As such, it is mobile and easily translocated down the profile to be deposited at the limit of translocation.

Soils containing an accumulation layer of iron or aluminum compounds are known as pedolfers.

Azonal Soils

Physical weathering results in the production of very-young soils containing very little clay material and, more generally, consisting of relatively coarse particles. It is responsible for the formation of the azonal (or very young) soils possessing no well-developed profiles and included within the great soil groups under lithosols (rock fragments), sands (dry) and alluvial (mud) soils. Their characteristics are similar to those of their parent materials.

As the size of these particles decreases,

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[4] Information supplied by Hans F. Winterkorn (letter)

the surface area per unit of mass increases, and they become more susceptible to chemical weathering. Continued weathering, now of a more-selective nature, gradually reduces the size of the smaller particles. With the accompanying rapid increase in surface area, chemical weathering is greatly accelerated and the tendency is for the finest particles to be converted into material of clay size, though not necessarily yet possessing the properties of colloidal clay. In due course the clay fraction emerges as a clay mineral, generally with the properties of a colloidal clay.

The effects of rain or floodwater on young soils will depend upon whether drainage is relatively free or impeded.

If the drainage is impeded, or initially free but changing to imperfect, the effect is to translocate the finer particles from the surface layer, or Horizon A, to form an accumulation layer, or Horizon B, below. The density depth, and thickness of this accumulation layer will depend upon the intensity and amount of rainfall, or on the frequency and amount of floodwater, the permeability of the parent material, and the stage in weathering reached. Intrazonal soils, or those soils in which normal weathering under the appropriate processes of chemical weathering is hindered by poor drainage or by the absence of free drainage or those soils whose properties are still largely controlled by those of the parent material, are now in the process of formation. They are generally to be found associated with two or more zonal, or mature, soils formed under reasonably good drainage conditions.

Intrazonal Soils

Most of the soils of stunted growth are characterized by an accumulation layer forming a hard pan or clay pan which may be cemented by oxides of iron or silica, by calcium carbonate, or by organic matter. An important factor in such soils is that any clay pan formed may not be sufficiently weathered to exhibit appreciably the properties of a colloidal clay. This particularly applies to the great soil group of planosole.

From here on the weathering of most soil-forming material is mainly chemical and becomes very much a function of the climate (temperature, rainfall, and humidity), which controls the type of vegetation and organic matter produced, and of drainage conditions.

As long periods of geological time are involved in weathering, it must be realized that changes in topography may occur and that the climatic and drainage conditions under which a soil is initially formed may change with a change in the active weathering process. This results in the ultimate production of a matured soil having characteristics differing from those to be expected from a consideration of the forces originally operative.

Soil profiles, i. e., the soil down to the unweathered parent material, may or may not include a water table. If they do, then in humid regions the controlling movement of the weathering solution is in a downward direction and of an acidic nature. Under such conditions a range of intrazonal soils, included in the great soil groups under the names of "alpine meadow soils" and "meadow soils" (Wiesenböden) are formed under mixed gleization and calcification processes; bog soils are formed under gleization planosols formed under mixed podsolization or laterization and gleization processes; ground-water podsols are formed under podsolization; and ground-water laterites formed under podsolization and laterization processes are formed where the particular soil formed will depend upon the temperature, rainfall, degree of imperfection of drainage, and profile age.

Under semiarid and arid climates a permanent ground-water table has less opportunity of forming within the depth of the soil profile than under humid conditions. The tendency is then for the downward and upward (from a temporary perched water-table) weathering solutions to be of equal importance or even for the upward leaching to be more important than the downward weathering. Though weathering proceeds, it now tends to do so in an alkaline medium and to maintain the salts within the profile. Under such conditions, the intrazonal soils include the great soil groups of solonchaks or saline soils (salinization), solonetz or alkaline soils (alkalization) and soloths or solote or degraded alkaline soils (dealkalization).

Among the intrazonal soils are also included two very young soils formed under good drainage conditions whose parent material contains sufficient free calcium to enable the clay fraction to remain base saturated under the prevailing weathering system. They are the brown forest soils, (Braunerde), formed under forest cover of the temperate climates and the rendzinas formed over calcareous material under grass cover in cool to hot, humid to semi-arid climates. In neither instance is there any appreciable accumulation of clay material within the profile.

Zonal Soils

Under original or subsequent conditions of good drainage, the appropriate soil-forming processes may operate unhindered. The particular process or, sometimes, processes, active in any specific area will depend upon climatic conditions and the age of the soil. In the latter instance one or more processes may have been completed to be followed by the process now observable. These soils have little in common with the parent material from which they are formed.

Consideration of the great soils groups, on the basis or those formed under the

TABLE 1

Rainfall	Temperature	Great Soil Group
	Calcification	
Arid	Temperate to cool	Desert Soils
Arid	Temperate to cool	Sierozems
Arid	Warm-temperate to hot	Red Desert Soils
Arid to semi-arid	Temperate to cool	Brown Soils
Arid to semi-arid	Temperate to hot	Reddish-brown Soils
Semi-arid	Temperate to cool	Chestnut Soils
Semi-arid	Warm-temperate	Reddish-chestnut Soils
Subhumid	Temperate to cool	Chernozem Soils
	Calcification with weak Podsolization	
Humid	Temperate to cool	Prairie Soils
Humid to Subhumid	Warm-temperate	Reddish Prairie Soils
	Calcification followed by Podsolization	
Sub-humid to humid	Temperate and cool	Degraded Chernozems
	Weak Podsolization and some Calcification	
Sub-humid to semiarid	Temperate to warm	Shantung Brown Soils
	Podsolization	
Humid	Cool-temperate	Podsols
Humid	Cool-temperate	Brown Podsolic Soils
Humid	Temperate	Gray-Brown Podsolic Soils
	Podsolization with some Laterization	
Humid	Warm-temperate to tropical	Yellow Podsolic Soils
	Polsolization and Laterization	
Humid	Warm-temperate to tropical	Red Podsolic Soils
Humid (with wet and dry seasons)	Tropical	Yellowish-brown, Reddish-brown Lateritic Soils
Humid (with wet and dry seasons)	Tropical	Laterite

Where:

Arid	infers only sufficient rainfall to support sparce vegetation.
Humid	infers rainfall sufficient to support forest vegetation
Semiarid	infers an intermediate condition of rainfall between arid and sub-humid supporting only scattered short grass or shrubs
Semihumid	infers a rainfall intermediate between semi-arid and humid supporting a moderate to dense growth of short to tall grasses
Wet	Infers a rainfall sufficient to support rain-forest vegetation.

various soil forming processes, gives the information in Table 1.

An important factor in design is to recognize, and this is possible from a knowledge of the great soil group to which the soil in question belongs, whether or not the subgrade layer or embankment or cutting is likely to become desiccated.

THE SOIL-GROUP MINERALS

The colloidal clay silicate minerals may be divided within three groups depending upon their activity or ability to adsorb soil bases including moisture:

Clay Mineral Group	Base Exchange Capacity
Kaolin-like	3 - 15 m. e.
Illite-like	20 - 40 m. e.
Montmorillonite-like	60 - 100 m. e.

The secondary clay minerals produced in any soil profile depend upon the nature of the weathering process and the pH of the soil solution.

Minerals of the kaolin group are produced under intensive acid hydrolysis and may characterize such soils as the podsols, podsolic, laterite, and lateritic soils. Those of the montmorillonite group characterize such young soils formed under an alkaline weathering system, active in warm to hot climates, as the chestnut, chernozem, degraded chernozem, and solonetz soils.

Illite is more usually found in the soils of intermediate weathering, especially under colder climates. There are beliefs that its presence may be partly due to its existance in the parent material and that under some unknown conditions it may change to montmorillonite (and vice versa). Illite may be present in lateritic, podsol and podsolic soils as an alternative to montmorillonite. Both illite and montmorillonite may be present together in chernozem, gray semi-desert and chestnut soils. Montmorillonite may be found together with kaolin in some laterite and lateritic soils.

The clay minerals can be identified by methods involving X-ray diffraction, dehydration curves, and thermal analysis. These methods, are sometimes used by soil-engineering researchers, but more generally it is considered that the time necessary for their performance is unjustified. Short cuts are therefore adopted.

With the possible exception of those instances when it is possible to estimate the clay mineral present on the basis of data available from, say, agricultural-research stations, this short-cut approach is to be deplored, for it can lead to false conclusions being drawn from the short-cut data.

If only one clay mineral is present, it can be given a group-name from its base-exchange capacity. In general, however, there is more than one soil mineral present, and properties of the soils may be so different, as a mixture of kaolin and montmorillonite, that it is impossible to determine presence of the soils from such data unless, of course, one is present only in very-small quantities.

The method used more generally is the determination of silica-sesquioxide ratio. This is not so much for recognizing the clay minerals present as for giving an idea of the overall effect of those clay minerals. Thus, a value of less than 1.33 indicates a well-formed laterite and the presence of a clay mineral of the kaolin group; whereas a high value in excess of three will indicate the presence of a mineral of the montmorillonite group and that the clay will have the properties of a montmorillonitic clay. It does not necessarily preclude the presence of other minerals.

In practice, it is not necessary that there be more montmorillonite than kaolinite, for example, to ensure that the clay has the overall properties of a montmorillonitic clay. Very-small proportions are generally sufficient for this.

The base-exchange capacity of a pure clay material containing only one clay mineral is a measure of the electrostatic charges carried by the colloid at the particular pH, usually 7, at which the base-exchange capacity is determined.

The more active the clay, the higher is the base-exchange capacity and the greater the charge held. It would appear that the highly active clays, which are the most-difficult to stabilize, should be fairly easy to stabilize provided a suitable stabilizer is added in sufficient quantities to the soil to neutralize these charges and turn the mixture into a hydrophobic material.

It must be remembered, however, that the amount of stabilizer added is more normally a function of the surface of soil aggregates than of the surface of the clay particles and that design provides for a controlled amount of moisture absorption for supplying a reserve of cohesion to be drawn upon during dry spells. The number of charges, or the base exchange capacity varies with the pH of the solution in contact with the clay particles and increases with increase in the pH.

It would appear possible that a stabilized mixture found satisfactory under laboratory test might not be found satisfactory in the field. This might be expected to apply most specifically to highly active soils in which detrimental replaceable sodium is most likely to be encountered.

Some lateritic-type soils contain a highly active clay mineral coated with ferric oxide. Such soils are likely to be amphoteric. It is important to select a stabilizer for such soils which will be effective under the pH conditions likely to pertain in the completed pavement or, perhaps, to add a catalyst which will ensure the pH remaining within an optimum range.

ORGANIC MATTER AND HUMUS

Organic matter, or the residue of living matter, consists of carbohydrates (cellulose, hemicellulose, and lignin); proteins, fats, resins, waxes, ash, etc. Cellulose, hemicellulose, and the proteins are readily decomposed, whereas lignin, fats, resins, waxes, and ash are more resistant to decomposition.

Under optimum conditions these residues will rapidly mineralize into water, carbon dioxide, and mineral salts. Under other than ideal conditions, however, an intermediate product is formed known as humus, which may be very resistant to further decomposition.

Humus is a mixture of complex materials, some of which act as natural soil stabilizers. Winterkorn (4) has fully appreciated this possibility and has made use of it in stabilization proposals.

Organic matter may be destroyed by oxidation or other chemical processes. Destruction by oxidation is brought about mainly by the action of such microorganisms as bacteria and fungi.

Bacteria flourish best in a moist but well-aerated soil containing free calcium, which is one of their principle sources of energy and which implies a granular soil structure promoting good aeration. Since oxidation increases with increase in temperature, it is likely to be more rapid in tropical than temperate climates. Excess moisture prevents good aeration, supports a less-active type of bacteria, and delays decomposition.

Under ideal conditions the decomposition of carbohydrates and proteins is rapid, and little humus is formed. Acids resulting from the percolation of water through such decomposing organic matter are, thus, in the main, restricted to carbonic acid. Under other than ideal conditions, decomposition is slower, and the acids of the percolating soil solution may also include the very-strong humic acid.

Under anaerobic conditions chemical changes occur in the residues of aquatic vegetation, irrespective of the presence or absence of free calcium, to produce the often-acid humus, or the peat, of bog soils. Such humus may contain little or much cellulose. The rate of humus formation increases with temperature.

Under humid conditions favoring natural coniferous forest, i. e., for soils deficient in soil bases, there is a collection of considerable organic matter on the soil surface, all of it poor in bases, and some accumulation down the soil profile from decaying roots. Decomposition occurs in an acid medium and is promoted by fungi rather than by bacteria. Mineralization is rapid, and little humus is formed. What humus is formed is frequently of the nature of semidecomposed raw humus dispersed throughout the profile. It differs in properties from that formed under anaerobic conditions.

Under a semiarid climate promoting a grassland-type of vegetation, lower moisture contents increase aeration. Decomposition is thus potentially more rapid. This is so under semiarid forest cover, and even less humus is produced than for forests in the more-humid regions. With decreasing rainfall, steppe- and prairie-type vegetation replace forest land. Decomposition, still potentially rapid, is halted by cold and frosty or hot and dry seasons and tends to be limited to that of the celluloses. These interruptions allow humus to form and to accumulate. The high base status of such soils enables the acid nature of the humus to be neutralized by base-exchange phenomena. Such soils as the chernozems, black earths, rendzinas and prairie soils are typical. The black color of such soils is not a reliable indication of the amount of humus contained in them, however.

Under hot desert conditions supporting shrub vegetation, the small amount of organic matter produced is rapidly decomposed. Such humus as is produced is dispersed down the profile by virtue of the long roots characterizing such conditions.

The heavier soils of the humid tropics are not so low in organic matter as might be imagined. The humus produced apparently combines with iron to form humates and is preserved as such on dehydration. Lighter soils, however, are low in organic matter.

The constitution of humus is still somewhat of a surmise. It contains a number of acids, the most important of which is generally referred to as humic acid, and various partly decomposed compounds of organic matter.

Organic matter is believed to be reduced to humus by three processes: the oxidation of lignin, the fixation of ammonia, and the combination of the resulting colloidal complex, lignin-humus or humic acid, with protein and hemicellulose colloids by adsorption.

The oxidation of lignin requires air and takes place more rapidly in an alkaline medium. During oxidation the number of carboxyl groups, causing acidity, is increased. This results in an increased base-exchange capacity. The fixation of nitrogen from ammonia imparts a resistance to further decomposition.

Hemicellulose and proteins are hydrolysable; but on combining with humic acid, they appear to be held tightly to the surface, as if they were exchangeable bases, and in such a way that they are orientated to make the hydrolysable groups inaccessible to organic catalysts. Some polymerisation may occur. It will be noted that such complex compounds are not only ionisable but contain strong carboxyl dipoles, whose effects will become more obvious with decreasing base-exchange capacity.

The properties of humus will thus depend upon the nature and ratios of its components and will vary with the soil with which it is associated. Thus, lignin-humus is believed to predominate in chestnut soils and proteins in gray soils. Hemicellulose is found in the former but not in the latter. The base-exchange capacity varies with the pH at which the humus is formed. The humus of chernozems, formed under neutral to slightly acid weathering, has a higher base exchange capacity than that of the podsols formed under acid conditions. For the same reason the humus of the chernozems would be expected to adsorb more moisture than that of the podsols.

Humus is electronegative and strongly acid when its bases are removed. Iron and aluminum colloids may impart to it amphoteric properties if sufficient basic groups be present.

Humus is only slightly soluble in water, but it may adsorb sufficient water to render it mobile. Translocation is thus, in the main, mechanical or in suspension. It is, however, very mobile, if not soluble, in an alkaline solution and is found fairly uniformly distributed to considerable depths in some tropical soils, where it appears to act as a protecting sheath around soil particles aggregates. It becomes very water-resistant once dehydrated.

It is adsorbed by clay particles, probably through the medium of calcium cations or by base exchange and dipole orientation.

Humus has an important modifying effect on soil structure by promoting, in the presence of replaceable calcium, a granular structure. It also effects the consistency and moisture relationships by increasing the liquid limit and plastic limit but not materially affecting the plastic index.

Cellulose and hemicellulose, found in soils, as the podsolic soils, formed under low bacterial activity, are detrimental to bitumen, lime, cement, and resin stabilization by virtue of promoting bacterial activity, which extends to the decomposition of the stabilizers themselves.

SOIL STRUCTURE

Soil structure includes macrostructure and microstructure.

The macrostructure of a cohesive soil refers to the natural elements into which the virgin soil breaks down into on drying, i. e., periodic desiccation is essential if the macrostructure is to be well developed.

On wetting from the dry condition, soil cracks or the air spaces around the boundaries of these elements tend to disappear. Part of the space is filled by extraneous material and slaked soil, and the remainder as the elements adsorb moisture and swell. Nevertheless, it is doubtful whether these boundary faces do, in general, disappear to permit the soil structure to reform a continuous whole. It seems more likely that the size and shape of the elements is maintained and that subsequent drying causes the same boundary surfaces to reappear. There are exceptions, of course, as with the less-cohesive soils and some slaking alkali clay soils.

The presence of these air spaces, or cracks, is an important factor. They permit percolation during the early stages of rewetting and offer the possibility of organic matter being deposited on the surfaces of the elements.

The shape and size of these elements is a function of the soil group to which the soil belongs and of the horizon from which the element be taken. Both are believed to be dependent on the soil's chemistry, though the shape is sometimes modified by shear stresses as in glei structure. The shape is also thought to be, in some way, connected with the clay particle shape and with differences between the coefficients of horizontal and vertical swelling (5).

Weathering processes conductive of natural consolidation and the nature of the soil's microstructure, i.e., volume changes and shear resistance, are presumably responsible for the size of the elements of dispersed clay soils.

Under Baver's classification system (6) for soil macrostructure, there are three main types: cube-like (caused by granulation), prism-like and plate-like (caused by fragmentation), and single grain (caused by deflocculation). Structures within, or between which, he classifies, as examples, chernozems; laterites and humid prairie soils; brown forest and red and yellow forest soils; desert soils; and gray forest soils.

The difference between the coefficients of horizontal and vertical shrinkage would appear to be the basis of the Nikiforoff (7) system of: (1) plate-like, in which the natural cracks are mainly horizontal; (2) prismatic, in which vertical cracks predominate, the surfaces are smooth and the edges are sharp; (3) blocky, in which horizontal and vertical cracks are equally well developed, the surfaces are smooth and the edges are sharp; and (4) granular, as blocky but surfaces are more rounded and not so smooth and edges are not sharp.

The microstructure, or the structural skeleton of a cohesive soil material, is the result of weathering processes productive of leaching (causing eluviation of clay particles and cementing colloids), flocculation and deflocculation (causing clay particles to be deposited in a random way or in an orientated way between coarser particles), natural consolidations as by drying and wetting cycles (where by its colloidal clay particles may be brought, if slaking occurs, into closer contact and possibly within the range of intermolecular forces), cementation (on dehydration of natural cements) which result in the creation of a resistance to water-deterioration.

On the above basis, Kubiena (8) has classified soil microstructure as shown in Table 2.

Natural soil cements are believed to include orientated clay particles; oxides of iron and silica; organic matter residues as fats, waxes, resins, and certain other humic compounds which are considered to become attached to the surfaces of clay particles largely through the process of base- and dipole-exchange-adsorption effects (some authorities consider the attachment to be through the medium of calcium and aluminum ions); certain gum products of microbial metabolism; certain microörganisms; and certain grass roots.

The nature of the soil micro-structure will thus tend to vary with the soil group and the particle grading.

Microstructure tends to increase the soil's resistance to shear deformation and to make the soil material water resistant. Some soil structures are quickly reestablished after disturbance; some are never reestablished; while others may occupy some intermediate condition.

Some structures lead to fine-aggregate soils, which are normally permeable; some to coarse-aggregate soils, which may or may not be permeable; while others lead to the formation of dense, impermeable soils. The various soils represent gradations between the conditions of permeability and impermeability, between aggregation and dispersion in their natural densification, in their amount of cementation and in their resistance to water deterioration.

So long as wetting is slow, as it normally is under a road pavement, soil macrostructure does not usually disintegrate, though it may develop the already incipient internal planes of weakness when cementing agents are weaker than the disintegrating forces. Exceptions may occur in soils rich in replaceable sodium and, possibly, magnesium, when the clay particles may disperse and in doing so repel each other with a force greater than the resistance of any cementing agents. The displacement of air from a dry soil by water at a faster rate than it can escape increases slaking effects. Clays low in organic matter tend to form hard clods, especially when exchangeable sodium is high and the free salt content is low, whereas those rich in organic matter are aggregated into crumbs possessing pronounced microstructure.

When a soil possessing appreciable microstructure is incorporated into an embankment and compacted, the small aggregates may still retain much of their microstructure and their water repelling properties.

SOIL STRUCTURE AND VOLUME CHANGE

Volume or density changes are affected by soil structure, as evidenced by both density and microstructure and by desiccation.

Powdered and Unrestrained Cohesive Soils

As long as the air content is small, it would appear that the normally determined shrinkage curve (volume in cc. per 100 gms. of dry soil versus moisture content in percent of dry weight) for a saturated sample closely follows and parallells the basic shrinkage curve for a pulverized saturated sample containing no air.

For such conditions, the rewetting curve will retrace the drying curve, provided the soil is not dried to a moisture content below the plastic limit, i. e., provided air does not enter during the drying process.

Should the soil dry out to a moisture content below the plastic limit, so that some air enters and desiccation occurs, then the rewetting curve moves away from the basic curve and the engineering shrinkage limit decreases. The soil opens out and the permanent structure is in the process of being formed.

When the air content increases to a value above a critical value of about 4 percent (at the maximum density and optimum moisture content), the form of the drying and wetting curves becomes pronouncedly sigmoid and there is hysteresis. An equilibrium condition is eventually reached, which is apparently determined by the activity of the clay fraction and the neutralization effects of the various forms of molecular forces, cementation, and other structure-forming processes. Some of these, as the presence of organic matter and oxides, may be already inherrent in the mixture. The minimum density is thus stabilized, provided conditions favorable to slaking are absent.

For densities within the range of field relative compactions and for a soil material with a well-defined upper limit to moisture adsorption (no slaking), it would appear that the overall volume change per unit weight of

TABLE 2

Coating of mineral grains	Arrangement of fabric	Occurrence
Grains not coated	Grains embedded loosely in a dense ground mass	Lateritic Soils
	Grains united by intergranular braces	Chernozems, Brown Earths, Lateritic Soils
	Intergranular spaces containing loose deposits of flocculent material	Sandy Prairie Soils
Grains coated	Grains cemented in a dense ground mass	Desert Crusts Podsol B-horizons
	Grains united by intergranular braces	B-horizons of Podsolized Brown Forest Soils
	Inter-granular spaces empty	B-horizons of iron and humus Podsols.

a cohesive, but not too-highly plastic, soil between zero and saturation moisture contents, is the same irrespective of the relative compaction.

By contrast, the density changes between these maximum and minimum moisture-content limits increases with increase in density, while the moisture-content range increases to some maximum value and there is a decrease in the value of the engineering shrinkage limit with decreased density.

For this constancy of volume change to occur, air must be free to escape during wetting. Air cannot always freely escape from some highly plastic soils, however. The volume change per unit weight may then increase with decreased density. Such volume change curves tend to be irregular and may include a coarse capillary range, i. e., a change in moisture content may occur without a change in volume.

Density and desiccation would thus appear to affect the shape of the volume-change curves, while density, desiccation and microstructure affect the value of the maximum moisture content and the value of the site shrinkage limit.

Restraint and static loading tend to reduce volume changes, while dynamic loading tends to increase the density by forcing out the contained air.

Undisturbed Cohesive Soils

If the soil is always completely or nearly completely saturated, volume-change curves are apparently of the same type as those for powdered samples containing little or no air. On drying and wetting over a range above the effective plastic limit, the curve closely follows and parallells the zero-air-void curve for a powdered sample. The difference between the two conditions of undisturbed and disturbed is that the structure of an undisturbed sample has built up a resistance to volume change, reducing the maximum moisture content, and the site shrinkage limit is in excess of the test value.

Under conditions of desiccation, the curves would seem, according to the author's views, to follow either the basic or the sigmoid pattern, according to the properties and the amount of the clay fraction, while the maximum and minimum dry densities would appear to be determined by structural forming processes and, in particular, may be influenced by the value of the test shrinkage limit.

As long as the natural soil structure is not allowed to deteriorate during the construction of a road, it does not appear that applied loading and restraint exert any appreciable influence over the unloaded natural volume-change cycle.

The internal structure of a road pavement may be considered, as regards volume changes, to be rigid for base courses of the macadam type, semirigid for base courses of the stabilized-soil type, while that of compacted-embankment soils may be considered as flexible and generally subject to measurable volume change.

STABILITY REQUIREMENTS

The stability of any road structural unit will depend upon: (1) overall properties of the unit as controlled by the internal climate at the time of study; (2) properties of each of the various individual constituents making up the structural unit; and (3) variations in the internal climate which, apart from having a direct influence on the overall properties, may be accompanied by variations in the properties of one or more of the constituents and thus exert a further indirect influence on the overall properties.

The field of soil mechanics embraces the study of stability in relation to the overall properties for some assumed condition of internal climate considered to represent the worst possible condition. This approach, on a global basis, may not: (1) be sufficiently comprehensive as there may in effect be more than one worst condition. (2) give, for some soils, the worst condition which it is assumed to represent. (3) give, for other soils, an economic solution, i. e., the worst site condition may not be so severe as it is assumed to be.

The field of pedological engineering embraces a study of the natural laws operative in any structural unit, the requirements for satisfactory design, how far these requirements are met by orthodox design (i. e., an interpretation of the assumptions inherent in orthodox design), the properties of the various constituents of a structural unit, and the effect of variations in climate on the properties of these constituents.

The overall properties of any structural unit of given composition will vary with moisture content and, sometimes, with internal temperature. External temperature may have a direct effect on moisture content conditions.

Orthodox design is frequently based on a condition of minimum shear resistance as represented by some laboratory condition of saturation as, for example, attained during the expansion test used in conjunction with the C. B. R. determination.

Such an approach assumes four stipulations which may or may not be fulfilled: (1) minimum shear resistance is the only criterion; (2) minimum shear resistance measured is not greater than the minimum shear resistance existing in the field; (3) minimum shear resistance is not appreciably less than the minimum shear resistance existing in the field; and (4) there is a factor of safety involved in any correlationship, between test data and satisfactory performance, used.

SWELLING PRESSURE MAY HAVE TO BE CONSIDERED

Pedological studies have shown that a condition of minimum shear resistance is not always the only design criterion. The reverse condition, implying the possibility of differential raising of the structure instead of the more-common differential sinking of a structure may be of even greater importance (9).

Whether design should be based on the former or on the latter or on both possibilities depends on the nature of the materials comprising the unit structure and on the external climate. In other words, in addition to the natural laws controlling moisture changes, the natural laws governing changes in internal-pressure conditions occuring during moisture-content changes is, or may be, a subject for consideration. The phenomenon to be measured and controlled is the effect of a change in internal-pressure conditions resulting in swelling pressure.

The seat of swelling is, more generally, in any clay material present and swelling pressure is liberated when dry clay takes up moisture. The maximum value of the swelling pressure will vary with the nature of the clay mineral and of the adsorbed bases. It will increase with the activity of the clay mineral. In general terms, it will tend to increase as the external climate changes from temperate to tropical. The amount of swelling pressure released, for any given clay material will increase with decreased initial moisture content, i. e. with desiccation, and appears to approach a maximum for the shrinkage limit as determined from the site shrinkage curve, which depends on the structural condition of the material. It is a minimum or zero when swelling ceases, which is at some moisture content value approaching the field capacity determined for the particular structural condition.

In practice, however, the maximum value appears to be dependent more on the amount of desiccation and possibly on the effects of the replaceable bases than on the nature of the clay mineral (10).

Under temperate climates, where it has been shown by experience that the moisture content under a pavement seldom falls much below the plastic limit, except possibly during droughts, the pressure effect is usually unimportant, and the minimum shear resistance is the design criterion. Under tropical conditions, swelling pressure may be the controlling design factor for some desiccated A-6 soils, and both factors may have to receive consideration in any design involving the presence of certain A-7 soils.

The implications of minimum moisture-content conditions may be as important as those of maximum moisture content.

MOISTURE MOVEMENTS AND MAXIMUM MOISTURE CONTENT UNDER A PAVEMENT

An investigation into the causes of moisture movement was one of the first tasks of the pedological engineer. It was stimulated by the report in 1935 of the Imperial Valley road failure in California, (11), when free water was found under the pavement although the yearly rainfall did not exceed 1 or 2 inches; and by some moisture content data in 1936 from Burma (9), showing moisture increases occurring towards the end of the long, hot, dry season preceeding the monsoon rains.

It was generally suspected that moisture under a pavement might reach some equilibrium value approximating the plastic limit in temperate climates but that under tropical climates the maximum value might reach considerably greater values. There was, however, no way of estimating the magnitude of the equilibrium value.

The study of the natural laws under which moisture moves in soils was inaugerated almost simultaneously in England and the United States. In the latter country the date is fixed by the setting up of the Highway Research Board Subcommittee on "Nongravitational Flow of Water Through Soils" in 1944. Its object was the determination of the laws controlling moisture movement and the equilibrium value of the moisture content under a pavement.

In the United States the problem was undertaken by Winterkorn and Eyring in 1945 (12) and by Winterkorn in 1947 (13). Working on the basis that moisture equilibrium was determined by energy considerations, they considered that, of the various ways in which moisture could flow, i.e., by surface-tension differences, vapor diffusion, vapor convection, and by film flow, the most likely and most generally applicable way was in the film phase, though it was possible that some moisture also moved in the capillary and vapor states. Of the energies which could be responsible for moisture movement, i.e., surface energy caused by the presence around the colloidal clay particles of replaceable ions, capillary energy arising out of surface-tension forces, kinetic energy or vapor-pressure differences, and thermal energy caused by a temperature gradient. Winterkorn considered that the most important cause of moisture flow, when applicable, was a thermal gradient which resulted in film flow due to change in the water affinity of particle surfaces with change in temperature. Winterkorn thus supported the Bouyoucos concept and developed this concept into his theory of the thermoösmotic flow of moisture in soils (14).

Though many had denied that an appreciable movement of moisture could occur in soils under a temperature gradient, as had been suggested by Lebedev, Chaptal, and Bouyoucos, this natural law was clearly and cleanly confirmed, by the Road Research Laboratory in England (15), as an important factor in determining moisture movements in desiccated soils.[5] The results agree with those of Bouyoucos, showing that there was an optimum moisture content approximately equal to the plastic limit at which a maximum amount of moisture is moved, but the Road Research Laboratory differed from Bouyoucos and Winterkorn in believing that the moisture movement occurred in the vapor phase and not in the film phase.

The mode of attack then changed. The emphasis was now more on the magnitude of the equilibrium moisture content than on the means by which such a value was reached. The United States, as represented by Spangler in 1945 (16) and England, as represented by the Road Research Laboratory (15), turned to the development of the more thermodynamic aspect as originally developed mathematically in California by Edlefsen and Anderson in 1943 (17).

This approach has been so developed in England that it is believed that it is now possible to estimate the moisture equilibrium values in saturated and nonsaturated soils under any condition of loading. The approach used, based on suction-pressure differences, appears to apply more to a study of the natural laws controlling moisture-equilibrium values than to the natural laws promoting moisture movement. As regards the latter, the mechanism is presumably considered to be some sort of vapor-pressure gradient.

It is apparently not believed that thermoösmosis plays an important part in moisture flow in England, as most of the studies made in that country seem to have been based on conditions of constant temperature. This may well be so in England where conditions (relatively high water table, relatively high minimum soil-moisture contents, and relatively low temperatures) tend to be more conducive of capillary than film or vapor flow; but it is not likely to be so in tropical regions where conditions (low water tables, relatively low minimum soil-moisture contents, and relatively high temperatures) tend to favor movement in the vapor or film states.

Though the ultimate soil-moisture-content distribution does not appear to be dependent on the state in which moisture flows, the actual state in which it moves in desiccated soils, i.e., in the vapor or film phase, is nevertheless of importance to road engineers. If movement occurs in the film phase, then the total amount of water moved within a given time is proportional to the particle surface area per unit volume and hence increases with density (12); whereas if it occurs in the vapor phase, vapor movement occurs less readily in well-compacted soils than in loose soils of the same moisture content (15).

From a purely soil-science point of

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[5] See also "Climate and Highways," H. F. Winterkorn, Trans A. Geophys U, June 1, 1944

view, it would appear reasonably certain that, in the instance of a powdered and saturated sample being compacted under natural drying forces, the sample will, on slowly rewetting from a moisture content not less than the shrinkage limit, take up moisture by vapor flow to a moisture content just below the plastic limit and thereafter by film flow till the sample be wet enough to enable capillary flow to occur. But in the instance of artificially compacted and undisturbed cohesive soils, all containing appreciable imprisoned air, it is believed that the film-flow stage may extend over a considerable range and persist up to the last few percentages of saturation for some soils.

From the point of view of the stability of the soil material, Winterkorn (18) has pointed out that the rate of soil structural deterioration is really more important than the rate of water penetration.

For predictory purposes, a fraction of the field moisture equivalent (which varies from about 0.7 for compacted A-6 sodium clays to about 0.9 for A-7 calcium clays) appears to give a good estimate of the maximum moisture content of a cohesive subgrade soil likely to occur under the center of a covered pavement.

FACTOR OF SAFETY IN DESIGN

It is known that standard testing procedure may not give the minimum C.B.R. or the maximum moisture intake of some A-7 soils (as for some calcium black-cotton soils) because of the limited time factor involved in the soaking tests; and it is known that some A-6 soils (as some sodium black-cotton soils) will never be in the disadvantageous condition implied by the standard method used for determining the soaked C.B.R.

Any factor of safety inherent in the C.B.R. method of design obviously depends on whether the minimum site C.B.R. is greater or less than the test value.

Pedological thought suggests that, under the meticulous care and conditions of design and construction prevailing in California at the time this design method was developed, there was an implied factor of safety which may be absent under more-modern methods of design. These latter would seem to permit the use of poor A-7 soils in embankments and under conditions which do not presuppose any control over embankment height.

It has been suggested (3) that the minimum value of the factor of safety may, with reservations, be indicated by

$$\frac{\text{L. L.} - \text{P. L.}}{\text{F. M. E.} - \text{P. L.}} \quad \text{or} \quad \frac{\text{L. L.} - \text{S. L.}}{\text{F. M. E.} - \text{S. L.}}$$

according to climatic conditions. Under conditions of good design, an adequate height of subgrade above ground-water table and as imprevious surfacing, the factor of safety would be expected to increase above the values quoted.

PROPERTIES OF THE STRUCTURAL CONSTITUENTS

The most-active constituents of a road structural unit are the clay and organic-matter contents. Other constituents are active but to a lesser degree, and their effects are not so obvious. It is only now that the properties of these others are being given more considerations.

Colloidal clay is composed of secondary clay minerals, with a crystalline structure, formed from the parent material during its weathering and the process of soil formation. Because of boundary conditions, the nature of and the way in which ions are arranged, a number of valences or electrolines of force associated with the ions situated in external and internal (cleavage-plane) surfaces of the minerals are unsatisfied by ions forming part of the crystalline material and are free to adsorb adventitious ions.

There are, in general terms, two classes of clay minerals: the silicate minerals (as of the montmorillonite, illite, and kaolin groups) and the oxide minerals (as hydrated ferric, aluminum, and other oxides).

The silicate minerals are found more commonly under temperate climates and the oxide minerals in tropical areas, though the most active of the silicate minerals (montmorillonite) is encountered more usually in the tropics.

The clay minerals to be found in any soil are a function of the soil-forming processes and of the great soil group to which the soil belongs.

The silicate minerals are electronegatively charged colloids and dissociate adsorbed positive ions when in aqueous solution, whereas the pure oxide minerals are electropositively charged and dissociate adsorbed negative ions. In the latter instance, however, the intensity of the anionic field may be so low that the effect of any dipole groups situated on the surface of the minerals may be more-effective electrically than that of the dissociated ions, i. e., the oxide surfaces may behave more as a solid surface than as a colloidal surface.

The complete range of soil clay minerals varies between the highly active minerals of the montmorillonite group, carrying an intense field of negative charges, down to the slightly active members of the kaolin group, carrying but a few negative charges, and on to the positively charged oxides.

Among the members of the oxide group are sometimes included lateritic soil minerals, which occupy a place in both classes. They may be amphoteric. Though basically silicates (possibly of the montmorillonitic group) and electronegatively charged, their micelles are sheathed with a coating of iron oxide. Under alkaline conditions the iron is insoluble and the colloids act as if electronegatively charged and dissociate cations; but when the soil solution is acid, as is more usual for such soils, the iron becomes soluble, goes into colloidal solution, and ionises. The colloid then acts as if electropositively charged and will dissociate anions. The dividing line, as regards sign of resultant charge, does not necessarily occur when the soil solution is neutral but may occur on the acid side of neutrality at a condition of the soil solution known as the insoelectric point. The resultant charge carried is then zero.

This amphoteric property is believed to be an important one in the understanding of some stabilization phenomena. It possibly supplies an explanation of the successful part played by some catalysts in certain soil-resin combinations (19).

Colloidal organic matter acts much in the same way as an electronegatively charged clay colloid. The intensity of the surface field may, however, be several times greater than that surrounding the most-active clay colloid.

When a silicate clay or organic matter colloid is in aqueous solution, the adsorbed ions will dissociate or partly leave the colloidal surface, where the firmness with which they are held generally decreases with decrease in the valency of the adsorbed ion. If the adsorbed ion happens to be a sodium ion, it becomes almost free from surface restraint and is easily displaced.

In general, two phenomena are likely to occur in a soil-water solution: (1) the adsorbed ions become hydrated and are separate from immediate contact with the particle surface and (2) this ionic hydration, by partially releasing the ions from the colloidal particle surface, permits the surface itself to become hydrated. The former process is referred to as hydration, or ionic hydration, which is accompanied by the release of the heat of wetting; the latter is referred to as osmotic hydration, since osmosis is considered to be involved at this stage with the hydrated ions forming an osmotic membrane. The amount of each form of hydration will depend upon the chemistry and the shape of the clay mineral micelles.

Oxide minerals possess relatively little attraction for water, and pure laterite is insoluble and nonplastic.

If the solution be an electrolytic solution other than a water solution, then the adsorbed cations on the silicate clay surface may be replaced by cations from the electrolyte according to known laws of cationic, or base, exchange. By leaching with a suitable electrolytic solution, a sodium-clay colloid with a high affinity for water may be changed to a calcium clay with less affinity for water and lower plastic index.

Base exchange features in a number of stabilization processes.

Similar laws of replacement apply to electropositively charged clay colloids, but the process involved is not yet fully understood.

The deleterious properties of clay depend upon the amount and activity of the colloidal clay and colloidal organic matter present. While some forms of organic matter are beneficial by virtue of acting as a waterproofer, other forms and all clay material are in varying degrees detrimental.

Primarily, the potential activity of the clay material (its capacity for adsorbing bases including water) increases with its base-exchange capacity or as the clay mineral changes from an almost inactive kaolin to a highly active montmorillonite.

Secondly, the potential activity varies with the adsorbed bases, which are mainly those of calcium, hydrogen, sodium, and magnesium.

Thirdly, the effective activity will appreciably depend upon the structure of the soil fabric, i. e., upon whether the soil be undisturbed, disturbed, naturally compacted or artificially compacted and, if so, the relative compaction.

In addition to the phenomenon of ionic exchange, there is another and somewhat similar form of exchange which (though not of such importance in the field of soil science, because of the lower forces involved and the slower rate at which it occurs) may often be of prime importance in the field of soil stabilization. That is dipole exchange by which an ion containing a dipole group will replace another ion containing a dipole group of lower dipole moment. This phenomenon is believed of prime importance in the field of stabilization involving the use or organic compounds containing carboxyl and similar groups, which are able to replace those of water possessing a lower dipole moment.

Stabilization, in the chemical sense, consists essentially of adding to a soil material an optimum percentage of another material of opposite polarity to produce, by exchange phenomena, a mixture of greater cohesion or a mixture in which the deleterious clay particles or aggregates are surrounded by an insoluble sheath resistant to moisture penetration. The problem is essentially a surface-chemical problem. Thus, an optimum percentage of laterite clay will increase the cohesion of a podsolic clay, and a cationic detergent will stabilize an acid and siliceous soil, i. e., an electronegative clay colloid.

STABILIZATION

Stabilization in the sense of maintaining a minimum desirable shear resistance may be divided into three categories: (1) stabilization by compaction along; (2) mechanical stabilization not specifically providing for moisture stabilization; and (3) moisture stabilization involving the incorporation of some chemical substance.

Stabilization by Compaction

The process of artificial soil compaction may have a stabilizing effect only when suitable soil material is used and when climatic conditions are favorable.

Much depends, as regards the suitability of the material, on the nature of the microstructure, i. e., on the nature of the clay mineral and of the replaceable bases, on the presence and nature of protecting sheaths of organic matter and oxides, and whether these media are dispersed around single grains or around compound particles forming aggregates. Whether or not the cementing agents are reversible is another factor to be considered.

Many a laboratory possesses as an exhibit compacted soil material which has been kept in water for a long period without showing any sign of disintegration. But this characteristic does not apply to all compacted soils. It is possible with some naturally stabilized soils and with some A-6 clays but is less likely to be possible with cohesionless materials, silts, and A-7 clays.

There is an upper limit to moisture adsorption dependent on energy conditions. If the water table is very high, it is determined by the surface energy of the particles plus the capillary energy and corresponds to complete saturation. If the water table is very low, this upper limit is reduced to almost that corresponding to the surface energy of the particles. Between the two extremes is a moisture content known as the lentocapillary point, representing the upper limit to saturation, above which, under constant temperature conditions, moisture moves slowly in the fine capillaries and in the film or in the vapor phase.

Highly active clays, especially of the high-surface-energy calcium variety, may, after compaction at optimum moisture content, adsorb moisture from a nearby free-water surface so that ultimately the cohesive bonds between the clay micelles and aggregates are weakened and some slaking occurs irrespective of the percentage compaction. This condition is sometimes brought about or accelerated by the presence of appreciable percentages of carbonates and certain types of organic matter. Such soils, when compacted to maximum density and unloaded, will take up moisture, if available, to approximately their liquid limit. The possibility of some loss of structure occurring appears to be indicated by a relatively high field moisture equivalent. Such soils are not appreciably stabilized by compaction.

The maximum subgrade moisture content in an embankment is controlled to some extent by external loading, tending to suppress moisture adsorption and swelling, and by the length of the path the moisture has to travel, whereby it may be removed by evaporation, transpiration, etc., before it has an opportunity to accumulate in dangerous amounts. But, with such highly active subgrade materials, there always exists the possibility of a loss of stability not necessarily indicated by the 4-day C.B.R. value. Such soils may continue to adsorb moisture and lose stability for periods far in excess of 4 days.

The presence of replaceable sodium in both A-6 and A-7 clays delays and may prevent the condition of incipient slaking by closing the soil pores with swollen clay, provided external energy is not available to work the moisture into the soil material. This delaying action is frequently sufficient to prevent structural deterioration· by reason of the time interval during which moisture is available to cause damage.

With other clays, more common in the A-6 group and believed indicated by a relatively low field-moisture equivalent, the maximum moisture content, as controlled by the effective energy available to cause moisture adsorption, is not so greatly in excess of the optimum moisture content and decreases from the maximum value as the density of compaction approaches a maximum. Compaction tends to stabilize such soils.

When the energy available to cause moisture adsorption is satisfied, any excess moisture which may tend to accumulate (as by percolation through blacktop) will drain away if enabled. If the excess moisture is unable to drain away by virtue of the densification of a dispersed type of subgrade material, some additional softening of the subgrade may occur.

If the moisture content of the soil be appreciably below the standard optimum, or the plastic limit, when ample free water becomes available, few soils (unloaded), other than certain A-6 active sodium clays, can resist the disruptive action of rapid water entry, and slaking occurs by a dynamic action irrespective of the value of the field moisture equivalent. Under conditions of slow adsorption, as from a low water table, the colloidal cohesive braces can often adjust themselves to the slow volume changes without severe deterioration. One of the first to draw attention to this phenomenon was Henry Porter of the Texas State Highway Department (20).

The above discussion specifically refers to soils which have been disturbed before testing. Many soils with high moisture-adsorption characteristics by virtue of their physical and chemical structure will remain at a safe moisture content when disturbed, provided their natural structure is not destroyed and they are not allowed to dry out before being covered.

Some soils tend to be stabilized by compaction. This stabilization increases with increased density. It must be remembered however, that the amount of densification attainable in highly plastic clays is often limited by a kind of elastic phenomenon consequent on the imprisonment of air within the soil structure during compaction. Also, the load-carrying capacity of other less-plastic soils may depreciate on over-compaction, due to the breaking down of their natural microstructure.

A climate is favorable to stabilization by compaction when it results in the production of soil material resistant to water adsorption and when it does not encourage excessive pavement distortion.

It appears that, for all soil material normally resistant to slaking, the volume change occurring during cycles of slow wetting and drying, between saturation and the air dry conditions, does not vary appreciably with density. H. Porter of Texas is also believed to have been the first to note this characteristic (20).

For soils compacted at optimum moisture content under temperate climatic conditions, i. e., where the moisture content in the subsoil does not usually fall much below the plastic limit, the moisture intake, above the optimum, and the accompanying volume change for all but the highly active clays and silts, decreases as the percentage compaction increases. This is a stabilizing factor. When soil desiccation occurs, however, this stabilizing effect of decreased volume change with increased density disappears, insofar as the volume change becomes less dependent on density and there is a tendency to introduce slaking conditions and excessive differential volume changes or pavement distortion.

Mechanical Stabilization

The requirements for a satisfactory design of a road structural unit may be expressed in terms of the suitability of the material in the particular unit as well as in terms directly implying adequate shear resistance, such as in the empirical manner based on material recognition, classification, and rating (mechanical analysis and plasticity data) which has become the standard, though not necessarily the most popular, method used in the United States.

Such a method relies on the statistical analysis of an immense amount of experience and data for its correlationships between test requirements and satisfactory performance.

The method in mind is that originally introduced by H. Allen of the Bureau of Public Roads in 1942 for the classification and rating of embankment materials and for the determination of the total thickness for subbase, base, and surfacing relative to the various ratings. This method was built up, extended, and developed to give the present standard specifications of the American Association of State Highway Officials for materials for embankments, base courses, and surface courses.

The various specifications for surface and base-course materials include a control over the grading of the material and over the liquid limit and plastic index of its fines content. Specifications for subbase courses generally require that the materials used should give good densities when compacted and that their plastic index should not exceed some stipulated value. For embankment materials the control is mainly concerned with relative compaction. For all structural units a plastic-index control features either directly or indirectly.

The implications of such design recommendations were examined pedologically in a letter to Winterkorn by Wooltorton in 1947 (22), which concluded that: (1) a plastic index control was in effect a direct control over the maximum possible volume change (i. e., under conditions of slaking); (2) a liquid limit control expressed the same requirements on a moisture content change basis and that the permissable maximum value of the liquid limit varied with the specific gravity of the particles, the apparent density and the percentage of fines; (3) such design recommendations could only logically apply to the particular "average" or leached podsolic soils for which the statistical data, leading to such recommendations, referred; and (4) the general problem could not be solved for the want of some additional condition or control. The missing condition was considered to be the logical one of limiting the actual volume change to some permissable value.

Thus for base courses or any structural unit in which no overall swelling would be permissible: Plastic index times fines content (measuring the maximum value of possible swelling) should not exceed the volume of voids available within the compacted material to accommodate such swelling. For other structural units: plastic index times fines content should not exceed some predetermined value implying some overall swelling is permissible in the unloaded condition but which may in part or in whole be suppressed by the external loading.

The logical conclusions from this analysis were that: (1) under certain conditions where slaking was not encouraged as it is within the fines of a coarse granular base course, some allowance could be made for the nature of the soil structure in fixing permissible limits for the plastic index or maximum possible volume change and (2) the permissible maximum value of the fines content varied inversely as the plastic index, i. e., for any particular material so that, ignoring any structural effects, this permissible percentage, which features in present design recommendations or grading tables, varies with the nature of the soil colloids and decreases with increase in the colloidal activity, i. e., the tendency is for any permissible percentage to change with the soil group to which the soil belongs.

The validity of this reasoning was investigated by Winterkorn (18, 21) in connection with the addition of portland cement, lime, asphalt, tar, aniline-furfural, Resin 321, and abietic acid to synthetic soils composed of 65 percent sand, 25 percent silt (ground feldspar), and 10 percent clay (illite, kaolinite, montmorillonite) to which percentages of salt were added. He found that for small percentages of additives the best results were not obtained when kaolinite formed the clay fraction but when the more active illite was used.

The low surface activity of untreated kaolinite mixtures resulted in a high air-voids volume when they were fully saturated, whereas the higher surface activity of untreated illite lead to a small air-voids volume on saturation. Mixtures containing kaolin would thus require considerably more additives to prevent detrimental moisture adsorption leading to softening, stripping, etc. The moisture adsorption for the samples containing bentonite was sufficient to exceed the pore space available and to cause overall swelling, thereby weakening binding and water-proofing effects.

Chemical Stabilization

The principles governing the electrochemical theory of stabilization have already been referred to. It is not proposed to consider further the development of this approach as so much has yet to be learned about the properties of the various complex compounds used and the various reactions which occur, during stabilization, between these compounds and the colloidal clay particles. The intent is only to refer to those aspects of the development concerning which something has already been learned by experiment and experience.

It should be kept in mind during the following discussions that the efficiency of any particular stabilizer, as a stabilizer, and the economy of using such a stabilizer are two considerations which the author believes should be kept separate initially.

In general, the more active the clay fraction and the finer the soil material be pulverized, i.e., the greater the number of forces associated with particle surfaces to be stabilized, the greater will be the percentage of additive required. For this latter reason, cohesive materials are not usually completely pulverized but are broken down into small aggregates of an optimum size. Unfortunately, factors other than those of surface chemistry have to be considered with the result that expectations arising out of the electrochemical approach have often to be modified.

Cement Stabilization. The additive cement is used for: (1) reducing, by surface-chemical phenomena, the moisture affinity of clay materials and (2) promoting cementation sufficient to create a semirigid framework within the structural unit, though not for creating a completely solid or rigid structure as characterizes a cement-concrete pavement.

In the instance of cement stabilization, two requirements often have to be fulfilled. In such instances the surface-chemical effects are not necessarily the controlling factors.

From the electrochemical point-of-view cement would be expected to be very effective in the stabilization of those organic-matter-free clays acting as electronegatively charged materials, though this does not imply that any such clay can be stabilized by the incorporation of only a small percentage of cement. On the contrary, the percentage would be expected to increase with increase in the activity of the clay. Small percentages are only likely to be applicable with low-activity clays or with nonactive soil material where little more than cementation or mechanical stabilization is required and not moisture plus mechanical stabilization.

In addition to the nature of the colloidal complex itself, that of the exchangeable ions must be considered, though the actual part played by these ions is not yet fully understood. Tests (23) indicate that the effect of the replaceable bases increases with the silica-sesquioxide ratio, whereas the most-desirable controlling replaceable base depends upon the base-exchange capacity and is most likely to be a monovalent ion. Such a base appears (25) to be indicated by that cation which, for a monionic variation of that cation, gives a minimum value for the expression: optimum moisture content less the shrinkage limit.

The general effect of organic matter is one of modifying the effective activity of the clay particles where this may be accompanied by reactions, of a nature dependent on the chemistry of the organic matter, which may deter the process of stabilization. Some acid organic matter, especially that associated with podsols and podsolic soils, is detrimental by delaying or preventing the setting of the cement, while that organic matter associated with some lateritic soils has not been found to be detrimental.

It may be added here that it is often difficult to find a satisfactory stabilizer for some lateritic soils. It is believed that there are a number of reasons for this, among which the amphoteric nature of such materials must feature.

In addition to the considerations of surface chemistry and chemistry controlling the efficiency of soil-cement stabilizations, there are also certain physical factors having an influence on the process. Grain-size distribution, pore space, moisture-density relationships and, compressive strength are of importance. The gross effect of these appears to be a modification of the results to be expected from the electrochemical approach. This grows in importance as the activity of the clay material increases and results in there being an optimum value of the activity for most-efficient stabilization.

Some salts (e. g., sulphates of sodium and magnesium) are detrimental by virtue of interacting with the cement to form expanding compounds leading to disintegration of the mixture.

Much pioneer work has been done on the pedological study of soil-cement stabilization by Hicks (24), Mills (25), Catton (26), and Winterkorn (23). Much of this has been used by the Highway Research Board (26a).

Bitumen Stabilization. Bitumen may be used for the purpose of supplying cohesion for nonmechanically stabilized granular materials, for waterproofing mechanically stabilized granular mixtures or for waterproofing cohesive soils.

The principle controlling the stabilization of granular materials is one of coating such particle with a film of bitumen, of an optimum thickness, sufficient to supply adequate cohesion without, at the same time, destroying the natural frictional resistance of the particles.

For cohesive materials, the principle is of adding sufficient bitumen to block the capillaries within the small soil aggregates to hinder moisture changes within these aggregates and partly to sheath these aggregates. Complete waterproofing of the aggregates is considered to be neither necessary nor desirable.

The amount of bitumen required depends essentially on the affinity of the bitumen for the soil material. As bitumen predominantly acts as an electronegatively charged colloid, the amount of bitumen required increases with the number of electronegative charges located on the surface of the material to be stabilized. In the instance of a clay soil and a given particle grading, the percentage of bitumen required increases with the activity of the clay material as represented by its base-exchange capacity or its silica-sesquioxide ratio. It also increases with decrease in the valency of the adsorbed ions and with increase in the amount of organic matter.

Of the possibly adsorbed ions, sodium is usually detrimental. The presence of acid organic matter from leached forest soils is frequently so; but the basic organic matter associated with the chernozems is not. Free calcium leads to detrimental conditions by promoting bacterial activity leading to the decomposition of the bitumen. This particularly applies to heavy clay soils (37).

The percentage of bitumen required varies with a number of other factors, e. g., voids-ratio, climate, and type and the source of bitumen used.

The relatively hydrophobic soils and rocks, containing carbonates and oxides, are the most-easily wetted by bitumen to produce good adhesion and (when once protected by bitumen) a substantial resistance to moisture penetration into the bitumen-soil material interface.

Adhesion between bitumen and the relatively hydrophilic materials (as clay and rock silicates) is generally poor, and water, if it can reach the bitumen-soil material interface, quickly replaces the bitumen as a sheath around such particles. Therefore, the incorporation of a small percentage of wetting agent whose function is a surface-chemical reaction to change the sign of the electrostatic force field on the particle surfaces is usually essential with such materials. Types of wetting agents used include lime, cement, aniline-furfural, and the like. Bitumens containing a high wax content, cracked oils, and high-temperature tars may sometimes be used without a wetting agent. The desired percentage of wetting agent has been found to vary with climate and the base-exchange capacity of the clay material.

The pedological approach to soil-bitumen stabilization has been furthered by a number of investigators, including McLeod (27), of Canada; the Road Research Laboratory, England (15); and H. F. Winterkorn (28). The Highway Research Board has made use of some of the results of such research (29).

Resin Stabilization. Resins are used more generally for moisture stabilization. Resin and resin derivatives do not add to soil stability by providing cohesive resistance, though some synthetic resins (e. g., phenol-formaldehyde) may lead to an increase in strength.

Natural rosins have a constitution somewhat similar to abietic acid. The molecules contain hydrophilic acid and bulky hydrophobic hydrocarbon groups (the latter acting as strong dipoles). They are insoluble in an acid solution and almost insoluble in pure water, but their solubility increases in an alkaline medium.

Rosin derivatives, such as the complex compounds of sodium abietate and abietic acid, apparently can feature in both base-exchange and dipole-orientation phenomena. The same applies to the synthetic resins which have commonly featured in stabilization research.

In a water medium, partly neutralized, powdered natural resins break down into smaller particles of ultramicron size. These ultramicrons attach themselves to or orientate themselves about the surfaces of clay aggregates or particles so that, in the process of stabilization, the hydrophobic groups are directed outwards from the clay surfaces, thus repelling the entry of water. The presence of a film of moisture around the clay particles appears to be necessary for the success of this operation, i. e., presumably to enable effective orientation to take place—particularly so when natural rosins are used.

Natural resins are known to stabilize some acid soils and chalk. The mechanism by which stabilization is so achieved is not fully understood, and it has been said that several phenomena are probably involved in the operation. As a surmise it appears that natural resin particles carry a residual negative charge by which they can become attached to the surface of, say, a particle of chalk which carries secondary positive charges.

Similarly, in the case of the very-acid soils (pH 4 to 6) of an amphoteric nature or which have been treated to obtain hydrogen, aluminum, or iron modifications, the surface of the clay silicates act as if positively charged and thus encourage the attachment to themselves of particles of insoluble resins.

In the instance of partly soluble sodium derivatives, anionic exchange may occur to produce an insoluble sheath around the soil material.

Synthetic resins, as aniline-furfural, appear to attach themselves to an electronegatively charged surface by a process more similar to base exchange. They may be efficient as moisture stabilizers over a wide pH range, but they appear to be most efficient in an acid medium.

Research on the use of resin stabilizers has been largely sponsored by Winterkorn (30) and his co-workers. Some useful basic research has been reported on by the Road Research Laboratory, England (15). The practicability of the use of aniline-furfural has been described by Mueller (31).

Use of Electrolytes in Stabilization. A number of electrolytes, as calcium chloride, sodium chloride, hydrated lime, ferric chloride, and sodium silicate, can play an important, though generally limited, role in stabilization. The common principle involved is ionic exchange, though such others may feature as a change in hygroscopicity, a lowering of the freezing point, the possibility of increased maximum densities and shearing resistance, and retardation of evaporation by crystallization within the capillaries.

In addition to its deliquescent properties, calcium chloride can, by leaching, change a highly water-retentive clay, as a sodium clay, into a calcium clay by base-exchange phenomenon, thereby reducing the plastic index and volume change. The benefit achieved by this process increases with increase in the silica-sesquioxide ration.

Hydrated lime may behave similarly with the added advantage of acting as a wetting agent to change the effective sign of highly active clay colloids. In the presence of natural pozzolanic materials, there is also a cementation effect.

The effect of the addition of a suitable percentage of silicate of soda to appropriate soils is to increase water-stable aggregation and resistance to crushing and to decrease the plastic limit, plastic index, and swelling. The process involved is apparently that of anionic exchange, resulting in the formation of insoluble silicate sheaths around the soil particles and accompanied by cementation effects.

There is a considerable field of literature on the use of electrolytic semistabilizers, but it is only proposed to refer to a believed important publication by Laws and Page which appears to have escaped attention (32). It is felt that there is scope for silicate of soda stabilization, as distinct from its sole use as a cementation agent, in the tropics.

As the silicates are electronegatively charged, they could not be expected to stabilize active clays. This is borne out by Reference 32, in which it is shown that the efficiency of the additive increases with decrease in the activity of the clay, i. e., with decrease in the silica-sesquioxide ration. It is likely to be most efficient with those soils of an electropositive character or with those active clays which have had imposed on them an electropositive nature by virtue of coatings of iron compounds.

In conclusion, attention is drawn to the work of Winterkorn and his associates Moorman Chandrasekharan, Silberg, and Goodpaster, of the problems associated with the stabilization of soils belonging to specified soil groups, i.e., podsolic (33), loess pampeano (34), lateritic (21), saline (35), alkaline (36), and beach soils (30). Attention is also drawn to Winterkorn's more-general paper on the stabilization of cohesive soils (37).

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Soil Stabilization and Colloid Science

ERNST A. HAUSER, Professor of Colloid Science,
Massachusetts Institute of Technology

Introductory Remarks by the Chairman: A great problem is to scientists like a medieval fortress to beleaguering soldiers. The walls are scouted for weaknesses, and effort is concentrated on what is considered to be the weakest spot. If a breach is made, most of the men will use it to enter the fortress. A scientific pioneer, as any other pioneer, does not like crowds. Even though he had originally recognized the weakness and was the first in the breaching of the wall, he will leave the crowd now seeping through in order to attack other problems and break new walls.

Hauser is such a true pioneer and a worthy representative of colloid science, a discipline that is always out in front in attacking complex problems and that leaves the conquered country to be organized by her socially more-accomplished sister, physical chemistry. Hauser is well known all over the world for his pioneering accomplishments in colloid science and its application to the chemistry and technology of rubber and other clastomers and of clays and other silicate minerals. His many contributions to science and technology are too numerous to even list in this place; several of these were of prime military importance during the Second World War.

It is gratifying that Hauser consented to enrich this symposium with an interpretation of the relations between soil stabilization and colloid science.

● COLLOID science is always needed in order to explain the interaction of matter present in the colloidal range of dimensions with other substances. Any substance, organic or inorganic, falls within the colloidal range if it is present in at least one dimension in the range between one micron and one milimicron.

In soil stabilization, clays must always be considered as very important and often determinant soil components; therefore, a sound knowledge of their structure, composition and morphology is of great importance.

A vast amount of information on clays has been accumulated since 1925, when it was possible, for the first time with the aid of new research tools and the formulation of new concepts, to embark on a truly fundamental study of clay minerals. This gigantic work has resulted in a good, though still not complete, understanding of the occurance, composition, and colloidal properties of all clay minerals (1).

This contribution will cover the most-important facts of the colloid science of clay minerals and the most-recent concepts pertaining to them. Despite the emphasis placed here on the clay minerals and their properties, it should not be overlooked that clay minerals in natural soils frequently have adsorbed on their surfaces organic matter resulting from the decomposition of plants or animals. For this reason, the surface activity of these clays will differ pronouncedly from that of pure clay.

This condition has been aptly described by Winterkorn (4) as follows: "Clay minerals in natural soils are not as clean as a scraped bone; rather the mineral surface is normally in as close a relationship with adsorbed and synactive organic matter as a bone in a living being is with cartilage and muscle tissue." Also, microörganisms are normally present in natural and often in stabilized soils. Their bearing on soil stabilization has been recognized by only a few workers in the field (5). An explanation of the variability in properties of clays will be offered before the influence exerted on soil stabilization by organic matter is discussed.

Genesis of Clay Minerals

Of all the theories pertaining to the origin of clay minerals, geologists and mineralogists have largely accepted the "residual clay" and the "transported clay" theories. The former is based on the assumption that the formation of clay minerals is the result of surface weathering of fresh rocks or is due to the action of solutions. From a colloid-scientific point of view this theory deserves special attention. However, it does not offer a truly satisfactory explanation of the genesis of the two basic components of clays, silica gel, and alumina or magnisia gel.

Although some of the latest contributions to the chemistry of silicates admit that colloid science plays an important role, the majority of scientists interested in the colloid science of silicates are seemingly still not familiar with the fact that colloid science had already made basic contributions to the theory of the genesis of silicates over a quarter of a century ago (2).

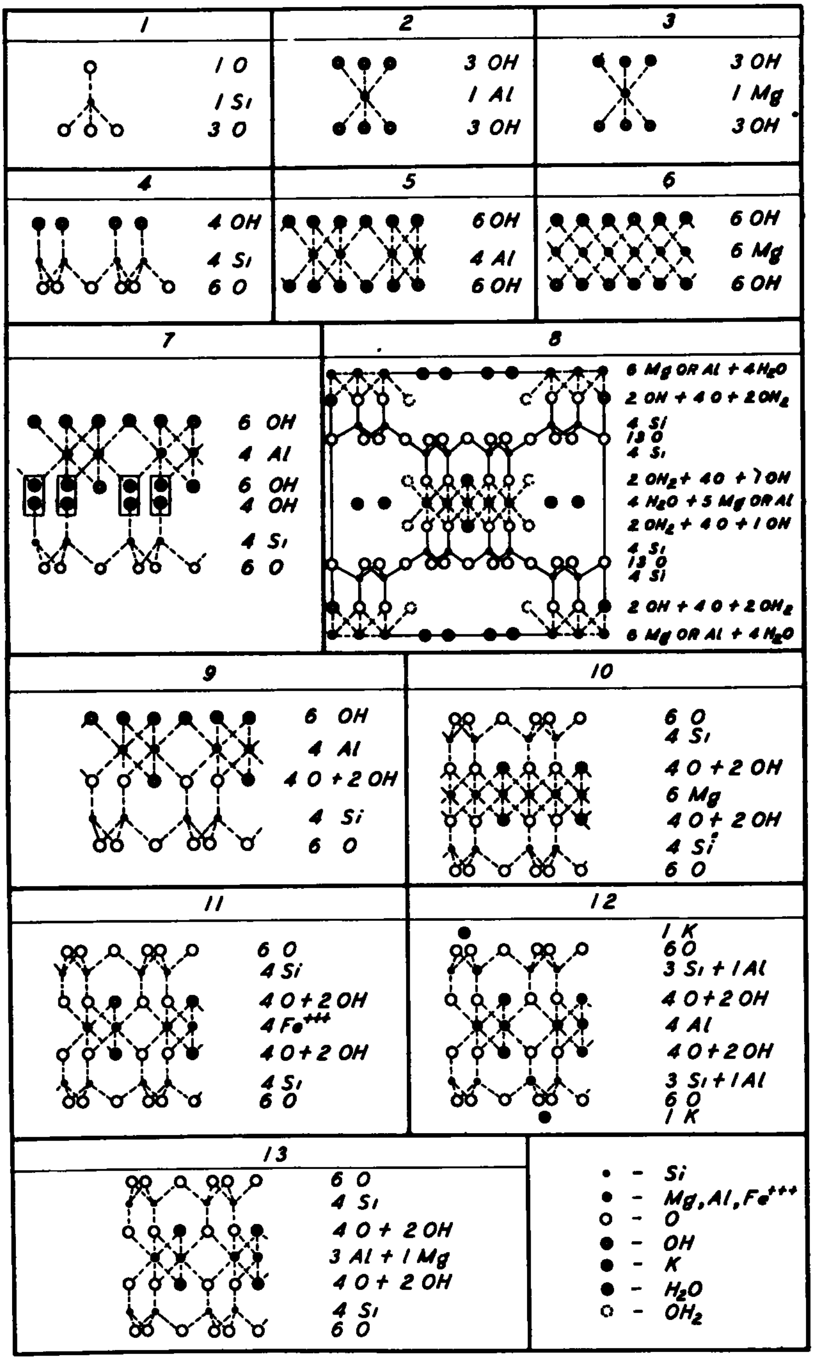
Even as early as 1779, research was done in the field of what is known today as the colloid science of siliceous matter. In his paper, "De Terra Silicea," the Swedish scientist, Torbern Bergman, stated:

"Finally I must still think of those incomplete phenomena which depend on a seeming solubility. This siliceous liquor (alkali silicate solution--E. A. H.) is precipitated by all acids, because the alkali prefers to hang onto them rather than to the gravel. This precipitated gravel has a very expanded and loose texture and is filled with water, so that it is twelve times as heavy when moist than when dry. If more water is added before adding acid, however, the solution remains clear, even if more acid is then added than would be needed to neutralize the alkali. This is a peculiar phenomenon and the reason for it is probably the following: Through the dilution with water the siliceous particles are very much separated from each other, or made finer and better distributed throughout the liquid. Although the particles should settle out, being heavier than liquid, they cannot in this case overcome the resistance due to friction, because a greater force will be needed to accelerate sedimentation than that resulting from the difference in specific gravity. The silica particles remain suspended in the liquid but at the same time are invisible due to their fineness and transparency."

The more-modern work which has been overlooked dates from 1927, when the German geologist, Schornstein, made a fundamental contribution to the colloid science of siliceous matter. The following is a passage from one of his basic papers (2):

"During the decomposition of silicates the following must be borne in mind: In the magma, silicic acid is in chemical equilibrium with the bases. It is able to form complex molecules which, however, are chemically well-defined; this is demonstrated by the fact that all these silicates are capable of separating into crystallographically well-defined particles. Silicic acid owes this property to its high acidity at elevated temperatures. At low temperatures the acidity of silicic acid is so small, however, that it can form a salt only with the strongest bases, as for example Na_2O and K_2O. It will be precipitated from these by the weakest acid, however, even by water alone. This implies that most of the silicates are present in a metastable equilibrium, even at normal temperatures. When water decomposition sets in, the equilibrium is disrupted. In the absence of acid, oxides or hydroxides of the silicate bases are formed and silicic acid is liberated. In the presence of carbonic acid, humic acid, or sulfuric acid, primarily those bases will be attacked which will form soluble salts with these acids. The moleculer structure of the silicate is disrupted and at certain locations of the crystal lattice the acid-insoluble bases and silicic acid remain. When liberated they are present in a molecular disperse condition. Since they are insoluble, they are not capable of forming a molecular solution with water; oversaturation results, causing polymerization. This can either occur in space-lattice fashion, resulting in the formation of crystals, or, if the oversaturation is too great in comparison to solubility and the ability to crystallize too small, colloidal gels are formed. The formation of colloids due to the decomposition of the silicate is a condensation reaction."

We still have no truly satisfactory answer to the question of how clay minerals were actually formed or how their genesis can be explained in a way which takes into account all the knowledge now at our disposal. Many years ago the alteration of pure silica and alumina to kaolinite or montmorillonite was demonstrated. In all probability we are dealing here with a combination of the chemical reactions attendant upon leaching and a devitrification of the glass, the reaction then proceeding through a hydration and adsorption of ions. In all probability the silica and alumina gels reacted first to form halloysite; then, by condensation, kaolinite or montmorillonite, talc, mica, and other minerals, depending on the ions present. In any event, clays



The schematic drawing in the first column of each section represents the composition of the unit cell of the respective building unit or complete crystal lattice. All atoms have been projected into one plane The second columns give the number and type of atoms or groups for every lattice plane. 1 = silicon tetrahedron, 2 = aluminum octahedron, 3= magnesium octahedron, 4 = hydrated silica, 5 = gibbsite; 6 = brucite, 7 = halloysite; 8 = attapulgite; 9 = kaolinite, 10 = talc, 11 = nontronite, 12= mica (illite); 13= montmorillonite (substituted)

Figure 1. Structural data of the most-important clay minerals and their building units.

owe their genesis to the presence of silica and alumina or magnesia gels and the changes they undergo with time or temperature (1, 2, 3).

A fact which until quite recently has not found the attention it deserves is that, in spite of the inertness of most siliceous matter, the chemical reactivity of the silica surfaces surpasses that of most stable oxides. Systematic research on the surface chemistry of silicates has led to the discovery of unexpected chemical reactions which result from the dehydration or polymerization of silicic acid (2).

Structure of Clay Minerals

In 1923 Hadding and, a year later, Rinne were able to prove by X-ray-diffraction studies that clays are not composed of matter in the amorphous state, as had been assumed, but that they are crystalline in structure. Since then our knowledge of the structure of clays has been greatly increased. Colloid science has offered a better understanding of some of the phenomena which are so characteristic of clays and which could not be explained by physico-chemical reasoning alone.

Bearing in mind the fact that all clays are composed of silica and alumina or magnesia, and first taking the unit cells of these components into consideration, it will immediately be realized how these structures can increase in dimensions. It will then also be seen how they can combine to form either simple double layers or triple layers of silica and alumina or magnesia (Figure 1).

A silicon atom binding four oxygen atoms forms a tetrahedron; the radii and distances between the silicon and oxygens permit the four oxygens to touch, leaving a cavity just large enough to include the silicon (Figure 2, left).

The second building unit is a combination of aluminum with six oxygens, where the latter completely enclose the aluminum atom (Figure 2, right). The aluminum can be exchanged for magnesium, but since magnesium is a larger atom it does not fit exactly into the cavity; therefore, a slight strain in the structure is set up. These two combinations are known as aluminum-oxygen and magnesium-oxygen octahedra.

These building units are held together by the sharing of electrons. Since each silicon atom can share four of its outer electrons, whereas each oxygen atom needs two electrons for saturation, a silicon-oxygen tetrahedron is unsaturated, lacking four electrons. If the oxygens are replaced by hydroxyl groups, however, the system is saturated, since the missing electron has been supplied by the hydrogen of the hydroxyl group.

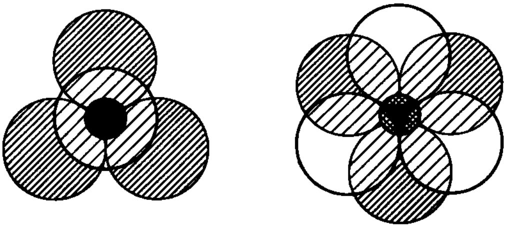


Figure 2. Left: A silicon-oxygen tetrahedron (schematic). The oxygen atom on top has been drawn as a transparent sphere to show the location of the silicon atom in the cavity formed by the four oxygen atoms. Right: An aluminum- (magnesium) oxygen octahedron (schematic). The oxygen atoms constituting the upper layer have been drawn transparent to show the location of the aluminum (or magnesium) atom in the cavity formed between the oxygen (hydroxyl) layers.

Starting with orthosilicic acid we find that in forming a silica sheet every oxygen atom located in one plane is shared by two silicon atoms, except the oxygens located at the edges of the sheet, which remain unsaturated; the oxygens in the vertex position of the tetrahedra have a hydrogen attached. This structure is saturated, therefore, and is known as a hydrated silica sheet (Figure 3). If we join nonhydrated silicon tetrahedra to form such a sheet, we find that all oxygen atoms are saturated except those in the vertex, edge, and corner positions. Consequently, any unsaturation occuring due to edge or corner positions of atoms will be negligible compared to the surface unsaturation of such a structure.

The second building unit--the aluminum or magnesium octahedron--is capable of forming similar sheets. Assuming that all the oxygens are saturated by hydrogen, then two layers of hydroxyl groups, each in close hexagonal packing, can be cemented together by magnesium atoms, so that six hydroxyl groups share three magnesiums. Except for edges and corners, such a sheet is saturated and neutral. Several of these composite layers can be held together by secondary forces, such as van der Waals' forces.

It is interesting to note that by cementing two of the hydroxyl sheets together with magnesium, the high symmetry of a hexagonal configuration is disturbed and reduces to a monoclinic one, since the hydroxyl sheets are staggered. The mineral built up according to the above structure is known as brucite (Figure 1).

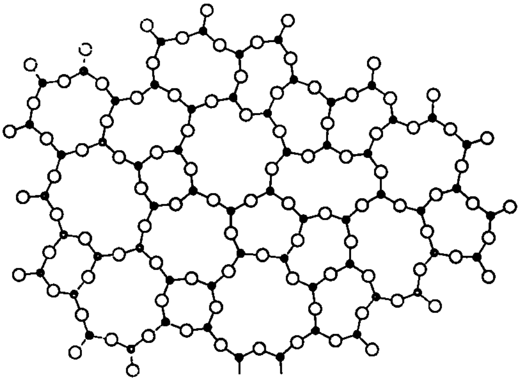


Figure 3. Structure of an unsymmetrical silicon-oxygen sheet; silica glass (schematic)(after W. H. Zachariason).

If aluminum replaces the magnesium in the octahedral sheets, changes have to be made in order to retain a neutral sheet. Aluminum can share three electrons, magnesium only two. Consequently, one third of the places formerly occupied by magnesium must remain vacant. Therefore, the aluminum octahedral sheets will not be so densely packed as the magnesium octahedral sheets. This mineral is known as gibbsite.

Just as silicon-oxygen tetrahedral sheets are built up from single silicon-oxygen tetrahedra, gibbsite from the aluminum-oxygen octahedra, and brucite from magnesium-oxygen octahedral sheets, the minerals of the kaolinite, montmorillonite, and illite groups are built up by joining gibbsite or brucite sheets to silicon-oxygen tetrahedral sheets (1).

Some Colloid-Scientific Considerations

According to its crystallographic classification, kaolinite is a monoclinic system which displays good cleavage. In the formation of fragments the fracture occurs along the basal cleavage plane and also normal to it. Fracture along the cleavage plane does not rupture primary valence bonds, and this results in the formation of thin hexagonal platey particles. If the fracture is parallel to the c axis of the crystal, however, the bonds between Si and O, Al and OH, or O must be broken. These broken bonds are the basis for the preferential adsorption of hydroxyl ions. The adsorbed hydroxyl ions are hydrated; that means that they carry with them water molecules which become part of the water hulls. The particle and the cations located close to it constitute the Gouy-Freundlich diffuse double layer.

If a small trace of an electrolyte, e.g., sodium hydroxide, is present, profound changes in the forces connected with the micelle will occur as a result of the preferential ion adsorption on the particle. This results in the setting up of a considerable repulsive force at the edge of the lyosphere. The charge density on the particle and its potential are high, and from a colloidal point of view the particle is at maximum stability. In this state the kaolinite slip is deflocculated or dispersed.

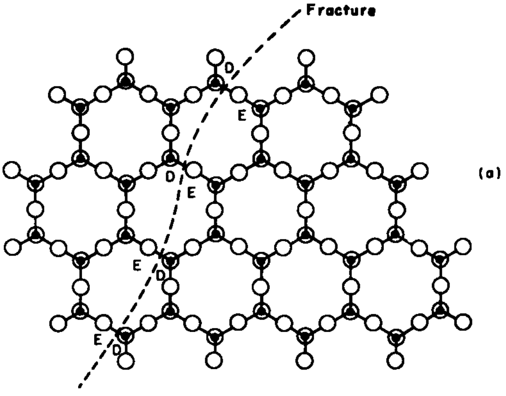


Figure 4. The formation of E units and D units at the fracture surface of crystalline SiO_2 (after W. A. Weyl).

If we now look at the structure of a clay mineral of the montmorillonite group, we find that these substitutions result in establishing net residual negative charges on certain sections of the lattice, which in turn cause attraction. Sodium is the most-common cation found in naturally occurring montmorillonites. The crystal units of montmorillonite are held together quite loosely by weak O-O bonds. The sodium ions are primarily adsorbed on the surfaces of the silica sheets and to a negligible extent on the fracture along the c-axis. When hydration occurs these ions pry the particles apart and a double layer of appreciable thickness is formed. Montmorillonitic clays thus exhibit a high degree of swelling (6).

Silica in its various modifications represents a three-dimensional network of SiO_4 tetrahedra in which the O^{2-} ion is shared by two Si^{4+} ions.

In the interior of silica the building unit is the $Si^{4+}\left(\frac{O^{2-}}{2}\right)$ group. In contrast thereto, two types of groups must be expected in the surface, namely $Si^{4+}\left(\frac{O^{2-}}{2}\right)^3$ and $Si^{4+}\left(\frac{O^{2-}}{2}\right)\,{}^3O^{2-}$. The first, known as a deficiency unit, or D unit, shows an oyxgen deficiency resulting in an excess positive charge; the second, known as an excess unit, or E unit, has oxygen in excess over the stoichiometric ratio and carries a negative charge (Figure 4) (2).

This may lead to an electron transfer from the E unit to the D unit so that the potential field is reduced and with it the energy content of the nascent surface. For silica such an electron transfer does not seem very probable, since this would lead to the formation of Si^{3+} ions having eight-plus-one outer electrons. It is the instability of this electron configuration which most probably excludes such an electron transfer from contributing to the stability of the nascent surface of silica.

This information indicates the complexity of the problems pertaining to the properties exhibited by clayey substances and demonstrates one phase of the importance of colloid science for soil stabilization.

In dealing with colloidal clays it is impossible to explain their properties by rigid mathematical laws, as so successfully applied in physico-chemistry. This should not imply that the use of any mathematics in this field is of no value at all; however, there is little to be gained by drawing conclusions based on mathematics alone, disregarding the peculiarities of the colloidal state of matter. The state involved in colloid science is characterized by a high surface-over-volume ratio. In accordance with the electronic theory of valency, an ion located in the surface of a particle must be far-more reactive than one in the interior. Therefore, matter present in the colloidal state must be much-more reactive than its chemical composition alone would indicate. This also explains why most of the properties characteristic of colloids cannot be harnessed into rigid mathematical concepts.

Ion Exchange

Of all the phenomena exhibited by minerals which form colloidal micelles when in contact with water, the most important is the ion-exchange reaction. This reaction was originally recognized by Thompson in his investigation of the properties of soils (2).

The ultimate clay crystal carries a net negative charge. This is the result either of anion adsorption onto its surface or of an unbalanced crystal lattice. Whatever the basic cause, the individual ultimate clay particle may be pictured as a very-complex anion. To balance the charge the particle will tend to adsorb the necessary number of cations available in the environment. These cations then act as the so-called counter ions. When dispersed in water they will hydrate to a degree depending on their valency and hydratability, dissociate to a certain extent from the surface of the particle, and thereby form a diffuse electric double layer, thus giving rise to the formation of a colloidal micelle. Such a clay particle may therefore be compared with a dissociated electrolyte, the size of one of its ions falling within the colloidal range of dimensions.

For example, just as a soap-like sodium oleate in hard water will exchange its sodium ions for the less-hydrated calcium ions, the counter ions of the clay particle will be exchanged with ions from the dispersion medium if the resulting micelle would have less tendency to hydrate and carry a lower charge, or both. This reaction is, of course, the more pronounced the more ions of a high degree of hydration are present in the clay under investigation. The magnitude of this ability to adsorb cations depends primarily on the structural configuration of the nucleus of the colloidal clay micelle. It is usually expressed in milliequivalents of cations per hundred grams of clay.

By the same reasoning, however, clays may also adsorb anions where net positive charges are set up in the crystal lattice or where the hydrogen of a hydroxyl group is exchanged for a stronger ion, like PO_4^{---}. Generally speaking, therefore, it may be stated that ion exchange will follow the Hofmeister or lyotropic series, at least for cations, i.e., the higher the atomic weight of an ion, the more firmly it will be held by the exchanger. Therefore, the exchange reaction for monovalent ions will follow the series $Li < Na < K < NH_4 < Rb < Cs$, and for bivalent ions the series $Mg < Ca < Sr < Ba < La$ (Figure 5).

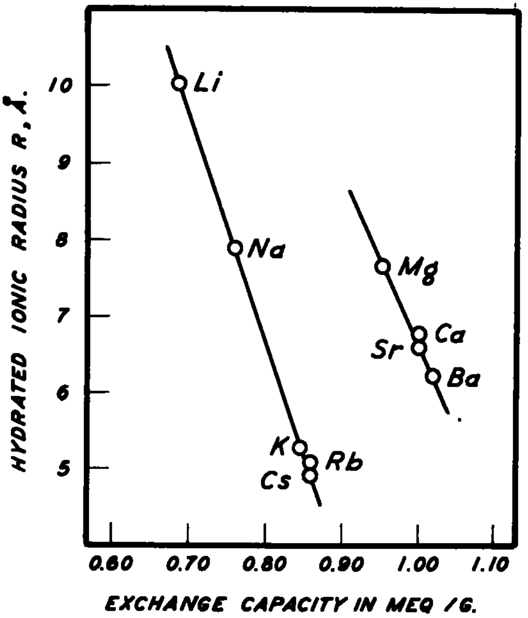


Figure 5. The radii of hydrated ions versus exchange capacity at equilibrium.

There is one other fact which should not be overlooked, however. Besides valency and hydration, the size of the exchanging ion is also important; it is difficult to exchange with ions which have an apparent diameter of at least 2.64 Angstrom units, the diameter of the circle which can be inscribed within the hexagonal net of oxygen atoms forming the silica sheet. It immediately becomes evident why it is extremely difficult, if at all possible, to exchange with a potassium ion, which has an apparent diameter of 2.66 A. (Figure 6).

For a long time it was assumed that ion-exchange reactions were limited to inorganic ions. It has been shown, however, that organic ions can react in the same way (7). Organic constitutents of soils play a large part in the exchange capacity of normal soils and thereby influence the soil-water relationships. Bearing this in mind, it becomes clear that the exchange capacity of actual soils cannot be predicted from their mineralogical composition only, but that careful experimental research is essential to obtain the correct information.

Soil Stabilization

The term "soil stabilization" includes all physical, chemical, electrical, thermal, and other methods by which soils can be improved to serve better their intended purposes (8). Stabilization has been achieved by the use of bituminous materials, especially for road and airstrip construction; much work has also been carried out with tars, asphalts, portland cements, lime, calcium chloride, sodium silicate, lignin derivatives, and various other natural and synthetic resins.

The problem of stabilizing soil is particularly important in dealing with fine-grained clays, most of which contain montmorillonite. In all these clays water will have a marked effect, since such soils lose their shear strength when very wet.

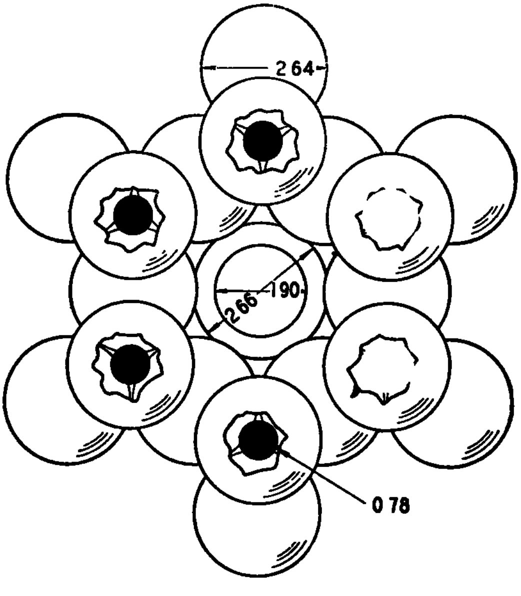


Figure 6. Hexagonal arrangement of silicon-oxygen tetrahedra in a silica sheet (schematic). The top oxygen atoms have been cut open to show the location of the silicon atoms. The large circle in the center space with a diameter of 2.66 A. corresponds to the diameter of a potassium atom, the smaller one to a sodium atom. The actual dimensions in Angstrom units (1 A. - 0.1 millimicron) for the different atoms are indicated on the drawing.

Most of the treatments now in use encounter increasing difficulties as the size of the dominant soil particles decreases.

For this reason, the search for better stabilizers for fine-grained soils is being pursued quite actively.

A primary military problem is to obtain an almost immediate improvement of the structural properties of naturally occurring soil. The treatment must be effective with various naturally occurring soils even if they differ pronouncedly in their water content, so that they will show an appreciable strength both in shear and in tension without being affected by water after they have been reacted. It must be possible to incorporate the stabilizer into the naturally occurring soil without actually removing the soil from the site; also, the stabilizer should be effective in very small proportions.

One approach toward the solution of this problem is the incorporation into the soil of a monomer which can be polymerized thereafter at normal temperature, thereby forming effective bonds or links with the soil as a basic part of the new structure.

The monomer to be employed must be able to enter into ion exchange with a base-exchangeable soil or to provide an effective bond with soils exhibiting no base-exchange properties. In the latter case the polymerized monomer will form long-chain polymers into which soil particles are interwoven.

The monomer should be ionizable and represent a water-soluble salt. If the organic part of the monomer is negatively charged, the cation of the monomer must be polyvalent, so that a positively charged complex monomer ion becomes available for base exchange with the negatively charged soil particles. Furthermore, a polyvalent cation is needed whenever the cation is to serve as a link in cross-polymerization.

It must also be borne in mind that a certain quantity of free soil water is needed when chemicals of this type are added to the soil, so that they become ionized and immediately enter into the exchange reaction. Generally it can be stated that most soils have enough water present to meet the requirements for polymerization. Any monomer which is not dissolved and base exchanged with the soil particles remains available for polymerization in the final phase of the treatment.

Most important is the formation of an insoluble polymer in which the soil particles become part of the polymer. If this is accomplished the permeability of the soil decreases, because a continuous soil-polymer-water structure is formed. The soil water is also held more closely to the soil structure by the formation of the polymer structure and its dipolar alignment.

On June 4, 1946, United States Patent 2,401,348 was issued to E. A. Hauser and E. M. Dannenberg for molding compositions using base-exchange solids like bentonites in finely divided form to which an ionizable salt of a polymerizable carboxylic acid was added. This compound contains the polymerizable olefinic group of the salt. In Claim 1 of the patent the use of acrylic acid salts was claimed specifically (2).

Although this patent did not refer to the use of a polymerization catalyst which would permit the reaction to take place at normal temperature, it must be considered, nevertheless, as the original idea for working out the acrylate and similar methods for soil stabilization.

Whenever a base-exchange solid like sodium bentonite is used, the reaction has to be carried out in such a way that the sodium ions which act as counter ions in the clay micelle may be exchanged, so that the clay becomes of the nonswelling type. As work continued it was found that salts of multivalent cations, e.g., lead acrylates, generally give satisfactory results.

As a result of this work a patent on "Stabilization of Soils" was granted in 1953 (2). According to this patent, the dissociation of calcium acrylate is accompanied by a base-exchange reaction in which the ionized calcium acrylate ions replace sodium ions associated with the soil particles.

After the soil has been intermixed with calcium acrylate and a base-exchange reaction has taken place, the soil-monomer mixture can be polymerized at normal outdoor temperature through the use of an appropriate catalyst, e.g., one comprising an oxidizing agent and a reducing agent.

Calcium acrylate will not ionize to an appreciable extent; therefore, a complex colloidal electrolyte forms. Calcium ions have a diameter greater than that of the "inscribed circle" (see Figure 6), which prevents their penetration of the silica layer of the clay particle; therefore, the calcium ion will be attached to the gibbsite layer of most clay particles. If a very-moist soil is involved in the reaction, however, it becomes questionable whether this can be accomplished. In most cases, therefore, only a skeleton of calcium acrylate surrounding the soil particles will actually be formed and not a truly chemiically combined structure.

What is still being widely overlooked today is the fact that, as previously mentioned, soils can never be considered as being absolutely pure; the presence of decomposed plants (which act as nutrients for bacteria) and of other microörganisms must be taken into consideration (4, 5). Therefore, any work carried out with pure clays or sands is only of theoretical importance (9). Far more attention must be paid to the reactivity of the different soils with their surrounding media to obtain truly successful results. Only to make tensile and shear tests on specially molded samples can never result in a truly satisfactory answer to the problems involved in soil stabilization. Soils should first be subjected to systematic chemical and bacteriological investigation.

Without paying more attention to these factors, it will never be possible to stabilize soil in situ satisfactorily.

Conclusions

Colloid science is of extreme importance for any work pertaining to the problem of soil stabilization. For this reason the most-important facts pertaining to the structure of clay minerals have been discussed. On the basis of the clay structures, the colloidal properties of clays have been referred to with specific attention to the phenomenon of ion-exchange reactions. Based on this information, the application of acrylic resin exchangers is discussed and the problems involved in the use of calcium acrylate are pointed out.

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Prediction of the Consistency Limits of Soils and Soil Mixtures

MANUEL PIMENTEL PERIERA DOS SANTOS, Chief Engineer,
Laboratory for Material Testing and Soil Mechanics
Mozambique, Portugese East Africa

Introductory Remarks by the Chairman: Satisfactory prediction of consistency properties of soil mixtures from the respective properties of the component soils can save a large amount of soil testing and is, therefore, of great economic importance. On the other hand, since all disturbed soils can be considered as mixtures of two or more component soils, a method resulting in satisfactory prediction of the consistency properties of soil mixtures is of basic importance for the understanding of the factors determining consistency properties of natural soils.

This problem has been attacked in different ways by different agencies and countries. Representative of our own empirical and pragmatic genius are the well-known Michigan method and also the more or less successful attempts to correlate consistency properties with the minus-2-micron clay fraction. Russian ideology has led to the "substantial true plastic limit" of Vasilieff and Goldstein, which is, mainly a function of the surface area and the water affinity of the component soil fractions.

These approaches have overlooked the importance of the granulometric factor, despite the fact that the relationship between granulometry and consistency of mineral-water systems has been known and used in the design of concrete mixtures ever since the pioneering work of Abrams. It is gratifying and very important for our science that Santos has finally made good this omission and that he has developed a granulometric soil constant which permits not only a satisfactory prediction of consistency properties of soil mixtures but which goes to the heart of the consistency problem itself.

● LABORATORIES engaged in large-scale projects of granular soil stabilization are forced to occupy themselves constantly with the study of soil mixtures that must comply with strict specifications with respect to their granulometry, liquid limits, and plasticity index. One can, therefore, understand the interest in developing a method that gives directly the liquid limit and plasticity index of a mixture of two or more soils if the values of these properties are known for the component soils. Such a method to be really useful must produce results that are sufficiently close to permit dispensing with the check tests on the adopted mixtures, leading thus to a great economy in time and money.

The Portuguese province of Mozambique, situated on the east coast of Africa, has under execution a road-construction program amounting to the paving of 5,500 kilometers. In this program low-cost road construction methods are utilized and different types of soil stabilization are employed. Whenever possible, preference is given to granular soil stabilization. When this is not feasible, recourse is taken to soil-cement or to macadam. Bituminous soil stabilization, which is more expensive, is reserved for road toppings and for the surface treatment of bases stabilized by means of other methods.

The Laboratory for Material Testing and Soil Mechanics of Mozambique; aided by mobile field laboratories constructed on trucks, is confronted, within a relatively short period of time, with an enormous volume of tests that amounts to the examination of many tens of thousands of soil samples. This was one of the determinant reasons for the necessity to develop methods which, in the case of soil mixtures, would permit an appreciable reduction of the testing load.

On the other hand, the fact that one can predict with sufficient approximation the properties of a soil mixture from those of the component soils permits the study of certain other important problems. One can plot diagrams of the variation of the Atterberg limits with the variation of the percentages of the different soils in the mixtures. Furthermore, it is possible to arrive at corresponding consistency limits of granular soils by mixing them with a reference clay of known properties and testing the properties of the mixture. Since the liquid and plastic limits of granular stabilized mixtures fall within the range of lower numerical values, one possesses an important instrument for the study of the properties and behavior of these mixtures.

Finally, since it is possible to predict the properties of a soil mixture by means of the introduction of a new granulometric constant, the question occurs whether or not it is also possible to establish a clear correlation between the granulometry of a soil and its Atterberg limits. This problem has been studied for several years in our laboratory. Results have been obtained that justify our best hopes.

SOIL MIXTURES

Several authors have occupied themselves with the problem of developing approximate equations which permit the selection of a soil mixture suitable for granular soil stabilization. Among these we shall cite those referring to the index of plasticity of Bonnenfant (1), Escario (2), Hogentogler (3), and of the Highway Department of Michigan (4). None of these can be considered as satisfactory and as of general applicability.

In the course of our investigations on this problem, we have introduced a new soil constant which depends on the granulometry of the sand fraction. This constant is defined by (5).

$$a = \frac{\Sigma\ y}{100\ n} \text{ - - - - - - - - - - - -(1)}$$

in which: y = ordinate (percentage passing) of the granulometric curve of the soil, using the sieves of the British system Nos. 200, 100, 52, 25, 14, 7 (the corresponding ASTM sieves are Nos. 200, 100, 50, 30, 16, and 8);

n = number of ordinates (six).

We have shown that the introduction of this constant permits, with sufficient approximation to be practical, to solve the problem of prediction of the characteristics of a mixture of two or more soils by means of the general expression (6,7).

$$K = \frac{A_1a_1K_1 + A_2a_2K_2}{A_1a_1 + A_2a_2} \text{ - - - - - (2)}$$

in which: K = characteristic property of the mixture (liquid limit, plastic limit, shrinkage limit, plasticity index, linear shrinkage, field moisture equivalent)

K_1 and K_2 = corresponding characteristics of the soils to be mixed;

A_1 and A_2 = weight percentages (divided by 100) of the soils in the mixture

a_1 and a_2 = constants defined by Equation 1.

Expression 2 has already been proven by many thousands of determinations with statistically satisfactory results. As shall be seen later, this expression is considered as a first approximation of a more rigorous equation which, on its part, is the consequence of the relationship existing between the limits of consistency of soils and their granulometry.

Figure 1 represents a typical case studied in the laboratory: a mixture of a sand (Soil No. 6093) with a clay (Soil No. 4424). The values of the liquid limit and the plasticity index of the sand which because of its granular nature cannot be determined directly, were derived from the respective values of a mixture of equal parts of the sand and of the reference clay (Soil No. 4447). The characteristics of the four soils are given in Table 1.

The mixture gave a liquid limit of 46 and a plasticity index of 25; therefore:

$$46 = \frac{0.50 \times 0.98 \times 61 + 0.50 \times 0.48 \times LL}{0.50 \times 0.98 + 0.50 \times 0.48}$$

giving a LL = 15; and

$$23 = \frac{0.50 \times 0.98 \times 33 + 0.50 \times 0.48 \times PI}{0.50 \times 0.98 + 0.50 \times 0.48}$$

giving a PI = 4.

CONSISTENCY PROPERTIES OF GRANULAR SOILS

The usual methods for the determination of the liquid and plastic limits cannot be applied to granular soils. However, as we have seen, mixing such soils with a reference clay permits extrapolation and calculation of the respective constants. The question arises whether values determined by this method possess and physical significance.

Increase in the percentage of sand in a soil shows itself by a decrease of its index of plasticity through narrowing of the difference between the liquid and plastic limits, signifying a decrease in the plastic zone in the behavior of the soil towards water. However, for completely sandy soils the index of plasticity does not become zero, but attains a negative value, a phenomenon that has been verified previously by Bonnenfant (1). This is the case, for example, with the sand (No. 4281; see Table 1) for which a LL = 6 and a PI = -2 were determined from a mixture with a reference clay. As shall be seen later, statistical interpretation of the relationship between the granulometry of soils and their consistency limits gives the same indication. Figure 7 shows that for $t < 0.19$ (in which $t = \frac{x}{a}$, x being the percentage of soil that passes the No. 200 sieve) the indices of plasticity are normally smaller than unity.

OBSERVATIONS CONCERNING THE CONSTANT a

It can easily be seen that the constant a, which has been introduced for the prediction of the characteristics of soil mixtures, is of more-general significance and is endowed with certain properties that are worthy of note.

In the first place, it can be proven easily that this constant is proportional to the area enclosed by the granulometric curve, by the axis y = o, and by the ordinates corresponding to the sieves No. 7 and No. 200 (British system). The constant a, therefore, possesses a certain resemblance to the concepts introduced by Abrams (8), Turnbull (9), and Burmister (10).

However, for a given soil the constant a cannot take any arbitrary value.

The constant depends, in the first place, on the percentage of fines x (passing through sieve No. 200). This is very definitely shown in Figure 2, in which the relationship between x and a is represented for 1,029 soils from Mozambique. One can see that the number of soils in which a is less than 0.5 is very limited; these soils are predominately composed of gravel.

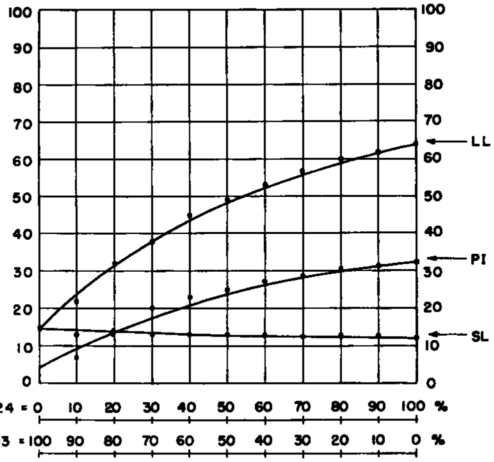

Figure 1. The L. L., P. I., and S. L. of Binary Soil Mixtures as Functions of the Component Soils.

If in place of a we consider $t = \frac{x}{a}$, we obtain the relationship between t and x shown in Figure 3 for the same 1,029 soils. The dispersion of the values is even less accentuated, and one can establish the statistical relationship:

TABLE 1

Soil No		4424	4447	6903	4281
Granulometry					
% passing sieves No					
U S	British				
8	7	100	100	100	100
16	14	99	99	92	97
30	25	99	99	59	67
50	52	99	98	20	9
100	100	98	97	10	--
200	200 (x)	96	96	7	--
<0.005 mm		70	67	--	--
<0.002 mm		59	58	--	--
<0.001 mm		52	53	--	--
liquid limit		64	61	15 (a)	6 (a)
plasticity index		33	33	4 (a)	-2 (a)
shrinkage limit		12	12	15	16
sp gravity		2.67	2.68	2.67	2.69
% organic matter		1	0	1	0
Classification					
Textural		clay	clay	sand	sand
H R B		A-7-5 (20)	A-7-6 (20)	A-1-b (0)	A-1-b (0)
Constants					
a		0.99	0.98	0.48	0.46
$t = \frac{x}{a}$		0.97	0.98	0.15	0

(a) determined from a mixture with a reference clay

$$t = 0.9x^3 - 2.1x^2 + 2.0x \quad - - - \quad (3)$$

or, with a lesser degree of accuracy:

$$t = x^3 - 2x^2 + 2x \quad - - - - - - - \quad (4)$$

Thus, the constant a determined exclusively from the ordinates of the sand fraction of the granulometric curve, appears to be directly influenced by the percentage of fines. What is the influence of the percentage of clay less than 0.002 milimeters?

Figures 4 and 5 give a definite answer to this question; for each value of the percentage of clay finer than 0.002 milimeter exists a perfectly defined minimum of a or t, and probably also a maximum. Tentatively the values can be considered as the outer limits in our respective graphs.

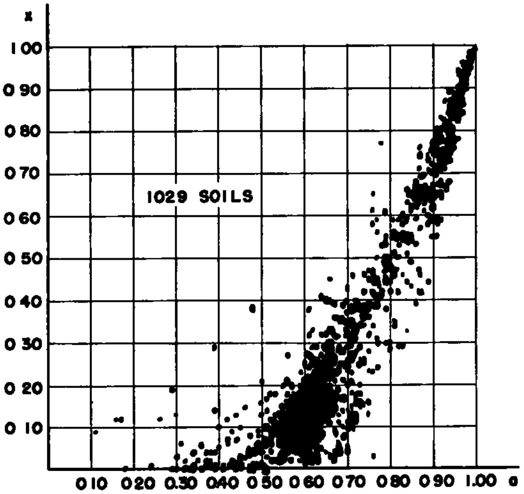

Figure 2. Relationship Between the Percentage of Fines X (-0.076 mm) and the Constant A.

So fár it is not yet possible to draw all the possible conclusions from these results; however, everything leads one to believe that in natural soils the percentages of clay and sand are not independent but are mutually related within certain limits.

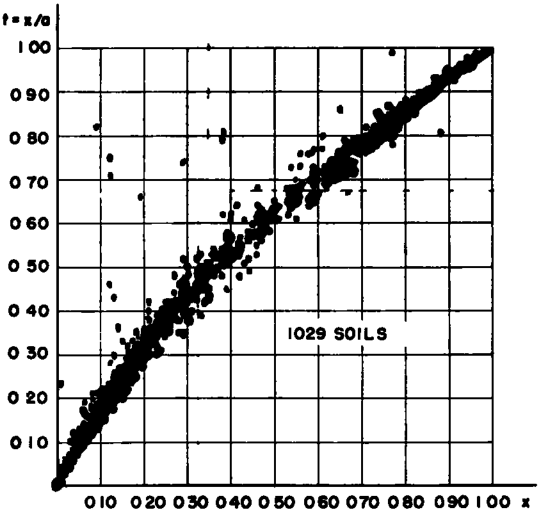

Figure 3. Relationship Between the Constants T=X/A and X.

On the other hand, as t and x are definitely related, one can predict the existence of a similar relationship between the percentage of the total fines x and the percentage of clay particles finer than 0.002 millimeters.

It appears necessary to introduce as a working hypothesis that, the granulometric curve of a natural soil cannot take an arbitrary form: on the contrary, the different fractions composing it are strictly related to each other.

CORRELATION BETWEEN THE ATTERBERG LIMITS AND THE GRANULOMETRY

The existence of Relationship 2 lets us predict that there must exist a certain relation between granulometry and the magnitude of the studied properties.

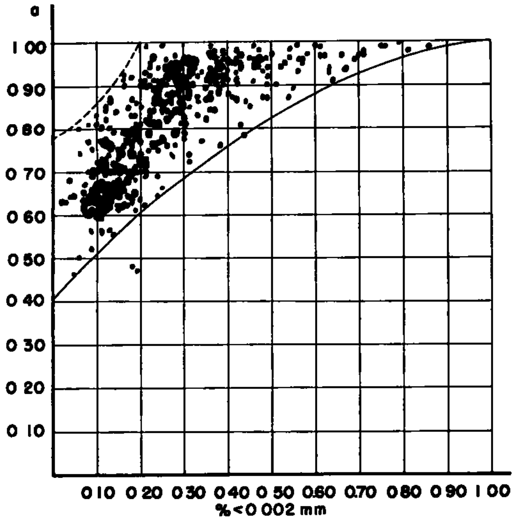

Figure 4. Correlation Between the Constant A and the Percentage of Clay (-0.002 mm).

As a matter of fact, every soil can be considered as an ideal mixture of two other soils; the latter are obtained by separating with a No. 200 sieve the fines (clay and silt) from the coarse material (sand and gravel).

This ideal mixture would be composed of x percent of fine and (1 - x) percent of coarse material. The granulometric curves for each of these ideal soils, obtained by the bipartition of the initial soil, would be completely defined. In effect if:

y_i = an arbitrary ordinate of the granulometric curve of the simple soil;

y'_i = corresponding ordinate of the fine fraction passing through No. 200 sieve;

y_i'' = corresponding ordinate of the fraction of the coarse material retained on the No. 200 sieve;

then: $y_i = xy_i' + (1 - x)\, y_i''$ - - - - - - - - (5)

Since for diameters equal to or smaller than the openings of the No. 200 sieve: y_i'' = 1, and for diameters equal to or larger than the openings of the No. 200 sieve: y_i = 0, we obtain:

$$y_i' = \frac{y_i}{x} \quad \text{and} \quad y_i'' = \frac{y_i - x}{1 - x}$$

Since $\underline{a}$ is the granulometric constant of the simple soil and $\underline{a_1}$ and $\underline{a_2}$ are the corresponding constants of the ideal component soils, we obtain:

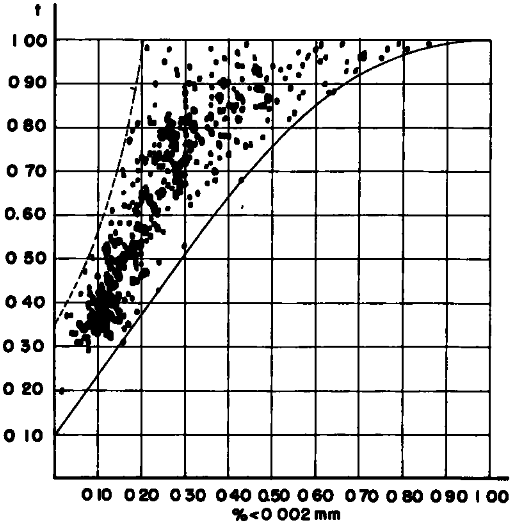

Figure 5. Correlation Between T=X/A and the Percentage of Clay (-0.002 mm).

$$a = xa_1 + (1 - x)\, a_2 \text{ - - - - - - - - } (8)$$

with: $a_1 = 1$ - - - - - - - - - - - - - - - - (9)

$$a_2 = \frac{a - x}{1 - x} \text{ - - - - - - - - - - - - } (10)$$

or: $a = x + 1\,(1 - x)\, a_2$ - - - - - - - (11)

From (2) results:

$$K = \frac{xK_1 + (1 - x)\dfrac{a - x}{1 - x} K_2}{a} = K_2 + (K_1 - K_2)\frac{x}{a} = K_2 + (K_1 - K_2)\, t$$

$$= K_2 + f.t. \text{ - - - - - - - - - - } (12)$$

$\underline{f}$ being a function of unknown variables.

Actually, for the cases already studied, and respective to the liquid limits and plasticity indexes (and also the plastic limits) Equation 12 takes the form of:

$$K = b_1 + b_2\, t + b_3\, t^2 \text{ - - - - - - - } (13)$$

Figures 6 and 7 show exactly this relationship for 787 Mozambique soils. The constants of these curves, viz:

$$LL = 0.067 + 0.303t + 0.309t^2 \text{ - - } (14)$$

$$PI = 0.026 + 0.097t + 0.240t^2 \text{ - - } (15)$$

have been previously adjusted by the method of the least squares for presentation at the Soil Mechanics Congress at Zürich in 1953 (5) for 353 different soils. For these a factor of correlation R = 0.988 for the liquid limits and an R = 0.983 for the plastic limits had been obtained. This correlation factor is defined by

$$R = 1 - \frac{U}{n\,\sigma^2}$$

in which $\underline{U}$ is the sum of the squares of the residues, n the number of observations and σ the standard deviation.

From Equation 13 one obtains:

$$LL \text{ or } PI = b_1 + b_2\frac{x}{a} + b_3\frac{x^2}{a^2}$$

$$= b_1 + b_2\frac{A_1 x_1 + A_2 x_2}{A_1 a_1 + A_2 a_2} + b_3\frac{(A_1 x_1 + A_2 x_2)^2}{(A_1 a_1 + A_2 a_2)^2}$$

$$= \frac{A_1 a_1 \left(b_1 + b_2\dfrac{x_1}{a_1} + b_3\dfrac{x_1 x}{a_1 a}\right) + A_2 a_2 \left(b_1 + b_2\dfrac{x_2}{a_2} + b_3\dfrac{x_2 x}{a_2 a}\right)}{A_1 a_1 + A_2 a_2}$$

This equation would coincide with Equation 2 if one took

$\dfrac{x_1^2}{a_1^2}$ in place of $\dfrac{x_1 x}{a_1 x}$ and $\dfrac{x_2^2}{a_2^2}$ in place of $\dfrac{x_2 x}{a_2 a}$.

The difference

$$\Delta = \frac{b_3}{a}\Big[A_1 x_1 (t - t_1) + A_2 x_2 (t - t_2)\Big] \text{ - } (16)$$

is not appreciable for practical purposes.

The dispersion of the results indicates the influence of factors other than the granulometry. A large-scale investigation is presently being conducted in our lab-

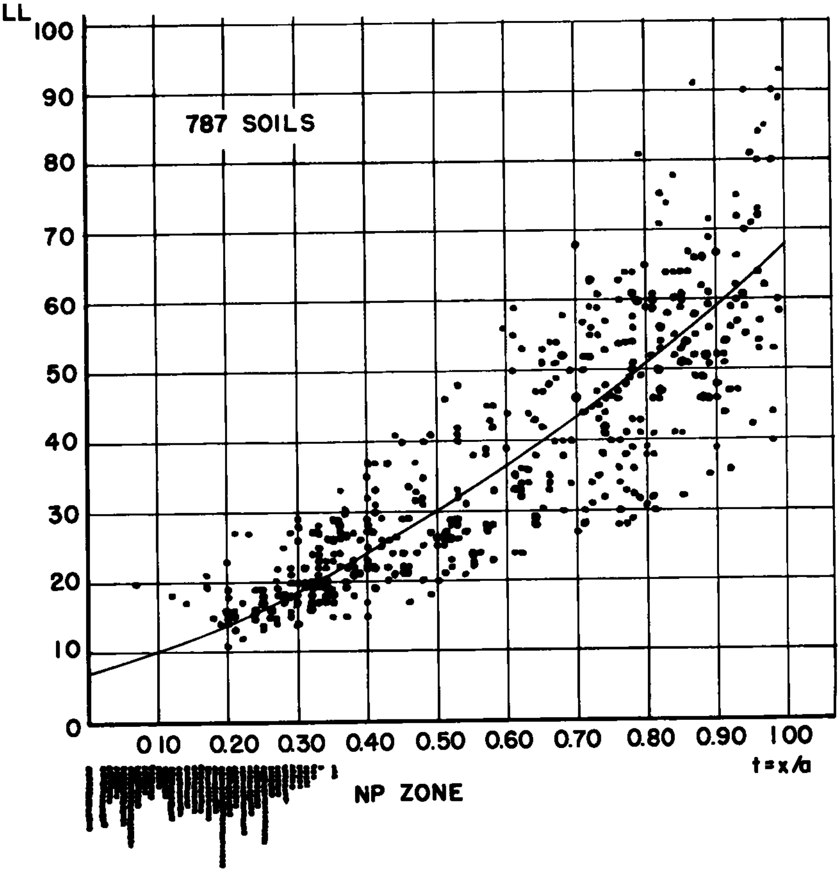

Figure 6. Correlation Between the Liquid Limit and T.

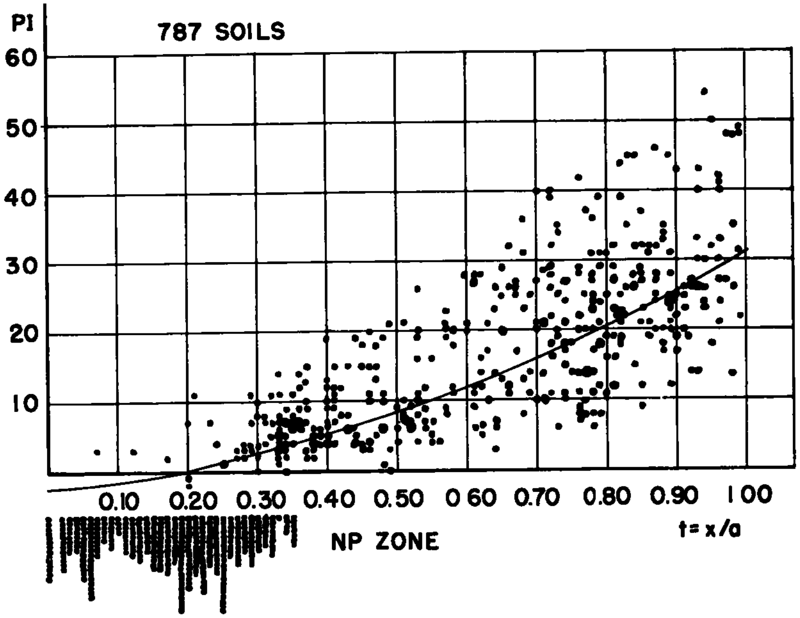

Figure 7. Correlation Between the Plasticity Index and T.

oratory for the purpose of studying the influence on the Atterberg limits of: (1) mineralogical nature of the clay fraction; (2) percentage of colloids; (3) pH; (4) organic material; (5) calcium carbonate; (6) base exchange capacity; and (7) R_2O_3; CaO; MgO; Fe_2O_3; Al_2O_3; Na^+; K^+.

One must, of course, realize that many of the Mozambique soils that were examined possess lateritic properties. This aspect has been taken into due account. The mineralogical identification of the clay fractions is made by simultaneous use of differential thermal analysis, direct observation under the electron microscope, and electron diffraction. The results already obtained in this area will be published in the near future.

In the appreciation of Equations 14 and 15, one must keep in mind that the coefficients were determined exclusively on Mozambique soils. It would, therefore, be interesting to apply them to the soils of varied origin which were listed by Lambe and Martin (11). Unfortunately these authors give as granulometric data only the percentages smaller than 0.07 and 0.002 mm. Considering the former as corresponding to x and using the graph in Figure 3, we determined t. In this determination the most-favorable value of t compatible with the data of the graph was employed.

TABLE 2

Sample No.	Soil	x -0 07 mm	t	Liquid Limit Determined	 Calculated	 Deviation	Plasticity Index Determined	 Calculated	 Deviation	Composition of Silt and Clay portion in percent by weight
38	Tan silty clay	0 41	0 49	29	29	0	11	8	3	Quartz = 50, Illite = 20, Kaolinite = 15, Gibbsite = Trace O M = 0 5
5	Marl	0 45	0 50	28	30	-2	8	8	0	Calcite = 75 Montmorillonite = 10
28	Gray boulder	0 44	0 60	34	36	-2	15	12	+3	Quartz = 20, Fe_2O_3 = 0 5 (Illite = 60), (Mica = 25), O M = 1 7
30	Brown boulder clay	0 28	0 35	19	21	-2	7	4	+3	Quartz = 45, Fe_2O_3 = 0 6 (Illite = 40), (Chlorite = 10 +),
19	Clay	0 97	0 96	68	65	+3	22	28	-6	Quartz = 15, Montmorillonite = 29 Carbonate minerals = 10, Fe_2O_3 = 4 0 Illite = 26, O M = 0 9
23	Gray clay	0 98	0 97	62	65	-3	25	29	-4	Montmorillonoid = 45 Illite = 25 O M = 5 6
16	Red clay	0 86	0 93	66	62	+4	42	27	+15	(Illite = 50) (Montimorillonite = 15) Fe_2O_3 = 3 8, Gibbsite = 5
21	Red clay	0 83	0 94	58	62	-4	38	28	+10	Quartz = 40, Fe_2O_3 = 2 3 Illite = 30 Montmorillonite = 30
24	Brown sandy silt	0 55	0 68	30	36	-6	10	12	-2	Quartz = 25, Fe_2O_3 = 3 1 (Montmorillonoid = 50) (Illite = 10), Dolomite = 8
32	Gravelly sandy clay	0 43	0 48	19	26	-7	9	8	+1	Carbonate minerals = 45 (Illite = 20), Chlorite = 30 (Mica = 3), Fe_2O_3 = 0 7
40	Tan sandy silt	0 86	0 87	44	57	-7	15	24	-9	Quartz = trace, Illite = 4, Calcite = 50, (Montmorillonoid = 6) Dolomite = 20, Attapulgite = 13,
6	Grayish white silty clay	0 97	0 96	57	65	-8	22	29	-7	Kaolinite = 90
26	Dark brown clay	0 79	0 88	69	58	+11	39	25	+14	Magnesite = 5, (Illite = trace) Fe_2O_3 = 4 5, O M = 0 7 (Montmorillonoid = 70)
37	Red and gray clay	0 78	0 78	35	49	-14	15	19	-4	Quartz = 50, Fe_2O_3 = 4 2, Kaolinite = 25, Illite = 25
11	Yellowish tan silt	0 73	0 75	31	47	-16	8	18	-8	Quartz = 5, Calcite = 90 Montmorillonoid = 5
36	Red sandy clay	0.65	0 69	23	42	-19	9	15	-6	Quartz = 50, O M = 2, Dolomite = 15, Fe_2O_3 = 0.9, (Illite = 20), Vermiculite = 5
29	Gray clay	0 82	0 83	33	54	-21	15	22	-7	Quartz = 30, Fe_2O_3 = 1 9 (Illite = 30) (Chlorite = 5+) O M = 0 4
12	Dark gray shale	0 70	0 83	75	53	+22	29	22	+7	Feldspar (clay sizes) = 10 (Illite = 13), Montmorillonoid = 12) O M = 2 7, Fe_2O_3 = 1 4, Dolomite = 10
18	Clay	0.92	0.92	37	61	-24	13	26	-13	Quartz = trace (Illite = 13), Carbonate minerals = 15 O.M. = 2 7, (Montmorillonoid = 12) Fe_2O_3 = 1.4
31	Gray clay	0 95	0 94	37	63	-26	18	27	-9	Quartz = 35 O M = 0 7, Illite = 45 Fe_2O_3 = 1 6, Chlorite = 15+
10	Stratified clay	0 85	0 86	28	55	-27	10	23	-13	Quartz = 35, Illite = 5-15, Fe_2O_3 = 1 5
35	Brown silt	0 90	0 90	32	59	-27	6	25	-19	Quartz = 35, (Vermiculite = 27) Illite = 20, O M = 1.2, Fe_2O_3 = 1 3
34	Gray silt	0 95	0 94	25	63	-38	6	27	-21	Quartz = 30, (Illite = 25) Calcite = 10, Swelling chlorite = 15, Fe_2O_3 = 0.9
27	Yellow clay	0 81	0 90	99	59	+40	71	26	+45	Montmorillonoid = 70, Illite = 5 Fe_2O_3 = 2 3, Quartz = trace
9	Brown silt	0 97	0 96	24	65	-41	4	29	-25	Dolomite = 35, Quartz = 30 Illite = 15, O M = 1 3, Fe_2O_3 = 1 5
39	Light brown clay	0 55	0 76	100	48	+52	66	29	+37	Quartz = 15, Attapulgite = 12 Calcite = 60, Illite = 12, O M. = 0 9

Table 2 presents the data obtained, in order of increasing deviations between the experimental and calculated liquid limit values.

Of the 23 soils considered, 11 gave liquid limits with deviations of less than 10 between the experimental and calculated values; 16 gave plasticity indices also with deviations smaller than 10.

OTHER APPLICATIONS TO GRANULAR STABILIZATION

Very probably the specifications rela-

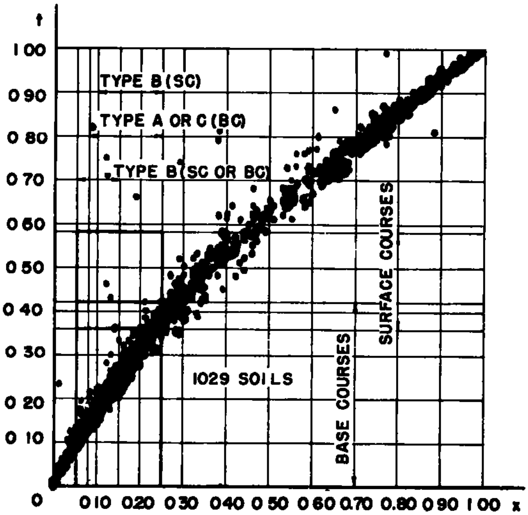

Figure 8. Plot of T Versus X Applied to Granular Soil Stabilization.

ting to granular soil stabilization (such as those of ASTM, which are based on the establishment of limiting values for the granulometry, the percentage passing the No. 200 sieve, the liquid limit and plasticity index, contain restrictions that are more than sufficient.

At the present state of our work, there appears to be good reason for the hope that rational justification may be developed for these empirical specifications.

This possibility was pointed out in previous work of the author (5) in which tentative diagrams were presented, based on values of x and t, for relating soils to the most suitable type of stabilization.

In Figure 8, another type of diagram is presented, based on our values of $t = \frac{x}{a}$ and x, in which are plotted data on the 1,029 soils employed in Figure 3. The extreme values for t were deduced from Equations 14 and 15 for the liquid limit and plasticity index values specified by ASTM. It is apparent that only a relatively limited number of soils satisfy these specifications.

ACKNOWLEDGMENT

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Grateful acknowledgment is made to Hans F. Winterkorn for reviewing this paper and for translating it from the Portugese.

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Exchange Adsorption by Clays of Large Organic Cations

J. E. GIESEKING, Professor of Soil Physics,
Department of Agronomy, University of Illinois

Introductory Remarks by the Chairman: Scientific study of the exchange adsorption on clays of large organic cations goes back to the early thirties of this century. The results of these studies are presently employed with great success in the manufacture of paints, lacquers, stains and lubricants to give superior consistency properties of these materials and improved resistance of the protective films formed by them.

It should be a source of satisfaction to the highway engineer that his own study and practical application of the exchange reactions of large organic cations also go back to the early thirties and that present limitations in general use are due not to lack of success but to the economic factor of lower unit prices of stabilized soils and other structural materials as compared with those of paints, lacquers, and greases.

Gieseking, a pioneer in the study and practical application of these reactions, has prepared an authoritative and precise paper on this subject which promises to become of increasing importance in the field of soil stabilization.

● Clays and clayey soils having high cation-exchange capacities also have high water-adsorbing capacities. These clayey materials normally become gelatinous when associated with large amounts of water. They retard the flow of water through soil materials, they swell when they become wet, they shrink upon drying, and when wet, they tend to flow and shear when stresses are applied to them.

The above properties are useful in a few engineering applications, but there are many engineering applications where the above properties would make a material unfit for use. Saturation of clays with large organic cations destroys the forces that attract water molecules and may possibly be a means of ameliorating clays for certain types of engineering uses.

ALTERATION OF THE PROPERTIES OF CLAYS BY ORGANIC CATION ADSORPTION

The clays in natural soil materials are normally saturated with exchangeable calcium, magnesium, hydrogen, or sodium or any combination of these cations. These cations can be exchanged by any other cation. Such reactions are equilibria reactions which can be forced to completion by any of the chemical methods used in forcing equilibria reactions to completion.

Cernescu (1) was the first to show that the exchangeable calcium in Grossalmeroder clay could be exchanged by large organic cations. In 1933 Gieseking (4) noted that a number of large colored cations (positive dyes) were strongly adsorbed by clays and that clays saturated with these cations no longer associated with water to form gels.

During the time these organic cationic clay complexes were being prepared, Winterkorn (12) demonstrated to the author that clayey soils saturated with large organic cations associated readily with asphaltic materials to give soil materials which could be formed in strong, somewhat resilient masses which were resistant to the attack of water. The observations of Winterkorn were in line with the observations of Harkins (7) in which he showed that "like dissolves like."

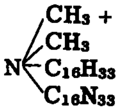
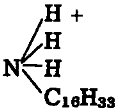
Gieseking and Jenny (6), Gieseking (5), Hendricks (8), Jordan (9, 10) and Jordan, Hook, and Findlayson (11) have presented sufficient data to show that any soluble organic cationic compound can be expected to be strongly adsorbed by clays by exchanging the organic cation for the inorganic cations on the clays. Jordan and his associates have published many data to show that clays saturated with organic cations have hydrophobic (water-hating) yet organophilic (organic-loving) complexes.

The cheapest and by far the most-common organic cationic compounds are the amines. The amine cations are ammonium ions in which one, two, three, or four of the hydrogens of the NH4 ion have been replaced by organic radicals. The radicals may be single carbon methyl radicals, they may be (within the limits of solubility, geometric or steric compatibility, and reactivity) composed of long chains of carbon atoms, or they may be composed of various combinations of phenyl, benzyl, or naphthyl rings.

The analogy of the exchange between a natural sodium clay and the ammonium ion and the completely substituted organic ammonium ion can be shown by the following reactions:

$$\text{Na Clay} + NH_4Cl \longrightarrow NH_4\ \text{Clay} + NaCl$$

$$\text{Na Clay} + NR_4Cl \longrightarrow NR_4\ \text{Clay} + NaCl$$

Jordan (10) has shown that the more completely the clay particles are covered with organic cationic material the more organophilic the clay becomes. He has calculated that  $N\begin{matrix}CH_3 +\\ CH_3\\ C_{16}H_{33}\\ C_{16}N_{33}\end{matrix}$ gives 40 percent in excess of the amount of organic material necessary to completely cover the surfaces of Wyoming bentonite crystals when saturated with this organic ammonium ion. At saturation with  $N\begin{matrix}H +\\ H\\ H\\ C_{16}H_{33}\end{matrix}$ his calculations show only 72 percent of the surface covered with organic material.

Jordan and associates (11) have shown that, when the surfaces of Wyoming bentonite are completely covered by large adsorbed organic cations, the bentonite readily forms gels with toluene a nonpolar liquid. Other observations showed that bentonite incompletely covered with organic cations requires the addition of a small amount of a polar substance, such as an alcohol, to the toluene before good gelation is obtained. Similar systems gave poor gelation when large amounts of alcohol were added to the toluene. These observations show that organic ammonium bentonites can be prepared that are extremely organophilic.

Winterkorn (13) has shown that a number of soils were stabilized against the effects of alternate wetting and drying and alternate freezing and thawing by treating the soils with a mixture of 70 parts aniline to 30 parts furfural. Under the weakly acid conditions which he used, it would be expected that his postulation of the formation of a Stenhouse dye is correct. Since the Stenhouse dye tends to act like a divalent cation in acid solutions, Winterkorn's 6-percent treatments should have been sufficient to saturate all of the soils which he used with this organic cation and the 2-percent treatments should have been sufficient to saturate all of the soils except those with the very highest cation-exchange capacities.

The author has found that Wyoming bentonite, nontronite and Ipava clay, when saturated with various organic cations do not adsorb as much hygroscopic moisture (25 C. and 96 percent relative humidity) as the corresponding natural clays. These clays are montmorillonitic clays. The results are shown in Table 1.

TABLE 1

HYGROSCOPIC MOISTURE ADSORBED BY UNTREATED AND ORGANIC AMMONIUM TREATED WYOMING BENTONITE, NONTRONITE, AND IPAVA CALY. STORED AT 25°C. AT 96% RELATIVE HUMIDITY FOR 10 DAYS.

Treatment*	Wyoming bentonite	Nontronite	Ipava clay
	% H_2O	% H_2O	% H_2O
None	20.2	19.3	8.5
Aniline	9.5		6.2
Pyridine	10.4	11.2	5.9
B-naphthylamine	4.0	8.2	6.1
Brucine	12.9		5.7
Methylene Blue	9.2		
Malachite Green	7.1		
Janus Green	6.6		
Magdala Red	5.9		

* All of the organic substituted cations were used in association with chloride anoins except magdala red which was used in association with acetate anoins.

The differences between the treated and untreated Ipava clay are not as striking as those for Wyoming bentonite and nontronite. This is true because the natural Ipava clay contains organic materials and is already partly saturated with organic cations formed during the decay of nitrogenous vegetative materials.

THE RESISTANCE OF CLAY ADSORBED ORGANIC AMMONIUM CATIONS TOWARD MICROBIOLOGICAL DECOMPOSITION

If the organic ammonium cations are to be used in ameliorating clays for engineering uses, they must resist microbiological decay for a reasonable time. No definite answer is available to this problem, but the work of Ensminger and Gieseking (2) and Erickson (3) indicate that clay-adsorbed cations would be expected to be more resistant toward decomposition than the unadsorbed cations. Ensminger and Gieseking found that clay-adsorbed proteins were more resistant to enzymatic hydrolysis than unadsorbed proteins.

Erickson found that clay-adsorbed amino acids are more resistant to enzymatic and chemical hydrolysis, oxidation, deaminization, and decarboxylization than unadsorbed amino acids. The proteins and amino acids are organic ammonium ions. They are easily hydrolyzed and decomposed by the enzyme systems of microörganisms when not protected in some way.

The studies of Ensminger and Gieseking and Erickson indicate that other, more-stable organic ammonium cations may possibly be still more resistant to decomposition when adsorbed on clays.

COST OF SATURATING CLAYS WITH ORGANIC AMMONIUM CATIONS

Aniline hydrochloride was quoted in the December, 1953, markets at 22 cents a pound. A clayey soil with a cation-exchange capacity of 25 milligram equivalents per 100 grams, or 25 millipound equivalents per 100 lbs. of soil requires 65 lbs. of aniline per ton to saturate the clay with this organic ammonium cation. This amounts to approximately $14 per cubic yard. Dimethyldioctadecyl ammonium chloride, which Jordan found to be most effective in "waterproofing" clays, was quoted at 37 cents (75 percent purity) in December of 1953.

Since dimethyldioctadecyl ammonium chloride is a larger molecule, and allowing for impurities, calculations show that it takes six times as much of this compound as aniline hydrochloride to saturate a clay. From this we find that it costs approximately $77 to saturate a cubic yard of soil with dimethyldioctadecyl ammonium chloride, if the soil has a cation-exchange capacity of 25 milliequivalents. Soils with lower exchange capacities would naturally require proportionately smaller amounts of the organic ammonium salts to saturate them.

Winterkorn's results (13) suggest that only a fraction of the exchange capacity would need to be occupied by organic ammonium cations in order to render soils more easily stabilized with asphaltic materials. If this should be true, the cost of the combination might possibly be within practical limits, especially in cases where other materials for construction were not available.

PRESENT USES OF ORGANIC AMMONIUM CLAY COMPLEXES IN INDUSTRY

Manufacturers of paints, lacquers, and stains are using the organophilic dimethyldioctadecyl ammonium bentonite to inhibit settling of pigments and to give a tougher and more-resistant film after the paint has dried. Lubricant manufacturers are using this same organic cation: bentonite complex to thicken greases and to prevent the melting of greases at high temperatures. Asphalt manufacturers are adding this compound to asphaltic materials to make their product resistant to summer heat.

CONCLUSIONS

It is evident from the results reported by Jordan and associates, Winterkorn, and Gieseking that organic-ammonium-saturated clays are more hydrophobic than natural clays. Jordan has shown that these complexes are more organophilic than the natural clays. Winterkorn has shown that aniline-treated soils are more stable toward alternate wetting and drying and alternate freezing and thawing than the corresponding untreated soils.

Saturation of natural clays with organic cations is an expensive operation. It appears that the presence of only small amounts of organic cations on the clay surfaces might prove to be beneficial in certain soil-stabilizing operations. Further studies should be made to show the effects of various degrees of organic-cation saturation on the clay-water relationships of a number of clayey soils. Prohibitive costs might be avoided by these treatments if only a low degree of saturation with the organic cation were required. The organic ammonium clay complexes are already in widespread usage in high unit cost materials such as paints, lubricants, and asphaltic materials.

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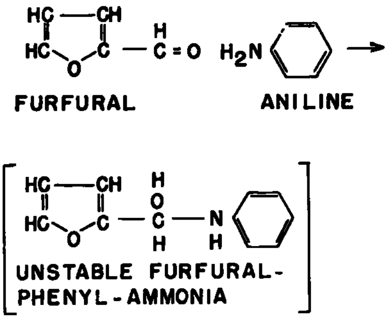
Discussion

ERNST H. GOEBEL on leave Technische Hochschule, Munchen, Germany, at present Visiting Scholar, Princeton University, Princeton, New Jersey — Complementing Gieseking's contribution to this symposium it may be of interest to discuss certain features of the aniline-furfural method, in which the principle of stabilization by means of large organic cations is combined with that of synthesis in situ of artificial resins. As exemplified by the Navy's beach-stabilization method, this is so far the only method of stabilization with synthetic resins that has proven itself in large scale operations (1) and possesses the advantage of relatively low cost of the materials employed.

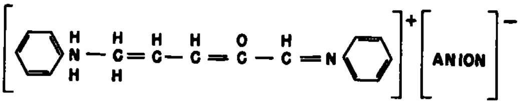
As Geiseking pointed out, the Winterkorn method of stabilizing soils with aniline-furfural seems to be largely directed to the use of a mol ratio of 2 aniline to 1 furfural i. e., 2 to 1, which is approximately 70 percent to 30 percent by weight. Under neutral conditions it can be expected that one aniline molecule reacts with one molecule of furfural to give a furfuraldehyde-ammonia type of compound of the formula:



This compound contains an -OH- group close to a very-active N atom which has the tendency to fill its electron shell to an octet by assembling electron donors. The influence of this nitrogen is so strong that even the neighboring alcoholic -OH- group reacts acidic. It is obvious that an unstable compound like this has a great tendency toward polymerization. In this case even the furfural ring is opened during the formation of the resin.

As this reaction requires just one molecule of each constituent, the weight ratio of the two reactants is very closely 1 to 1, (the molecular weight of aniline being 93, and that of furfural 96). This means that with a percentage ratio of 70 to 30, about half of the aniline would be free to be used as a large organic cation for exchange reaction with the soil exchange complex.

On the other hand, Winterkorn's work was concerned mostly with naturally acid or acidified soils and beach sands. Winterkorn states that according to literature and to his own experiments it is probable that using the 2-to-1-aniline-furfural ratio, the following molecule results:



The structure of this molecule indicates, on one hand, the cationic nature of the substance which fills Geiseking's demand for large organic cations that are able to replace natural exchange cations in the soil. On the other hand, it is also obvious that a polar compound of this type tends to polymerize due to its large number of double bonds, since double bonds are always a sign of great chemical activity because of their unsaturated character.

It may be of interest that this compound contains even a vinyl-alcohol group - $\overset{H}{C}$ =
H
O - which is known as one of the most-
C
reactive groups for modern plastics.

From the point of view of the very-strong trend of aniline-furfural towards condensation and polymerization it seems to be more likely that in a mixture of 70 to 30 (aniline to furfural) the 2-to-1 compound is actually formed instead of the 1-to-1 furfuraldehyde-aniline compound. The 2-to-1 compound is, of course, a large organic cation and acts in accordance with Gieseking's explanation of the effect of large organic cations on the water affinity of soils. Consequently, a compound of the above-mentioned character is able to accomplish two principle purposes: (1) giving the soil particles water repellant properties by base exchange and (2) cementing the particles together and giving to the system strength by polymerization.

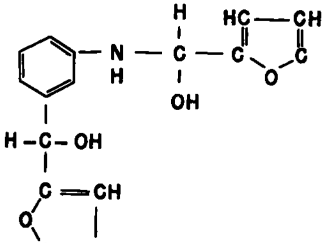
It must be mentioned that the 70-to-30 combination in amounts from 2 to 6 percent by weight of the soil had given good results with respect to compressive strength and weathering resistance, of soils treated with it (2).

As aniline is higher in price than furfural and will probably always be more important as a valuable material in other chemical branches, it must be considered as desirable to reduce its proportion to the lowest possible limit. Of special interest in this connection is the work of A. Tager (3), who extended the use of the aniline-furfural method to alkaline-reacting soils. He obtained good results in stabilizing soils with a combination of 1 percent of 3-to-2 aniline-furfural and 1 percent of hydrated lime. This shows the ability of furfural to react in various proportions with aniline and other aromatic amines at different hydrogen-ion concentrations.

Furthermore, Winterkorn had already reported (2) that besides the maximum strength reached with an aniline-furfural ratio of 70 to 30, reasonable compressive strength is obtained already with an aniline-furfural ratio of 30 to 70, and that there probably exists another strength maximum in the range of about 30 to 50 percent of aniline, provided that a proper amount of catalyst is used. This catalyst should influence first of all the pH value of the system.

As results of Winterkorn's work show, addition of a certain amount of acid increases enormously the compressive strength of soils mixed with two percent of a 35-to-65 (aniline-to-furfural) combination. Though this catalytic effect is not uniformly observed for all the different ratios of aniline to furfural, it may be of special importance within the range of 35 percent to 65 percent of aniline.

A ratio of 35 to 65 of aniline and furfural in weight percent approximately equals a molecular ratio of 1 to 2. The structure of a chemical compound with such a molecular ratio of the two constituents is probably the following:



This compound must also be considered as an intermediate product, tending to form spontaneously resins of higher molecular weight. The analogy with phenol-formaldehyde condensation is obvious.

Although this 1-to-2-aniline-furfural compound has not given optimum results as a stabilizer, according to Winterkorn this reaction deserves further study.

The excellent results obtained by Tager with aniline-furfural-hydrated-lime combinations is another consequence of the variegated reactivity and compatibility of aniline-furfural. Similar results have been found in the case of combinations with other inorganic cementing materials (4). Of course, the excellent compatibility of aniline-furfural resins with other organic cementing and waterproofing agents has been well known for a long time. It may be recalled that almost 20 years ago Winterkorn developed a most-effective method of improving the adhesion of bitumen to hydrophilic aggregate by means of small admixtures (about 1 percent by weight of the bitumen) of aniline-furfural (5).

References

1. Kravath, Fred F., Commander CEC, U. S. N. "Stabilization of Beach Sand". The Military Engineer, Vol. XLVI, No. 310, pp. 128-131 (1954).

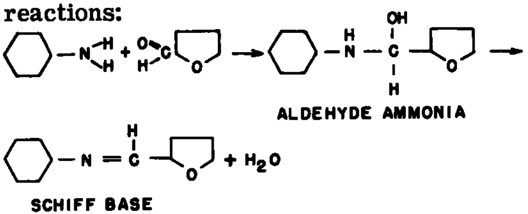
2. Winterkorn, Hans F., "A Laboratory Study of the Soil Stabilizing Effectiveness of Artificial Resins with Special Emphasis on the Aniline-Furfural Resins". Techn. Dev. Note No. 41, Civil Aeronautics Administration, Washington 25, D. C. (1946).

3. Tager, A., Doctor Thesis on Soil Stabilization, Paris (1951).

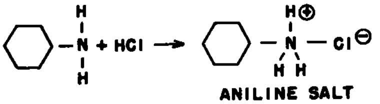
4. Winterkorn, Hans F., "Desarrollo del Metodo Anilina-Furfural para la Estabilizacion de Playas". Bulletin No. 2 Winterkorn Road Research Institute, Princeton, N. J. (1954).

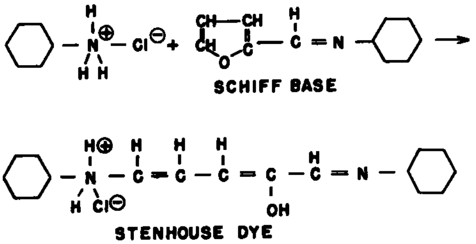
5. Winterkorn, Hans F., "Affinity of Hydrophilic Aggregate for Asphaltic Bitumen. Use of Furfural and its Resinous Derivatives for Improving Affinity". Ind. and Engineering Chemistry, 30, pp. 1362-1368 (1938).

J. E. GIESEKING, Closure — Furfural reacts with aniline in the absence of acids to give a Schiff base according to the following reactions:

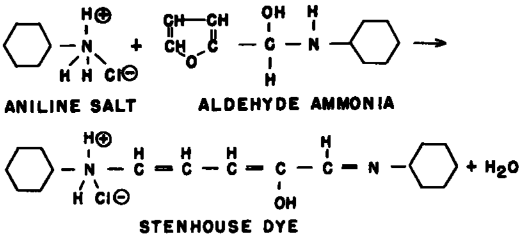


If enough acid is present to change the aniline completely to a salt, the above reactions will not take place. If a smaller amount of acid is present in the system the aniline salt will react with the above Schiff base as follows:





Possibly we should go back to the reactive aldehyde ammonia and write the reaction for the formation of the Stenhouse dye as follows:



This Stenhouse dye is the compound which Winterkorn suggested as the active agent in his soil stabilization process. He worked at the proper hydrogen-ion concentration to get a high yield of this compound. It should be remembered that this ion will tend to act as a divalent ion in acid solutions.

The Stenhouse dyes contain a conjugated system of double bonds and are, therefore, subject to further polymerization.

E. H. Goebel mentioned Tager's work on the stabilization of soils with aniline and furfural in the presence of calcium hydroxide. The reactions involved in Tager's work must be polymerizations of a type differing from those in acid solutions. However, the Schiff base mentioned in the first part of this letter would be expected to form in Tager's process.

Furfural enters into many different types of reactions. One wonders if anyone is able to comprehend all of the possibilities and all of the conditions that might tend to alter the courses of the reactions. We can be sure, however, that some of the products of the reaction between furfural and aniline will be large organic cations.

Microbiological Factors in Soil Stabilization

P. C. T. JONES, Rothamsted Experimental Station,
Harpenden, Hertfordshire, England

Introductory Remarks by the Chairman: It is a maxim in the carbon economy of the biosphere that all carbon compounds formed must be destructible in order to avoid carbon starvation and to maintain life as we know it. A great role in this destruction is played by bacteria and fungi. During the last 20 years the probable contribution by microörganisms to destruction under certain conditions of stabilized soils and of other construction materials containing organic matter has been mentioned a number of times in the engineering literature. But this awakened little interest and less reaction among highway engineers.

The problem became acute, however, during World War II, when airfields successfully waterproofed by means of small amounts of natural resin derivatives returned to their previous unstabilized condition within relatively short periods of time. It was on that occasion that the Road Research Laboratory in Great Britain sponsored a thorough study of this problem at the famous Rothamstead Agricultural Experiment Station. The man entrusted with this investigation was Jones, who did a beautiful job in identifying and isolating the microbial culprits and in exposing their working methods.

Jones, who has long since left the field of soil stabilization research, has taken time out from pressing and important medical research to give us an account of the important role microbiology plays in certain types of soil stabilization.

● IN recent years, interest in the problem of soil stabilization has arisen in two seperate contexts: (1) on account of the intimate relation between soil structure and the important problem of erosion (and in relation to the water and nutrient supply to the living plant) and (2) on account of its relationship to civil-engineering problems of road construction.

In both these cases, variations in the microflora (and microfauna) of the soil will occur and will be of particular importance when foreign readily decomposable materials (such as the resins currently used) are added. The use of cohesive soil materials for construction purposes is of considerable antiquity. Adobe construction is still widely used in the world, even at the present, and (as archaeological findings disclose) was of yet greater importance in times past.

One important feature of the soil and mud materials used was soon established: under wet conditions, mechanical strength is soon lost. Cohesive soils will carry a heavy load as long as they are dry, but when they become wet, the individual soil particles become surrounded by a water film which acts as a lubricant between the platy surfaces of the soil clays and so renders them susceptible to shearing motion, and the soil degenerates into mud.

If the pressure applied to such a soil by a heavy load is spread over a large area, as with the concrete mattress frequently used in road construction, sufficient support is present. Otherwise, depression of that area will occur on account of the plasticity of the soil.

We are all familiar (in Conan Doyle at least) with the bog of legend, which when dry will support a heavy load, but when wet offers a tempting hazard to the novelist and the unsuspecting. This, of course, is an extreme case of the phenomenon of clay lubrication as outlined above.

Again, it is apparent that such soils, which collapse under conditions of high precipitation are of little agricultural value, as well as being readily eroded. If, however, some means are found for rendering these soils water-stable, the solution of both these problems becomes possible.

The stability of natural soils of good structure is maintained by the cohesive action of the so-called humic acids (34), and a greatly enhanced stability can be achieved by the addition of natural or synthetic resins.

It is important here, however, to distinguish between agricultural and civil-engineering requirements. In road construction the object of stabilization is to prevent, as far as possible, the movement of water within the soil once it has been compacted. However, in agricultural practice, the object of such immobilization is conservative and the maintenance of a structure that allows sufficient aeration for the respiring roots of the living plant. Thus, materials such as gum rosin and Vinsol, which can be used for road- or airstrip-stabilization are unsuitable for agricultural purposes; whereas materials such as Krilium are admirable for the latter purpose.

In road construction the soil foundation must be maintained at some relatively constant moisture content at which the soil possesses adequate mechanical strength (stability). This is achieved by various means, such as cementing the soil particles together by binders of some sort, or maintaining the natural cohesion of the soil by waterproofing it.

Substances capable of effecting these ends are known. In the first case, cement and bitumen are well known, familiar in the case of large aggregates as concrete and macadam respectively; and in the latter case, recent research has indicated the use of various compounds, generally possessing a free polar group attached to another, non-polar in character, which have the property of minimizing the tendency of the soil moisture to increase under wet conditions.

One group of these includes certain natural resins, effective when added to the soil in powder form in quantities of the order of 1 percent by weight. In soil-microbiological studies, it almost goes without saying that whatever material one adds to a soil under natural conditions there will be present among the tremendous diversity of soil microörganisms some that will be capable of attacking and decomposing the material added. Indeed, were this not so, the surface of our globe would be much more encumbered with the dead bodies and waste products of past creatures. Indeed, it is only the casts, calcareous or silicious skeletons or the more resistant lignin-like materials that remain.

It is to this process of ubiquitous decomposition that we owe our coalfields and, in all probability, our oilfields. It is almost a sine qua non that energy-rich materials, such as gum rosin, will be decomposed; and at an early stage it was suspected that they might be susceptible to microbial attack in the soil.

REVIEW OF LITERATURE

Although little work is extant on the microbial attack on natural or synthetic resins, a considerable literature exists on the decomposition of chemically related compounds. The decomposition of hydrocarbons, both aliphatic and aromatic, has been extensively studied. If the views on biogenesis offered by the Russian Biochemist Oparin are accepted (23), it would appear that the ability to metabolize this type of compound is primitive indeed. From his reasoning, based on stellar and planetary spectra and on petrological examination, it would appear that compounds of this type were widespread in his suggested original anaerobic environment.

The available literature on hydrocarbons is well reviewed by ZoBell (41) and by Bushness and Haas (4). Microbial attack on methane was first demonstrated by Söhngen (27). Aiyer (1) demonstrated the use of this hydrocarbon in swamp water by fluorescent rods. Söhngen (28) showed that mycobacteria and pseudomonads were capable of metabolizing gasoline, kerosene, and paraffin wax; while Baldwin (3) showed that the soil flora was modified by the addition of crude petroleum. Büttner (5) used paraffin wax to isolate saprophytic mycobacteria.

Hopkins and Chibnall (16) described a strain of Aspergillus versicolor which would metabolize odd- and even-numbered paraffins up to a chain length of C_{34}; however, they were unable to establish the means of breakdown beyond suggesting the possible occurrence of a polyketone as an intermediate to shorter-chain fatty acids. (It is of interest to note that the present author has isolated a strain of this fungus, which is an active resin-attacker, and has demonstrated the presence of carbonyl compounds in ethereal extracts of culture fluid).

Haag (12) correlated the use of paraffin wax with a decrease in its iodine number, while Grieg-Smith showed that paraffin was readily metabolized if adsorbed on a particulate surface, such as kieselguhr, while the importance of solid surfaces has been justly emphasized by ZoBell (40).

In a natural soil, an extremely large surface is available, and if the substance to be decomposed has some surface activity, such as the resins used in soil stabilization, then a mono- or poly-molecular film is quickly spread over the surface of the soil clays (the process known as "curing") and ideal conditions for decomposition occur.[1]

Attack on aromatic hydrocarbons has been demonstrated by many authors. Störmer (31) demonstrated the microbial decomposition of toluene and xylene, Tattersfield (35) and Jacobs (18) that on naphthalene. The latter author demonstrated the occurrence of phthalic acid in culture filtrates. Strawinski and Stone (32) showed one main fraction to be orthosalicylic acid; they also showed that bacteria were capable of metabolizing such diverse hydrocarbons as cetane, naphthalene, biphenyl, a-methyl naphthalene, and tetralin.

Sen Gupta (24) noted the disappearance of phenol and cresol from soils under natural conditions, while Gray and Thornton (11) successfully demonstrated bacterial attack on naphthalene, phenol, and cresols and found such bacteria to be present in 146 out of 245 soils tested. Wagner (37) isolated from soil and faeces bacteria that could attack phenol, catechol, and phloroglucinol, while Fowler, Ardern, and Lockett (10) suspected the occurrence of phenol attackers in sewage beds. More recently, Happold (13) isolated these organisms from many sources, including sewage.

Happold and Key (14) also isolated a gram-negative vibrio from gasworks liquor, which attacked phenol. Allen, Naghski and Hoover (2) recently studied the decomposition of the resins associated with the retting of the Guayule Shrub (Parthenium argentatum) in order to obtain the dispersed rubber latex of that plant. They observed that the process of retting patented by Spence (29) led to a diminution in the acetone soluble resinous material associated with the rubber, and they described a number of bacteria and fungi that would readily attack the resin, one identifiable as a cresol-attacker isolated by Gray and Thornton, (11). The microbiological decomposition of cholesterol, another closely related compound, has been noted and investigated by Turfitt (36).

Hence, from a survey of the available literature, there is considerable reason to assume that natural resins would be decomposed by soil microörganisms. This belief is sustained by a consideration of the amazing adaptability of these forms of life, c. f., Dubos (7), and from the fact that large quantities of natural resins have not (with the possible exception of amber) accumulated on the surface of the earth.

Since soil conditions will obviously affect the microörganisms concerned, some reference may perhaps be made here to papers on that extensive subject. An encyclopaedic work on environmental factors is that of Van Suchtelen (30) who showed that microbial activity (as expressed by CO_2 evolution) showed an increase with soil moisture content from a minimum at about 4 percent to a maximum at 15 percent. This would seem to indicate that a partial breakdown of resin stabilization would only serve to accelerate this process.

However, on the other hand, Hubbell and Gardner (17), in a paper most pertinent to the present problem, showed that soil compaction produced a considerable reduction in both aggregation and soil flora, both of these showing some correlation with the noncapillary pore space. From this it would seem that compaction itself tends to reduce microbial attack, an indication which has also been given in the present author's results.

Engberding (8) and Skinner, Jones, and

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[1] Note by H F W Whether adsorption on a solid surface makes an attack easier or more difficult depends on whether the susceptible parts of the molecules are directed towards, or adsorbed by, the solid surface or are on the exposed side of the adsorbed layer (see paper No 6, by Gieseking)

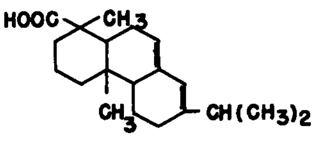
Mollison (26) have shown correlations between moisture content and bacterial numbers, the former using the direct count technique of Jones and Mollison (20), in addition. Winterkorn (39) noted effects of microbial activity on the strength of earth embankments and of bitumen-treated calcium soils.

It seems that there is considerable reason to assume that natural resins are decomposed in soils and that modifications of the local physical conditions materially affect the nature of the attack. The following material describes some aspects of this work and attempts to relate it more particularly to the practical problems of soil stabilization and road construction.

The practice of stabilizing, or more simply, waterproofing, road foundations with resins was initiated during the late war, and was initially developed for military purposes. When the resinous compounds used were incorporated into soil, the mixture resulting (ca. 1 percent resin) was compacted by rolling in order to obtain a mechanically substantial foundation. It was also realized that an optimum moisture content existed for each soil treated, and that the presence of excess (more than about 5 percent) of organic matter was deleterious to successful stabilization. Moreover, frequent failures occurred which were not readily ascribable to any of the known physical variables. It was then realized that some other, possibly biological, factor was involved. This article is intended to indicate how such factors may be elucidated and what problems they subsequently pose; with an indication to the most likely means of their resolution.

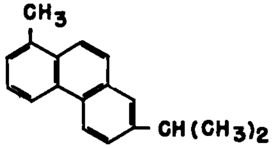
In this work, the three resinous materials investigated were (1) Vinsol, manufactured by the Hercules Powder Company, Wilmington, Delaware., (2) the natural residue from oleoresin distillation (viz, gum rosin), the principal constituent of which is abietic acid ($C_{20}H_{30}O_2$) and its isomers; and (3) "321" resin, being a proprietary mixture of gum rosin and sodium rosinate in the proportion of 3 to 1.

The work of Ruzicka and of Fieser has demonstrated the most-likely formulas of abietic acid to be



the rings being saturated with the exception of the double bonds indicated.

This is manifestly related to the aromatic hydrocarbon retene



which is methyl-isopropylphenanthrene.

It will be seen that abietic acid is monobasic, and by virtue of its hydrophobic abietyl group forms a mono- or polymolecular film on a water surface, a free-surface energy of about 10 dynes per centimeter being available. However, the sodium salt, by virtue of the high hydration capacity of the sodium ion, will be soluble at an alkaline pH. But the free acid and alkaline earth salts both form mono- or poly-molecular rigid films, which later can easily be demonstrated by powdering a surface film of resin on a water surface with talc when the whole surface is seen to move on disturbance as a single rigid unit.

Vinsol is an extract of pine stumps made with an organic solvent and contains some abietic acid in addition to other terpenelike materials. It will be seen below that non-carboxylic $^-$OH is present, the acid number being smaller and the molecular free energy less. A summary of the salient chemical and physical characters of these compounds is set out below. (From Appendix B. Cement-Durability Program, First Interim Report, War Department, U. S. Corps of Engineers, U. S. Army, Mount Vernon, New York, June, 1942-).

A theory of the stabilization of soils based on the formation of a surface film in the soil moisture has been developed by Clare (6) and others, who have demonstrated effective stabilization below pH 7, failure near neutrality, and film formation (presumably of the calcium salt) at pH values higher than 7.5.

METHODS OF INVESTIGATION

As few techniques were available for the investigation of this problem of microbial attack on resins incorporated in natural soils, the development of new and modified techniques seemed essential. These comprised: - (1) methods of isolating resin-attacking bacteria; (2) a method for selectively counting these bacteria; (3) methods for the detection and approximate estimation of resins in soil; and (4) a standard method of studying water absorption as used at the Road Research Laboratory, Harmondsworth, Middlesex, England.

Several other lines of inquiry of a more strictly biochemical nature were pursued, such as the study of resin attack by pure cultures of bacteria by micromanometric methods. But since these form the subject of another paper and are not strictly relevant to the immediate problem of the engineer, they are not dealt with at length below.

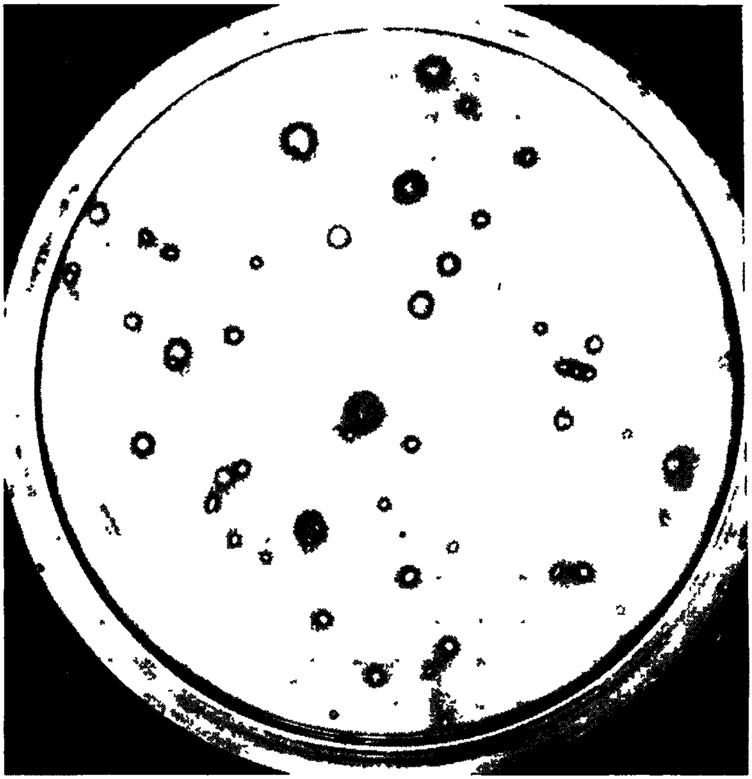


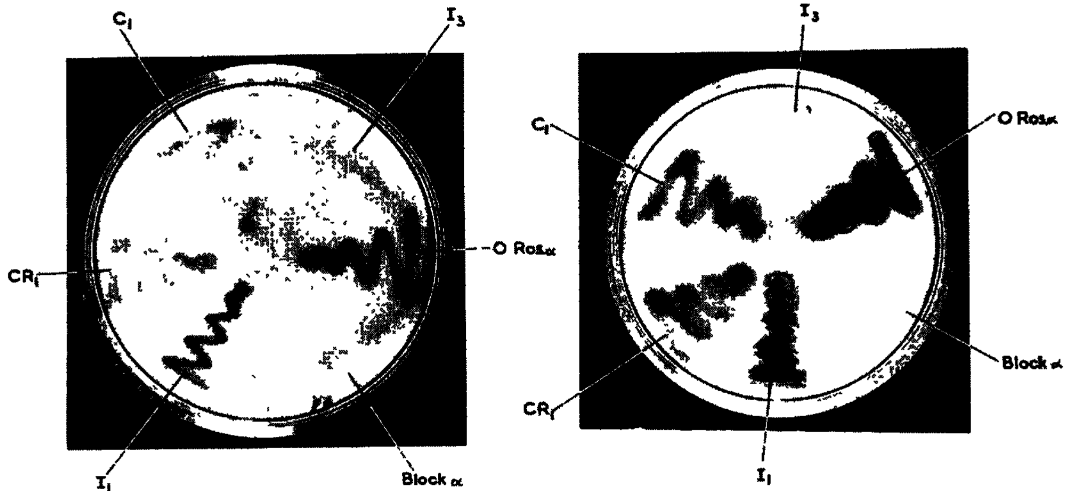
Figure 1. Sample plate of a dilution from a resin enriched soil showing bacterial colonies and their zones of halation or resin breakdown.

Isolation of Resin Attacking Bacteria By Enrichment Culture

The standard method of isolating pure cultures of bacteria is that of elective culture. This is effected by enrichment cultures in which a small amount of the substance under investigation is added to a large amount of soil and the admixture incubated. If the substance should not be too toxic in the concentration used, a microflora capable or metabolizing that particular substrate develops, when further addition of the substrate may be made. Finally, a small quantity of the bacterially enriched soil may be transferred to a soil-free mineral salt medium in which the substrate (in this case a resin) is the only organic foodstuff. If under these conditions bacteria multiply, then they are eminently capable of metabolising the substance concerned and, after repetitions of the above process, may be isolated by the technique (plating) of diluting the culture with a melted nutrient agar jelly and subculturing from one of the immobilized colonies that develop during cooling, solidification, and incubation.

Selective Counts by a Plating Technique

If the agar medium described above be so prepared that it contains only a resin as an available foodstuff (assuming agar to be almost nonnutritive, as it is to most terrestial organisms); then this can be used selectively to count the resin-attacking microbes present in soil and for their isolation and culture. Gum rosin and Vinsol are both insoluble materials and cannot be directly incorporated in a form sufficiently dispersed to be uniformly available to microörganisms. However, a fine emulsion in poured agar plates (as above) presents a much-greater surface for attack than larger aggregates.



Bacterial strains streaked on resin agar 7 days

Growth after 14 days

Figure 2. Pure culture streaks of resin attacking bacteria, showing halation and dissimilation of a colloidal resin agar.

In their studies on Guayule resins, Allen, Naghski, and Hoover (1) obtained a resin emulsion by dispersing an acetone solution of the resin into a mineral salt solution. In the opinion of the author, an emulsion is better obtained by prior saponification of the resin and dispersion into a buffer mineral-salt solution.

This is thought to be preferable, since in the former method, despite boiling to expel the acetone or ethanol, sufficient may still be present to affect the bacterial metabolism; the use of only nonorganic mineral solution precludes this.

The selective medium is prepared thusly: The resin is saponified with the calculated quantity of caustic soda, or, in the case of gum rosin, the commercially available sodium rosinate can be used. It is assumed that the relatively mild treatment of saponification will not affect the abietic acid nucleus. The sodium salts are then dispersed into the basal salt solution described below at a concentration approximately 0.2 percent, weight by volume at 90 C.

If, as was generally the case, the agar medium was being prepared, the agar was previously dissolved by boiling, since its subsequent peptising effect helped to ensure the stability of the resulting emulsion. The result was a stable milky-white opaque medium, the reaction of which was subsequently adjusted with normal NaOH to a pH of 7.0 - 7.2.

BASAL SALT SOLUTION

K_2HPO_4	0.35 gm.
KH_2PO_4	0.15 gm.
NaCl	0.10 gm.
$MgSO_4 \cdot 7H_2O$	0.10 gm. per litre
NH_4NO_3	0.10 gm.
$CaSO_4$	0.05 gm.
$FeCl_3$	Trace

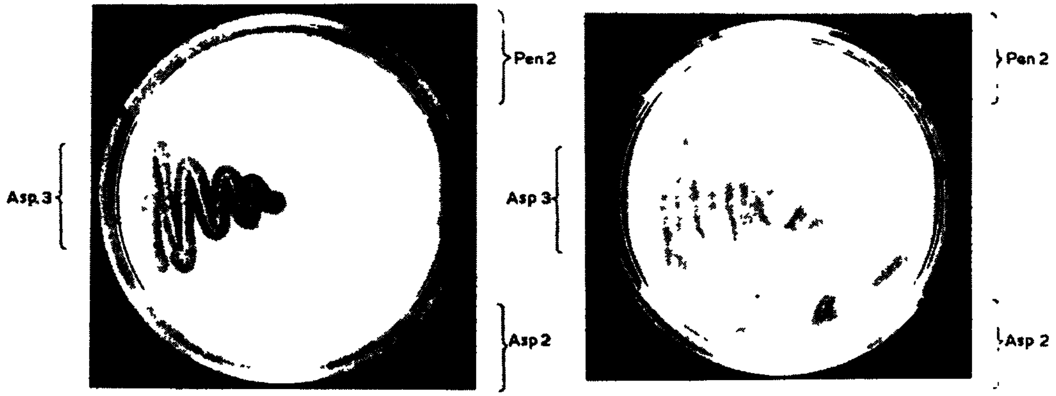
The agar emulsions thus obtained did not break on sterilization and could be stored. Since the dissimilation of the resin emulsion by the bacteria or fungi clarified the agar, the presence of a clear zone or halo around a bacterial or fungal colony on a resin-agar plate was indicative of attack, and the medium was thus suitable both for isolating and for counting resin-attacking organisms.

Examples of the use of this medium are seen in Figures 1, 2, and 3, which are respectively photographs of a count plate of resin attacking bacteria from soil, of pure culture streaks of resin-attacking bacteria, and of resin-attacking fungi. Other insoluble stabilizing agents were always investigated in a similar manner.

Methods for the Detection and Estimation of Resins

The classical test for substances having the menthadiene structure of an isopropyl unit attached to a ring structure with two conjugated double bonds is the Liebermann-Storch reaction (9), which is based on the color obtained when concentrated sulphuric acid or a saturated solution of $ZnCl_2$ in glacial acetic acid is added to a chloroform or acetic anhydride solution of these substances in the presence of an acyl- or aryl-chloride or anhydride (21).

This author has found the test given with benzoyl chloride and 40 percent of $ZnCl_2$ in glacial acetic acid to be fairly reliable when used either with experimental resin emulsions or with ether extracts of soils. However, the color obtained is fugitive, and unless techniques for accurate colorimetric estimation are available, it is better either to weight a simple ether extract, if the ether soluble solids of the natural soil are not too high, or to use the nitration method of Donath (25), which (although not so specific) gives permanent colors for comparison.



Fungi inoculated on to colloidally dispersed resin agar
7 days

Development after
14 days

Figure 3. Pure culture streaks of resin attacking fungi, showing halation and dissimilation of a colloidal resin agar.

Methods for Studying Water Absorption

The methods adopted for studying water-absorption by resin-soil mixtures under various microbiological conditions were those used by the Road Research Laboratory.

Two different types of specimen were prepared. The majority were made in the laboratory compacting apparatus described by Little (22). In this method, 150-gram batches of moist soil-resin mixture are compacted into cylindrical specimens 2 inches in diameter and approximately 1½ inches high and are allowed to cure in a moist atmosphere for 24 hours (i. e., to allow the spread of the resin film). The specimens are then removed, and the cylindrical surfaces coated with wax.

In the present investigations they were allowed to absorb water through their lower flat surface for varying periods of time, either by standing on saturated sand contained in a sieve or on the surface of a perforated metal sheet maintained ½ inch below the surface of water in a closed metal tank.

Specimens containing less soil than the above were also used; they were cylinders 1 inch in diameter, containing about 30 grams of moist soil-resin mixture. These were compacted in a small steel mold of the appropriate dimensions by means of a plunger which was subjected to 8 blows of a 5½-lb. hammer, (22) falling through a height of 1 foot. The specimens thus produced were about 1½ inches high and had a compacted soil density of the same order as that obtained in the standard compaction test. These specimens were cured and tested as described above.

RESULTS OF INVESTIGATIONS

Isolation of Resin-attacking Organisms

A large number of resin-attacking isolates were obtained from various sources, both by elective culture and by the use of the selecting-plating media described above. It was thought that the intestinal flora might contain some organisms that would attack sterols and the closely related abietic acid. This proved to be the case, and two very-active resin attackers were isolated from human faeces. All the bacteria isolated except those latter, show a morphological resemblance to the phenol, cresol, and naphthalene attackers isolated by Gray and Thornton (11). Moreover, two active resin-attacking fungi have been isolated, strains of Aspergillus nidulans and of Aspergillus versicolor. A proactinomycete was also isolated but was lost out of culture before it could be fully investigated. It is perhaps an unwarranted generalization to state that these organisms are of ubiquitous occurrence, but their presence has been demonstrated within a wide range of soils. These have comprised clayey soils at Rothamsted, sandy acid loams, and the calcareous brickearth from Harmondsworth (pH 8) that has frequently been used as an experimental material for this study. Even a heavily polluted London soil has given isolates of these organisms. However, no resinolytic organisms were demonstrated in marine muds.

The actual numbers of organisms present varied, and although no survey of relative numbers of organisms has been made, it would seem that it is in those soils in which a considerable zymogenous flora might be supposed to be present that the highest numbers of resin attackers were found. These comprised heavily manured farmyard and allotment soils, whereas a light chalky (calcareous) downland soil near Dover, Kent, proved to have a relatively small flora. Moreover, in a private communication, Yao Tsen Tchan of the Institut Pasteur, Paris, has also indicated that he has isolated a bacterium that appears to attack gum rosin.

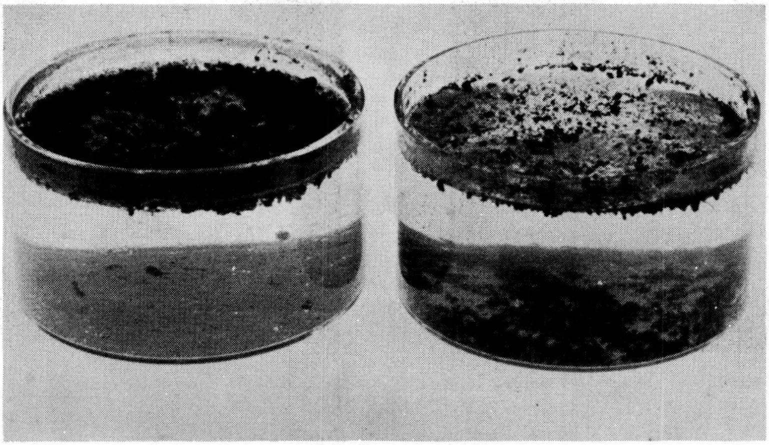


Figure 4. The effect of mercury in maintaining resinous soil stabilization (see Text).

Development of a Resinolytic Flora

The graphs shown in Figure 5 demonstrate the growth and increase in numbers of resin-attacking organisms, both in pure culture and in natural soil. (It is of interest to note the availability of blue copper rosinate in this context, which might a priori be supposed to be toxic.)

Demonstration that Increased Bacterial (and Fungal) Action Reduces the Effectiveness of Resins in Preventing Water Absorption

In general, bacterial counts made from samples of resin-stabilized roads showed an increase in resin-attacking organisms, particularly when failure was apparent. The introduction of 10 percent of cement in addition to the resin reduced bacterial numbers practically to extinction, but it is the opinion of the author that rather than using cement as a resin preservative in the proportion suggested (or slightly higher), it would be itself a treatment of choice.

LABORATORY EXPERIMENTS

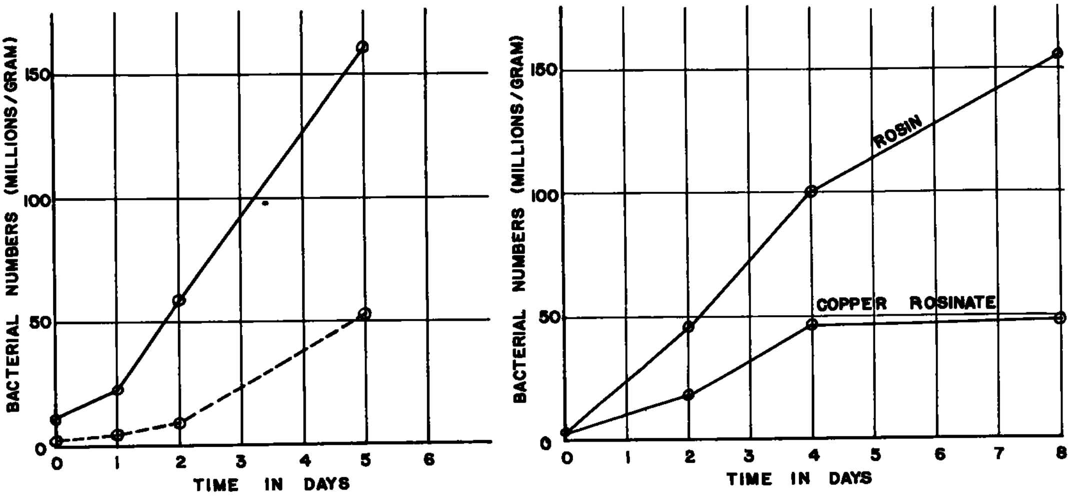
In some early experiments, the effect of alternate wetting and drying on small specimens was investigated. In this work, half the specimens were allowed to absorb water from a culture of known resin-attacking bacteria, and the others from a 0.2-percent solution of mercuric chloride, a vigorous sterilizing agent. In Figure 6 the effect of this wetting and drying can be seen; this effect was uniformly repeatable. Probably two factors are here present, both that of the mercuric ion in stabilizing the resin film, and its effectiveness as a sterilizing agent. (The bacterial numbers otherwise obtained from resin-enriched soils are greatly in excess of those from comparable rich natural field soils.)

In Table 2 are seen the results of another typical experiment in which 150-gram specimens of an unsterilized brick-earth (air dried) were stabilized with 1 percent of gum rosin at a moisture content of 12 percent and compacted in the Dietert compactor. Three sets of specimens were respectively uninoculated, inoculated with resin-attacking bacteria, and with resin attacking fungi. After curing and waxing, they were allowed to

TABLE 1

CHEMICAL AND PHYSICAL CHARACTERISTICS OF VINSOL RESIN AND ROSIN

	Vinsol resin	Gum rosin
Acid number	90-100	150-165
Methoxyl content	5-6%	Neg
Melting point (drop)	110-115°C	75-85°C
Noncarboxylic -OH (Zerewitnof)	6 %	Neg
Gasoline-soluble material	8-15%	90-100%
Approximate molecular weight	450	305
Combustion analysis (approx)		
Carbon	73%	79%
Hydrogen	7%	9.5%
Oxygen	20%	10.5%
Approximate formula	$C_{27}H_{30}O_8$	$C_{20}H_{30}O_2$



GROWTH OF RESIN-ATTACKING BACTERIA IN PURE CULTURE.

SUBSTRATE:
—— GUM ROSIN
---- "VINSOL"

DEVELOPMENT OF A RESINOLYTIC FLORA IN SOIL.

SUBSTRATES USED:
GUM ROSIN AND COPPER ROSINATE
MEDIUM USED:
SELECTIVE ROSIN MEDIUM

Figure 5.

TABLE 2

WEIGHT OF RESIN-STABILIZED SOIL SPECIMENS THAT HAD BEEN INOCULATED WITH RESINOLYTIC BACTERIA AND FUNGI

(Average weight of four specimens) (Standard errors given)

Time (Days)	Control	Inoculated with bacteria	Inoculated with fungi
1	149 0 ± 1 3	150 0 ± 1 5	149. 5 ± 1 2
2	150 0 ± 1 4	151 5 ± 1 5	151 5 ± 1 4
3	151 0 ± 1 3	154 0 ± 1 5	154 0 ± 1 4
4	152 0 ± 1 3	156. 0 ± 1 0	156 0 ± 1 6
5	152. 5 ± 1 4	159 0 ± 1 6	159 0 ± 1 9
7	155 0 ± 1 2	164 5 ± 1 9	168 0 ± 2. 0
8	156 0 ± 1. 5	166 0 ± 1 9	171 0 ± 2 1

absorb water from a free water surface. It will be seen that the addition of such massive starters results in increased water uptake, presumably on account of the breakdown of the resin film. (Sterilized soil was not used, on account of the physical changes that probably take place during such pretreatment, although resin attack, per se, is easily demonstrable if such a soil be inoculated.)

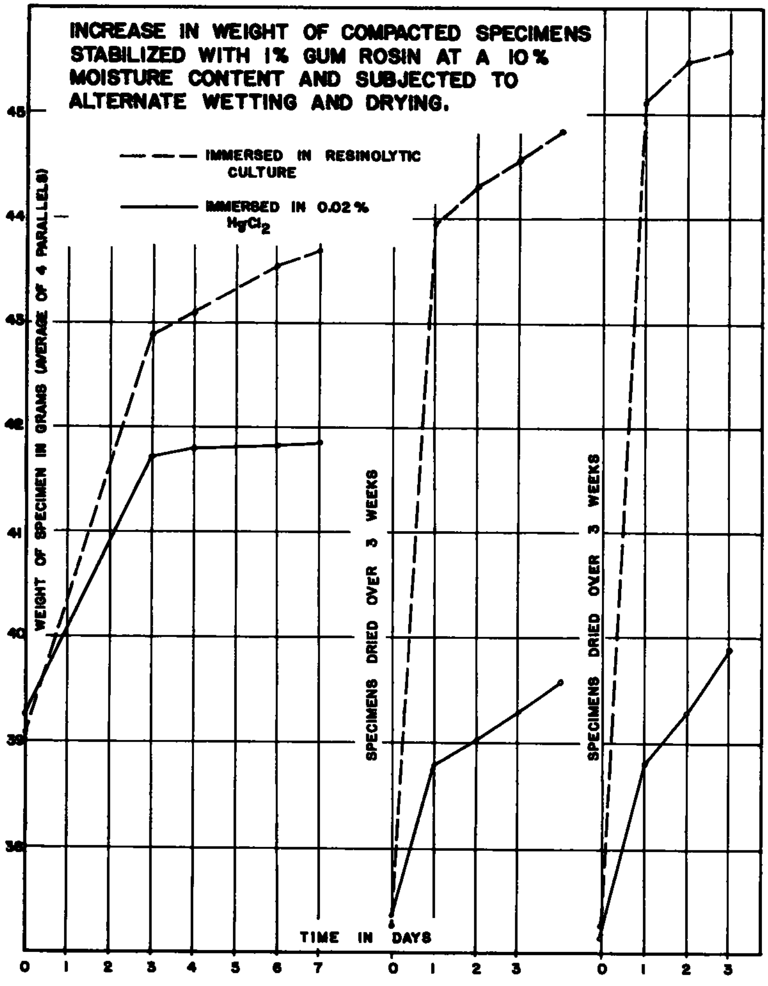


Figure 6.

TABLE 3

COUNTS MADE FROM SOILS THAT HAD BEEN INCUBATED (25°C) BOTH AEROBICALLY AND ANAEROBICALLY FOR FOURTEEN DAYS

Numbers of organisms (millions pergm.)

	Soil Alone	Soil + 1% Gum Rosin	Soil + 1% "Vinsol"
Aerobic	0. 64	31. 19	6 25
Anaerobic	---	18. 90	0. 63

These results suggest that the presence of resins in natural soils results in a great increase in the bacterial population of that soil and that the introduction of an extremely high resinolytic population into soil results in greatly impaired soil stabilization.

EFFECT OF PHYSICAL CONDITIONS AND OF ANTISEPTICS

The effect of physical conditions has been discussed above, and it is not proposed to deal with this subject at length here. However, a few results of interest will be mentioned. The effect of partial anaerobiosis (likely to be the case in the pore spaces of soil) was investigated by partially replacing the atmosphere of a dessiccator (used as a humidity chamber) with CO_2. The effect of such partial anaerobiosis is seen in Table 3, resin-attacking organisms being counted by means of the selective media described above.

An even-more-marked effect was seen with the two strains of resin-attacking fungi. Hence, as has readily been seen from other work, the resin-attacking organisms are aerobes, and their numbers can be reduced by lowering the oxygen tension[2]. From this observation it would immediately seem that the effect of compaction in reducing the pore space will reduce the available oxygen and thus (under active metabolic conditions, where microorganisms are respiring freely) a low oxygen tension will soon be achieved.

The results of Hubbell and Gardner (17) are of particular interest in this matter, since they observed a reduction in microbial numbers upon compaction in soils to which no addition had been made; while the results of the present author have tended to confirm this observation (though not without exception).

It would seem that further investigation of the effect of microenvironmental conditions might be of value in the conservation of resin stabilized foundations. Moreover, the observations made by the present author (26) and by others, such as Van Suchtelen (30), Engberding (8), and Jensen (19), have shown the dependence of bacterial numbers upon soil moisture content, although, as shown by the present author (Skinner, Jones and Mollison (26), there is a strong interaction with a temperature effect. Nevertheless when this is eliminated, the effect of soil moisture is still apparent. As may be expected, however, once no capillary pore space remains, a condition of waterlogging and of partial anaerobiosis occurs, and the bacterial numbers are reduced, according to Subrahmanyan, (33). Moreover, Hoffman (13), has suggested that the optimum conditions for soil bacteria occur when half the pore space is filled with water. (In the case of sandy soils, 8 to 10 percent; and with heavy clays 20 percent.)

The above observations may serve to give some indication of the mutually opposing factors that have to be resolved in this problem. For instance, to recapitulate, low moisture content upon compaction will tend to control the bacterial numbers within limits, but the effect of water ingress will be a cumulative one on the microflora up to the point of waterlogging, when the road foundation will not be useable in any case. Apart from the limited measure of control that can be achieved above, in the present work an investigation of the effect of antiseptics was made which will be briefly described below.

EFFECT OF ANTISEPTICS[3]

It was early realized that for antiseptics to be a practicable proposition in preventing the breakdown of resins in soil, they must be cheap in the quantities that would have to be used and preferably inorganic in nature; since the ability of soil flora to metabolize any substance that affords a source of energy is amazing indeed. Moreover, unless the substance is strongly surface active, it will be difficult to achieve a toxic concentration throughout the soil at an economic cost.

The attentions of the present author turned therefore to a selection of antiseptics as follows; sodium pentachlorophenate, bleaching powder, (calcium hypochlorite) copper sulphate, mercuric chloride (corrosive sublimate) and many others. The effect of soil and of resin pretreatment was investigated to some extent.

Laboratory experiments with media of colloidally dispersed copper rosinate and mercuric resinate, made by adding the calculated quantity of $Cu^{\cdot\cdot}$ or $Hg^{\cdot\cdot}$ to the basal salt solution in preparing the select-

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[2] Note by H. F W · Experiments performed at Princeton University on resin and asphalt stabilized lateritic soils showed strong microbial action in completely immersed specimens accompanied by reduction of iron from the trivalent to the bivalent form

[3] Note by the Editor See also "Fundamental Approach to the Stabilization of Cohesive Soils" by Hans F Winterkorn Proc 28th Annual Meeting, Highway Research Board, pp. 415-422 (1948)

TABLE 4

BACTERIAL NUMBERS OBTAINED BY PLATING SOILS INCUBATED FOR 10 DAYS AT 25°C - ADDITIONS AS IN TEXT

Specimen	Average No. of organisms (Millions per gram)	
	Nutrient	Resin
1% Gum Rosin + inoculum	332.69	283.75
Inoculum alone	97.87	23.87
1% Gum Rosin + inoculum + 1% $CuSO^4$	116.12	90.56
1% Gum Rosin + inoculum + 0.02% $HgCl_2$	0.31	0.19
1% Gum Rosin + inoculum + 0.02% sodium pentachlorophenate	0.37	0.12

ive media as above, showed that aspergillus nidulans and aspergillus versicolor would grow readily on a medium of blue copper rosinate, that bacterial attack was slightly reduced, and that neither would grow on the mercuric-rosinate medium. However, it has been noted above that an increase in bacterial numbers occurs on the addition to soil of copper rosinate. It therefore seemed that copper would be of little avail in preventing resin decomposition, despite its known antimicrobial use (in wood preservation and against powdery mildews for instance.)

These results are apparent in Table 4, in which a number of loose specimens of sterile Harmondsworth sandy loam were made up to a moisture content of 10 percent, a resin content of 1 percent, and a standard inoculum of resin-attacking organisms added. It will be noted that sodium pentachlorophenate was also effective in reducing bacterial numbers. The results of a similar experiment with gum rosin and Vinsol are shown in Table 5, where with a moisture content of 12.5 percent similar results are seen.

Some further tests were then made on the influence of the antiseptic of the stabilizing agent. Two series of compacted specimens of 2.5 cm. diameter were made from the same soils with eight blows of a 5½-lb. hammer. In the first series (A) the specimens were made concurrently with the bacterial count, and in the second (B) the soil was stored over a period of six weeks and then rewetted to 7.5 percent moisture content. All the specimens were then allowed to absorb water on a wet sand surface for 4 days; the final results of this experiment are given in Table 6.

It will be noted that a significant effect of $HgCl_2$ is only evidenced in Series B, where loose samples had been incubated. In all cases, adequate opportunity for the establishment of a resinolytic flora was present (the soils were inoculated), so that compacted specimens showed less tendency to break down than those stored loose. This is consistent with the hypothesis that the resinolytic organisms are aerobic. It may also be noted that too high a proportion of sodium pentachlorophenate affects the waterproofing of the resin used, but a low concentration reduces the number of resin attackers. However, it is known that chlorophenols in low concentration (as might be expected) are decomposed in soils (38) and thus any ameliorative effect is likely to be short lived).

Again, in the search for economically suitable antiseptics, bleach (calcium hypochlorite) was used. The results of a test using this, $HgCl_2$, and sodium penthachlorophenate are given in Table 7. It will be seen that calcium hypochlorite is useless, despite its antiseptic activity, since stabilization is impaired by the antiseptic agent.

An attempt, moreover, was made to effect antisepsis by chlorinating the resin molecule (the resins readily absorbed wet chlorine, with a rapid exothermic reaction); however, once again, the introduction of chlorine into the abietic acid molecule (presumably, by saturation of

TABLE 5

Specimen	Count on nutrient agar (millions/gm)	Count on rosin agar (millions/gm)	Count on "Vinsol" agar (millions/gm)
12.5% moisture + 1% Gum rosin + inoculum	267.76 (spreading present)	(412.20) insufficient plates	-
"	431.04	412.40	-
12.5% moisture + 1% Gum rosin + 0.1% S.P.C.P. + inoculum	0.32	nil	-
12.5% moisture + 1% Gum Rosin + 0.78% $HgCl_2$ + inoculum	0.32	nil	-
as above but with 0.25% and 0.5% S.P.C.P.	no colonies whatever		
12.5% moisture + 1% "Vinsol" + inoculum	538.88	-	122.88
ditto	429.44	-	123.60
ditto	503.04	-	101.20
12.5% moisture + 1% "Vinsol" + 0.1% S.P.C.P. + inoculum	137.32	-	nil
12.5% moisture + 1% "Vinsol" + 0.078% $HgCl_2$ + inoculum	2.28	-	nil

the conjugated double bonds) reduces the hydrophobic nature of the anionic group and thus defeats the intended purpose.

However, a different story always held with mercuric chloride, which was invariably effective, even in low concentrations. However, the high price and high toxicity of this product would seem to invalidate its use in the field. However, it would seem that some hope can be offered should Hg^{++} be so incorporated in the resin as to reduce the toxicity of the product to the handler, if the cost were still economic and the surface properties of spread and stabilization unaffected, then this alone would seem to offer any hope for the future.

EFFECT OF SOIL PRETREATMENTS

Various methods of field soil sterilization were tried in this context (e.g., steaming, formalin-and-sulphuric-acid treatment, and combinations of these). With the exception of sulphuric-acid treatment (which was most probably a pH effect (Final pH 4) none were efficaceous, and indeed gave the surprising result of inducing a vigorous resinolytic flora to such a marked extent that the recent use of formalin in a resin-free soil could readily be traced by observing the increase in resin-attacking organisms by plating on the selective media described.

TABLE 6

WATER UPTAKES OF SPECIMENS TREATED WITH VARIOUS ANTISEPTICS

(Mean weight of four specimens)
(Significant effects indicated with asterisks)

Treatment	Series A (compacted)			Series B Incubated loose (then compacted)		
	Air Dry Wt	Final Water Uptake	Percent Water Uptake	Air Dry Wt	Final Water Uptake	Percent Water Uptake
1% Rosin	23 80	2 13	8 95	42 24	6 37	15 08
" + $HgCl_2$ 0 078%	22 23	1 92	8 64	39.93	3 21	8.04
" + SPCP 0 1%	23 28	1 87	8 03	42 95	2 80	6 66
" + SPCP 0 25%	23 68	2 72	11 48	39 80	5 13	15.40
" + SPCP 0 5%	23 51	5 19	22.08	40 75	6 61	16 21
1% "Vinsol"	24 07	1 55	6 44	35 13	3 65	10 39
" + $NgCl_2$ 0 078%	24 33	1 37	5 63	36 50	2 99	7 77
" + SPCP 0 1%	24 64	2 03	8 24	39 86	3 32	8 31
" + SPCP 0 25%	22 60	3 04	13 45	40 24	4 88	12 13
" + SPCP 0 5%	23 95	4 11	16.77	41 11	7 50	18 46

TABLE 7

EXPERIMENT ON A SANDY LOAM SOIL
Schedule of different treatments

(S)	Soil alone
(V)	Soil + 1% "Vinsol"
(VP)	Soil + 1% "Vinsol" + 0 1% Na pentachlorophenate
(VH)	Soil + 1% "Vinsol" + 1% Calcium hypochlorite
(3)	Soil + 1% "321" resin
(3H)	Soil + 1% "321" resin + 1% Ca hypochlorite
(3M)	Soil + 1% "321" resin + 0.016% $HgCl_2$

TABLE OF RESULTS

(Initial weight of soil - 150 gms)
(moisture content 10%)
(Average of 2 specimens only, but 5% variation was maximal)

	Weight after drying (gms)	Weight after 1 week (gms)	Total water Uptake (gms)	Percentage Uptake
S	129 2	171 2	42 0	32 5
V	130 2	138 4	8 2	6 3
VP	131 2	136 4	5 2	4 0
VH	132 2	168 1	35 9	27.3
3	129 0	133 6	4 5	3 4
3H	132 1	141 4	9 3	7 1
3M	132 8	134 9	2 1	1 6

In some other experiments the effect of Ca hypochlorite invariably was to render the specimen unmanageable

It would therefore seem that these organisms are an integral part of the normal soil balance and that resin decomposition is only one of their talents. Thus, from the above investigation, admittedly very limited, it would seem that within the small range of antiseptics tested, mercury alone presented any advantage. How striking this may be is demonstrated by the following simple final experiment.

Two similar specimens, both stabilized with Resin 321 and one containing 0.016 percent of $HgCl_2$, were prepared in the Dietert compactor at a moisture content of 10 percent. After incubation for two weeks, both were dried in an oven and then powdered. Five grams from each were then taken and spread uniformly over a distilled-water surface in a 4-inch crystallizing dish and left overnight. At the end of 12 hours, the surface soil was agitated in both cases; the soil from the specimen that contained no mercury wetted at once (A), whereas that from the specimen that had contained mercury did not wet at all.

This experiment is illustrated in Figure 4. The effect is probably dual: that of the mercury in stabilizing the resin film, and its effect in preventing microbial attack. (Some of this work forms the subject of a patent in the author's name invested in the Imperial Trust.)

DISCUSSION

From the above, it is apparent that when complex decomposable substances are added to natural soils, unless they have a very-unusual structure or are present in high concentration, then their decomposition over a period of greater or less time by the soil microflora is virtually assured. However, this unfortunate fact must always be viewed from an economic point of view. If the useful life of a resin-stabilized foundation, with or without antiseptic, is suited to the economics of the situation (such as a temporary road as a precursor to some more-permanent foundation) then it has everything to recommend it. Moreover, it has the advantage that temporary disturbance is easily repaired (one reason for wartime development).

However, for longer term planning, the relative cost of more-permanent means of consolidating a foundation should be considered at the present state of knowledge, where the cost of an adequate quantity of a suitable stabilizing or binding agent should always be a primary factor. But a more-vigorous attack on the problem of resin decomposition would probably pay dividends, particularly where road construction in areas away from suitable stone is considered.

If the microbiological problem can be solved, then immense areas of lateritic soils in the tropics may well be opened to trade by road stabilization using the native resins. The produce thus freed to a hungry world would aid the development of such backward areas and pay for more-permanent roads to provide the means of permanent contact with the outside world, the spread of civilization, and the defeat of that bogey of the near future: world famine.

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Stabilization of Laterite Soils

A. REMILLON, Director
Brazzaville Branch of the Laboratories for
Construction and Public Works of
The French Government

Introductory Remarks by the Chairman: Laterite soils are typical for the wet tropics and are widely distributed in such areas as India, Indonesia, Indochina, Malaya, Burma, Western Australia, Madagascar, Central Africa, the Guianas, Brazil, Cuba, and others. They normally possess good tilth, and with plenty of solar energy and water available, they are, if properly fertilized, capable of excellent crop yields and may well be destined to contribute in a major degree to the food supply of a rapidly growing world population. A great need exists for a suitable network of low-cost roads in these areas already in their present underdeveloped condition and even more so if their proper agricultural development is to proceed.

Laterite soils differ widely from the soils of the temperate regions with respect to their physical and chemical properties. It is in these soil areas that the greatest discrepencies have been observed when soil-stabilization methods developed for certain temperate climates have been unjudiciously applied. This is especially true for granular soil stabilization.

Remillion, who is director of the Brazzaville Branch of the Laboratories for Construction and Public Works of the French Government, has attacked the problem of stabilizing laterite soils from the point of view of the scientifically well-trained, responsible executive engineer. Significantly, he has gone to the heart of the problem. Instead of trying to adjust for his conditions standards developed for temperate climates and soils of such climates, he makes a fresh analysis based on the actual properties of his soil materials and on the classical work of Talbot in the design of concrete mixtures. The analytical conclusions are compared with actual road performance, and the resulting design recommendations are as scientific and as practical as one can wish for.

● LATERITE formations cover an important portion of the tropical and equatorial zones. They may be present in the form of red clays or of laterite proper.

The latter may or may not be covered with an iron-oxide carapace.

Laterites form natural soil-concretes that are utilized widely for the construction of runways and highways. In the following we shall consider only the laterites proper, excluding the indurated oxide shield or carapace and also the lateritic clays.

GENERAL CONSIDERATIONS

Mineralogic Composition of laterites

Microscopic examination of laterites derived from the alteration of rocks in place shows that these materials are composed of: pure silica (SiO_2) from eolian sands; and hydrous oxides of aluminum ($Al_2O_3 \times 3\ H_2O$) and of iron ($2\ Fe_2O_3 \times H_2O$ - Goethite).

Processes and factors of alteration

Mineralogic examination reveals that two distinct and independent alteration processes occur simultaneously in laterite formation. On one hand, the normal weathering of feldspars leads to the formation of clays; on the other hand, the specific lateritic alteration leads first to hydrates and then to oxides.

This lateritic alteration is a consequence of the hydrolysis of the feldspars and of the elutriation of the alkalis and of part of the iron in the form of the bicarbonate. This elimination is favored by the great purity of the waters (hydrotimetric degree generally lower than 3) and by an elevated temperature (about 86 F). The hydrates tend to transform into oxides under the effect of solar radiation (ultraviolet) and possibly also under the action of iron bacteria.

The concretions or pisoliths that constitute the skeleton of the laterites seem to have formed around hydrargillite crystals that served as crystallization nuclei.

These nodules always have a higher specific gravity than the average of the material.

Several examples of these are given in Table 1.

These results and the analyses made on the different fractions show that the nodules are almost exclusively composed of the oxides, while the fine particles are still in the form of the hydrates.

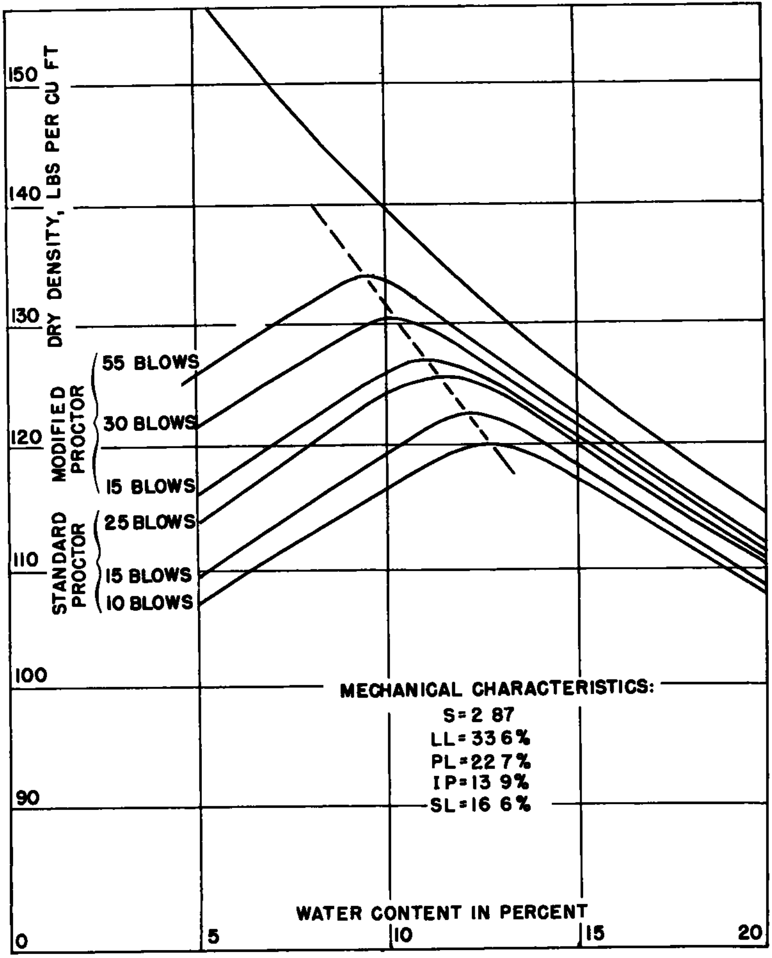

Figure 1. Proctor Test.

TEST METHODS AND PROPERTIES OF LATERITES

All laterites possess peculiar mechanical properties; this necessitates modifications in the test methods.

The nodules are always friable and of porous structure.

The fine particles are always flocculated.

Granulometric analyses and Atterberg limits

For the separation of the fine particles from the nodules wet sieving is necessary. In addition, for the hydrometer analysis deflocculation of the fine constituents is required.

Although some laterites can be easily dispersed by means of ammonium or sodium phosphate, in many cases this is not successful.

Regarding the Atterberg limits, it must

TABLE 1

Values of specific gravities:	Ex 1	Ex 2	Ex 3
. Fraction			
>0.5 mm	2.76	3.13	3.13
0.5 - 0.1 mm	2.73	2.75	2.65
0.1 - 0.05 mm	2.71	2.64	2.63
<0.05 mm	2.65	2.63	2.60

be noted that the mixing of the particles during the determination of the liquid limit leads, on one hand, to a breaking up of the fines and, on the other hand, to a slight deflocculation.

These deflocculated fines absorb an additional amount of water. It is, therefore, difficult to give exact values for the Atterberg limits, since the limits obtained by experiment depend on the modifications which the material has undergone in the course of the determination[1].

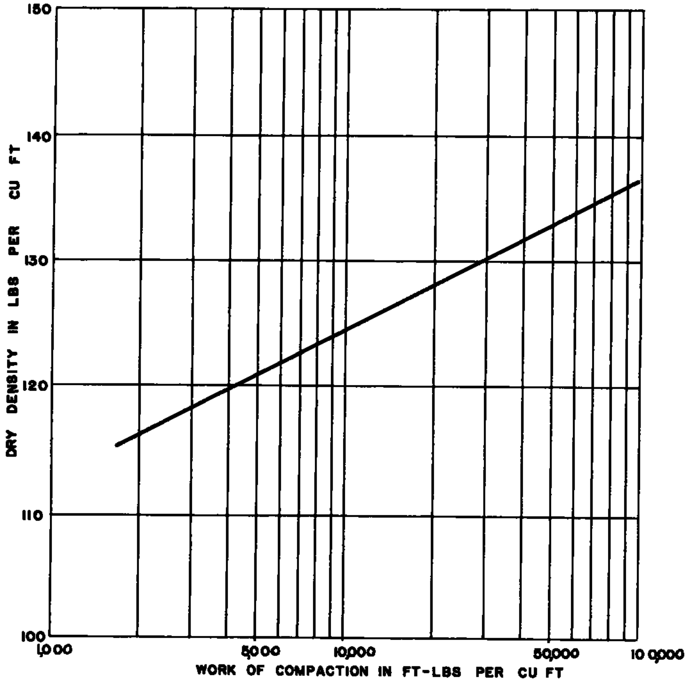

Figure 2. Variation of Maximum Dry Density as a Function of the Work of Compaction.

All granulometric curves of laterites show a discontinuity between 2 and 0.5 mm. This discontinuity indicates that the nodules do not form in a continuous manner from the fines, but that they form around crystallization nuclei as has been indicated above.

In addition, the majority of granulometric curves show that laterites contain an appreciable quantity of fine windblown sand.

Proctor Test

The fragility of the laterite nodules is such that it is necessary to use new material for each point of the Proctor curve.

Figure 1 shows Proctor curves for different compactive efforts. Figure 2 shows that, if plotted on semilog paper, the variation of the dry density as a function of the compactive effort can be expressed by a straight line.

Classification of Laterite Soils

The majority of laterites generally possesses the following characteristics: (1) less than 35 percent passing the number 200 sieve; (2) liquid limit less than 40; and (3) plasticity index larger than 10.

The general types of laterites, therefore, fall into Class A 2-6 (AASHO Designation M 145-49). The fraction smaller than five microns usually has a liquid limit from 50 to 60 and a plasticity index from 15 to 30. These fines, therefore, correspond to an A-7 clay. Although classes as A 2-6 soils, laterites generally rank in road construction at least equal with soils of a Class A 1-b.

UTILIZATION OF LATERITES FOR THE CONSTRUCTION OF UNPAVED ROADS

Many tests have been made on sections that during the dry season formed cor-

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[1] See: "Laterite Soils and Their Stabilization" by Hans F Winterkorn and E. C. Chandrasckharan, Proc. 30th Ann Meeting, Highway Research Board (January 1951).

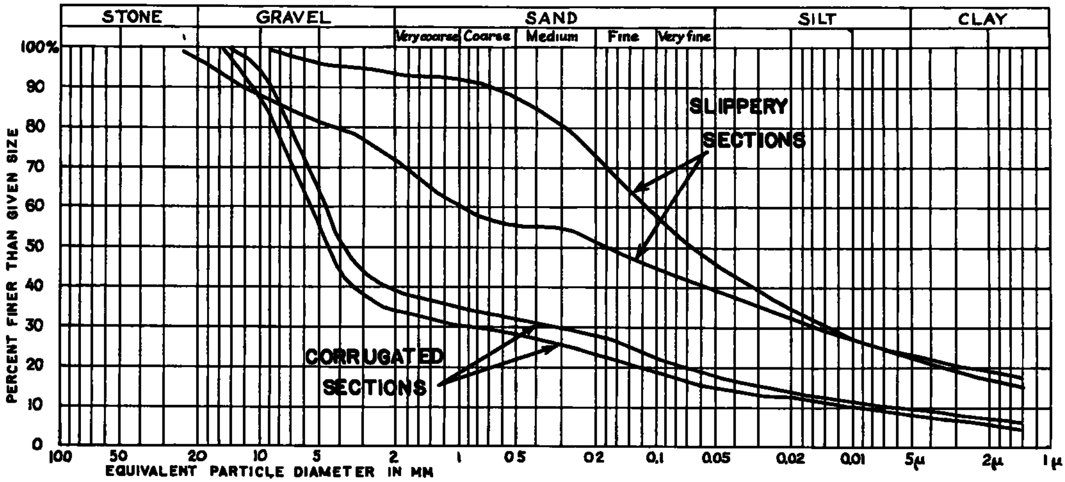

Figure 3. Granulometric analyses: Samples from unsatisfactory sections of unsurfaced roads.

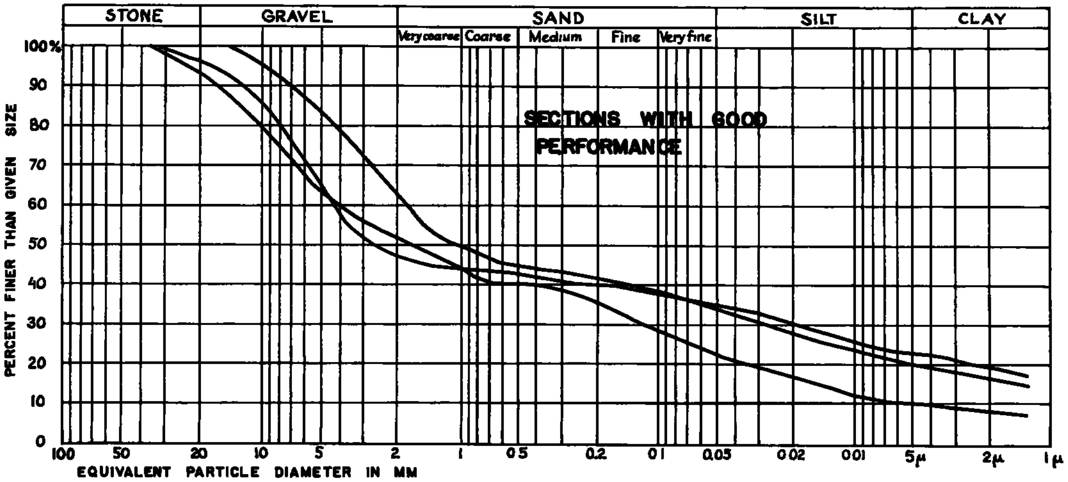

Figure 4. Samples from satisfactory sections of unsurfaced roads.

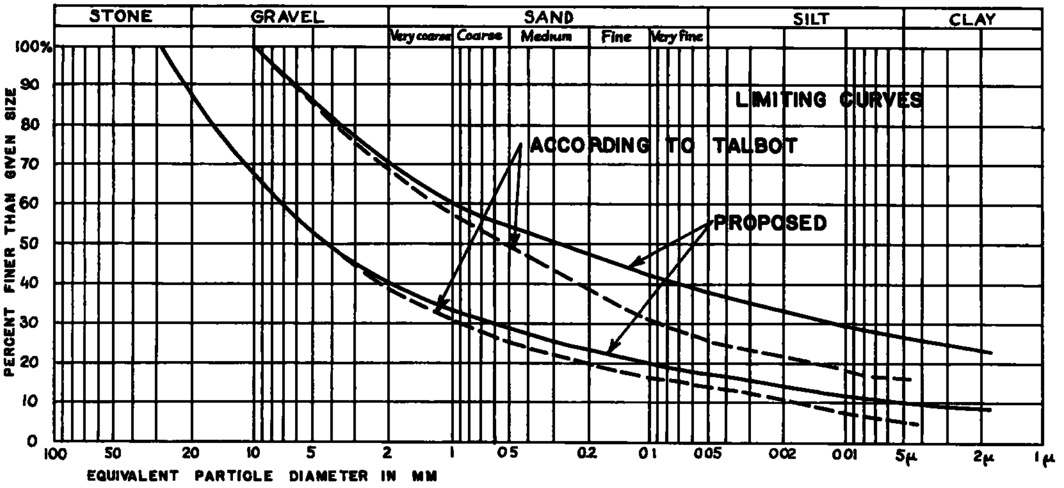

Figure 5. Granulometric analyses: Unsurfaced Roads.

rugations and also on sections that during the rainy season either showed lack of stability or excessive slipperiness. Typical curves for each of these conditions are reported in Figures 3 and 4. The curve envelopes of materials that give satisfactory performance have been derived from a number of granulometric analyses and are reported in Figure 5.

In the small-size range, these envelopes differ from the formerly accepted curves of Talbot. The mortar content must exceed the values indicated by the Talbot curves if the road surface is to retain a good riding quality. The clay content must lie between 10 and 25 percent. The plasticity indices of satisfactory materials have all been found to lie between 15 and 25 percent. One must notice, however, that this plasticity condition is not sufficient by itself, since materials showing formation of corrugations possessed correct plasticities. As a matter of fact, the two conditions of granulometry and plasticity cannot be separated.

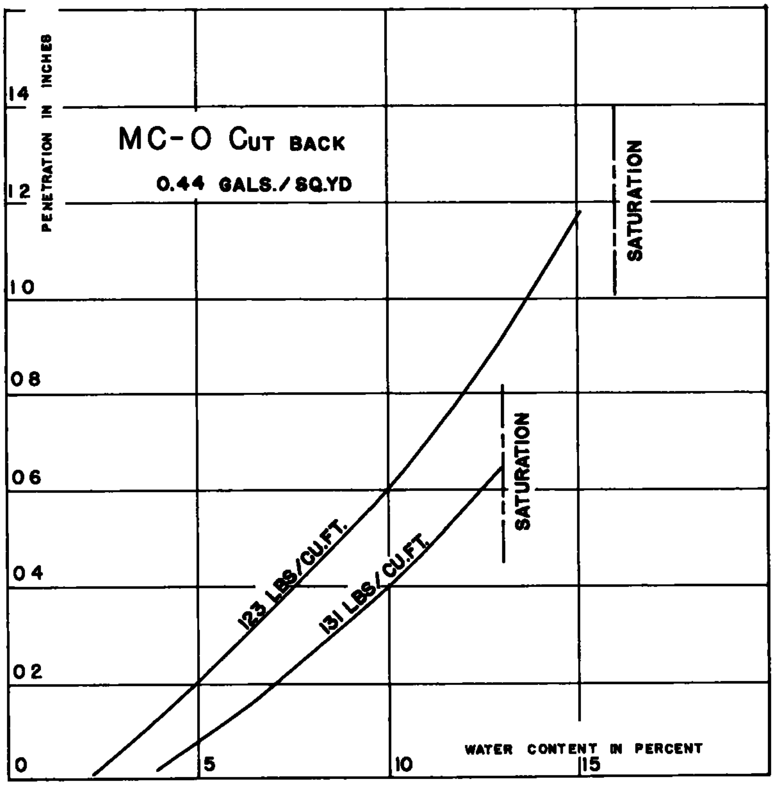

Figure 6. Influence of water content on the penetration of cut-back asphalt.

Maintenance of Nonpaved Highways

The maintenance of nonpaved highways is generally done by motor grader. This treatment leads to a disintegration of the upper portion of the highway. If the respective materials are in too advanced a stage of alteration, it is no longer possible to shape the affected portion without incorporation of clay.

This phenomenon is due to the fact that the laterite placed on the road surface loses part of the fines in the form of dust and also to the fact that certain laterites placed on roads age and lose their plasticity by formation of oxides. One says then that the laterite has gone dead.

Generally speaking, unpaved highways that are expected to be maintained by motor grader must be constituted of "live" laterites, that means of such whose fines are still rich in hydrates. Before placing materials on the highway one should check the state of evolution of the fines by determination of their specific gravities and their chemical composition.

UTILIZATION OF LATERITES FOR PAVED HIGHWAYS

Impregnation of Laterites

Numerous tests have been performed over a number of years on roads made of compacted, impregnated, and paved laterites. The principal difficulty encountered in the course of construction was in the impregnation of the laterite. As a matter of fact, impregnation is only possible after thorough moistening of the surfaces. Figure 6 shows the influence of moisture content on the penetration of the binder. Experience has proved that even in the case of laterites having a high plastic index, penetration is satisfactory if the surface is thoroughly moistened.

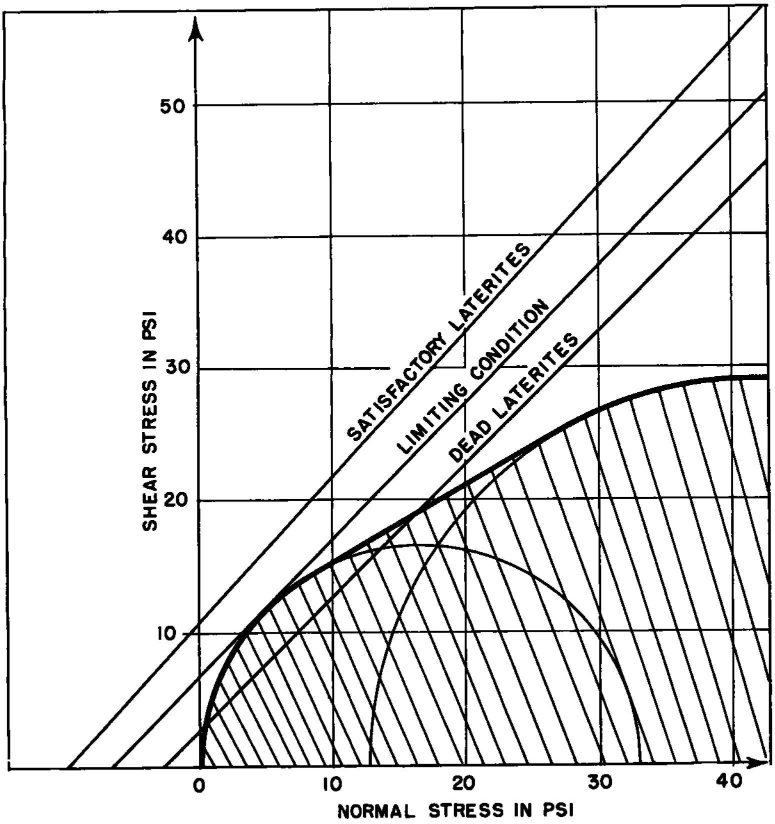

Figure 7. Stress envelope for 5-ton load.

Trafficability of Paved Highways

Simply impregnated and paved laterite roads have been constructed only within the last few years, and data on their permanence and performance under heavy traffic are still insufficient for complete evaluation. However, some investigations have already been performed on highways that have deteriorated under traffic. Particularly the intrinsic curves for compacted laterites have been determined, and have been employed for working out standards regarding the quality of laterite highways.

In Figure 7 is shown the stress envelope corresponding to the effect of a 5-ton wheel load on the road surface. For a satisfactory strength of the road bed, it is sufficient if the intrinsic curve of the material does not cut the stress envelope. Since the majority of laterites have angles of friction of at least 40 deg., it is sufficient if the cohesion is at least equal to 7.1 psi. Before constructing paved highways on laterites, one must make certain that at least this amount of cohesion is obtained.

ACKNOWLEDGMENT

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Soil Stabilization with Lime-Flyash Mixtures: Preliminary Studies with Silty and Clayey Soils

T. Y. CHU, Assistant Professor of Civil Engineering,
D. T. DAVIDSON, Associate Professor of Civil Engineering,
W. L. GOECKER, Research Assistant, and
Z. C. MOH, Research Assistant,
Iowa Engineering Experiment Station, Iowa State College

Introductory Remarks by the Chairman: One of the oldest inorganic cementing materials used by the Western World is a combination of hydrated lime with volcanic cinders. Many venerable structures still standing in Western Europe two thousand years after their construction attest to the quality of this hydraulic cement. While volcanic ashes are still employed where available for this purpose, modern industry is producing a similar material in the form of flyashes.

Unfortunately, all flyashes are not alike or equally suited for cementing and stabilization purposes. For proper evaluation of the soil stabilizing ability of flyash-lime mixtures dependable testing methods must be developed and standardized. Davidson and his associates at the Iowa Engineering Experiment Station have undertaken to develop a tentative test method for this purpose and are presenting it together with a preliminary evaluation of the merits of lime-flyash stabilization with a number of silty and clayey soils.

●THE use of admixtures for the stabilization of soils has been a subject of great interest to highway engineers in recent years. Various organic and inorganic materials have been investigated for possible use as stabilizing agents in the construction of subbase, base, or surface courses. Mixtures of lime and flyash are among those that have shown promise (1, 2). This paper presents results of laboratory studies of lime-flyash stabilization of silty and clayey soils sampled in Texas, Virginia, Iowa, and Kentucky (see Table 1). The main objectives of the studies were to (1) develop a test method for the preliminary evaluation of lime-flyash-stabilized silty and clayey soils and (2) make a preliminary evaluation of the merits of lime-flyash stabilization with the soils sampled.

MECHANISM OF LIME-AND-FLYASH STABILIZATION

Flyash is the gray, dust-like ash which results from burning powdered coal. The coal is burned while in suspension in air, and the resulting ash consists largely of tiny spheres of silica and alumina glass. The ash is similar to volcanic ash used in early Roman construction (3) and is a pozzolanic material, that is, it is not in itself a cement, but it reacts with lime and water to form a cement. This cementitious material may be regarded as a calcium silicate, but since good pozzolans ordinarily contain small percentages of alkalies, sodium and potassium, it is likely that more-complex compounds are also important. On the other hand, on the basis of free-lime determinations in hydrated mixtures of portland cement and flyash (4), some engineers believe that lime acts only as a catalyst to hydrate the flyash.

It is the reaction of lime and flyash which is utilized to stabilize soils. Theoretically, the stabilized soils should be compacted to a maximum to make maximum grain-contact areas available for cementing. Such a maximum compaction for any given compactive effort is obtained at an optimum moisture content which, however, may differ from the moisture content needed for a complete lime-and-flyash reaction.

FACTORS AFFECTING LIME-FLYASH STABILIZATION

The stability of lime-flyash-soil mixtures is affected by many variables. The variables listed in Figure 1 are the more-important ones affecting the stability of a processed soil in which lime and flyash are the only additives. The use of a small amount of a third additive to improve the effectiveness of lime-flyash stabilization will introduce still other variables. It is planned ultimately to evaluate as many of the variables as possible in the Iowa Engineering Experiment Station research on lime-flyash stabilization, of which the work presented in this paper is a part. The studies with silty and clayey soils have been primarily concerned with: (1) amount of lime and flyash to be added, (2) ratio of lime to flyash, (3) moisture content during mixing and compaction, (4) length of curing, and (5) curing conditions.

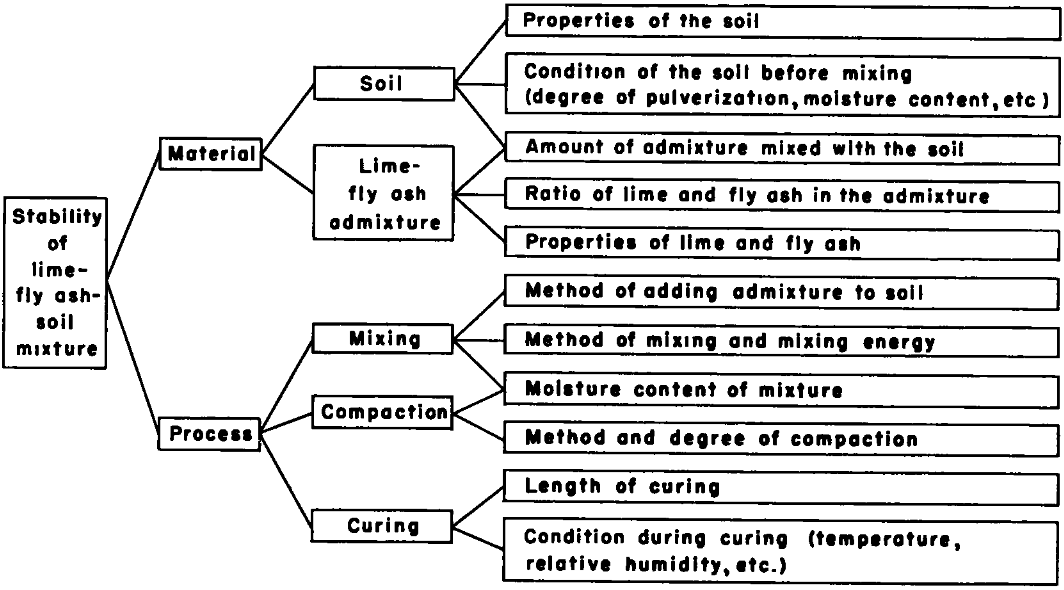

Figure 1. Factors effecting the stability of lime-flyash-soil mixtures.

TEST METHOD FOR PRELIMINARY EVALUATION

One of the first things needed for conducting the preliminary evaluation studies of lime-and-flyash-stabilized silty and clayey soils was a simple method of test to provide data for determining benefits to the stability of the soils processed and for selecting the more promising combinations of lime, flyash, and soil for further studies. Other features desired in the test were: (1) use of small test specimens molded to near standard Proctor density, (2) use of curing conditions similar to those obtainable in the field, (3) testing of specimens after immersion in water, and (4) attainment of a fairly high degree of reproducibility of test results.

Test procedures utilizing the unconfined-compression test and specimens 2 inches high and 2 inches in diameter were developed to meet these requirements.

The experiments conducted in developing the test procedures deal with many of the previously mentioned variables affecting the stability of lime-flyash-soil mixtures.

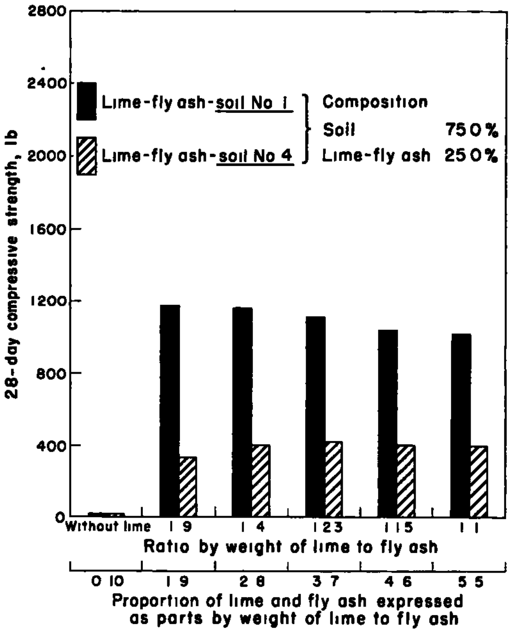

Figure 2. Effect of variations in the Ratio of lime to flyash on the compressive strength of two lime-flyash stabilized soils.

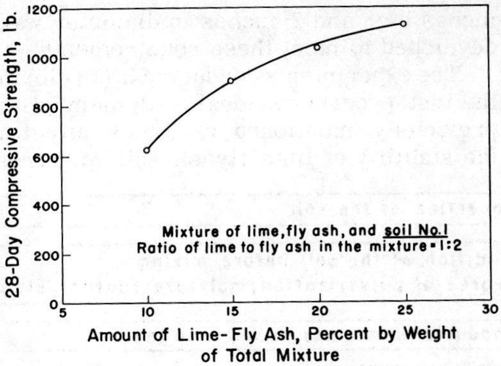

Figure 3. Effect of variations in the amount of lime and flyash on the compressive strength of a lime-flyash-stabilized soil.

They are discussed in Appendix A. On the basis of these experiments, the method of test presented in Appendix B is recommended for the preliminary evaluation of lime-flyash-stabilized silty and clayey soils.

The main steps in conducting the test are as follows:

1. Preparation of mixtures. Lime, flyash, and pulverized soil are dry mixed. The proper amount of water is then mixed with the three materials to obtain a uniform mixture at optimum moisture content for standard Proctor density.

2. Molding of specimens. Immediately after mixing, the moist mixture is used for the molding of specimens 2 inches in diameter by 2 inches high. The molding apparatus shown in Figure 4 was used to compact specimens to near standard Proctor density.

3. Curing of specimens. The specimens are cured in a moist cabinet capable of maintaining a temperature of 70±3 F. and a relative humidity of not less than 90 percent. Some of the specimens are cured for 7 days and others for 28 days.

TABLE 1

BRIEF DESCRIPTION OF THE FOUR SOIL SAMPLES

Soil	No. 1	No. 2	No. 3	No. 4
Source	Texas	Virginia	Iowa	Kentucky
Geological Origin	Coastal plain deposit, largely deltaic (Beaumont clay)	Residual soil on diorite	Friable loess from near Missouri River floodplain	Natural levee deposit from Ohio River
Soil Series	Lake Charles	Davidson	Hamburg	Melvin*
Horizon	C	B	C	C*
Engineering Classification (AASHO)	A-7-6(20)	A-7-5(18)	A-4(8)	A-6(8)

*There is some question as to whether the soil should be classified in Melvin or Lindside series. The sample is probably from the C horizon.

4. Testing of specimens. Cured specimens are completely immersed in distilled water at near 70 F. for 24 hours and then are tested for compressive strength by the unconfined-compression test. The maximum test load causing failure of the specimen is taken as its compressive strength.

PRELIMINARY EVALUATION OF STABILIZED SILTY AND CLAYEY SOILS

A main objective of the overall lime-flyash-soil investigation is to evaluate combinations of lime and flyash as stabilizing agents for a wide variety of soils

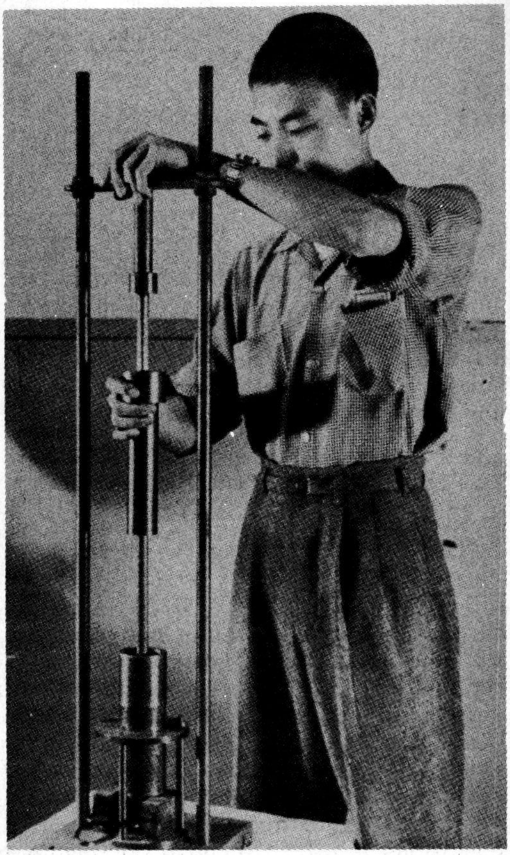

Figure 4. Compaction apparatus for molding test specimens 2 inches in diameter by 2 inches high. This apparatus was also used for determining moisture-density relations of raw soils and lime-flyash-soil mixtures.

TABLE 2

PROPERTIES OF THE FOUR SOIL SAMPLES

Soil		No. 1	No. 2	No. 3	No. 4
Textural Composition, Percent	Sand-Size	7.7	3.4	0.7	7.7
	Silt-Size	48.2	12.0	78.3	55.7
	Clay-Size	44.1	84.6	21.0	36.6
	Colloidal-Size	36.8	72.9	15.8	19.4
Textural Classification[a]		Clay	Clay	Silty clay loam	Silty clay
Liquid Limit, Percent		57.4	75.2	31.8	33.0
Plastic Limit, Percent		19.9	51.2	24.6	22.4
Plasticity Index		37.5	24.0	7.2	10.6
Shrinkage Limit, Percent		14.4	27.3	25.2	22.9
Centrifuge Moisture Equivalent, Percent		21.2	29.5	15.2	21.4
Field Moisture Equivalent, Percent		21.2	47.2	26.4	25.5
Specific Gravity, 25 C./4 C.		2.67	2.91	2.68	2.69
Predominant Clay Mineral[b]		Montmorillonite	Halloysite	Montmorillonite	Montmorillonite or illite
Cation Exchange Cap., m.e./100g		25.5	11.3	13.4	11.1
pH		5.9	4.1	7.8	4.5
Carbonates, Percent		2.7	1.0	10.2	1.2
Organic Matter, Percent		0.62	0.52	0.17	0.94

[a] Textural Classifications are based on the Bureau of Public Roads System except that 0.074 mm. was used as the lower limit of the sand fraction.

[b] Determined by the differential thermal analysis of the fraction passing no. 200 sieve.

occurring in different parts of the United States.

The four soil samples used in the studies reported in this paper are silty and clayey textures soils sampled in Texas, Virginia, Iowa, and Kentucky. The sources and a brief description of the samples are given in Table 1 and important properties are compared in Table 2.

Especially noteworthy are the difference in clay mineral composition of the two clay textured soils (No. 1 and No. 2) and the high carbonate content of Soil 3. A major portion of the carbonates in this soil occur in the silt-size range. The properties of the hydrated lime and the flyash used are given in Table 3. Future studies will include quicklime and other varieties of lime and flyash.

Preliminary evaluations of the lime and flyash with each of the four soils were made by using the previously described test method. The lime-and-flyash content of a mixture is expressed in percent by weight of the total mixture. The proportion of lime and flyash in the mixture is expressed as a ratio by weight of lime to flyash. Compressive strength values reported are the

TABLE 3

PROPERTIES OF THE HYDRATED LIME AND FLYASH USED

Properties	Hydrated Lime[a]	Flyash[b]
Specific gravity	----	2.67
Fineness		
Material passing no. 325 sieve, percent	99.0	94.3
Specific surface area, sq. cm. per g. (Based on the specific gravity of 2.67)	----	3,470
Chemical analysis		
Silicon dioxide, percent	0.80	38.90
Aluminum oxide, percent	----	22.92
Iron and aluminum oxides, percent	0.82	----
Magnesium oxide, percent	0.49	0.52
Sulfur trioxide, percent	----	2.00
Calcium carbonate, percent	0.77	8.36
Total calcium hydroxide, percent	97.82	----
Available calcium hydroxide, percent	97.38	----
Loss on ignition	24.56	2.10

[a] The hydrated lime and the test data were furnished by the Linwood Stone Products Co., Inc., Buffalo, Iowa.

[b] The flyash was from the Paddy's Run Station, Louisville Gas and Electric Co., Louisville, Kentucky, the tests on flyash were made by the Robert W. Hunt Co., Chicago, Illinois.

average of three test specimens. The findings are as follows:

1. Moisture-density relations. Lime-flyash admixtures affect the moisture-density relationship. As shown by the illustrative data in Table 4, maximum dry density is decreased and the optimum moisture is usually slightly increased. The decrease in density observed may not be very important since the stability of lime-flyash-stabilized soils also depends on many other factors.

2. Compressive strength. The addition of lime and flyash to the soils materially improves their stability as indicated by the compressive strength of 24-hr. immersed specimens. Untreated specimens fail during the immersion period. As shown by the data in Table 5, the compressive strength of stabilized soil specimens range from about 200 lb. to over 700 lb. after 7-day moist-curing at

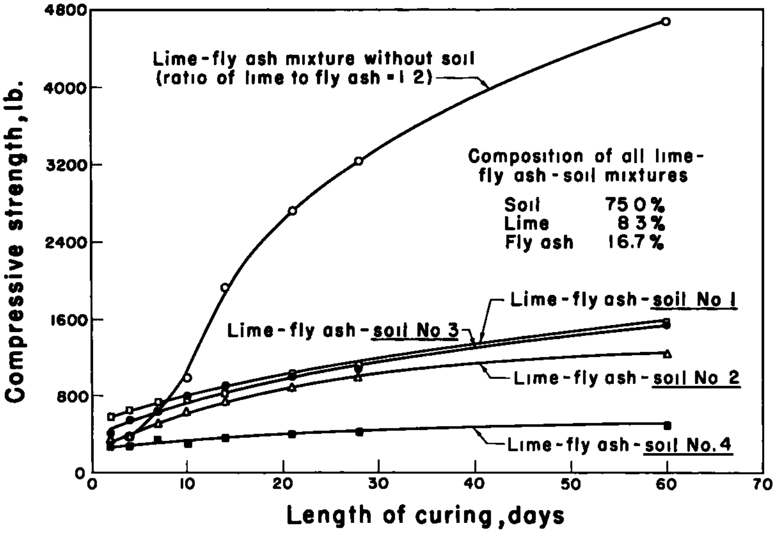

Figure 5. Relationship between length of curing and compressive strength. The curves compare the rate of increase in compressive strength of mixtures of lime and flyash with each of the four soils. A curve representing a lime-flyash mixture without soil is also shown.

TABLE 4

DATA ILLUSTRATING THE EFFECT OF LIME AND FLYASH ON MOISTURE-DENSITY VALUES[a]

Soil	Lime and Flyash Admixture,[b] Percent	Maximum Dry Density, lb per cu ft	Optimum Moisture Content, Percent
No 1	33	97.8	22.5
	No admixture	109.5	18.3
No 2	33	87.8	32.2
	No admixture	85.6	37.1
No 3	33	97.7	20.6
	No admixture	109.9	18.2
No 4	33	97.3	22.6
	No admixture	108.6	17.8

[a] Determined with the molding apparatus shown in Figure 4 The molding procedure was correlated to give moisture-density values closely approximating those obtainable by the standard Proctor density test (ASTM Designation D698-42T)

[b] Ratio of lime to flyash is 1 to 1

TABLE 5

COMPRESSIVE STRENGTH OF THE FOUR SOILS STABILIZED WITH LIME AND FLYASH ADMIXTURES

Soil	Lime-Flyash Admixture[a]	Compressive Strength	
		7-day	28-day
	percent	lb	lb
No 1	15	640	900
	25	740	1,130
No 2	15	230	350
	25	525	1,005
No 3	15	485	735
	25	640	1,045
No 4	15	225	255
	25	350	430

[a] Ratio of lime to flyash is 1 to 2

near 70 F. and increases to as much as 1,130 lb. after 28-day curing. Criteria for judging the adequacy of these strength values must be determined by additional studies.

The data on hand indicate that the texture of fine-grained soils may not be the controlling factor in their response to lime-and-flyash treatments. Test results for Soils 1, 2, and 3 show that about the same degree of stabilization can be obtained with silty or clayey textured soils. Sufficient data are not yet available to explain the comparatively low compressive strengths obtained with Soil 4.

3. *Ratio of lime to flyash.* Variations in the ratio of lime to flyash will affect the compressive strength of mixtures of lime, flyash, and soil. The effect may not be great for some soils when the variation is within a certain range. This is illustrated for Soils 1 and 4 by the data in Figure 2. The variation in compressive strength is small when the lime-and-flyash ratio is within the range 1 to 1 to 1 to 9. The same general relationship was found for a mixture of lime and flyash without soil. Further studies of the ratio of lime to flyash are planned.

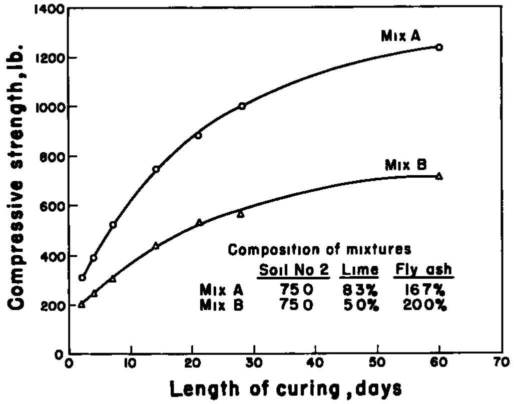

Figure 6 Curves comparing the rate of increase in compressive strength of lime-flyash-soil mixtures containing varying proportions of lime and flyash. The ratio of lime to flyash in Mix A is about 1 to 2, that in Mix B is 1 to 4.

4. *Amount of lime-and-flyash admixture.* For a given ratio of lime to flyash, the compressive strength of the lime-flyash-soil mixture will increase with an increase in the amount of lime and flyash used. This is shown in Table 5; the mixtures containing 25 percent of lime and flyash have higher compressive strengths than those containing 15 percent. The relationship between amount of lime and flyash and compressive strength is further illustrated by the curve in Figure 3.

5. *Selection of compositions for further investigation.* Results of the preliminary evaluation studies may be used as a guide in selecting several lime-flyash-soil mixtures for additional study by test methods which have been more closely correlated with service behavior in the field. The wetting-and-drying and freezing-and-thawing test procedures used for soil-cement mixtures (ASTM Designation: D559 and D560) have been used for this purpose by other investigators (1, 2, 5).

ACKNOWLEDGMENT

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Appendix A

Development of Test Method for Preliminary Evaluation

In developing the test method, a series of experiments were performed to determine the most desirable ways of preparing, curing, and testing lime-flyash-soil specimens. The properties of the lime, the flyash, and the four soils used in the experiments are given in Tables 1, 2, and 3. Lime-and-flyash admixtures are expressed in percent by weight of the lime-flyash-soil mixture; ratios of lime to flyash are by weight. Most of the data reported are the average of results obtained from tests using three specimens.

PREPARATION OF SPECIMENS

The moisture content of a lime-flyash-soil mixture during compaction and the method and degree of compaction have an important effect on the stability of the compacted mixture. In the lime-and-flyash-stabilization studies with silty and clayey soils, specimens 2 inches in diameter and 2 inches high are molded to near standard Proctor density. The molding apparatus used is shown in Figure 4.

The lime-flyash-soil mixture is placed in the molding cylinder in one layer and compacted by a 5-lb. hammer dropping from a height of 12 inches. Experiments with a number of fine-grained raw soils and stabilized soils showed that five blows of the hammer on each side of the specimen are needed to compact the soil or stabilized mixture to near standard Proctor density. The details of the compaction apparatus and the procedure for its use will be given in another paper. ("Some Laboratory Tests for the Evaluation of Stabilized Soils", to be presented at the 1955 Annual Meeting of the American Society for Testing Materials by T. Y. Chu and D. T. Davidson).

The moisture-density relations and the moisture-compressive-strength relations were studied with mixtures of lime and flyash and each of the four soils. Specimens used for determining compressive strength were cured at 70 ± 3 F. in a moist cabinet for seven days and then were tested according to the procedure given in Appendix B.

Results of the experiments (see Table A) indicate that for most of the mixtures the amount of moisture required for maximum dry density is more than that required for highest compressive strength. However, the difference, if there is any, between the two moisture requirements is not great. Since the moisture content giving maximum dry density can be determined with less effort and in a much shorter time, it seems desirable to use this moisture content for preparing lime-flyash-soil specimens.

TABLE A

COMPARISON OF THE MOISTURE CONTENT GIVING MAXIMUM DENSITY WITH THAT GIVING HIGHEST COMPRESSIVE STRENGTH OF LIME, FLYASH, AND SOIL MIXTURES

Soil	Lime and Flyash Admixture,[a] Percent	Moisture Content, Percent	
		For Maximum Dry Density	For Highest 7-day Compressive Strength
No 1	25	21 6	20 8
	33	22 5	21 5
No 2	33	32 2	32 2
No 3	25	20 3	16 8
	33	20 7	18 8
No 4	33	22 5	19 0

[a] Ratio of lime to flyash is 1 to 1

CURING OF SPECIMENS

The stability of lime-flyash-soil mixtures is dependent upon the length of curing and such curing conditions as the temperature and the relative humidity during curing. On the basis of the following curing experiments, it appears desirable to cure specimens in a moist cabinet capable of maintaining a temperature of 70 ± 3 F. and

TABLE B

DATA ILLUSTRATING THE EFFECT OF VARIATIONS IN TEMPERATURE DURING CURING ON THE COMPRESSIVE STRENGTH OF LIME, FLYASH, AND SOIL MIXTURES

Soil	Lime and Flyash Admixture, Percent	Ratio of Lime to Flyash	Compressive Strength, lb.				
			Cured at 140±2 F[a] for 7 days	Cured at 110±2 F[a] for 7 days	Cured at 70±3 F[b]		
					7-day	28-day	60-day
No. 1	25	1:2	3600	1850	740	1130	1580
No. 2	15	1:2	430	315	230	350	320
	25	1:2	1590	1200	525	1005	1235
	25	1:4	765	590	310	575	725
No. 3	25	1:2	7040	N.D.[c]	640	1045	1550
No. 4	15	1:2	1575	265	225	255	310
	25	1:2	3125	550	350	430	505
	25	1:4	2330	335	300	380	425

[a] Specimens were kept in air-tight containers to prevent evaporation during curing.

[b] Specimens were cured in a moist cabinet capable of maintaining a relative humidity of not less than 90 percent.

[c] Not determined.

TABLE C

DATA ILLUSTRATING THE EFFECT OF VARIATIONS IN RELATIVE HUMIDITY DURING CURING ON THE COMPRESSIVE STRENGTH OF LIME-FLYASH-SOIL MIXTURES

Soil	Lime and Flyash Admixture,[a] Percent	Compressive Strength, lb.					
		7-day Curing at 70±3 F.			28-day Curing at 70±3 F.		
		32% R. H.[b]	65% R. H.	100% R. H.	32% R. H.	65% R. H.	100% R. H.
No. 1	25	530	480	540	765	770	770
No. 2	25	360	405	380	940	870	860
No. 3	25	810	735	590	1170	1120	980
No. 4	25	370	355	330	575	425	380
Lime and flyash mixture without soil, (ratio 1 to 2)		880	1110	740	880	2475	2885

[a] Ratio of lime to flyash is 1 to 2.

[b] Relative humidity.

a relative humidity of not less than 90 percent for periods of 7 and 28 days. In these experiments the compressive strength was determined by using 24-hr. immersed specimens, as discussed in Appendix B.

Temperature During Curing

A comparison of the compressive strengths of test specimens cured at various temperatures is shown in Table B. The data for 7-day specimens indicate that the use of elevated temperatures (140 ± 2 F and 110 ± 2 F) results in a compressive strength much higher than obtained by curing at 70 ± 3 F. For many of the mixtures tested, the compressive strengths of 7-day specimens cured at near 140 F. are even much higher than the compressive strengths of 60-day specimens cured at near 70 F. It is concluded that curing at near 140 F. as used by other investigators (2) gives results which may not be attainable in the field by conventional methods of curing.

Relative Humidity During Curing

The effect of the relative humidity during curing on compressive strength is illustrated by the data in Table C. For many of the mixtures tested, the highest compressive strength is obtained with relative humidities lower than 100 percent. Since no uniform trend of variation in compressive strength is indicated by the test data, it is not possible to select a relative humidity which will result in the highest compressive strength for all mixtures. Because of this, the curing of specimens at a relative humidity not lower than 90 percent is recommended.

Effect of Carbon Dioxide in the Air

The data of experiments with mixtures of lime and flyash and each of the four soils indicate that the effect on the compressive strength of cured specimens of carbon dioxide in the air during curing is not great. For this reason, it appears not to be necessary to control the amount of carbon dioxide present during the curing period.

Length of Curing

The compressive strength of mixtures of lime, flyash, and soil increases with an increase in the length of curing. Among various factors affecting the rate of increase in compressive strength are the kind of soil stabilized and the amount and proportion of lime and flyash used. Curves illustrating the influence of these variables on the rate of increase in compressive strength are given in Figures 5 and 6. The data shown were obtained by using specimens cured for varying periods at 70 ± 3 F. and in a relative humidity of not less than 90 percent.

Especially noteworthy in Figure 5 is the remarkably rapid increase in compressive strength of the lime-and-flyash mixture without soil during the 60-day curing period studies. These data and the data of related experiments suggest that the relationship between the compressive strength of a lime-flyash-soil mixture and the amount if lime and flyash used is as follows: If the ratio of lime to flyash is maintained constant, the larger the amount of lime and flyash, the faster the increase in the compressive strength of the lime-flyash-soil mixture. Experiments are being made to verify this relationship and to investigate curing periods longer than 60 days.

From the above discussion, it is apparent that a fairly long curing period is desirable in evaluating the stability of mixtures of lime, flyash, and soil. It may be that a curing period longer than one month may not be practical, especially when results are urgently needed in planning seasonal construction projects. A solution is to use the conventional 7-day and 28-day curing periods used for testing portland-cement concrete.

TESTING OF SPECIMENS

Specimens after curing are immersed in distilled water for 24 hours. The water absorption and the volume change of a specimen during immersion can be determined by weighing and measuring the height and diameter of the specimen before and after the immersion. The compressive strength after immersion can be determined by the unconfined-compression test.

Test data obtained in the experiments with silty and clayey soils indicate that the compressive strength values are usually sufficient for preliminary evaluation purposes. The compressive strength of a specimen may be expressed in terms of the total load causing failure of the specimen during the unconfined compression test; or it may be expressed as a unit stress (in psi.) computed from the total load causing failure. The first alternative is recommended in the test method given in Appendix B.

Appendix B

Recommended Test Method for the Preliminary Evaluation Of Lime-and-Flyash-Stabilized Silty and Clayey Soils

This method describes test procedures for evaluating the effectiveness of lime-and-flyash stabilization of silty and clayey soils.[1] The test may also be used as an aid in the selection of lime-flyash-soil mixtures for final evaluation tests.

APPARATUS

The apparatus used in this test consists of the following: (1) Mechanical Mixer, a mechanical mixer capable of producing uniform mixtures of soil, lime, flyash, and water; (2) Compaction Apparatus, a compaction apparatus capable of preparing stabilized soil specimens of uniform density. An apparatus found suitable for molding specimens, 2-inch diameter and 2-inch high is shown in Figure 4; (3) Moist Cabinet, a moist cabinet capable of maintaining a temperature of 70±3 F. and a relative humidity of not less than 90 percent; (4) Testing Machine, a loading device of more than 5,000-lb. capacity, capable of applying the load through a uniform motion of the testing head at a rate of 0.1-inch per minute; and (5) Balance, No. 10 Sieve, Etc.

SAMPLES

Representative samples of soil, hydrated lime, and flyash are used for the test. With some modifications in the test procedure, other types of lime may also be used.

PREPARATION OF SOIL

An air-dried sample of soil is pulverized and screened through a No. 10 sieve. The pulverization should be so done as not to reduce the size of the individual soil particles. The entire sample of many silty and clayey soils will pass through the sieve. It is believed that the test method also may be used for soils containing particles retained on the sieve, if the retained fraction is less than 15 percent by weight of the total soil sample.

PREPARATION OF MIXTURE OF LIME, FLYASH, AND SOIL

The pulverized soil is dry mixed with a predetermined amount of hydrated lime and flyash. The proper amount of distilled water is then mixed with the three materials to obtain a mixture at optimum moisture content for maximum density. Both the dry and the wet mixing may be done with a mechanical mixer.

MOLDING OF SPECIMENS

Immediately after mixing, the moist mixture is used for molding specimens 2 inches in diameter and 2 inches high. (Specimens having a height-diameter ratio of one are used mainly for convenience in molding. If desired, specimens 2 inches in diameter and 4 inches or more in height may be used.) Usually three or more specimens are prepared for each test result required. It is desirable to so mold the specimens as to simulate the compacted lime-flyash-soil mixture obtainable under actual field conditions. The density of specimens molded may be determined by weighing the specimens and by measuring their height, since the diameter of all specimens is constant.

CURING OF SPECIMENS

Specimens are cured in a moist cabinet capable of maintaining a temperature of

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[1] This test method is recommended mainly on the basis of experiments using several silty and clayey soils and one variety each of hydrated lime and flyash Experiments with a wider variety of lime, flyash, and soil are needed to verify the suitability of the test procedure and to make improvements in it.

70±3 F. and a relative humidity of not less than 90 percent. The length of the curing period may be 7 or 28 days.

TESTING OF SPECIMENS

After specimens are cured, they are completely immersed in distilled water at about 70 F. for 24 hours. Each specimen is then tested for unconfined compressive strength. The rate of deformation during testing is 0.1 inch per minute. The maximum test load causing failure of the specimen is taken as its compressive strength.

Soil Stabilization with Resins and Chemicals

R. C. MAINFORT
Highway and Construction Materials Department
Dow Chemical Company

Introductory Remarks by the Chairman: Many federal, state, and private agencies have made essential contributions to the practice and science of soil stabilization. Most noteworthy among these are the Bureau of Public Roads, the state highway departments, the engineering departments of the Army, the Navy, and the Airforce, the Civil Aeronautics Administration, and the Bureau of Reclamation.

It is natural that the quality and quantity of contributions of these and other agencies varied with time, one or another being leading at one particular period. At one time, there was a golden age for the Civil Aeronautics Administration with respect to soil and soil stabilization research when Grieme had assembled a staff of keen and active men. Among these were David S. Jenkins, now director of the Saline Water Conversion Program of the Department of the Interior; George W. McAlpin, now chief soils engineer of the State of New York; and R. C. Mainfort, presently with the Dow Chemical Company.

During this period the CAA not only sponsored at various universities research that was of utmost importance and has by now largely become classical, but organized its own laboratories for further evaluation and supplementation of this research. Mainfort, who remained with the Soils and Pavement Research Section of the CAA until this section was dissolved, has enriched this symposium with a concise and condesned review of the work he directed while associated with the CAA.

●FOR several years various organizations have investigated the possibility of using chemical additives to alter the characteristics of natural soils in order that they might be more successfully used as a structural material. The desired effect might be obtained by bonding, waterproofing, or otherwise modifying the natural soil so that the resulting mixture can withstand the detrimental forces of weather, moisture, and load application. From the standpoint of highway and airport construction, the application of such techniques should permit the economical and efficient utilization of soils for the construction of durable wearing surfaces and base courses.

The improvement of natural soil by the addition of chemical admixtures is generally referred to as "chemical soil stabilization." For the purpose of this report, the term is used to describe any method whereby the engineering properties of natural soils are improved by the chemical or physicochemical interaction between an admixture and a soil and includes the use of such general materials as cement and bitumen.

Most of these soil stabilizers fulfill their function by imparting their own properties to the resulting mixtures. Recent studies, however, indicate that a group of substances, referred to as "trace chemicals," are capable of altering soil properties almost entirely by interaction with the surfaces of the soil particles (1). Such surface-active substances show promise of being effective in quantities in the range of 0.1 to 0.5 percent by weight of the soil.

The overall testing of prospective soil-stabilizing agents has covered a considerable period of time and has been the work of different investigators. For this reason, and because of the fact that uniform test procedures were not employed, the relative effectiveness of various admixtures can be presented only in a quantitative way.

GENERAL TESTING PROCEDURES

Other than the methods used for evaluating the effectiveness of soil-cement mixtures, there are no recognized standard procedures for testing the acceptability of chemically treated soil samples. Different investigators have used different methods, depending upon their own requirements or particular definition of a successful soil-stabilizing agent.

In many of the earlier investigations, mild exposure tests were used, and in some cases the treated samples were subjected only to a few hours of capillary moisture absorption or immersion. Certain materials tested in this manner appeared to have stabilizing properties which, under more severe laboratory and field exposures, proved to be merely temporary. As additional test data have become available, the trend has been toward more-severe laboratory-testing procedures supplemented by small-scale field-weathering studies.

Observation of field-weathered samples indicates that a properly stabilized soil should be able to withstand not only moisture absorption but also numerous cycles of freezing and thawing, wetting and drying, or other temperature variations in the presence of moisture. One form of laboratory-testing procedure in which the results compare closely to those obtained by field weathering has been reported by the Civil Aeronautics Administration (2).

An important factor in any laboratory evaluation test is the type and amount of curing allowed a sample prior to its subjection to exposure. The principal methods that have been used are (1) moist-curing, (2) oven-drying to a constant weight at 140 F, (3) air-drying to a constant weight, and (4) partially drying to a predetermined moisture content.

A partial drying of the sample is probably nearer to the field conditions likely to be attained, but this procedure is difficult to control and duplicate in the laboratory. For this reason, air-drying to constant weight or moist-curing are generally used. Materials requiring moisture for proper hardening (e.g. portland cement) should be moist-cured prior to any desired air drying.

Oven drying is advantageous for obtaining a uniform drying rate which can be easily duplicated and is suitable for curing natural soils and those treated with some admixtures. For admixtures such as resins and bitumens, however, the application of heat results in an improvement to the treatment that cannot be duplicated during normal construction procedures.

In order that the general effectiveness of proposed soil-stabilizing agents can be determined, they should be tested with a variety of soil types. The differences in soil composition and their corresponding reaction to chemical treatments have been the largest variables encountered in soil-stabilization studies.

In most testing procedures the natural soil obtained from the field is air dried, screened to a selected top size (usually through the No. 4 or No. 10 screen), and classified by standard soil-testing methods. The maximum density and optimum moisture are determined, and treated samples are molded to predetermined density by dynamic or static compaction. For important evaluations of more-promising stabilizers, the densities should be determined with the required admixture in the soil. After molding, the samples are cured and then subjected to laboratory or field exposure. The relative effectiveness of the treatment can be evaluated at the end of the test by an inspection of the sample, measurement of its moisture content, and determination of its compressive strength.

INORGANIC CEMENTING MATERIALS

Several inorganic cementing materials are capable of hardening or otherwise modifying the physical characteristics of soils, but none have been found which will impart appreciable waterproofing characteristics to the resulting structure. Inorganic cementing agents are particularly susceptible to moisture attack during conditions of freezing and thawing. A soil that has been successfully bonded will absorb considerably less moisture than the same untreated soil. This condition is usually a result of the inability of the bonded sample to swell, rather than a result of any waterproofing action of the treatment.

A relatively high percentage of an inorganic cementing agent (10 percent or more) is normally required for properly bonding a soil. Smaller quantities, however, are sometimes effective in improving the structural properties of soils by modifying their physical and chemical characteristics. Portland cement, lime, sodium silicate, and crushed slag are particularly effective in reducing the plasticity of fine-grained soils.

Portland Cement

Portland cement probably has been used more widely and with greater success than any other soil-stabilizing material, and its properties in this respect have been reported by numerous investigators. Although an excellent bonding agent for many soils, portland cement does not impart water-repellent characteristics to the treated soil; and under certain climatic conditions, especially where wide temperature variations prevail, it has not always proved to be a durable admixture. In addition to the highly plastic soils, which are not recommended for cement treatment, there exist certain soils of good physical characteristics which cannot be successfully stabilized with cement (2, 3, 4).

Various additives have been used in an effort to waterproof soil-cement mixtures, but none of these have been significantly effective under severe testing conditions. Resins and salts of stearic and of oleic acid produce temporary water repellency but are not effective under prolonged weathering. Some of these additives reduce the strength of the soil cement. Calcium chloride has been successfully used to improve the susceptibility of certain soils to cement treatment (3).

Even without admixtures cement has proved to be the most-generally satisfactory material for stabilizing fine-grained soils. Additional study that might extend its usefulness appears to be fully warranted.

Lime

A considerable amount of work has been conducted by numerous investigators to determine the suitability of lime as a soil-stabilizing agent. Some bonding of soil has been obtained using lime treatments of 6 percent and higher, but the samples are particularly susceptible to the disintegrating action of cycles of freezing and thawing. Lime-treated samples have failed completely during one winter of exposure (2). Numerous additives have been used in an effort to improve the soil-stabilizing characteristics of lime, but none have proved to be particularly effective.

The physical properties of plastic soils are considerably modified by the addition of small quantities of lime. The plasticity is reduced and the soils become more friable and are much easier to mix and mold to uniform density.

Favorable results have been reported in which soils were treated with a combination of lime and flyash (5), Purdue University (6) and the Texas Highway Department (7) have recently conducted valuable studies concerning the soil-modifying properties of lime.

Sodium Silicate

The use of sodium silicate is another method which has received considerable attention for solidifying soils. Good bonding of sandy soils has been attained with treatments of 6 percent or more of sodium silicate, but such treatments will not withstand attack by moisture. Sodium silicate is not effective with fine-grained soils. Earlier studies (8) indicated that the addition of sodium aluminate increased the resistance to moisture attack of samples treated with sodium silicate, but later tests using a variety of soils and more-severe exposure tests have clearly shown this combination to be unsuitable as a soil-stabilizing agent.

The most-effective form of sodium silicate found for soil-stabilization purposes is a 40-percent solution, 40 to 42 degrees Baumé, with a ratio of 1 part of Na_2O to 3.25 parts of SiO_2.

Combinations of sodium silicate with a strong inorganic salt, such as calcium chloride, have been used successfully for stabilizing medium- and fine-sand masses in deep-foundation construction (9). In this method the two chemicals, which are usually applied separately by pressure injection, react within the sand pores to form an impervious gel which, upon hardening, binds the sand particles into a solid mass. Sodium silicate alone, and in combination with other chemicals, has also been used to temporarily harden sandy soils to such an extent that excavation work could proceed without the use of shoring.

Powdered Slag

Powdered slag provides only slight bonding when mixed with soils in quantities up to 20 percent. The addition of small quantities of crushed slag considerably modifies the characteristics of the more-plastic soils. In this respect, the admixture reacts with the soil in the same way that cement or lime reacts; it reduces the plasticity and allows the soils to be more uniformly and easily mixed.

Other Additives

Inorganic soluble salts, such as sodium chloride, calcium chloride, barium chloride, copper sulfate, and barium sulfate, are ineffective for stabilizing fine-grained soils where waterproofing or bonding qualities are required. The plasticity of some soils has been reduced by the addition of calcium chloride, but the permanency of such treatment is questionable. Both calcium chloride and sodium chloride have been used successfully to enable granular roads to retain a moisture content beneficial to stability and to facilitate further densification and increased stability of the road under the action of traffic.

Oxychloride cements, zinc oxide, and plaster of paris do not set up well when mixed with soils in reasonable quantities. Natural and air-entrained cements are not as effective as portland cement.

BITUMINOUS MATERIALS

Bituminous materials have been used extensively for soil stabilization, and their properties in this respect have been thoroughly investigated in the laboratory and field by numerous agencies. A brief summary of the methods used for testing and applying these materials has been compiled by the Highway Research Board (10).

Bituminous materials have not been particularly effective when used with fine-grained soils. It is difficult to obtain a thorough mixture of the bitumens and the finer-grained soils, even though mixing methods recommended by the individual producers are followed closely. In an effort to determine the most-effective mixing procedure, bitumens have been added to prepared soil under the following conditions: (1) air-dried soil, (2) soil at optimum moisture, (3) soil above optimum moisture (near liquid limit), and (4) soil at different intermediate moisture contents. None of these variations in mixing procedure appears to improve the resulting stability of samples. More-nearly uniform dispersion is obtained when bituminous materials are added to soil at a moisture content well above optimum and mixed in slurry form. In this case, the mixture should be dried back to near optimum moisture before molding. All bitumens provide a certain degree of water-repellency to treated soil, but the resulting mixture is particularly susceptible to the destructive action of alternate freezing and thawing.

The stabilizing properties of bituminous materials, particularly MC-2, RC-2 and asphalt emulsion, have been improved by the individual addition of small percentages of lime, resorcinol-formaldehyde resins, amiline-furfural resin, and zinc stearate (2).

Bituminous materials have been and are being successfully used to stabilize and waterproof sandy soils.

RESINOUS WATERPROOFING MATERIALS

For the purpose of soil stabilization, resinous waterproofing materials are considered to be those natural or synthetic resins whose chief function is to maintain the moisture content of a soil at or below optimum moisture by preventing entry of water into the treated and compacted mixture. Little or no cementing action is obtained from these materials. Unlike bonding agents, whose effectiveness generally increases with the quantity used, waterproofing agents usually attain maximum effectiveness when applied in small quantities—2 percent or less by weight of the treated soil.

Although imparting desirable characteristics to the soil and providing considerable waterproofing effect under mild exposure, none of the water-repellent resins appears to be a suitable soil-stabilizing agent at the present time. It has been suggested that soil-bacterial activity may have a detrimental effect on the permanency of organic soil stabilizers such as bituminous and resinous materials (11).

Stabinol

Stabinol is composed of 80 percent of portland cement and 20 percent of a complex resinous compound. When used in small quantities, the cement fraction cannot impart appreciable bonding to the treated soil but acts as a carrier for the resinous material and as a possible modifier for the soil to allow more effective use of the resin. A maximum 2-percent treatment usually has been recommended for water-repellent purposes.

Initial laboratory tests of the material were quite promising (12). In further studies involving field testing, more-severe laboratory exposure, and the use of a wider range of soils, Stabinol was not effective unless applied in sufficient quantities to enable the cement fraction to become active as a bonding material. Used in larger quantities, however, Stabinol cannot be considered as strictly a waterproofing material.

Resin 321

The effectiveness of Resin 321, a partially neutralized salt of abietic acid, as a soil-stabilizing material has been previouslv reported (2, 12, 13). These reports indicate that Resin 321 is an excellent waterproofing agent for certain soils and that its water-repellent characteristics can be utilized for improving the effectiveness of other stabilizing materials. However, the beneficial qualities of Resin 321 are reduced considerably under severe exposure.

Vinsol and NVX

Vinsol and NVX (a neutralized form of Vinsol) are powdered resins which, like Stabinol and Resin 321, function to impart water-repellent characteristics when mixed with soils. Vinsol is superior to NVX but inferior to both Stabinol and Resin 321 (2, 12).

Other Resins

A considerable number of natural and partially neutralized natural resins have been investigated. These include Belro, Copal, Kauri, Batavia Damar, and Elemi resins. All of these are slightly beneficial as soil-waterproofing agents but are recommended only for temporary emergency use in areas where they are locally available.

Other resinous materials which have been tested as soil stabilizers are Soilpak (approximately seven parts lime to one of resin), Valite (a sugar resin), commercial powdered rosin, lignin sulfate, and partially neutralized tall oil. None of these are particularly effective.

RESINOUS BONDING MATERIALS

The term "resinous bonding materials," as applied to soil stabilization, refers to those natural or synthetic resinous materials which cement or bond the particles of soils with which they are mixed. Some of these resins are also good water-repellent materials. Approximately 5 percent of the resin has been arbitrarily selected as a maximum treatment in most studies, but testing of higher percentages indicate that their effectiveness increases with the quantity used. Some resinous materials, however, show considerable effectiveness when used in quantities of less than 5 percent.

Although several of the synthetic resins show considerable promise as soil-stabilizing agents, this form of treatment is still in the experimental stage. The present high cost of resins, compared with conventional methods of soil treatment, is an important factor to be considered in their possible use as soil-stabilizing agents. Initial phases of the work with synthetic resins have been reported by several agencies (2, 14, 15, 16).

Aniline-Furfural Resin

The soil-stabilizing properties of the resin formed by the interaction in the soil of two parts of aniline and one part of furfural has been thoroughly investigated and reported (2, 14, 17).

Aniline and furfural begin reaction upon contact; therefore, it is necessary to add the chemicals to the soil separately. Unless special catalysts are used, the reaction between aniline and furfural in moist soil is slow. Treated samples are easily molded within at least 8 hours after mixing. Samples which were moist cured prior to testing are inferior to those permitted to air dry or oven dry, indicating that complete resinification of aniline and furfural does not take place under moist-cure conditions.

Laboratory and field testing indicate that aniline and furfural under air-dry conditions form one of the best soil-stabilizing agents, one which is effective over a wide range of soils. In some tests aniline-furfural-treated soil samples have proved to be superior to those treated with even higher percentages of cement.

Furfural plus aniline sulfate and furfural plus aniline chloride are also effective stabilizers, but to a lesser degree than furfural plus aniline. Two parts of aniline sulfate to one part furfural was the most effective of these combinations.

Aniline-furfural mixtures are the cheapest of the synthetic bonding resins which have been considered for soil stabilization. Aniline is toxic and must be handled with considerable care.

Resorcinol-Formaldehyde Resin

Certain resorcinol-formaldehyde-resin formulations are able to harden soil under moist-cure conditions at room temperature (2). Samples treated with this material and stored in a moist cabinet become quite firm within a few hours after mixing. Air-dried samples are equally as stable as those which are moist cured.

These resins are usually furnished in liquid form and require the addition of 15 percent, by weight of the resin, of an aldehyde hardener to accelerate and complete the resinification.

At least 5 percent of this treatment is required for effective soil stabilization, but higher percentages of treatment produce higher stabilities. Although not an outstanding stabilizing agent, resorcinol-formaldehyde resins have shown considerable promise, even under severe laboratory and field exposure.

At the present time this type resin is far too costly for consideration as a primary soil-stabilizing agent. However, it appears to be suitable for improving other more economical admixtures, particularly the bituminous materials. The stabilizing properties of MC-2 and asphalt emulsion have been particularly benefited by the addition of small quantities of this material.

Phenol-Formaldehyde Resin

Several forms of phenol-formaldehyde resin have shown promise as soil stabilizers (2). The outstanding material of this group was a two-stage-type resin, which required the addition of 10 percent of hexamethylenetetramine for obtaining proper set. Five percent treatment was used for most of the tests, although higher percentages were more effective.

This type reacts with the soil in a manner similar to resorcinol-formaldehyde resin, although it is not so effective. It sets well at room temperature during moist curing or air drying. Although quite promising in laboratory tests, all samples failed during field exposure. Numerous catalysts were used in an effort to improve its effectiveness, but none were particularly beneficial. Magnesium stearate, zinc stearate, potassium permanganate, and ammonium chloride were of slight benefit with certain soils.

Urea-Formaldehyde Resins

Several forms of urea-formaldehyde resins have been tested as soil stabilizers (2). The more effective of these are produced in powder form and are precatalyzed so that no activator is required to insure complete reaction. They set up with equal effectiveness under moist- and air-cure conditions.

The addition of urea-formaldehyde resin tends to dry a moist soil and to disperse or fluff the particles. This action greatly reduces the density and increases the optimum moisture obtained by a given compactive effort as compared with untreated soil.

This form of soil treatment was not applicable over a wide range of soil types.

Furfuryl Alcohol Plus Sulfuric Acid

Some stability has been imparted to sandy soils by the resin formed from the interaction of furfuryl alcohol and sulfuric acid (2, 8). In this method of treatment the furfuryl alcohol and the required acid are added to the soil in separate portions of the mix water. Sulfuric acid has been used for most tests, although other strong inorganic acids are also effective in forming the resin. The reaction of furfuryl alcohol with strong acid solutions is very rapid, requiring considerable care in handling. As mixed in the soil, however, the reaction is not violent; and the rate of resinification can be readily controlled by varying the concentration of the acid.

Promising results were obtained by this method of treatment in preliminary work, using a sandy soil, particularly when the samples were oven dried prior to testing. However, the treatment was found to be only slightly effective with other types of soil under air-dry or moist-curing conditions. Freeze-thaw and field-weathering tests were particularly destructive to samples treated with furfuryl alcohol.

The furfuryl-alcohol treatment is much-more expensive and far-less effective than that using the aniline-furfural mixture.

Calcium Acrylate Resin

An extensive program of research and development concerning the soil-stabilizing properties of calcium acrylate is at present under way at the Massachusetts Institute of Technology for the Army Research and Development Laboratories (18).

Calcium acrylate is a water-soluble white powder which forms a hard resin when mixed with certain activators. Present recommendations for its use as a soil treatment are 10 percent calcium acrylate plus 1.36 and 1.25 percent sodium thiosulfate and ammonium persulfate, respectively. All of these quantities are based upon the dry weight of the soil. The hardening time of the resin can be controlled by varying the quantities of the activators.

In some respects, particularly from a military standpoint, the action of this material approaches the ideal requirements of a soil stabilizer. It can be added to extremely wet soils and in a matter of minutes transform the mass into a dry solid structure capable of sustaining considerable load application.

When properly mixed, calcium-acrylate-treated samples are quite resistant to laboratory-weathering tests. The ratio of the activators to the calcium acrylate is critical and should be such that the combination does not begin to set before it has been thoroughly mixed with the soil. The treatment appears to be applicable to a wide range of soil types. With certain soils its effectiveness can be improved by the individual addition of small quantities of Resin 321 and calcium stearate.

Tung Oil

The results obtained from the laboratory testing of samples treated with tung oil have been reported (8, 19). Although the data contained in these publications indicate that tung oil is a promising stabilizer with certain soil types when air-dried, its use is not economically feasible for normal construction purposes in this country. The test results show that tung-oil-treated samples were not particularly effective under severe exposure. Tung oil is superior as a soil stabilizer to all other oils studied, including linseed oil, soya oil, tall oil, and several synthetic drying oils. The addition of calcium oxide improved the effectiveness of tung oil to a small degree.

Other Resins

Several other resins have been investigated for possible use in soil stabilization, but none are particularly effective. These include melamine-formaldehyde, styrene, coumarone-indene, polyterpene, and methyl-methacrylate resins.

CONCLUSIONS

At the present time there appears to be no entirely successful method for stabilizing soils by the use of chemical admixtures. Although several materials are suitable under specific conditions or with particular soils, none have proved to be universally acceptable.

Portland cement is the most-generally effective soil-stabilizing material, but its characteristics in this respect could be improved. A suitable waterproofing additive for soil-cement mixtures has not yet been found.

The resin formed by the interaction of two parts aniline and one part furfural is a very good bonding agent for soil and the most-effective waterproofing material studied so far. Under certain conditions this treatment is effective in quantities as low as 2 percent of the dry weight of the soil.

Calcium acrylate activated by sodium thiosulfate and ammonium persulfate is highly successful in transforming wet, plastic soils into firm, durable structures. Although only in the experimental stages of development, this material appears to have good possibilities as a soil modifier and stabilizer. Like other synthetic resins, however, it is too expensive at the present time to be considered for normal construction purposes.

Other resinous materials have shown some promise as soil stabilizers, but their principal value appears to be as additives to nonconventional and cheaper paving materials. Resorcinol and phenol-formaldehyde resins have considerably improved the soil-stabilizing effectiveness of bituminous materials.

Because of the extremely variable and complex chemical characteristics of different soils, it is probable that no universally suitable soil-stabilizing agent will be found. It may be that the most we can expect to develop is a chemical which, although not specifically furnishing bonding or waterproofing characteristics, will otherwise modify natural soil to such an extent that it will be more useful for structural purposes. Development of an economical and durable material which would reduce the plasticity and improve the shrinkage and swelling characteristics of soil would be a valuable contribution to soil science. Such materials as cement, lime, sodium silicate, aniline-furfural, and calcium acrylate have shown considerable promise in this respect.

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Effect of Calcium on the Continuity of Electroösmotic Flow Rate

RALPH L. ROLLINS, Engineering Experiment Station,
Iowa State College, Ames

Introductory Remarks by the Chairman: Large-scale use of the electroösmotic phenomenon appears to have been mainly in the drainage of fine-grained natural soils. For this, relatively low electric potentials are employed, and interference by chemical and polarization phenomena has been reported to be practically negligible. Chemical phenomena, however, have been utilized in electroösmotic stabilization of friction pile systems and have been observed in laboratory studies on soils in which the exchange ions were varied.

Much experimental work with chemically well-defined soils and employing a wide range of electric potentials remains to be done before electroösmosis can become a dependable tool in soil stabilization. It is gratifying, therefore, that the Engineering Experiment Station of Iowa State College has attacked this problem on a wide front and that Rollins is presenting us with the first results of this attack on the effect of calcium on the continuity of the electroösmotic flow rate.

● THE bearing capacity of a subgrade is a function of moisture content. A large number of pavement failures can be attributed to the loss in subgrade support which takes place when excess moisture accumulates. It follows that any method of controlling or reducing this accumulated moisture falls into the category of soil stabilization.

A number of investigators have shown that moisture content beneath pavements fluctuates throughout the year, increasing during the cold months and decreasing during the warm periods.

The most-critical time, so far as subgrade bearing support is concerned, is during the thawing period in the early spring. If some means were available for removing the excess moisture at that time, it seems reasonable to expect that the performance of the pavement would be considerably improved.

One possible means of removing this excess moisture which appears to have some possibilities is electroösmosis, or the movement of moisture by an electrical-potential gradient.

PREVIOUS WORK IN ELECTROÖSMOSIS IN SOILS

Space does not permit the inclusion of a complete review of all previous investigations in electroösmosis of soils; however Casagrande (2) has reviewed the subject in a recent publication. Unfortunately, in many cases too little attention has been paid to the physicochemical characteristics of the soil, and that makes it impossible to project the results to other areas.

Winterkorn (6) has presented data showing the relative electroösmotic permeabilities for homoionic modifications of a soil involving the usual cations likely to be associated with the electroösmosis of soils. The flow of water for constant amperage was very slow for Ca, Na, and K and contained a high percentage of suspended colloids. Mg and Fe soils gave a regular and clear flow up to approximately one hour, while the current density had to be reduced for Al and H systems early in the experiment when the resistance of the sample became so large that the power sup-

TABLE 1

PHYSICOCHEMICAL PROPERTIES OF THE SOILS

Sample No.	Cat. Exch. Cap. M. E./100g	pH	Ca. and Mg. Expressed as $CaCO_3$%	Conductivity of Sat. Ext. Millimhos/cm	Organic Matter
1	40.2	7.95	3.0	0.386	
2	24.4	6.7	1.6	0.277	0.37
3	18.2	7.0	1.9	0.342	0.18
4	30.4	6.25	1.6	0.145	
5	11.2	8.4	10.3	1.21	0.24
7	39.6	7.0	3.0	0.347	
8	23.0	7.75	9.8	1.09	

Sample No.	Atterberg Limits			Mechanical Analysis		
	L. L.	P. L.	P. I.	% Sand	% Silt	Clay%[a]
1	71.5	24.5	47	2.11	27.89	70
2	51.9	18.5	33.4	0.7	60.2	39.4
3	39.4	26.9	12.5	2.0	70.6	27.4
4	70.6	21.9	48.7	8.2	31.0	60.8
5	29.6	27.3	2.3	4	82.6	13.4
7	70	36	34	19	26	55
8	55	23	32	24	30	45

[a] Less than 5 microns.

ply was insufficient to maintain it. Economic yields were lowest for Ca, K, and Na modifications, medium for Mg systems, and greatest for H and Al soils.

Casagrande (1), who has been a prolific writer with respect to electroösmosis and electrochemical hardening in soils, summarizes his experience as to the continuity of flow in systems in which the electrodes are in contact with the soil as follows: "The rate of discharge of water at the cathode, continuous for many weeks at an almost constant rate, finally drops off gradually with a corresponding decrease in the electric current."

Winterkorn (7), using the Schmidt concept for micro systems, has developed equations relating electroösmotic permeability to porosity. He presents data to show that in general the experimental curves are quite similar to the theoretical curves; however in the expanding lattice type clays, such as bentonite, the electroösmotic permeability porosity relationship is affected by hydration of the clay micelles.

This paper represents the initial phase of a broad program of research having to do with electroösmosis in soils. Specifically this paper shows the effect of calcium and magnesium on the continuity of the electroösmotic-flow rate. An attempt is made to show the reason for this effect as well as to suggest a possible use of natural calcium and magnesium in soils as stabilizing agents.

TECHNICAL REQUIREMENTS FOR ELECTROÖSMOTIC DRAINAGE

The following three general questions associated with electroösmotic drainage must be answered before the technical feasibility of such an operation can be clearly defined, but it is not believed that they have been fully answered up to the present:

1. Under what conditions is it possible to remove moisture from a saturated soil by electroösmosis, and what pre-testing procedures need be adopted? Involved in this question are such things as: (1) What is the relative ability of the common soil cations themselves to transmit water in electroösmosis as differentiated from the

effect they have on hydration and swelling? (2) What is the effect of hydration and swelling on electroösmotic permeability? (3) What is the influence of the type of clay mineral on electroösmotic flow? (4) How do various types and quantities of soluble salts affect the movement of moisture in electroösmosis? (5) How does the electroösmotic-flow rate vary with the porosity?

2. Assuming that water can initially be removed from soil, is it possible to continue this removal over an extended period of time? This question is not completely independent of the first one, for it involves in addition a consideration of electrode reactions and over-potentials, type and kind of electrodes and the positioning of the electrodes. Solubilities of the various compounds of the normal exchangeable cations at various pH must be considered; since any electroösmotic operation on soils tends to replace the base ions with the hydrogen ions.

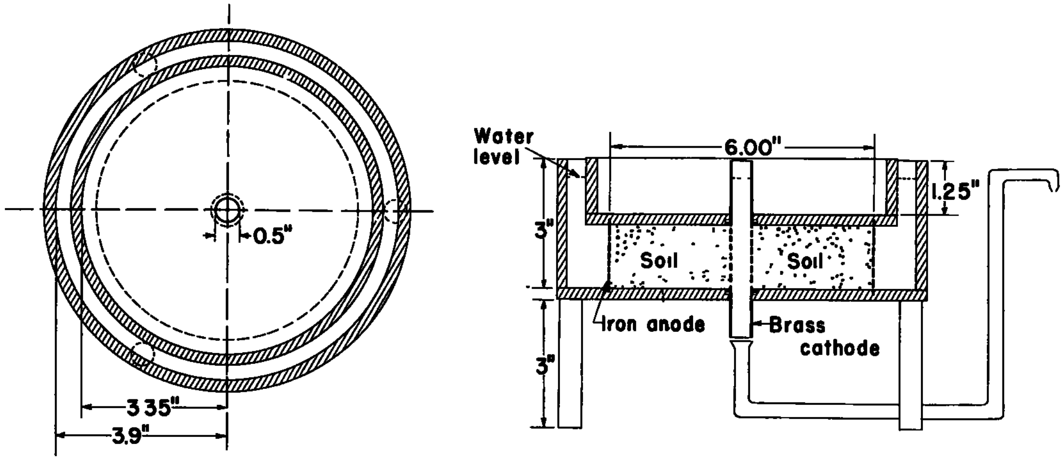

Figure 1. Electroösmotic-flow apparatus.

3. How much can the moisture content be reduced below the saturation point, and how is it affected by current density, porosity, clay mineral type, and adsorbed cation? The importance of this question is obvious. For a given dry density the supporting power of a subgrade is a function of the moisture content, and the reduction in moisture content below saturation is a measure of the improvement in the subgrade bearing capacity.

METHODS AND PROCEDURE

All of the soils used in these experiments were clays with the exception of two, which were silts. Physico-chemical data for these soils are presented in Table 1. The combined calcium and magnesium was determined by leaching the samples with 1.5N HCl and titrating with versinate. The results are expressed in equivalent calcium carbonate. Based on actual determination of exchangeable cations for some of the soils and a comparison of the calcium versus magnesium content previously determined by other investigators for soils found in similar areas, it is believed that the calcium content was considerably greater than the magnesium content for these soils.

It is observed that all soils have considerably more than enough calcium and

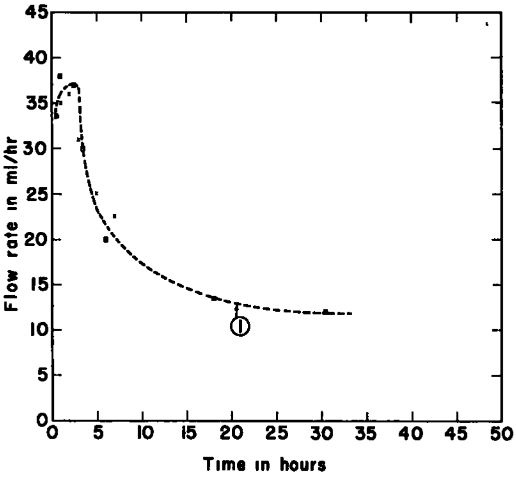

Figure 2.

magnesium to satisfy the exchange complex of the soil. This condition is found in many of the Iowa soils.

The radial flow apparatus used in these experiments is shown in Figure 1. Direct current was made available by using a transformer and rectifier. Holes were drilled in the top plate of the apparatus for observing changes in the electrical potential gradient. The soils were mixed thoroughly with water at approximately the liquid limit before being placed in the apparatus. The water level at the anode and cathode were maintained at the same elevation so there was no pressure flow. An applied voltage of 30 volts was used in all experiments, and periodic observations were made of the current and flow rate.

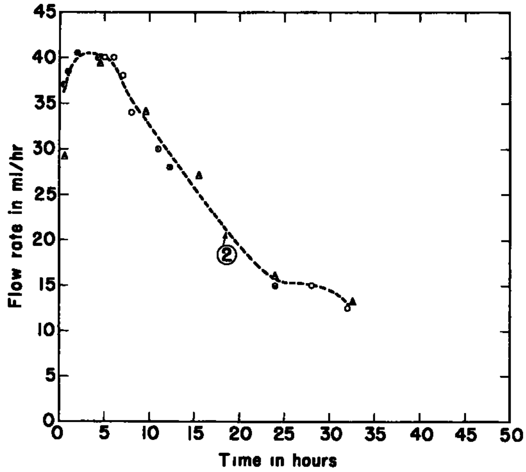

Figure 3.

SOME EXPERIMENTAL RESULTS

Relationship between electroösmotic flow rate, electrical current, electrical potential gradient and time.

Figures 2 through 8 show the flow rate in milliliters per hour as a function of time. In all cases there is a considerable reduction in the flow rate with time. Soils 1, 2, 3, 4, and 7 have equivalent calcium carbonate percentages ranging from 1.6 to 3 percent.

The reduction in flow rate is much less for the silty soils than for the clays. A comparison of Soil 4 with Soil 3 (a clay and silt, respectively, having approximately equal equivalent calcium carbonate contents) shows very clearly this effect. Soil 5 is a silt, and Soil 8 is a clay; and both have equivalent calcium carbonate percentages of about 10. In this example the reduction in flow rate also is greatest for the clay, although the reduction is lower for both than in soils with less initial calcium.

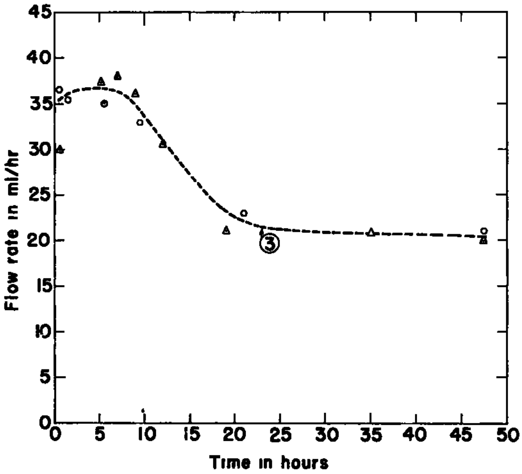

Figure 4.

The reduction in electrical current closely paralleled the reduction in the flow of water; the current typically rose during the first hour or so and dropped off thereafter. Figure 9 is a plot of accumulative flow in milliliters per hour as a function of the accumulative charge in coulombs. The linear relationship between flow and charge on this plot is characteristic for all soils, which indicates that the reduction in flow is primarily caused by the reduction in current, since electroösmotic flow is known to be directly proportional to the current.

Electrical potential measurements were made periodically throughout an experi-

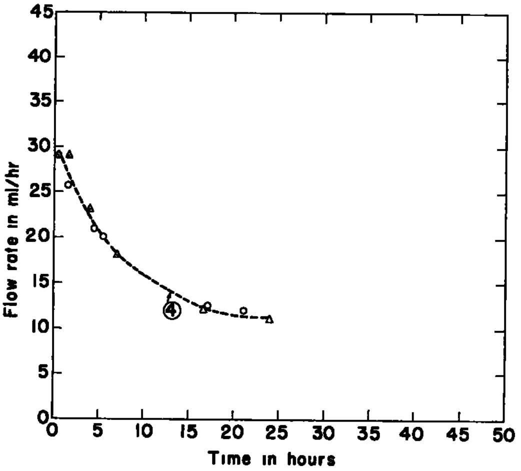

Figure 5.

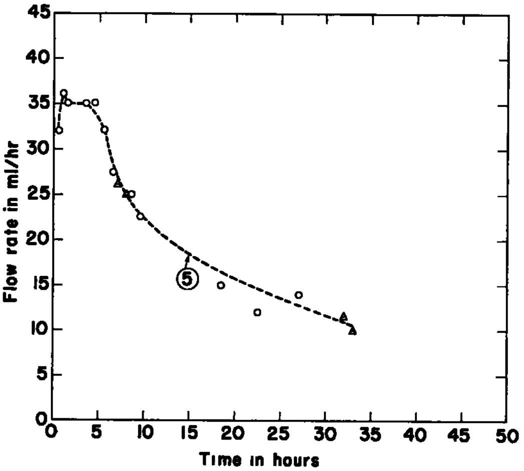

Figure 6.

ment. Figure 10 shows a typical plot of electrical potential versus the Log r/r_C at the beginning and end of an experiment, where r equals the radius from the center of the cathode to a point in consideration and r_C is the radius of the cathode. As might be expected such a plot is linear at the beginning of an experiment. As time elapses there is a continuous increase in the resistance toward the cathode in comparison with that toward the anode. This process continues until the majority of the resistance is concentrated in the vicinity of the cathode.

CHARACTERISTICS OF SOIL BETWEEN ANODE AND CATHODE AFTER ELECTROÖSMOSIS

Visual inspection of the soil in the electroösmotic apparatus at the completion of a run showed that a dense core developed around the cathode.

A sharp line of demarcation was evident between the remainder of the soil and the core. The moisture content within the core was considerably less than in the surrounding soil and the cores for the clays appeared to be more compact than those for the silts. Table 3 shows the equivalent calcium carbonate and the pH for the raw soil, the soil at the anode, and the soil at the cathode at the completion of the experiment. As might be expected, the pH of the soil at the anode decreases and that of the soil at the cathode increases with the pH of the raw soil. The table also shows that considerable amounts of equivalent calcium carbonate had collected at the cathode. Table 2 shows a comparison of the particle size distribution for the raw soil and for soil taken from around the cathode for Soil 1. It should be noted that the sample for this analysis had been allowed to air dry for several days. Comparative Atterberg limits are also listed in this figure.

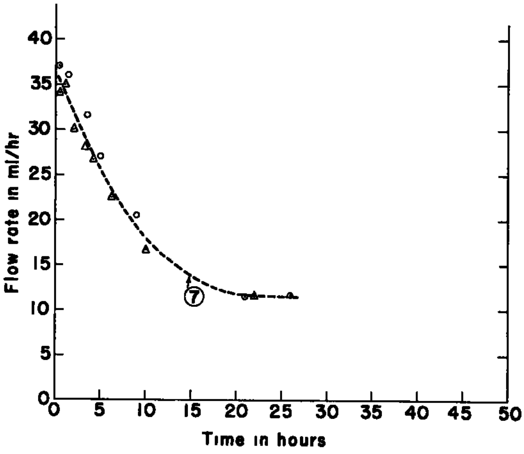

Figure 7.

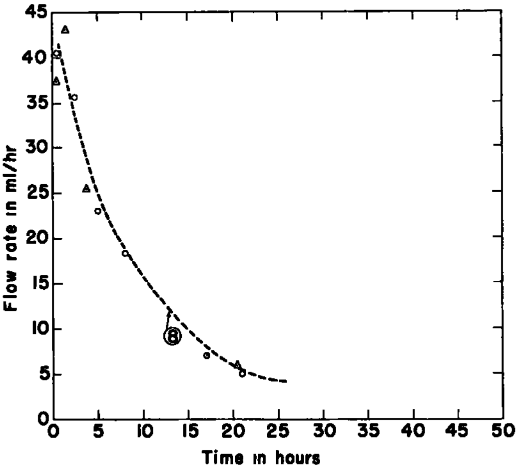

Figure 8.

DEMONSTRATION OF OHMIC OVERPOTENTIAL AT THE CATHODE

During normal experiments the cathode used was a ½-inch perforated brass tube directly in contact with the soil. This type of cathode was modified by inserting a ⅛-inch brass rod into the center of the old cathode. This rod when insulated from the half-inch tube then served as the cathode.

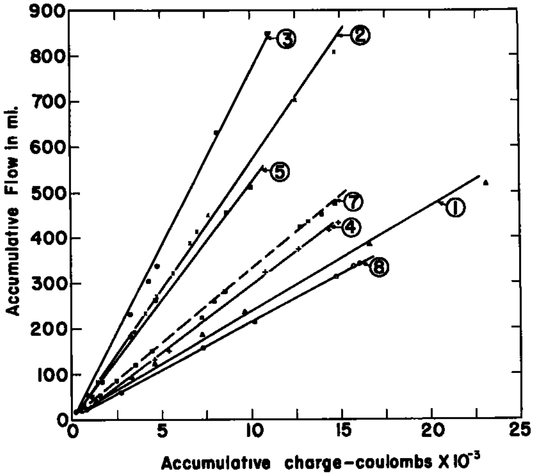

Figure 9. Relation between the accumulative electroösmotic-flow rate and accumulative charge.

The annular space between the rod and tube was filled with distilled water, and the same water levels as before were maintained. A series of experiments were carried out using this type of system. A typical plot of the results for Soil 1 is shown in Figure 11.

The current and the flow rate are plotted as ordinates and the time as the abscissa. An experiment was started with distilled water around the cathode. Initially the current rose as salts moved into solution around the cathode. Then calcium hydroxide was deposited on the ⅞-inch-diameter electrode with a consequent severe reduction in current and a simultaneous reduction in flow rate. At periodic intervals the cathode was removed and cleaned, after which the current and flow rate both increased and then decreased as the calcium hydroxide deposited. Such experiments were carried on for approximately 12 hours, and it was noted that there was a gradual reduction in flow as well as in current. Examination of the soil around the cathode showed that calcium had been deposited and that some cementation of the soil particles had taken place, which possibly accounts for the overall reduction in flow rate.

Figure 12 shows the relation between specific conductivity and time for Soils 1, 3, 4, and 5, which had been mixed at approximately the liquid limit with 4 percent of calcium hydroxide. Note that for all soils there was a decrease in conductivity with time which was greatest for the clays.

TABLE 2

A COMPARISON OF THE PHYSICAL PROPERTIES OF THE NATURAL SOIL AND THE CATHODE SOIL

Soil 1	Atterberg Limits			Mechanical Analysis		
				% Sand	% Silt	% Clay
Natural Soil	71.5	24 5	47	2.11	27 89	70 00
Cathode Soil	53 3	39.5	13 8	37.60	27 65	34 75

TABLE 3

A COMPARISON OF EQUIVALENT CALCIUM CARBONATE, AND pH FOR THE RAW SOIL, SOIL AT THE ANODE AND SOIL AT THE CATHODE

Soil	Equivalent Calcium Carbonate %			pH		
	Raw Soil	Anode	Cathode	Raw Soil	Anode	Cathode
1	3 0	2 0	10 7	7.95	3 93	10.10
2	1 6	1 5	8 0	6.70	4.10	10.20
3	1 9	1 3	5.8	7 00	4 50	8 43
4	1 6	1 1	7.3	6 25	3.90	9.10
5	10 3	8 0	13 4	8.40	7.60	9.60
7	3 0	1 4	6.5	7.00	4 20	9 20
8	9 8	7 8	10 2	7.75	7 40	9.50

DISCUSSION OF RESULTS

Many of the results given above can be explained by elementary electrochemistry. In electrochemistry the standard electrode potential of an element is a measure of the tendency of that element to form positive ions in solution. The more positive (American convention) an electrode potential is, the greater is its tendency to form positive ions; and the more negative the standard potential, the greater is the tendency to gain electrons to form negative ions. Standard electrode potentials hold only for unit activity; however the electrode potentials may be computed at other concen-

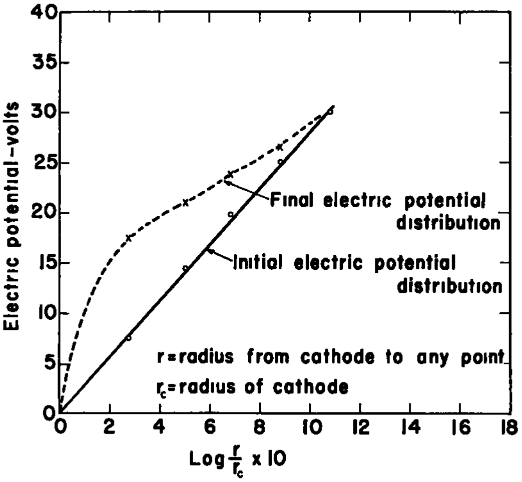

Figure 10. Relation between the electrical potential and the log r/r_c.

trations if the activity is known. If two cations compete at an electrode for electrons, the one with the more negative electrode potential tends to be discharged first.

Calcium has one of the highest standard electrode potentials of any of the elements, and it does not form metallic calcium at ordinary temperatures in aqueous solutions. The other most common cation likely to be available to accept electrons in the system described above is hydrogen. Hydrogen gas then is liberated at the cathode, and hydroxl ions tend to accumulate. Table 4 shows the pH at which the hydroxides and basic salts of various cations precipitate. Thus all calcium ions accumulating near the cathode will usually precipitate as calcium hydroxide. Other cations, such as the ferrous ion, will precipitate as hydroxides when they reach the appropriate pH value in the electro-osmotic system.

Figure 9 shows in addition that the reduction in flow is directly proportional to the drop in current. Figure 11 shows that the reduction in current is associated with the deposition of calcium hydroxide on the metal surface of the cathode, and Figure 12 shows that the current will also be reduced when calcium hydroxide is deposited in the soil. The cementing affect, indicated by the changes in particle size distribution and consistency limits, undoubtedly blocks some of the exchange positions and restricts the movement of ions in these areas.

TABLE 4

THE pH AT WHICH SOME METAL HYDROXIDES OR BASIC SALTS ARE PRECIPITATED

Metal	pH	Metal	pH
Ferric	2.3	Nickel	6.7
Aluminum	4.1	Magnesium	10 5
Zinc	5.2	Calcium	Sparingly Soluble
Cupric	5.4	Potassium	Soluble
Ferrous	5.5	Sodium	Soluble

Considerable densification developed around the cathode, and some of the re-

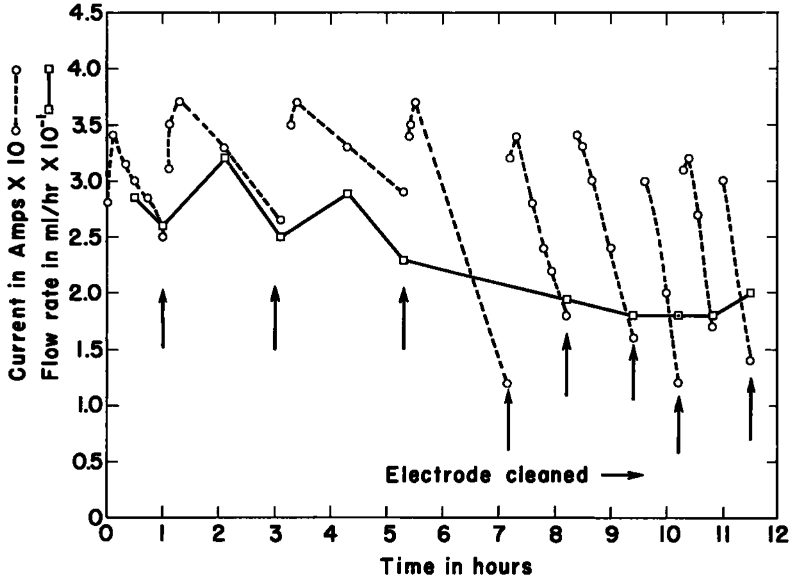

Figure 11. Relation between current, flow rate, and time for special cathode.

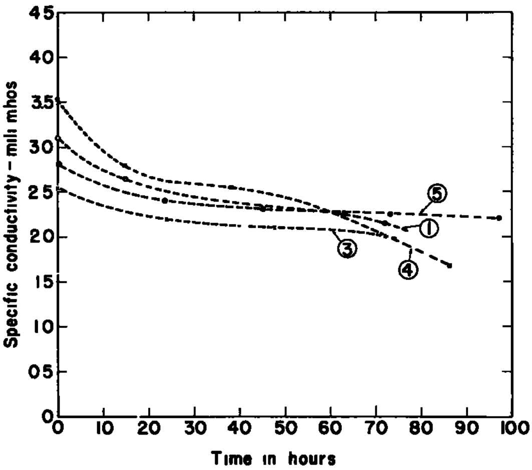

Figure 12. Relation between specific conductivity and time for soils treated with 4 percent of $CaOH_2$.

duction in flow might have been due to the reduction in the porosity at this position. Soils taken from around the cathode developed a fair amount of strength, even though the porosity at the liquid limit was quite high. The soils were also considerably waterproofed, as was evidenced by the fact that no slaking took place after extended periods of soaking. This stabilizing effect in soils high in calcium probably should be investigated further.

Note that the data so far presented pertain to radial flow. Experiments have been conducted with rectilinear flow in an apparatus approximately 6 inches square and 1½ inches deep. These experiments were carried on for approximately 30 hours for Soil 1. The densely calcified section around the cathode was approximately ¾ inch thick, and the flow rate and current dropped off to a considerable extent although not quite as rapidly as in radial flow. Iron electrodes were used, and the formation of a band of ferrous hydroxide was observed immediately behind the calcium hydroxide after a period of about 48 hours. This formation further reduced both the current and the flow.

These results indicate that before electroösmosis can be used to dewater soils of this type some method must be developed to prevent deposition of the calcium, or provisions must be made for cleaning the cathode. An alternative to this system might be to deposit the calcium in the surface of the subgrade followed by a relocation of the electrodes for dewatering.

From the above discussion, it is also evident that anodes such as iron should not be used for electroösmosis in soils similar to those used in these investigations. Experiments are now under way to determine the possibility of using as anodes plastic materials coated with various colloidal graphite material. Research directed toward the development of practical means of depolarizing the cathode is also continuing.

SUMMARY

The results of this investigation show that the electroösmotic flow rate for soils relatively high in calcium is characterized by a relatively high initial flow rate which drops off rapidly with time. The reduction in flow is greater for clays than for sand. Analysis of the soil between the anode and cathode at the completion of a run shows that the pH has decreased at the anode and increased at the cathode and that considerable amounts of calcium hydroxide has deposited at the cathode. This calcium hydroxide coats the metallic cathode, cements the soil particles, and densifies the soil adjacent to the cathode. It is shown that the first two effects considerably increase the electrical resistance and decrease the current flowing in the system. It is further shown that the reduction in flow of water is directly proportional to the reduction in electrical current.

ACKNOWLEDGMENT

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Elastic Behavior of Soil-Cement Mixtures

FRIEDRICH REINHOLD, Professor of Highway and Municipal Engineering,
Technical University, Darmstadt, Germany

Introductory Remarks by the Chairman: Modern soil-cement is a remarkable construction material. Aside from its excellent structural properties, it is noteworthy that from its early infancy in South Carolina to its present worldwide use, it was guided and supervised scientifically and that more precise records have been kept on its development than on any other large-scale construction material.

But do we know as much as we should even with respect to the mechanical properties of this material? Have we been completely fair to it and studied and developed all its potentialities? Or have we treated it as a dependable slave that can carry the loads we place on it without bothering how it does it?

Reinhold looks at soil-cement as a unique construction material that is not just a poor concrete or a stabilized soil but a material that may prove to be superior in certain highway and other applications than traditional materials presently employed for them. From this point of view, Reinhold has studied the elastic behavior of soil-cement as a function of cement and clay content and has given in his paper a precise account of the results so far obtained in his important work.

● THE modulus of elasticity and Poisson's number play an important role in the dimensional design of homogeneous pavements. Questions regarding the strength of a pavement, its reaction to temperature and moisture changes, and the distance and type of joints to be used are directly related to the elasticity constants. These constants must be determined for soil-cement if we want to understand its actual behavior under stresses caused by externally applied forces. From a knowledge of their magnitudes, a better utilization of the properties of this new construction material may result as well as an expansion of its field of application.

GENERAL REMARKS CONCERNING ELASTIC BEHAVIOR

For the purpose of defining fundamental concepts and of determining the type of experiments necessary to measure the elastic behavior of soil-cement mixtures, the following general considerations are presented.

The elastic behavior of homogeneous isotropic materials is essentially characterized by two coefficients, the modulus of elasticity E (or its inverse a) and the Poisson number m (or its inverse μ = Poisson's Ratio). In the general case these coefficients are not constants. They are determined by measurements of changes in longitudinal and lateral dimensions in uniaxial tension or compression, which are evaluated by means of the stress-strain diagram.

Stresses $s = \frac{P}{A}$ are forces per unit area that are caused by externally applied loads. Strains are defined by $\epsilon = \frac{\Delta l}{l}$ i. e., by the ratio of change in a dimension to the original dimension and, hence are dimensionless. Strains may be caused by compression, tension or shear forces.

Fundamentally, there exists no proportionality between stress and strain. The law established by Hooke in 1678 "Ut tensio, sic vis":

$$\epsilon = \frac{s}{E} = \alpha \times s$$

which for a long time had been considered as a natural law covering the behavior of all materials, is strictly true only for the lower stress range of steel and for a minority of substances, within certain limits. All other materials show larger or smaller deviations. Schülle, therefore, proposed the expression:

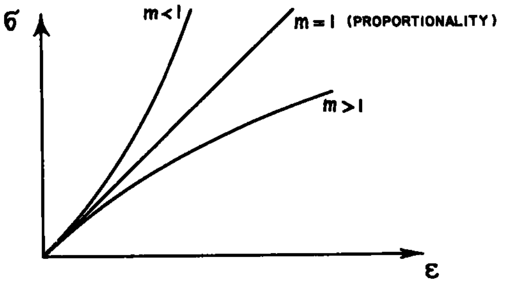

Figure 1. Graphical presentation of the equation $\epsilon = a\, s^{m}$.

$$\epsilon = a\, s^{m}$$

This equation, which also represents only an approximation of the actual behavior of materials, already causes such mathematical difficulties in the practical use of the theory of elasticity that one is forced to linearize in accordance with Hook's law in order to obtain mathematical solutions of actual problems.

For the definition and calculation of the modulus of elasticity E from measured data, there exist two possibilities: (1) the elasticity modulus is represented by the tangent at a chosen point of the stress strain curve

$$E_1 = \frac{d\, s}{d\, \epsilon}$$

(2) the modulus of elasticity is assumed to be constant for a certain stress range

$$E_2 = \frac{s}{\epsilon}$$

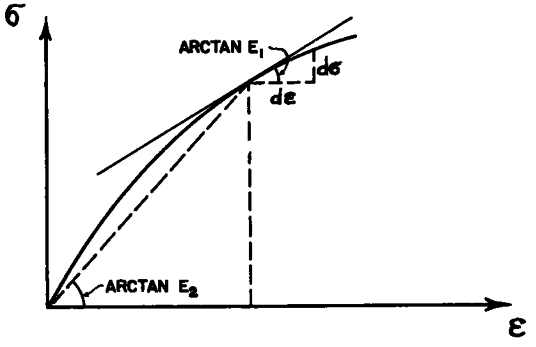

Figure 2. Graphical presentation of the modulus of elasticity.

Since calculation methods normally employed in civil engineering assume proportionality between stress and strain, the modulus of elasticity is usually determined according to the second method.

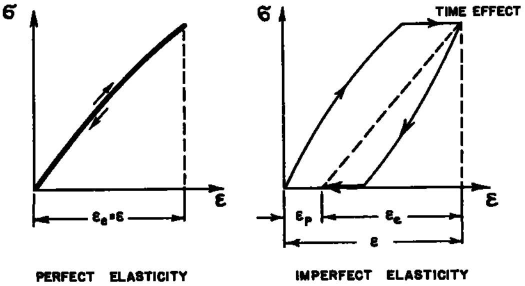

Figure 3. Perfectly and imperfectly elastic material.

Further differentiation is now made between perfect and imperfect elastic behavior. This renders the definition of the modulus of elasticity E considerably more complicated. In the case of perfect elasticity there exists a definite reversible relationship between stress and strain. More generally observed in imperfect elasticity in which the strains are a function not only of the stress but also of the type of loading, of the time, and of the repetition of load application. Thus, a difference exists between the corresponding strains and stresses in the case of loading and unloading, a phenomenon which is known as hysteresis.

The deformations ϵ_p remaining after unloading are called plastic, while the reversible deformations are called elastic ϵ_e. That portion of the deformation which is a function of time is called after effect. A measure for the degree of perfection of elasticity is:

$$\text{degree of elasticity } e = \frac{\text{elastic strain } \epsilon_e}{\text{total strain } \epsilon}$$

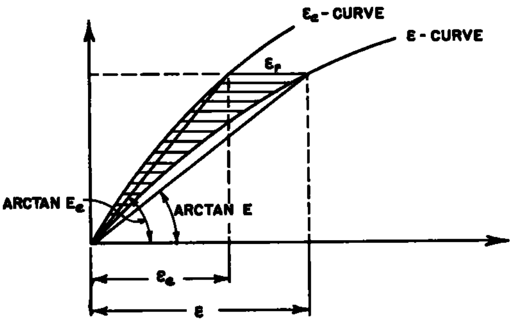

Figure 4. Graphical presentation of ϵ_e, ϵ, E_e, and E.

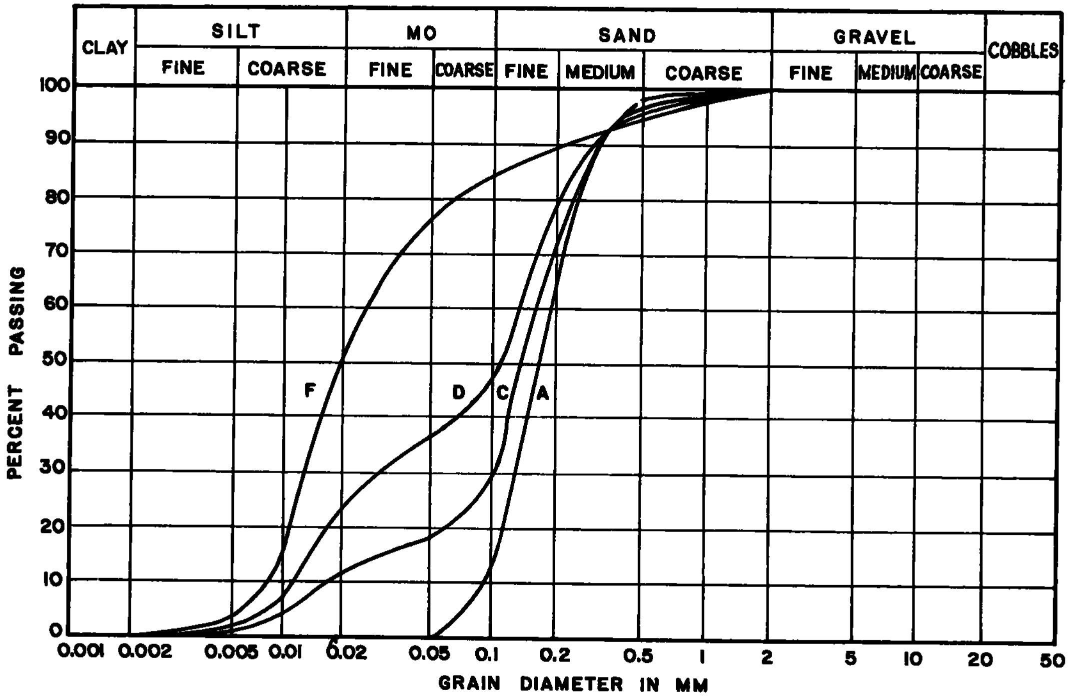

Figure 5. Granulometric distribution.

If e = 1, perfect elastic behavior exists; accordingly, this equation defines perfect elasticity.

Only the elastic strains are expressive of the elastic behavior of a material. Accordingly, the real modulus of elasticity E_e takes into account only the elastic strain while E is to be considered as a modulus of the total strain.

If ϵ_e and ϵ are plotted for equal loading stages in one stress strain diagram, then two curves are obtained (Figure 4). The cross-hatched area in Figure 4 represents the magnitude of the residual strain ϵ_p.

Strains obtained during the first application of loads of increasing magnitude are called virginal strains ϵ^0. The stress strain curve thus obtained represents extreme conditions and envelopes the $s - \epsilon$ branches that are obtained in repeated loading and unloading.

Under the influence of load repetitions, the strain lines shift in the ϵ direction. However, in most cases (for small stress increments earlier, and for large increments later) a stable state is usually reached in which the strains tend to approach constant ϵ values..

These phenomena have been thoroughly investigated in the case of portland-cement concrete. As will be shown later, they have also been observed in the case of soil-cement mixtures. Accordingly, the same methods of investigation and presentation of data can be employed for both materials.

The previous statements hold true, as a matter of principle, for both tension and compression, for longitudinal as well as lateral strains (ϵ_q). One must differentiate, therefore, between ϵq and ϵqe, ϵq and ϵq^0, and also between $m = \frac{\epsilon}{\epsilon q}$ and $m_e = \frac{e}{q_e}$. With respect to the exact determination of the stress actually present in the specimen, the tension experiment is, as a matter of principle, equivalent to the compression experiment even though the course of the stress-strain curve may be different.

In the case of brittle materials, i.e., also in the case of soil-cement, it is customary to investigate the elastic behavior under compressive stress, since by this means more-uniform uniaxial stress conditions may be realized, and also because a greater stress range can be investigated because of the higher compressive strength.

With respect to its structure, soil-cement may be considered as quasi-isotropic and homogeneous.

PREPARATION OF TEST SPECIMENS

Synthetic soils were employed for the manufacturing of the test specimens. The soils were made by mixing Griesheim sand, a fine noncohesive material of yellow color with Heppenheim clay. This clay is employed for the manufacture of roofing tile, and was available in finely ground condition.

Four different soil types were prepared which were mixed, at their optimum moisture contents, with three different proportions of portland cement. The ratio of cement to soil was taken as 1 to 6 for an economically justified upper limit, as 1 to 10 for a lower limit, and as 1 to 8 for an in-between amount.

The different soils are designated as follows:

Designation	Composition	
	Sand %	Clay %
A	100	0
C	75	25
D	50	50
F	0	100

The cement proportions are indicated as follows:

Designation	Mixing Ratio Soil/Cement
I	1:6
II	1:8
III	1:10

The granulometric composition of the four different soils is given in Figure 5. The liquid limits, plastic limits, and plasticity indexes are presented in Table 1.

Chemical investigation of the component soils A and F showed the absence of substances that could have a detrimental influence on the soil-cement. The cement employed (PZ325) fulfilled the German Standard Specifications, DIN 1164.

TABLE 1

CONSISTENCY CHARACTERISTICS OF THE SOILS

Designation	Liquid Limit	Plastic Limit	Plasticity Index
A	-	-	-[a]
C	16.80	-	-[a]
D	25.00	16.07	8.93
F	38.50	20.98	17.52

[a] Because of the granular nature of the soil, the plasticity index could not be determined.

The test specimens were made in accordance with the "Tentative Specifications for the construction of soil-cement roads."

Since, in accordance with Siebel (1), the ratio of height to width of test specimens used in strain measurements, should be about 3 to 1, specimens with the dimensions 7.07 by 7.07 by 23.21 cm. were manufactured.

In order to have a uniform basis of comparison, all specimens were cured in the same manner and for the same period of time before they were tested. The 28-day curing period was composed as follows: 2 days moist air, 12 days water immersion, 7 days dry air storage, and 7 days water immersion.

EXPERIMENTS AND TEST RESULTS

The following equipment was used for the stress-strain measurements: A 500-ton hydraulic testing machine provided with a 5-ton pressure stirrup for exact loading; a Martens mirror apparatus for the measurement of the longitudinal strains; and a tensometer for the measurement of the lateral strains.

For all longitudinal and lateral measurements the same arrangement was employed. This arrangement is briefly described in the following:

1. Mirror apparatus for measurement of longitudinal strains. The two mirror devices were fastened on two opposite sides of the prisms (normal to the compaction joints) in order to obtain dependable mean values. The gage length was 10 centimeters in the center portion of the test specimens in order to obtain exclusively the mono-axial stress condition, and to remain outside of the friction influence of the shear cone.

2. Tensometer for the measurement of lateral strains. Also arranged on the two opposite sides of the prism (parallel to the compaction joints) on the two free sides of the prism. The gauge length was 5 cm.; the gages were fastened at the elevation of the center of the gages measuring the longitudinal strains, and at a right angle to them.

3. Measuring stirrup for a range of 5 tons. Placed above the test specimen between a plane-parallel steel plate and the ball-jointed pressure head of the testing machine. The entire experimental arrangement of the test specimens, measuring appliances, and reading scales is given in Figure 6.

Details concerning the fastening of the instruments on the test specimens are shown in Figure 7. The strain determinations were based on the ϵ curve, with one repetition of loading for measuring the time effect. Previously conducted experiments had shown that this was sufficient for the determination of the elastic portion of the strain.

In the lower stress range, in which no influence of the time effect can be noted, the curve is identical with the virginal curve envelope ϵ^0, the upper portion of which could also be plotted if one would record the quick-reading values. The unloading usually was done to a stress of 5 kg. per sq. cm. in order to avoid slipping of the instruments.

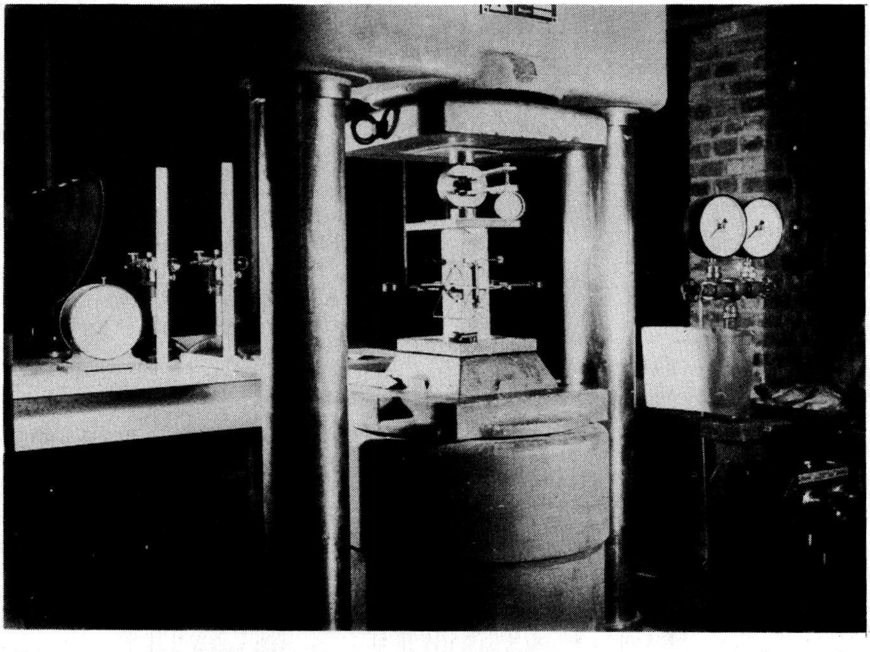

Figure 6. Arrangement for stress-strain measurements.

As examples of the graphic presentation of the experimental results, Figure 9 shows the ϵ and ϵ_q curves for longitudinal and lateral strains, as well as the moduli of elasticity, calculated therefrom, and the Poisson numbers for four test specimens made of the four soils types with a cement-to-soil ratio of 1 to 8. All measured values are given with reference to the condition of the test specimen before the first loading.

EVALUATION OF EXPERIMENTAL RESULTS

The stress-strain diagrams of all the different soil-cement compositions tested in compression, show the following basic characteristics: (1) The curve is always more or less concave with respect to the ϵ axis. (2) In a range up to about a third of the failure stress ($0.33 s_D$) the curve is almost a straight line; thus, for all practical purposes one can assume linearity in this range. The plastic deformations in this range are so unimportant that one can consider the material as possessing almost perfect elasticity; $e = 1$. (3) At a stress of $0.6 s_D$, the curvature increases rapidly. The curve becomes flatter and its tangent is often horizontal at the failure point.

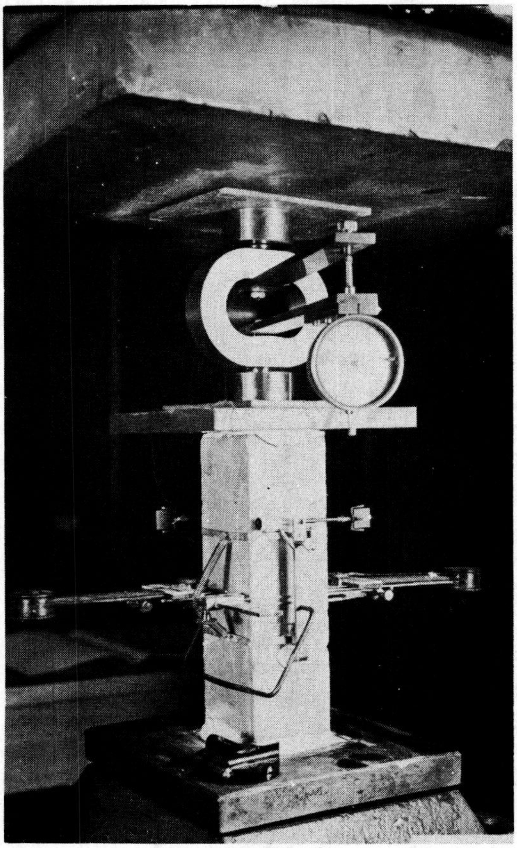

Figure 7. Details of experimental set-up.

These characteristics agree in principle with those of portland-cement concrete. Although soil-cement depending on its composition, possesses characteristics that are specifically its own, these characteristics are not alien to those of portland-cement concrete. This, of course, is not unexpected.

The stress-strain diagram for the lateral strains possesses characteristics similar to that for the longitudinal strains, with the only difference that the linearity is less developed but extends to about $0.6\ s_D$. From this point on, the lateral strains increase much more rapidly than the longitudinal strains. This is reflected in a decrease of the Poisson numbers. The reason for this behavior is according to Roŝ (2) a certain loosening of the internal structure (an indication of the beginning of failure as a result of fatigue).

Very generally, it can be said that the elastic behavior of soil-cement is a function of its strength. All factors that influence the strength properties also determine the elastic behavior of soil-cement. Fundamentally the following holds true: (1) the higher the failure stress the less curved is the stress strain diagram; (2) the higher is the range of linearity; and (3) the smaller is the proportion of plastic deformation and, therefore, the more perfect is the elasticity.

The determinant influence of the higher strength begins already with the work of compaction, as can always be seen from the density of the test specimens. Accordingly, of two specimens of the same composition, the one with the greater density will show smaller strains with the same loading increments.

In the following, the influence of the two components, cement and clay, shall be considered in greater detail. For their analysis, the total strains and the moduli of elasticity E_e shall be used. The ϵ curves are more suitable for comparison than the ϵ_e curves, since they cover a larger range (usually up to failure) and they are more differentiated.

Influence of Cement Content

Figure 11 shows the great influence of the cement content. As the cement content decidedly influences the strength of soil-cement compositions, the same way it influences the relationship between stress and strain. The point x which is plotted in the graphs gives the values for the stress of $0.33\ s_D$.

Influence of Clay Content

While in the case of cement it was the maximum content which gave the greatest strength and the related elastic properties, here it is the optimum clay content of the soil (about 25 percent). The characteristics are the same for all cement ratios. The different soils show different

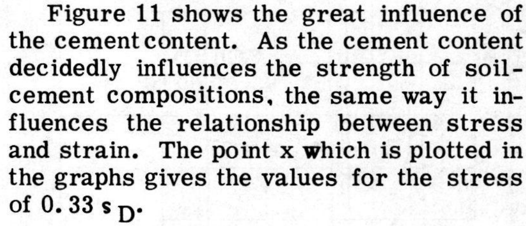

Figure 8. Presentation of ϵ^o and ϵ curves.

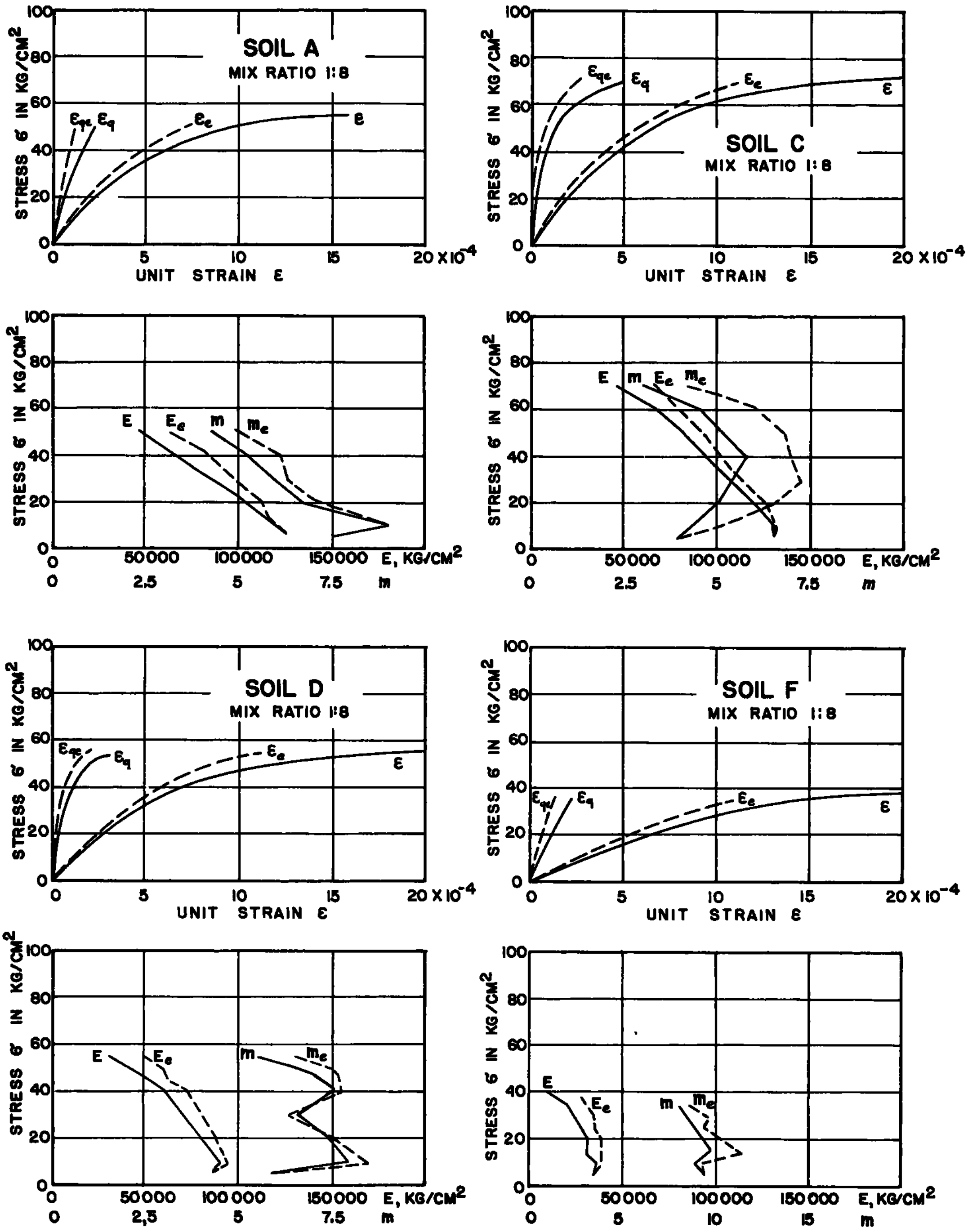

Figure 9. Graphical presentation of experimental results.

TABLE 2

Soil	Cement: Soil	$0.33\ s_D$ kg/cm²	E_e kg/cm²	m_e
A	1:6	29.8	138 500	8.35
	1:8	19.7	112 250	7.25
	1:10	14.2	91 000	7.05
C	1:6	40.0	143 000	8.00
	1:8	26.7	114 000	7.50
	1:10	19.2	93 000	7.35
D	1:6	24.2	93 000	9.00
	1:8	16.7	83 500	7.75
	1:10	13.5	66 500	10.56
F	1:6	17.85	46 000	11.1
	1:8	13.00	38 500	14.3
	1:10	10.00	29 650	18.75

angles of the linear portion of the stress-strain diagram with the strain axis. While the curve for the sandy soil starts with a relatively steep angle and then curves strongly into the upper flat part, the clay soils start with a flatter tangent, which is characteristic for them. One can say therefore:

The clay content is characterized by the angle of the tangent in the working range and by the curvature at $0.6\ s_D$. The influence of the clay contents is shown in Figure 12. The plotted points x give again the values for $0.33\ s_D$.

Practical Values for Modulus of Elasticity and Poisson's Number.

The $0.33\ s_D$ line seems to be especially suited for the fixing of practical average values for the modulus of elasticity and for Poisson's number. First, in this stress range lies the main use of soil-cement; second, all values converge quite strongly at this point and tend to become steady after initial wide fluctuations. In Table 2, average values from the experiments are given for the stress of $0.33\ s_D$. The following tendencies can be recognized: (1) the modulus of elasticity decreases with decreasing cement content and with increasing clay content and (2) Poisson number ($\frac{1}{\mu}$) increases with increasing clay content; the influence of the cement content appears to be indifferent or to depend upon the clay content, because in the case of sandy soils the m-values fall with decreasing cement content while they increase strongly in the case of clay soils.

An increase of the Poisson numbers indicates increasing compressibility of the material. The material can be compressed into itself with little lateral strain. Incompressible liquids, especially water, have the smallest possible Poisson number $m = 2$ ($\mu = 0.5$) which signifies volume constancy. Soil-cement with high clay content and low cement content possesses, therefore, a great compressibility in addition to a large range of elasticity.

SUMMARY AND PRACTICAL SIGNIFICANCE OF THE RESULTS

On the basis of the performed tests it could be established that soil-cement is basically similar to portland cement concrete, with respect to stress strain relationships.

Specifically, the following important results shall be stated: (1) determinant for the elastic behavior of soil-cement is its compressive strength; (2) this can be influenced by densification, cement content, water content, and clay content; (3) up to one-third of the compressive strength, one can assume a linear stress-strain diagram for soil-cement; (4) dependable values for moduli of elasticity and sufficiently accurate Poisson numbers are available for the range of practical application; and (5) they are functions of the stress and depend strongly upon the cement content and on the clay content; as average values may be taken: $E = 100,000$ kg. per sq. cm. and $m = 7.5$ ($\mu \simeq 0.13$).

On the basis of these investigations it appears possible to judge correctly the elastic behavior of soil-cement as a function of the cement and clay contents and to obtain dependable values for the dimensional design of pavement sections.

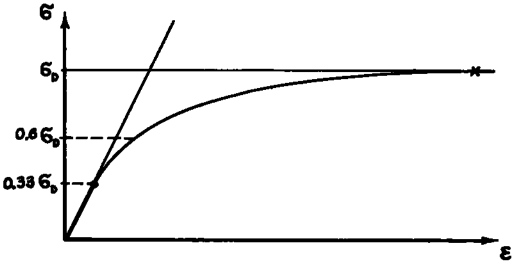

Figure 10. Characteristic stress-strain diagram of soil-cement.

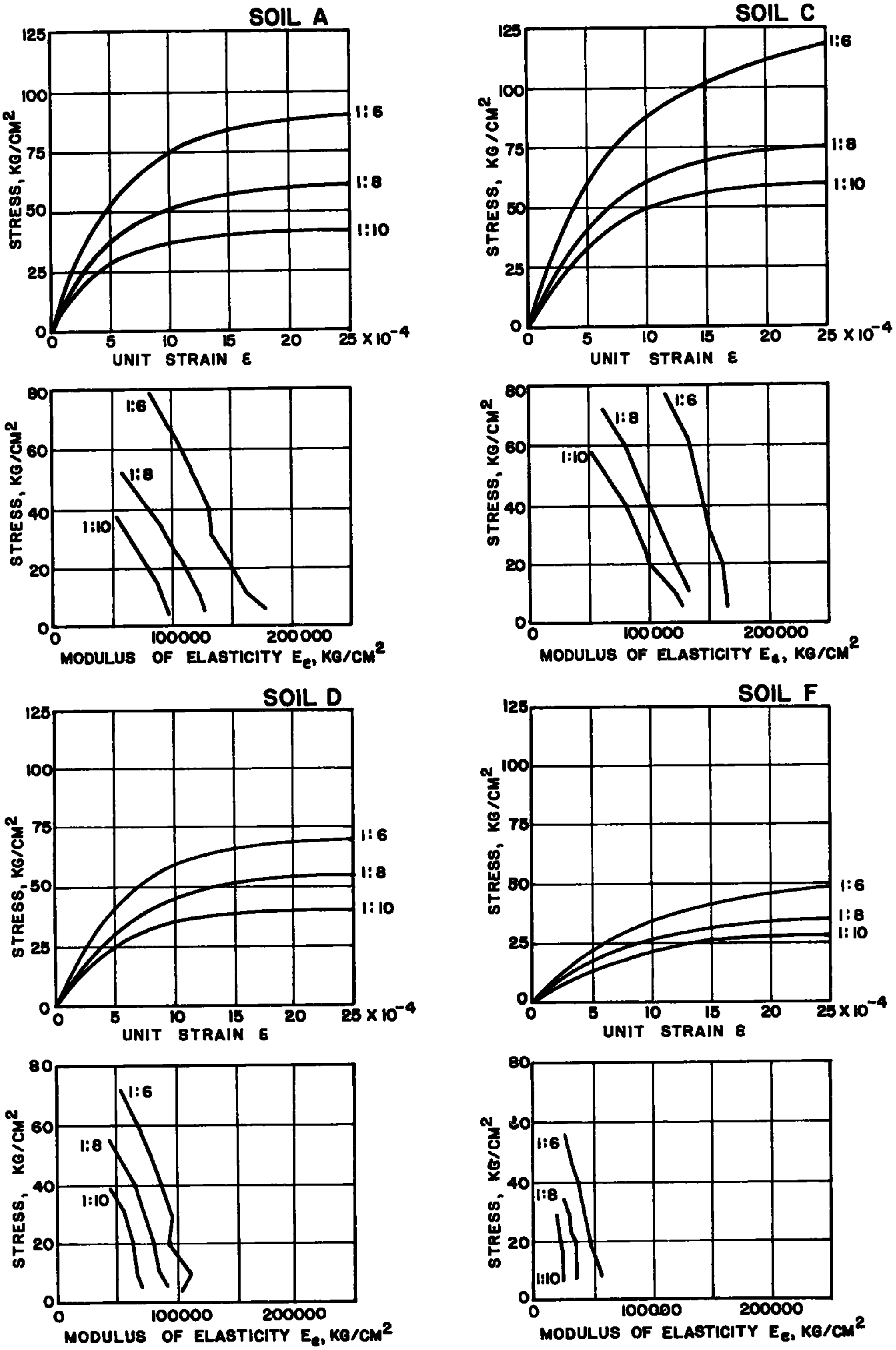

Figure 11. Influence of the cement content.

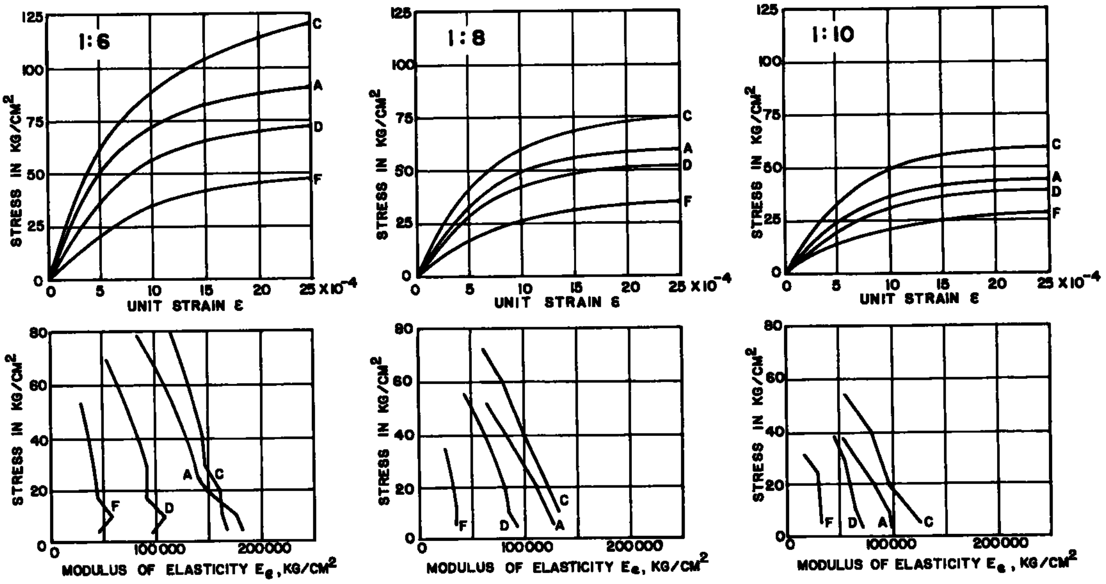

Figure 12. Influence of the clay content.

Acknowledgment

The investigational work presented in this paper was performed in the laboratories of the Experiment Station for Road and Municipal Engineering of the Technical University of Darmstadt mainly by G. Lammers with the coöperation of H. G. Ibelshaeuser of Darmstadt. To both, grateful acknowledgment is given for this work. Special thanks are due to Hans F. Winterkorn of Princeton University for reviewing this paper and translating it from German into English.

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Factors Influencing Physical Properties of Soil-Cement Mixtures

EARL J. FELT, Manager
Transportation Development Section, Portland Cement Association

Introductory Remarks by the Chairman: Felt has presented important results of the well-conceived and thorough investigations on soil-cement accomplished by the Portland Cement Association. It is especially gratifying to notice how in the development of soil-cement general pedological principles and specific physico-chemical concepts have been utilized at an early stage, together with the more-direct approach from soils and materials engineering.

The importance of this work for a better understanding and utilization of soil-cement is obvious. Less obvious (but perhaps of even greater significance) is the fact that the basic principles developed are applicable not only to systems composed of soil and portland cement but to soil stabilization with practically all types of inorganic cementing materials.

This paper may fill another need: Some have suspected a negative attitude in the emphasis given in the papers by our nonengineering contributors to the complexity of the science of soil stabilization and to the many problems that still remain to be solved. It is well for the scientist to point out these problems and for the engineer to listen. But it is also the glory of the engineer to view and measure these problems and to solve them with his creative genius. How this is done is well demonstrated in Felt's paper.

● THE properties of compacted and hydrated soil-cement mixtures are dependent to a great extent upon the types of soil involved. Certain differences in properties and in cement reaction are due to variations in the chemical composition of the soils. For instance, lateritic clays with a relatively low silica-sesquioxide ratio (colloids low in silica and high in the sesquioxides of iron and aluminum) pulverize more readily and react more favorably with cement than do northern podzolic or chernozem clay soils which have relatively high silica-sesquioxide ratios. Laboratory studies (1, 2) and field experience have shown the differences that may occur in the properties of soil-cement mixtures made with different clay soils.

Sandy soils, too, may react differently with cement depending upon their chemical makeup and surface chemical properties. For instance, a great variance exists between the relatively poor effectiveness of cement with the A horizon (topsoil) of many podzol or podzolic soils and the much greater effectiveness of cement with the C horizon (parent material) from the same soil profile. In these soils the organic matter in the A horizon is considered to be the chief offender, as it may retard or almost completely arrest the cement reaction.

Because of the tremendous effect organic matter may have on some soil-cement mixtures, special laboratory studies (3) and field experiments have been conducted. This work has shown that sandy soils of this type may be satisfactorily treated. Soil identification and careful sampling are, of course, prerequisites to effective soil-cement testing and construction. In this connection, the Pedological system of soil identification (4) which includes information on both the physical and chemical properties of soils is recommended for soil-cement work. Additional information of value in this field is available (5, 6).

Other factors which have pronounced influence on the physical properties of soil-cement mixtures include the quantity of cement and water added; the density to which the mixture is compacted; the length of time the soil, cement, and water are mixed prior to compaction; and the degree of pulverization of the soil if it is a clay.

The purpose of this report is to show how these last-named factors influence the strength and the relative durability of soil-cement specimens as indicated by their resistance to loss in weight resulting from alternate cycles of wetting and drying and of freezing and thawing in the laboratory. Although soil-cement may be used for a number of purposes (7), this report concerns compacted soil-cement mixtures (and cement-modified soils,[1] to a lesser extent) as used principally for base and subbase courses in highway and runway construction. It does not include data on "plastic" soil-cement, which is made with much higher water contents than the compacted type.

SCOPE OF PROGRAM

To obtain representative data, indicative of the performance of soil-cement mixtures made of different types of soil, tests were made on mixtures containing sandy, silty, and clayey soils. Seven series of tests were made to determine the influence of various factors upon the compressive strength and resistance to wetting and drying and to freezing and thawing of compacted, hydrated soil-cement mixtures. For convenience in this report these test series were identified as follows: Series 1, effect of density; Series 2, effect of molding moisture content; Series 3, effect of length of mixing time; Series 4, effect of degree of pulverization; Series 5, effect of air-entraining cement; Series 6, effect of the quantity of cement; and Series 7, effect of high early strength cement.

Data are also reported for three fine-grain soils treated with relatively low percentages of cement (cement-modified soils) and for three granular soils treated both with low percentages of cement as required for cement-modified soils and with higher percentages as required for soil-cement. These test series were designated: Series 8, properties of cement-modified fine-grain soils, and Series 9, properties of cement-modified granular soils compared with properties of granular soil-cement mixtures.

TABLE 1

GRADATION AND PHYSICAL TEST CONSTANTS[a] OF RESEARCH SOILS

Soil No.	Used in Test Series No.	Gradation - % of Total				Physical Test Constants[b]			Textural Class	U. S. BPR[c] Soil Group
		Sand		Silt	Clay					
		2.0 to 0.25 mm	0.25 to 0.05 mm	0.05 to 0.005 mm	0.005 to 0.000 mm	L. L.	P. I.	S. L.		
2a	1, 2	39	49	3	9	12	NP		Fine Sand	A-2
2a-2	3	24	49	8	19	26	11	18	Sandy Loam	A-2
2a-3	1, 2	35	46	8	11	13	NP	21	Loamy Sand	A-2
2a-4	1, 2, 3, 5	27	47	7	19	17	1	17	Sandy Loam	A-2
2a-6	6, 7	26	57	6	11	14	NP	20	Loamy Sand	A-2
4b-3	1, 2, 3,	2	17	57	24	38	13	25	Silty Clay Loam	A-4
4b-4	3	3	12	59	26	35	12	26	Silty Clay Loam	A-4
4b-5	1, 2, 3, 5	2	9	64	25	34	10	24	Silty Clay Loam	A-4
4b-6	4	2	7	67	24	37	12	20	Silty Clay Loam	A-4
4d	1, 2, 6, 7	1	8	67	24	34	14	16	Silty Clay Loam	A-4
6e	1, 2	2	10	35	53	49	26	17	Clay	A-6-7
6e-2	3	2	9	40	49	47	26	18	Clay	A-6-7
6e-4	1, 2, 3, 5	1	11	51	37	51	28	17	Silty Clay	A-6-7
7h	4, 6	8	9	36	47	37	18	16	Clay	A-7
7d	1, 2	0	14	18	68	118	83	14	Clay	A-7

[a] Obtained using Standard AASHO and ASTM Procedures

[b] L. L. = Liquid Limit
P. I. = Plasticity Index
S. L. = Shrinkage Limit
N. P. = Not Plastic

[c] Bureau of Public Roads

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[1] Cement-modified soil mixtures are those which contain less cement than the quantities required to definitely harden them to produce soil-cement.

The greatest use for soil-cement mixtures is in the construction of pavement base courses. For this reason, the cement contents used with the various soils when conducting Series 1, 2, 3, 4, and 5 were in the range that would normally be used in building pavement bases. These cement contents were 6 percent and 8 percent for the sandy soil, 12 percent and 14 percent for two silty soils, 12 percent for two clay soils, and 20 percent for one very-tough clay soil. Preliminary tests using AASHO Methods T135 (wet-dry test) and T136[2] (freeze-thaw test) were made to determine the cement requirements. In conducting Series 6, 7, 8, and 9, cement contents ranging from 1½ percent to 34 percent were included in the tests.

MATERIALS AND TEST METHODS

The physical properties of the soils used in the first seven test series are listed in Table 1. Similar data for soils used in Series 8 and 9 are given in discussion of those series. The first group in Table 1 is composed of five sandy soils all identified as Soil 2a. (The numbers after the 2a, i.e., 2a-2, 2a-4, etc., indicate different samples from approximately the same area.) Second is a group of five silty soils; third is a group of four clayey soils; and fourth is a single clay soil, very-heavy textured and extremely plastic. The numbers used in identifying these soils are also the AASHO soil-group numbers, thus Soil 4b and Soil 4d are A-4 silty soils, Soil 2a is an A-2 sandy soil, etc.

Soil 2a is a brown, fine, sandy soil from South Carolina. It is a mixture of the lower A, the B, and the upper C horizons of the soil profile and contains some organic contamination from the A horizon. This contamination varied in the different samples, depending upon the percentage of A horizon soil included. As previously mentioned, organic matter has a deleterious effect upon proper cement hydration, and thus some of the soil-cement made with samples of Soil 2a hardened at a slower rate and developed less strength than did soil-cement mixtures made with other samples of soil from approximately the same location.

Soil 4b (four samples) is a dark-gray, silty soil from a well-drained area in Illinois. It is from the lower A horizon of the soil profile and contains some organic matter. The presence of these organic compounds probably has some effect on the reaction of this soil with cement. Soil 4d is a brown silty soil from Illinois. It is from the lower B and upper C horizons and reacts in a normal manner.

Soil 6e (three samples) and soil 7h are brown clay soils from Illinois, from the B and upper C horizons of the soil profile. These soils react with cement in a normal manner.

Soil 7d is a light-brown, heavy clay soil from Mississippi. It is from the B and C horizons of the soil profile and is an unusually tough clay.

Type I portland cement, used in most of these tests, was a blend of four brands purchased on the open market. The soil-cement mixture was proportioned in the laboratory on a dry-weight basis, with cement contents selected to yield certain predetermined percentages expressed on a volume basis. Cement contents are reported, unless otherwise noted, in terms of volume of loose cement (94 lb. per cu. ft.) per unit volume of compacted soil-cement mixture. Thus, a cement content of 8 percent by volume indicates that a cubic foot of compacted mixture contains 0.08 sack (7.52 lb.) of cement.

Test Methods

In most of the work, three different tests were used in evaluating the influence of various factors upon the quality of the mixture. These were the wet-dry test, the freeze-thaw test, and the compressive-strength test. Specimens for the first two of these were 4 inches in diameter and 4.6 inches high. The testing methods were in accordance with AASHO methods T135 and T136, Wetting and Drying Test and Freezing and Thawing Test, respectively, except that the specimens were not in all cases compacted according to the AASHO standard procedure (T134) specified in these methods, and in some cases the tests were continued for more than the 12 cycles specified in the AASHO procedures. The compaction procedure was varied in several instances in order that the density and moisture content of the specimens could be varied and is described under each series of tests.

Specimens for the compressive strength tests were 2 inches in diameter and 2

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[2] ASTM Methods D559 and D560, respectively

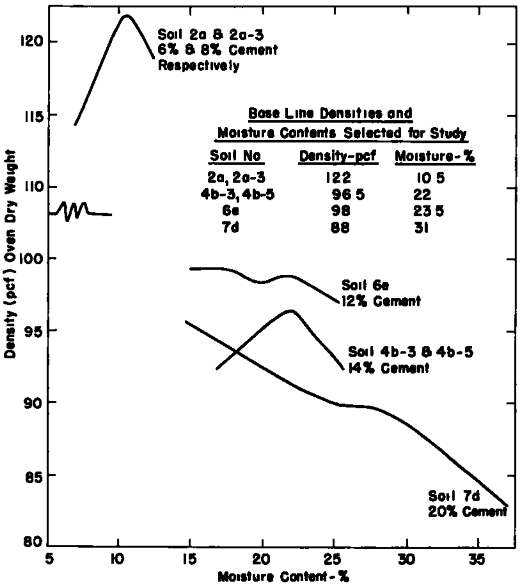


Figure 1. Moisture versus density of soil-cement mixtures used in Series 1 and 2.

inches in height. Specimens of this relatively small size were suitable as the soils used in the compressive-strength test were relatively fine textured and did not contain material retained on the No. 4 sieve. There is no AASHO standard method for making compression-test specimens of soil-cement mixtures.

Specimens were compacted from both ends using the double-piston method. A predetermined weight of mixture containing the proper moisture and cement content was placed in the cylindrical mold and compactive force applied through the pistons. When the pistons were separated by 2 inches, the specimen was at the designed density and contained the designed cement content by volume. After removing the specimens from the mold, they were stored for cement hydration in an atmosphere at 73 F. and 100 percent relative humidity. At the selected age for testing, the specimens were removed from storage, immersed in water for an hour, and then broken in a compression-testing machine.

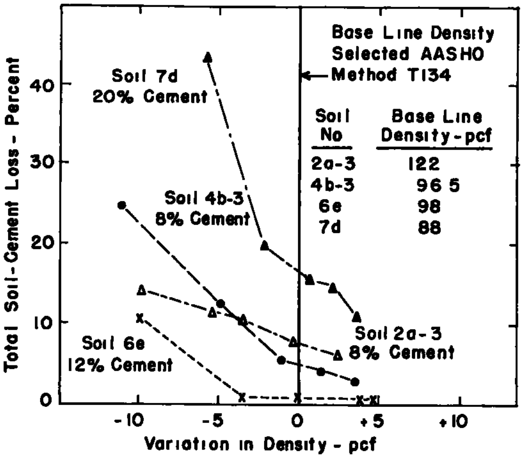


Figure 2. Effect of density on soil-cement loss from wet-dry test.

In Series 8 and 9, liquid-limit and plastic-limit tests, bearing-ratio tests, and soniscope tests (Series 9) were made to aid in evaluating the influence of cement in modifying the soil. Test procedures for these series are described later in the report.

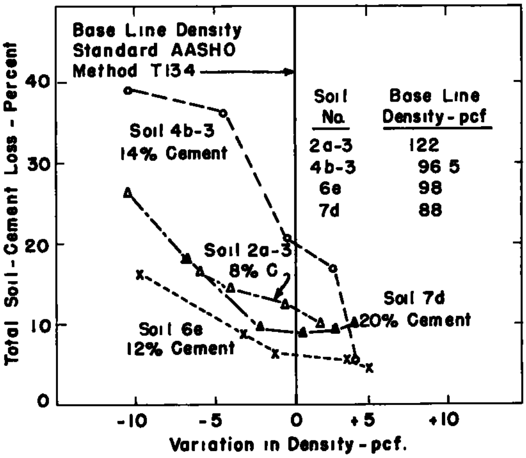


Figure 3. Effect of density on soil-cement loss from freeze-thaw test.

TEST RESULTS

Most soil-cement mixtures, when compacted in accordance with AASHO test T134, develop parabolic-shaped moisture-density curves, as shown for Soil 2a and Soil 4b in Figure 1, and a maximum density is indicated at an optimum moisture content at the peak of the curve. These values, obtained using the standard test, are called "base-line" values in Series 1 and 2, which were designed to study the effect of density and of moisture content on the properties of soil-cement mixtures.

The clay soil-cement mixtures used in the tests in Series 1 and 2 did not have parabolic-shaped moisture-density curves. These soils, because of their swelling characteristics as they become wet, tend to develop irregular moisture-density curves of ski-slide shape, as shown for Soil 7d in Figure 1. Experience (1, 2) has shown that, when a curve of this shape is obtained, the selection of a base-line density and base-line moisture content can be done most accurately after special tests have been made. The tests which will be discussed in Series 2 are of particular value with soils of this type, as they permit the engineer to select the moisture content which produces maximum effectiveness from the cement. The base-line optimum moisture content of soil-cement mixtures which have a secondary hump in the curve, as for instance Soil 6e in Figure 1, is generally taken about 2 percentage points above the water content at the second hump; the base-line density is that obtained at this particular moisture content.

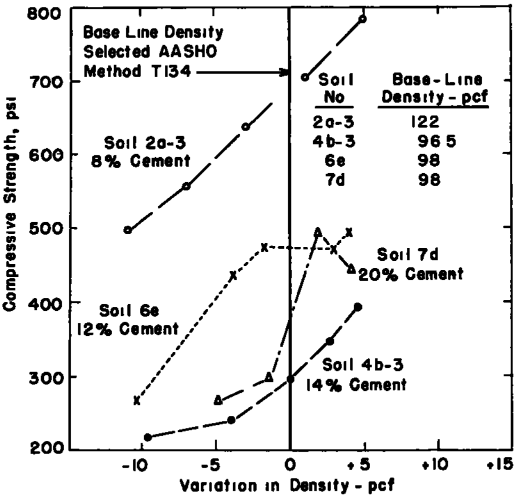


Figure 4. Effect of density on 28-day compressive strength.

Series 1: Effect of Density

Experience has shown that soil-cement mixture of good quality can be made by compacting the mixture to a density equal to that obtained by the AASHO compaction procedure. In Series 1 an investigation was made of the quality of soil-cement mixture compacted to lower and higher densities. In these tests the moisture content for each of the soil-cement mixtures was maintained constant and equal to the selected base-line moisture content. To vary the density at this moisture content, specimens for the wet-dry and freeze-thaw tests were molded according to the standard procedure, except that the number of blows of the compacting rammer per layer was changed from 25, and was set at 4, 9, 15, 20 and 50. This procedure produced specimens having a density variation of about 10 to 15 pcf. (pounds per cubic foot). Compressive-strength specimens 2 by 2 inches were then molded at densities and moisture contents equal to those of the wet-dry and freeze-thaw specimens.

Sandy Soil 2a-3, silty Soils 4b-3 and 4b-5, and clayey Soils 6e and 7d were used in the tests. Figure 1 shows the base-line densities and base-line optimum moisture contents selected for study. Figures 2, 3, and 4 show the base-line densities and also the variation in density from the base.

Results from the wet-dry test are shown in Figure 2 and from the freeze-thaw test in Figure 3. The ordinates show the weight loss from the specimens in 12 cycles of test. The effect of density is readily apparent; specimens had increasingly higher losses as the density decreased. Although all the different types of soil-cement mixture were benefited by increased density, the silty and clayey soil-cement mixtures were benefited the most. For the sand Soil 2a-3, each 1-pcf. increase in density reduced soil-cement losses about 1 percentage point. For silty Soil 4b-3 and clayey Soil 7d, however, the corresponding reduction in soil-cement losses varied from approximately 1.5 to 3.5 percentage points for each 1-pcf. increase in density in the low density range. (Soil 4b soil-cement contained 8 percent of cement in the wet-dry test, but 14 percent in the freeze-thaw test, which is critical for this soil.)

The compressive strengths of specimens molded at different densities are shown in Figure 4. Again, the value of high density is apparent. In the case of sandy Soil 2a-3, an increase in density of 1 pcf. resulted in an increase in compressive strength of approximately 20 psi. The compressive strengths of the silty and clayey soil-cements practically doubled as a result of increasing the density of these materials 10 to 15 pcf. For instance, with Soil 4b-3, at a density 5 pcf. below base-line density, the compressive strength was 235 psi., whereas at a density 5 pcf. above base-line density, the compressive strength was 400 psi. Soil-cement mixtures of clay Soils 6e and 7d showed similar performance. In general, with these soils, an

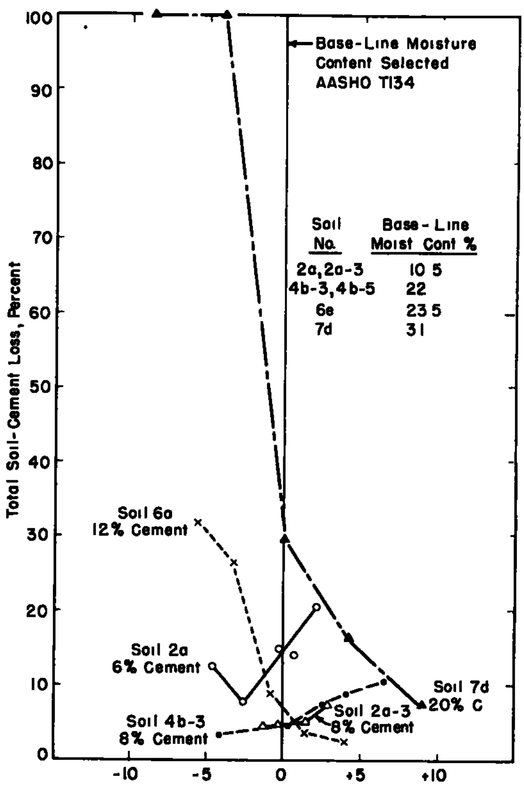


Figure 5. Effect of moisture content on soil-cement loss from wet-dry test.

increase in density of 1 pcf. in the low density range resulted in an increase in compressive strength of approximately 15 to 25 psi.

Series 2: Effect of Molding Moisture Content

To study the effect of molding moisture content on the quality of soil-cement mixtures, specimens for wet-dry and freeze-thaw tests were molded at moisture contents below and above the selected base-line moisture content, using the standard AASHO compaction procedure. Compressive strength specimens were molded at the same densities and moisture contents. Since the compactive effort was constant and the moisture content varied, the density of the specimens also varied, but the data indicate that the effect of moisture content overshadows the effect of the differences in density.

The data from the AASHO wet-dry and freeze-thaw tests are shown in Figures 5 and 6. For convenience, the data in Figure 6 are considered first; it is apparent from these data that soil-cement mixtures made of silty Soil 4b-3 and 4b-5 and clayey Soil 7d had much-less resistance to alternate freezing and thawing when they were compacted at moisture contents less than the base-line moisture contents. As seen in Figure 5, this was true in the wet-dry test for soil-cements of clay Soils 6e and 7d, but the soil-cement mixture made of Soil 4b was not affected significantly. (Soil 4b mixture contained only 8 percent of cement in the wet-dry test, but 14 percent of cement in the freeze-thaw test, which is critical for this soil.)

It is apparent from these data that to obtain high-quality mixtures from silty and clayey soils, the mixtures must be compacted at or above, never below, the AASHO T134 optimum moisture content when they are compacted to AASHO density. These data have been corroborated by other tests at the Portland Cement Association and are in agreement with results obtained by other investigators (2).

The effect of molding moisture content

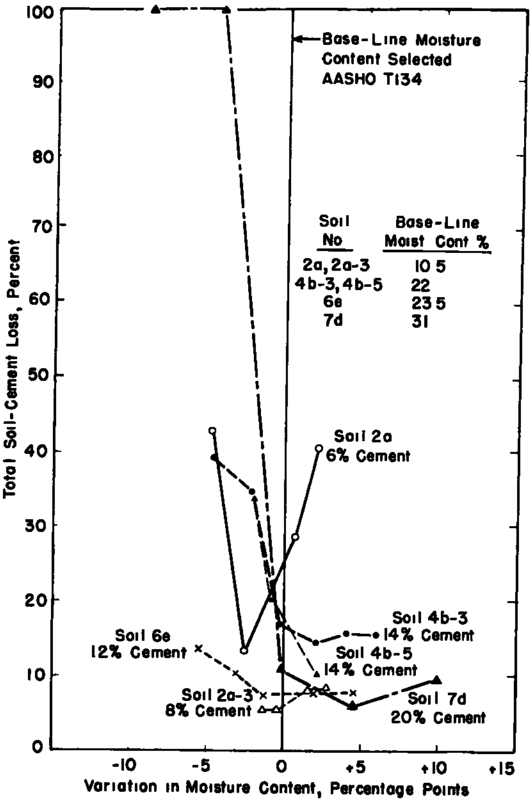


Figure 6. Effect of moisture content on soil-cement loss from freeze-thaw test.

on soil-cement mixtures made of sandy Soils 2a and 2a-3 was not the same as that found for the silty and clayey soils. In this case, moisture contents slightly on the dry side of base-line moisture content were favorable, suggesting that sand mixtures may follow to some degree the water-cement-ratio relationships for concrete.

The compressive-strength data for the various soil-cement mixtures are shown in Figure 7. Here it is seen that specimens molded at the base-line moisture content, or slightly below, except for Soil 7d, have maximum strength. The data for mixtures of Soil 7d indicate that the base-line moisture content selected for this soil was about 5 percentage points below the moisture content which produced maximum effectiveness from the cement.

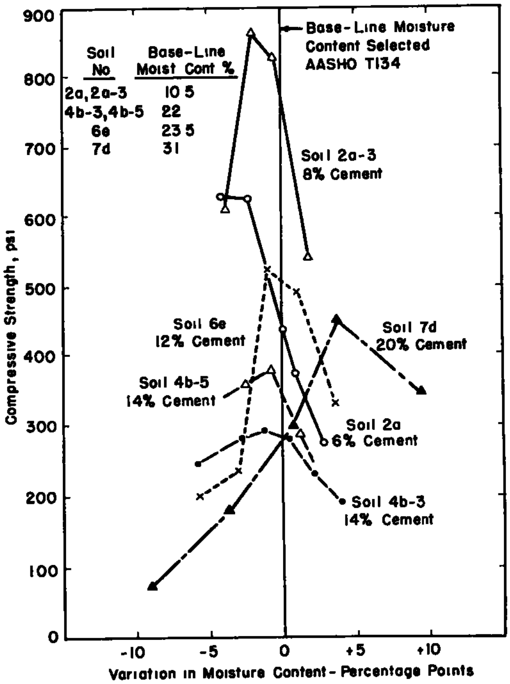


Figure 7. Effect of moisture content on 28-day compressive strength.

Based upon the wet-dry, freeze-thaw, and compressive-strength data combined, it appears that the best moisture content for Soil 7d soil-cement mixtures would be about 36 percent, or 5 percentage points above the selected base-line moisture content of 31 percent. As previously mentioned, the so-called optimum moisture content (that producing maximum effectiveness from the cement) for soil-cement mixtures having irregular moisture-density curves can be selected best after conducting special tests, such as these in Series 2.

It is to be noted that the newly selected optimum moisture content for mixture of Soil 7d (36 percent) is practically equal to the plastic limit (35 percent) of the soil, as indicated by the numerical difference between the P.I. and L.L. data in Table 1. It is of interest also to note that the plastic limit of Soil 6e (23 percent) is practically equal to the base-line optimum moisture of 23.5 percent, which was selected for Soil 6e soil-cement mixtures. Other tests on soils having irregular moisture-density curves confirm this generality.

The wet-dry, freeze-thaw, and compressive strength data, when considered together, indicate that for maximum effectiveness from the cement, sand mixtures should be compacted at optimum moisture content or slightly drier, whereas silty and clayey mixtures should be compacted at moisture contents 1 or 2 percentage points above optimum moisture.

Special Series 1 and 2

The test data for Series 1 and 2 give rise to questions as to the relative performance of soil-cement mixtures compacted to much-higher densities than those obtained in Series 1. In following the procedure of Series 1, i.e., molding specimens at different densities at the selected base-line moisture contents, the greatest density that was obtained was only about 5 pcf. above the selected base-line density. This is because the base-line water content plus air fills most of the voids in the mixture and thus limits the density.

If greater densities than these are desired, the moisture content must be decreased and a greater compactive effort used. The AASHO Modified method provides for this. In this method the standard 4-inch-diameter mold and the 2-inch-diameter rammer are used, but the material is placed in five layers, rather than in three as in the AASHO standard method, and each layer is compacted with 25 blows of a 10-lb. rammer falling 18 inches, rather than with a 5.5-lb. rammer falling 12 inches. This method produces a maximum density higher than AASHO base-line density at an optimum moisture content lower than AASHO base-line optimum moisture content.

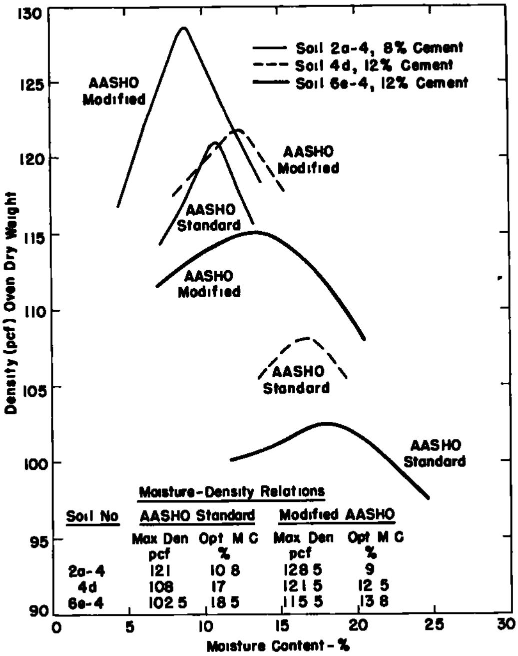
Soils used in these tests to determine the value of high densities included sandy Soil 2a-4, silty Soil 4d, and clay Soil 6e-4. The cement contents used were those that would commonly be used with these soils in pavement base construction.

As shown by Figure 8, the maximum densities of the soil-cement mixtures compacted by the AASHO Modified procedure were considerably greater than the maximum densities obtained with the standard method. For sandy Soil 2a-4 the AASHO Modified density was 7. 5 pcf. higher, and for the silty and clayey soils, approximately 13 pcf. higher. The optimum moisture contents were correspondingly lower with the AASHO Modified procedure, being about 2 percentage points lower for the sand mix and approximately 5 percentage points lower for the silt and clay mixtures.

Data in Table 2 indicate the relative quality of mixture produced using basic molding data obtained with the two methods. In this table, data are presented for specimens compacted at AASHO standard maximum density and AASHO standard optimum moisture content (A); AASHO Modified maximum density and AASHO Modified optimum moisture content (B); and AASHO standard maximum density and AASHO Modified optimum moisture content (C). The data obtained show that compressive strengths obtained using System B were considerably greater than those obtained using System A. This we might expect, as according to Series 1 the high densities are beneficial. Apparently at these higher densities less water is required in the mixture to effectively utilize the cement, and the acceptable moisture content is thus lower for the densities achieved by System B than for the lower densities of Series 2. As there is little difference between the wet-dry and freeze-thaw data for specimens compacted by Systems A or B and as the compressive strengths are much higher using System B, it appears that the soil-cement mixture compacted by System B is of the higher quality.

It is important to note that the density indicated by the AASHO Modified method as maximum for the silty and the clayey material is so great that it is difficult to obtain in construction practice. Methods of compaction which will produce this density are not generally available, and the density most likely to be obtained in construction is about that indicated as maximum by the ASTM method.

It is apparent that if only this latter density were obtained with a mixture compacted at the relatively low AASHO Modified optimum moisture content, the resulting quality of the mixture would be impaired. This is indicated in Table 2 by the compressive-strength data for the mixtures compacted using System C. For example, for the mixture using Soil 4d, the 28-day compressive-strength test for System C shows only 293 psi., as compared with 1,202 psi. for System B and 668 psi. for System A. System C also yielded inferior soil-cement mixtures with the clay Soil 6e-4. However, the sandy mixture compacted using System C appears to be satisfactory.



Moisture-Density Relations

Soil No	AASHO Standard Max Den pcf	AASHO Standard Opt M C %	Modified AASHO Max Den pcf	Modified AASHO Opt M C %
2a-4	121	10 8	128 5	9
4d	108	17	121 5	12 5
6e-4	102 5	18 5	115 5	13 8

Figure 8. Moisture-density relations of soil-cement mixtures, Special Series 1 and 2.

These data correlate well with the data in Series 1 and 2, where it was shown that for AASHO standard density the molding moisture content of the silty and clayey soil-cement mixtures must be equal to or slightly greater than the base-line AASHO standard optimum moisture content but for sandy soils a somewhat lower moisture content was satisfactory.

It appears, therefore, that if densi-

TABLE 2
COMPRESSIVE STRENGTHS AND WET-DRY AND FREEZE-THAW LOSSES OF SOIL-CEMENT COMPACTED TO AASHO STANDARD AND AASHO MODIFIED MAXIMUM DENSITIES AND OPTIMUM MOISTURE CONTENTS

Compaction System[a]	Density, pcf	Moisture Content, %	Compressive Strength, psi			Wet-Dry Loss, %		Freeze-Thaw Loss, %	
			Age in Days			No. of Test Cycles			
			2	7	28	12	24	12	24
			Soil 2a-4, 8% Cement by Volume						
A	121	10.8	265	665	800	3	5	5	7
B	128.5	9	435	732	1303	3	4	5	5
C	121	9	263	632	769				
			Soil 4d, 12 % Cement by Volume						
A	108	17	352	596	668	3	7	3	6
B	121.5	12.5	787	933	1202	3	27	3	6
C	108	12.5	229	277	293				
			Soil 6e-4, 12 % Cement By Volume						
A	102.5	18.5	203	417	486	31	86	9	35
B	115.5	13.8	326	427	709	28	97	5	6
C	102.5	13.8	70	138	149				

[a] A - AASHO Standard maximum density, AASHO Standard optimum moisture content
B - AASHO Modified maximum density, AASHO Modified optimum moisture content
C - AASHO Standard maximum density, AASHO Modified optimum moisture content

ties as great as those obtained by the AASHO Modified compaction procedure could be assured, in combination with their respective optimum moisture contents, soil-cement mixtures of superior quality would be produced. However, as these densities are not readily attained in practice, it usually is desirable to construct a soil-cement mixture near the AASHO standard optimum moisture content and to obtain as high a density as possible, preferably equal to AASHO standard density or greater. This procedure will provide sufficient water in the mixture to effectively utilize the cement at the particular density obtained.

Series 3: Effect of Prolonged Mixing

Soil-cement pavement bases are frequently constructed using mixed-in-place procedures in which the soil is pulverized in place, cement is added, and the dry mix is completed; water is then added and the damp mix is started. All of the required water is not added at one time, and several passes of the water equipment and of the mixing equipment may be necessary, with the result that the damp mixing may continue for 2 hours or more (8). Field experience has shown that mixtures of high quality may be produced in this way.* A laboratory study of the effect of this prolonged damp-mixing period was undertaken in the next series.

With the thought of simulating field conditions, soil-cement was damp mixed for periods of 2, 4, and 6 hours in the laboratory and then molded into test specimens. During the mixing period, water was added to the dry mix in equal increments at approximately 20-minute intervals. After each addition of water, the mixture was mixed by stirring for about 2 minutes. The water added in each increment was proportioned so that at the end of the specified time the mixture was at optimum moisture content. The optimum moisture content increases as the length of mixing time increases, so preliminary tests were made first to determine the altered optimum moisture content of the mixtures.

Under unusual field conditions, mixture that is damp may remain undisturbed for long periods without the intermittent mixing previously described. To obtain information on the effect of such treatment, test specimens were molded of mixtures that had been brought to optimum moisture content and had then been left in the loose condition undisturbed (without intermittent mixing) for periods of 2, 4, and 6 hours. Water lost by evaporation was replaced just prior to molding the specimens.

Wet-dry and freeze-thaw specimens were molded using the standard AASHO compaction method, and compressive-test specimens were molded at the same density and moisture content. Soils used in these tests were sandy Soils 2a-2 and 2a-4, silty Soils 4b-3 and 4b-5, and clay Soils 6e-2 and 6e-4. The cement contents used were those which commonly would be used with these soils in pavement base construction.

The AASHO optimum moisture contents and maximum densities of the various soil-cement mixtures subjected to intermittent mixing are shown in Figure 9. These data show that the optimum moisture content increased and the maximum density decreased as the length of mixing time increased. These effects suggest that a loose, damp mixture, probably through base exchange, develops a new structure and texture as it ages during the prolonged mixing periods. That this change starts soon after the soil, cement, and water are mixed is evidenced by data which show that the resistance to penetration of a small piston forced into a freshly compacted soil-cement specimen is nearly always greater than the penetration resistance offered by the compacted raw soil at the same moisture content.

As a result of 4 to 6 hours of intermittent mixing, the optimum moisture for the sand soil-cement mixtures increased 0.6 percentage point; that for the silt soil-cement mixtures increased about 1.3 percentage points; and that for the clay soil-cement increased 1 to 2 percentage points. The corresponding decreases in density ranged from 2 pcf. for the sand and silt mixtures to about 0.5 pcf. for the clay mixtures.

Experience has shown that corresponding differences in optimum moisture content and maximum density may be considerably more in the field during mixed-in-place construction. Thus, the data presented here, plus field experience, emphasize the necessity in field construction of obtaining moisture-density relations of soil-cement mixtures near the end of the moist-mixing period, just before compaction begins. These field optimum moistures and field maximum densities should then be used as the criteria for controlling the compaction of the mixtures, rather than laboratory or field tests made before the damp mixing has been completed. This is particularly important for mixtures made of silty and clayey soils, since it is imperative that they be compacted at moisture-contents at or slightly greater than the new optimum.

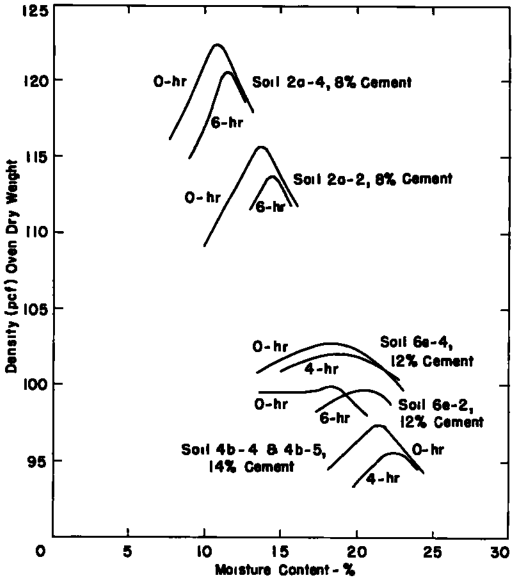


Figure 9. Moisture-density relations of soil-cement mixtures used in Series 3.

Wet-dry and freeze-thaw test data for these soil-cement specimens, plotted in Figures 10, 11, and 12, show that the soil-cement losses increased as the length of the damp-mixing period increased. This was most pronounced when the mixtures were not intermittently mixed during the period. For instance, for Soil 4b-4 (Figure 11) the losses after two hours were 13 percent for the mixture that was intermittently mixed but 40 percent for that which was undisturbed during the standing period. The data indicate that the least possible time should be consumed in damp mixing prior to compaction and that intermittent damp mixing of short duration, which is now common in soil-cement construction, is not seriously detrimental.

The compressive strength data are shown in Figures 13, 14 and 15. Figure 13 shows that the compressive strength for Soil 2a-2 decreased with the time of mixing, but for Soil 2a-4 it increased. (A similar observation will be noted in other test data involving prolonged mixing reported in Table 7.) Soil-cement mixtures made with Soil 2a-4 were stronger than those made with Soil 2a-2; although, as shown by Table 1, these two soils had practically the same gradation. However, Soil 2a-2 contained considerably more organic matter, and thus its reaction with cement was less effective than the reaction of Soil 2a-4. The data in Figures 14 and 15 for mix made of silty Soil 4b and clayey Soil 6e show that the strengths decreased with time of mixing. As in the case of the wet-dry and freeze-thaw tests, prolonged intermittent mixing was less harmful than an undisturbed delay.

Additional data showing the effect of prolonged mixing are presented in Series 5, Table 7; and in Series 8, Figures 20 and 21.

Series 4: Effect of Degree of Pulverization

Specifications for the construction of soil-cement base courses frequently require that 80 percent of the soil (exclusive of gravel, stone, etc.) be pulverized to pass the No. 4 sieve. Test Series 4 is an investigation of the effect of clay lumps in soil-cement, directed particularly toward the question of whether the quality of the mixture is adversely affected if less than 80 percent of the clay soil is pulverized.

Wet-dry and freeze-thaw soil-cement specimens were molded using Soils 4b-6 and 7h, each containing 0 percent, 20 percent, and 40 percent of lumps retained on a No. 4 sieve but passing a 1-inch sieve. The molding data for the specimens are given in Tables 3 and 4. In one set of specimens (A), air-dry clay lumps were added to the minus-No. -4 mixture which was at AASHO optimum moisture content. Specimens were molded immediately.

In the second set (B), air-dry clay lumps were added to the minus-No. -4 mixture, which was also air-dry. Water was then added to the total mix to bring it to optimum moisture content.

Thus, in Set A the clay lumps had less opportunity to absorb moisture during the mixing period than in Set B. In the latter case, some of the clay lumps were unintentionally pulverized during mixing. Immediately prior to molding specimens,

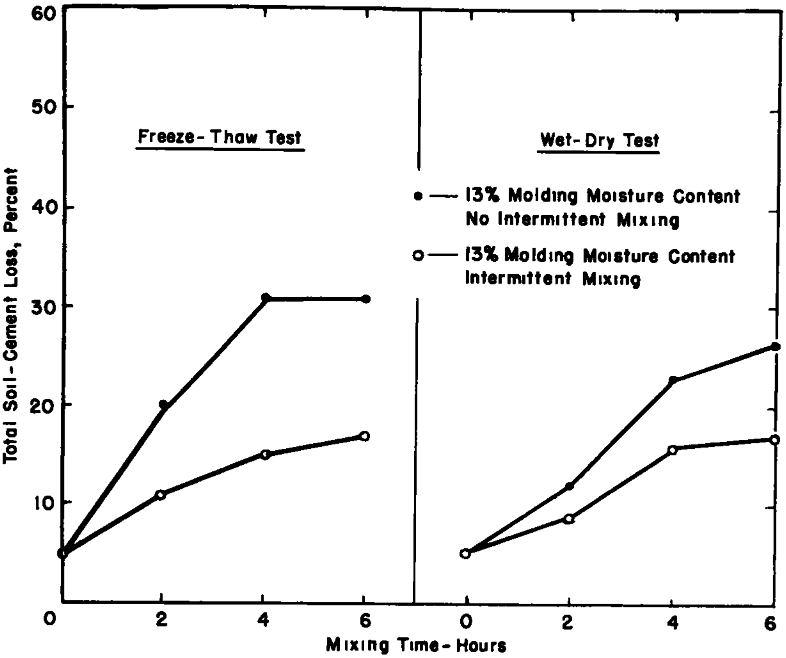


Figure 10. Effect of length of mixing time upon soil-cement losses: Soil 2a-2 plus 8 percent cement.

TABLE 3
MOLDING DATA FOR SOIL 4b-6, PLUS 14% CEMENT BY VOLUME
AASHO OPTIMUM MOISTURE CONTENT 19.8%, MAXIMUM DENSITY 102.5 PCF.
(SPECIMENS MOLDED ONLY FOR FREEZE-THAW TEST)

Set No.	Data on Clay Lumps			Moisture Content of Specimen, %		Density of Specimen, pcf.	Loss Due to 12 cyc. F-T, %
	% Included	Moisture Content, %					
		When Added to Mix	After Mixing	Minus No. 4 Mix	Total Mixture		
	0	–	–	19.9	19.9	102.5	2
A	20	3 to 4	4 to 5	19.8	16.4	103	8
	40	3 to 4	4 to 5	19.8	13.2	100	62
B	20	3 to 4	13 to 18[a]	–	20.1	101	6
	40	3 to 4	13 to 19[b]	–	19.7	101	8

[a] 14% clay lumps (unpluverized soil retained on No. 4 sieve) after damp mix completed.
[b] 22% clay lumps (unpulverized soil retained on No. 4 sieve) after damp mix completed.

TABLE 4
MOLDING DATA FOR SOIL 7h, PLUS 12% CEMENT BY VOLUME
AASHO OPTIMUM MOISTURE CONTENT 16.8%, MAXIMUM DENSITY 108.4 PCF.
(SPECIMENS MOLDED FOR WET-DRY AND FREEZE-THAW TESTS)

Set No.	Data on Clay Lumps			Moisture Content of Specimen, %		Density of Specimen, pcf.	Loss Due to 12 Cycles, %	
	% Included	Moisture Content, %						
		When Added to Mix	After Mixing	Minus No. 4 Mix	Total Mixture		W-D	F-T
	0	–	–	17.2	17.2	109	3	3
A	20	2	2	17	14.2	111	33	32
	40	2	2	17	11	109	100	100
B	20	2	9 to 13[a]	–	17.5	109.5	5	4
	40	2	11 to 16[b]	–	18	109.5	10	6

[a] 20% clay lumps (unpulverized soils retained on No. 4 sieve) after damp mix completed.
[b] 30% clay lumps (unpulverized soils retained on No. 4 sieve) after damp mix completed.

moisture content tests were made of the clay lumps and of the soil-cement mixtures.

As would be expected, the data in Tables 3 and 4 show that the clay lumps in Set A had gained little moisture by the time they were compacted into the test specimens, but the clay lumps in Set B had gained considerable moisture. The importance of this moisture factor is seen in the freeze-thaw and wet-dry test data, also presented in Tables 3 and 4. Here it will be noted that specimens of Set A had less resistance to alternate freezing and thawing and wetting and drying than Set B, and in some cases complete failure occurred by disruption of the specimens as the dry clay lumps absorbed water and swelled during the curing and testing periods. When the clay lumps were damp (Set B) and thus in a swelled condition at the time of inclusion in the test specimens, the unpulverized soil had little harmful effect.

These data show that the inclusion of damp clay lumps is not particularly harmful and may be permitted in soil-cement construction in accordance with properly written specifications. The inclusion of dry clay lumps, however, is harmful and should not be permitted. To eliminate the possibility of dry clay lumps, clayey soils, when necessary, can be pre-wetted a short time prior to construction.

Series 5: Effect of Air-Entraining Cement

In Series 5 a study was conducted to determine the comparative performance of mixtures made with air-entraining and non-air-entraining cements. Both cements were from the same mill, but one (Type IA) was with vinsol resin interground. Three soils were tested with each of the cements: Soil 2a-4 plus 8 percent of cement, Soil 4b-5 plus 14 percent of cement, and Soil 6e-4 plus 12 percent of cement. Test data were obtained for mixtures that were intermittently mixed for periods of 0, 2, 4, and 6 hours. For the sake of brevity, only the data for the 0-hour and 4-hour soil-cements are presented.

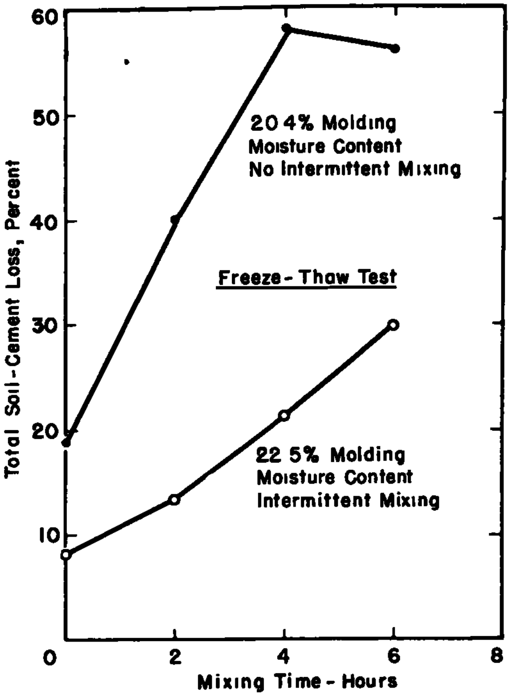


Figure 11. Effect of length of mixing time upon soil-cement losses: Soil 4b-4 plus 14 percent cement.

Moisture-density data, presented in Table 5, were obtained for the various mixtures using the standard AASHO method. The maximum densities and optimum moisture contents for each group were practically the same regardless of the type of cement, and the maximum densities generally decreased and the optimum moisture contents increased as the length of mixing time increased, which was also observed previously in Series 3.

TABLE 6
WET-DRY AND FREEZE-THAW TEST RESULTS FOR SOIL-CEMENTS CONTAINING TYPE I AND TYPE IA CEMENT

Soil No.	Approx. Cement Cont. by Vol., %	Type Cement	Wet-Dry Loss, % Mixing Time 0-hr	 4-hr	Freeze-Thaw Loss, % Mixing Time 0-hr	 4-hr
2a-4	8	I	5	10	8	16
		IA	4	10	6	13
4b-5	14	I	Tests not made		11	28
		IA			7	17
6e-4	12	I	17(8)[a]	26(18)	3	7
		IA	30(16)	25(16)	5	9

[a] Figures in () are soil-cement losses for specimens molded at a moisture content 4 percentage points wetter than ASTM indicated optimum moisture content.

Data from wet-dry and freeze-thaw tests are given in Table 6. Here it will be noted that there is relatively little difference in test data for the two cements; if there is any small advantage, it is in favor of Type IA.

Compressive strength results are given in Table 7. These data show only minor differences between the strengths obtained with Type I and Type IA cements.

Although these limited tests show little difference in the performance of mixtures made with Type I and Type IA cements, unusual effects have occasionally been noted in the laboratory with different soils and different cements. On this basis, it appears advisable to conduct the laboratory tests with the same type of cement that will be used in construction.

Series 6: Effect of Cement Content

The tests in Series 6 were made to investigate the effect of cement content on compressive strength and wet-dry and

TABLE 7
COMPRESSIVE STRENGTHS OF SOIL-CEMENTS CCNTAINING TYPE I AND TYPE IA CEMENTS

Soil No.	Approx. Cement Cont. by Vol., %	Type Cement	Compressive Strengths, psi. Mixing Time, Hours Zero Age, Days 7	 28	 180	 Four 7	 28	 180
2a-4	8	I	554	734	818	730	774	747
		IA	651	765	850	750	675	745
4b-5	14	I	358	440	503	325	429	486
		IA	322	411	516	314	401	489
6e-4	12	I	463	516	633	487	607	663
		IA	393	495	434	463	604	565

freeze-thaw resistance of soil-cement.

Moisture-density relations were established for mixtures of Soils 2a-6, 4d, and 7h with cement contents of approximately 8, 12, 16, 22, and 28 percent. From these curves, shown in part in Figure 16, the optimum moisture content and maximum density were obtained by interpolation and extrapolation for molding wet-dry, freeze-thaw, and compressive-strength specimens containing 6 to 34 percent of cement, as indicated in the tables and charts.

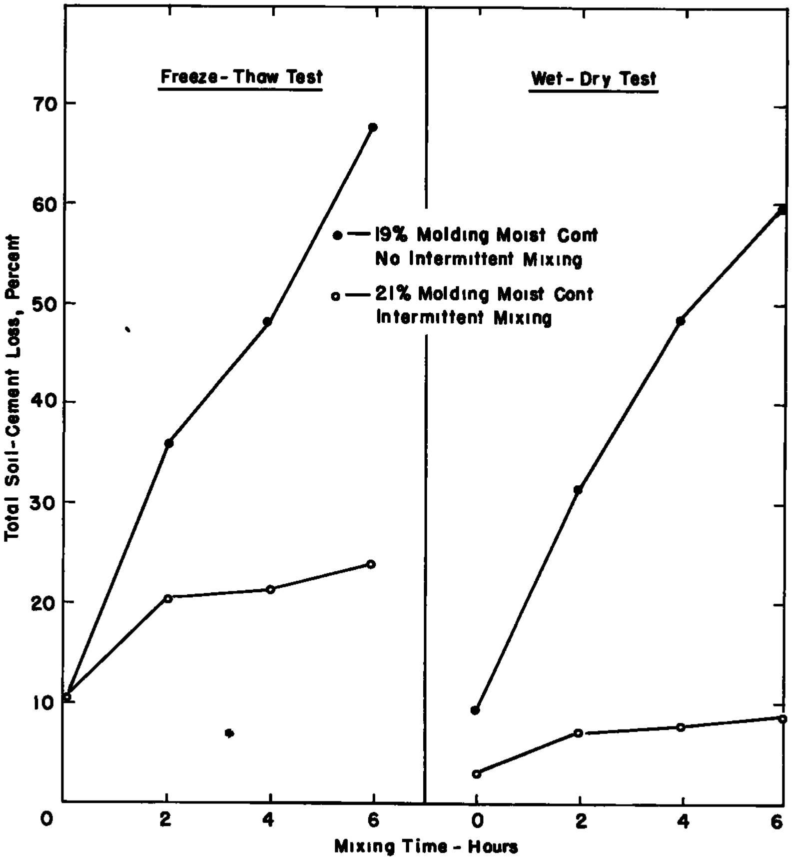


Figure 12. Effect of length of mixing time upon soil-cement losses: Soil 6e-2 plus 12 percent cement.

In this series, the wet-dry and freeze-thaw specimens were tested through 96 cycles. The data in Tables 8, 9, and 10 show that mixtures containing relatively high cement contents had great resistance to alternate wetting and drying and freezing and thawing. After 96 cycles many of the test specimens had practically no loss of material.

Soil-cement mixtures made of sand Soil 2a-6 were particularly resistant to wetting and drying and freezing and thawing, and as shown in Table 8, even the 8-percent-cement specimens gave fair performance for 96 cycles. All specimens containing 12 percent or more of cement were excellent, showing only a small loss after 96 cycles of test.

The data in Table 9 show that mixtures made of silty Soil 4d improved in quality as the cement content increased to 30 percent. Specimens containing 18 to 22 percent or more of cement showed good performance in the freeze-thaw and wet-dry tests for the full 96 cycles. In the wet-dry test several specimens split at the compaction planes (specimens were molded in three layers) and then cracked vertically into pieces which were hard and durable. Pieces less than ¾ inch in size were discarded and included in the material loss. The significance of this cracking has not been established; however, field performance of mixtures that crack

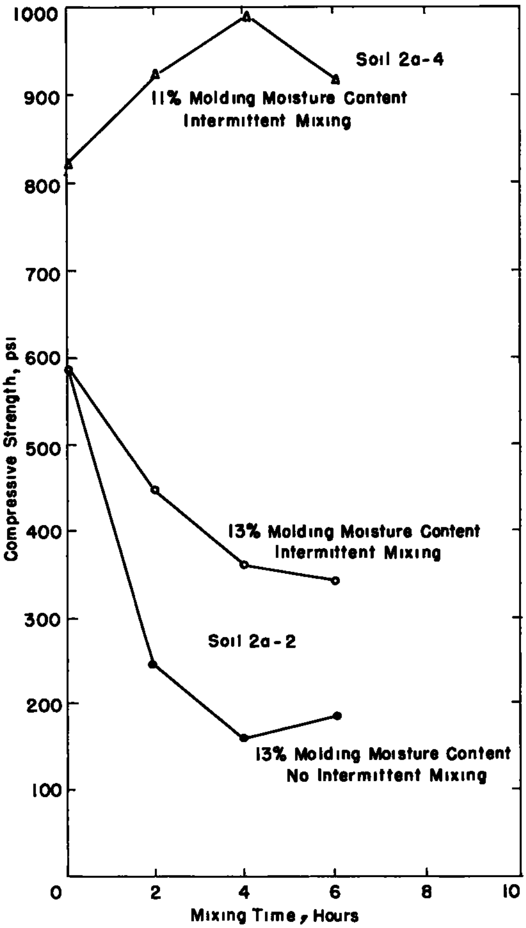


Figure 13. Effect of length of mixing time upon 28-day compressive strength: Soil 2a plus 8 percent cement.

in the laboratory wet-dry test has been satisfactory.

The data in Table 10 show that mixtures made of Soil 7h improved in quality as the cement increased up to 30 percent. Specimens containing 18 percent cement or more were good up to the 84th cycles. With this soil there was a sharp increase in quality as the cement content increased from 14 percent to 18 percent, particularly in the freeze-thaw test. As in the case of wet-dry specimens made of Soil 4d, Soil 7h specimens cracked in the wet-dry test but to a lesser extent.

The compressive-strength data for the various mixes are shown in Figures 17, 18, and 19. As might be expected, the specimens containing sandy Soil 2a-6 were considerably stronger than those containing silty Soil 4d, which were stronger than specimens containing clayey Soil 7h.

TABLE 8
Soil 2a-6
SOIL-CEMENT LOSSES % OF ORIGINAL WEIGHT

No. of Cycles	Wet-Dry Test								Freeze-Thaw Test							
	Cement Content By Vol., %															
	8	10	12	14	18	22	26	30	8	10	12	14	18	22	26	30
12	6	4	2	2	1	0	0	0	12	5	2	1	0	0	0	0
24	10	6	3	2	1	1	0	0	19	8	4	2	1	0	0	0
36	14	7	3	2	1	1	1	1	25	10	5	3	1	1	0	0
48	18	9	4	3	1	1	1	1	30	13	7	4	1	1	0	0
60	21	11	5	3	1	1	1	1	37	16	9	5	2	1	0	0
72	24	12	6	4	2	1	1	1	42	17	10	6	2	1	0	0
84	27	13	6	4	2	1	1	1	46	19	11	6	2	1	0	0
96	28	14	6	4	2	1	1	1	51	21	12	7	3	2	1	0

Soil 2a-6 showed a rather consistent increase in strength at all ages as the cement content increased from 6 percent to 30 percent. The other two soils showed a similar relationship to age 120 days, with some inconsistency in strength gain from 120 to 365 days. Maximum strengths obtained were 4,700 psi. for Soil 2a-6, 3,100 psi. for 4d, and 2,300 psi. for 7h.

Series 7: Effect of High-Early-Strength (Type III) Cement.

Compressive strength tests were made to study the effect of Type III cement in soil-cement mixtures. Specimens of sandy Soil 2a-6 and silty Soil 4d, each with 6, 10, and 14 percent of cement, were molded at AASHO optimum moisture content and

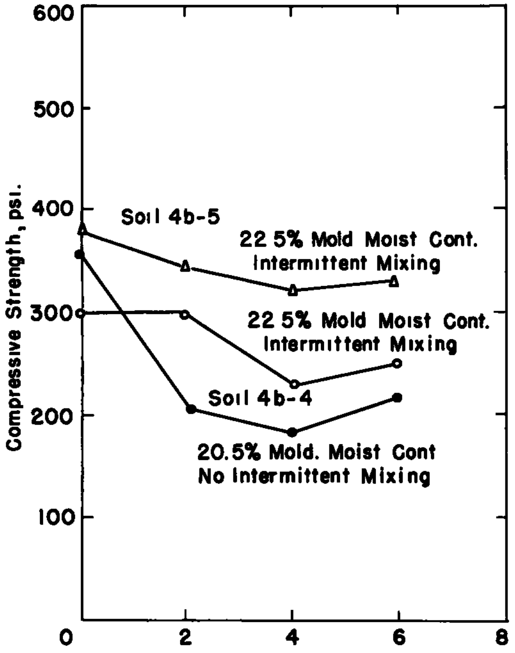


Figure 14. Effect of length of mixing tim upon 28-day compressivd strength: Soil 4 plus 14 percent cement.

TABLE 9
Soil 4d
SOIL-CEMENT LOSSES % OF ORIGINAL WEIGHT

No. of Cycles	Wet-Dry Test								Freeze-Thaw Test							
	Cement Content by Vol., %															
	8	10	12	14	18	22	26	30	8	10	12	14	18	22	26	30
12	7	5	4	2	2	1	1	1	21	7	3	2	2	2	1	1
24	29	25	21	9	4	2	1	1	58	24	6	4	2	2	1	1
36	53	43	33	27	11	7	1	1	70	33	9	9	2	2	2	2
48	67	54	48	35	18	8	3	2	82	53	16	14	3	3	3	2
60	72	63	56	46	22	9	4	3	92	66	21	18	5	5	4	3
72	75	70	62	47	38	12	5	4	100	74	40	20	5	5	4	4
84	80	76	67	52	47	17	7	4	100	88	45	21	5	5	4	4
96	87	83	78	60	56	20	9	5	100	93	55	24	5	5	4	4

maximum density. To determine whether prolonged intermittent mixing would have an unusual effect on soil-cement mixtures containing Type III cement, one set of specimens was molded after a 4-hour mixing time. (The same procedures were used as in Series 3.) The moisture-density relations in Table 11 show that the optimum moisture content and the maximum density for the mixtures containing Type I or Type III cement are practically the same. As was established in Series 3, a prolonged mixing period of 4 hours resulted in a general decrease in maximum density and an increase in optimum moisture content.

Compressive strength data for specimens broken at ages of 1, 2, 3, 4, 6, 7, 10, 14, 28, and 60 days are shown in Figures 20 and 21. For both soil types and both mixing procedures the early-age strengths were consistently greater for Type III than for Type I cement, and in nearly all cases the 60-day strengths were also greater for Type III.

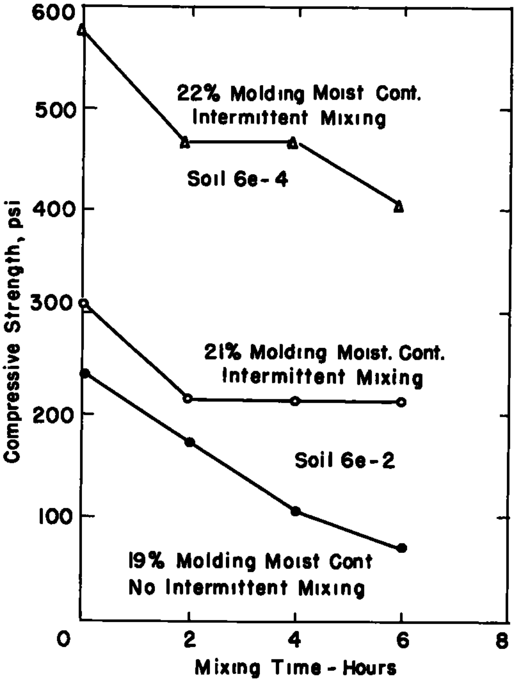


Figure 15. Effect of length of mixing time upon 28-day compressive strength: Soil 6e plus 12 percent cement.

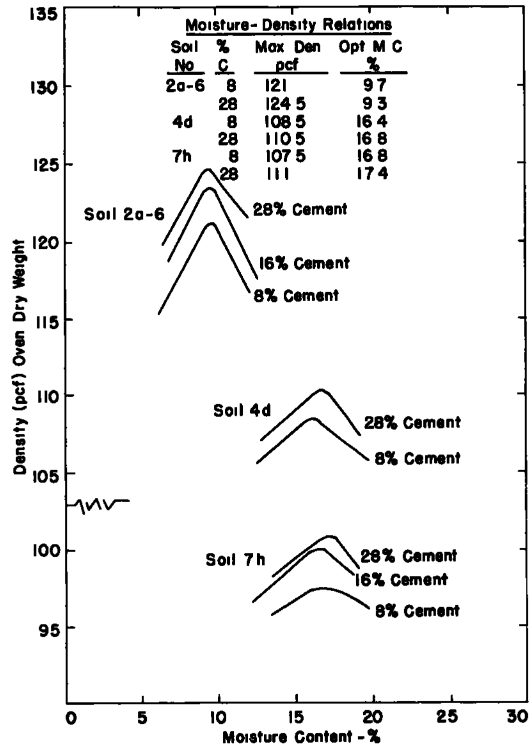


Figure 16. Moisture-density relations of soil-cement mixtures used in Series 6.

The prolonged intermittent mixing was not seriously detrimental with either of the cement types, although it resulted in some loss of strength.

Series 8 and 9: Cement-Modified Soils

Most clayey soils are volumetrically unstable, for they shrink when dried and expand when wetted; furthermore, their strength characteristics are unusually sensitive to changes in moisture content. Stabilization of these soils is an important field, and portland cement in quantities less than required for regular soil-cement mixtures has been used to reduce the extent to which the soils shrink, swell, and lose strength. The material thus produced is referred to as cement-modified soil. This type of soil stabilization is

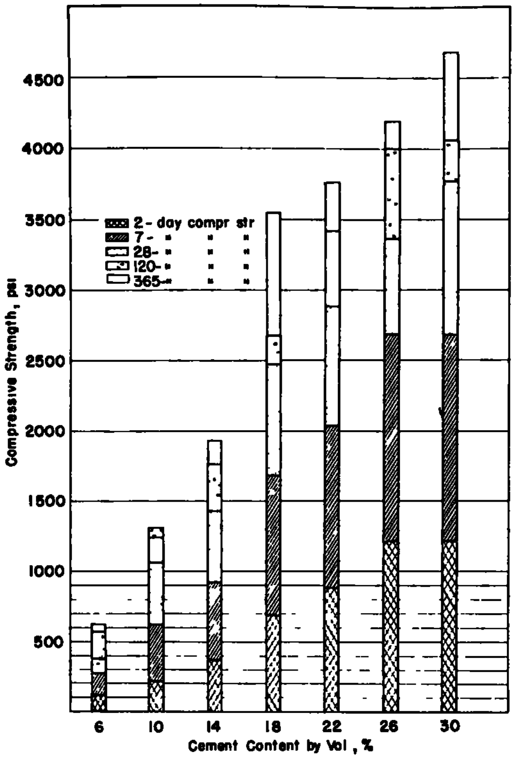


Figure 17. Effect of cement content and age on compressive strength: Soil 2a-6.

made possible through the surface-chemical effects of cement in reducing the water affinity and holding capacity of the clayey soil.

Cement modification may be applied also to granular materials which frequently in the natural state are poorly graded or contain so much clayey material that they become excessively plastic when wet. Of course these materials make excellent soil-cement mixtures with relatively low cement contents, but nevertheless there is considerable interest in modifying these materials rather than in definitely hardening them.

The optimum quantity of cement to modify a soil is not readily determined, as there are no standardized tests for evaluating the effect of the cement nor are there criteria for selecting the optimum cement content. The tests of Series 8 and 9 investigated the effect of cement on liquid limit, plastic limit, shrinkage limit, and gradation. A high plasticity index (liquid limit minus plastic limit) is normally associated with active clayey soils, whereas less-active clays, silts, or sandy soils have a lower index. Low shrinkage limits are associated with soils that show high shrinkage, and vice versa. Thus by determining these soil constants for cement-modified soils, some measure of the effectiveness of the cement in modifying the undesirable plasticity characteristics of the soil may be obtained. With some soils, bearing-ratio tests were made to evaluate further the effect of the cement.

TABLE 10
Soil 7h
SOIL-CEMENT LOSSES % OF ORIGINAL WEIGHT

No. of Cycles	Wet-Dry Test								Freeze-Thaw Test							
	Cement Content by Vol., %															
	8	10	12	14	18	22	26	30	8	10	12	14	18	22	26	30
12	9	7	3	2	1	1	1	1	10	6	3	3	2	1	1	1
24	24	18	10	4	1	1	1	1	19	18	4	4	2	1	1	1
36	40	33	16	9	2	1	1	1	36	26	7	7	3	2	1	1
48	61	45	27	18	8	2	1	1	61	41	9	8	3	2	2	2
60	72	48	39	24	11	6	1	1	84	86	41	10	4	3	2	2
72	78	49	43	31	15	7	1	1	100	100	64	25	4	3	2	2
84	80	52	45	33	19	9	3	1	100	100	83	43	4	3	2	2
96	92	56	46	35	24	11	5	1	100	100	97	52	34	27	3	3

Series 8: Cement-Modified Fine-Grain Soils

In this series three cement-modified clayey soils were studied to determine the influence of various cement contents in altering the properties of the soils. Samples to determine test constants and grain size were prepared by compacting, at optimum moisture content, sufficient soil and cement mixture to fill a third of a standard 4-by-4.6-inch specimen mold.

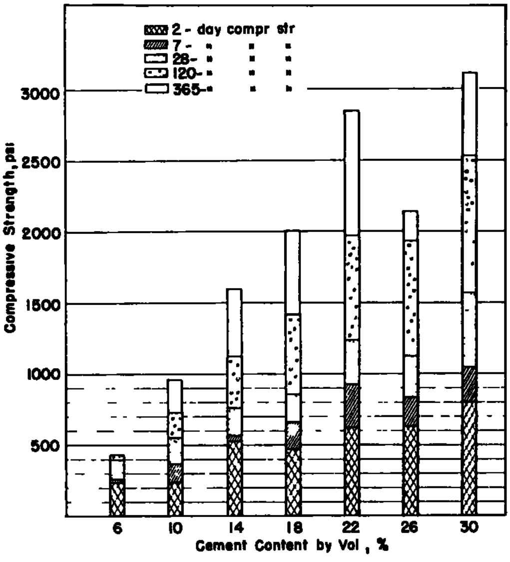


Figure 18. Effect of cement content and age on compressive strength: Soil 4d.

TABLE 11
MAXIMUM DENSITY AND OPTIMUM MOISTURE CONTENTS FOR SOIL-CEMENTS OF SOILS 2a-6 AND 4d

Soil No.	Soil 2a-6 with approx. 10% cement				Soil 4d with approx. 14% cement			
Cement	Type I		Type III		Type I		Type III	
Damp Mixing time, hr	0	4	0	4	0	4	0	4
Maximum density, pcf	122	119.5	122	119.5	107	107.5	106	105.5
Optimum moisture, %	9.7	11.0	9.7	11.0	17.8	18.3	17.8	18.2

These specimens were permitted to hydrate at 100 percent relative humidity for seven days; then part of the material was pulverized to pass a No. 10 sieve for a hydrometer analysis, and part was pulverized to pass a No. 40 sieve for determining test constants.

In one case, to see if repeated freezing and thawing affected the permanence of the cement influence, a set of specimens was subjected to 60 cycles of alternate freezing and thawing (following AASHO T 136). The materials were then pulverized to pass the sieves noted above.

The data in Tables 12 and 13 show that cement effectively reduces the plasticity index and increases the shrinkage limit of clayey soils. The small differences between the test constants before and after 60 cycles of freezing and thawing indicate that the effectiveness of the cement is not readily destroyed. The gradation analysis of the cement-modified soils shows that the percentage of clay-size particles is reduced by the cement action.

TABLE 12
TEST CONSTANTS AND GRADATION OF CEMENT-MODIFIED SOIL
Soil 6g, Clay Soil from Texas

% Cement by Volume % Cement by Weight	0 0	2 2.16	4 4.37	6 6.61	8 8.90	10 11.24
Liquid Limit	54	55	52	50	48	47
Plastic Limit	21	21	24	31	33	35
Plasticity Index	33	34	28	19	15	12
Shrinkage Limit	18	23	26	29	31	33
Coarse Sand, % (2.0-0.25mm.)	1	4	6	18	23	35
Fine Sand, % (0.25-0.05mm.)	11	7	25	24	26	27
Silt, % (0.05-0.005mm.)	40	37	33	29	31	24
Clay (less than 0.005mm.)	48	46	36	29	20	14

Specimens compacted to approximately 92 pcf. at 26 percent water, and hydrated 7 days, then pulverized for test constants and grain size.

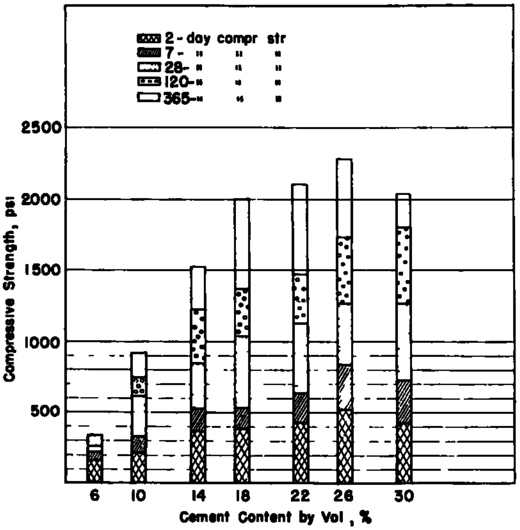


Figure 19. Effect of cement content and age on compressive strength: Soil 7h.

Additional data showing the effect of cement in modifying a silty clay loam soil are given in Table 14. In this case a bearing-ratio test was used as an aid in measuring the beneficial effect of the cement.

This test is the same as the California Bearing Ratio (CBR) test[3], but has been designated simply as a bearing-ratio test because at the high bearing values of some of the treated soils the ratio does not have quite the same significance as the CBR. The test is made by forcing a piston having an end area of 3 sq. inches into the top surface of a specimen confined in a steel mold 6 inches in diameter and 4.6 inches high. The cement-modified soil is compacted in the mold in three layers with the same compactive effort used for compacting soil-cement mixtures according to AASHO standard T 134. This requires 56 blows of the 5.5-lb. rammer on each of the three layers. In these tests, the soil and cement mixtures were hydrated in the molds for seven days at 100 percent relative humidity and then immersed in water for four days before testing. The load in pounds per square inch to force the piston into the specimen to a penetration of 0.1

---

[3] The California Bearing Ratio Test, reported by O J Porter (9) and by W H Jervis and Joseph B Eustis (10), is based on the observation that a force of 1,000 psi is required to push a 1 95-inch-diameter piston into a crusher-run, high-quality base-course material to a depth of 0 1 inch The relative strength or California Bearing Ratio of other base course materials is obtained by determining the penetration resistance in pounds per square inch at 0 1 inch and by dividing this unit load by 1,000 and multiplying by 100 to obtain a percentage value Thus a material having a penetration resistance of 500 psi has a CBR of 50

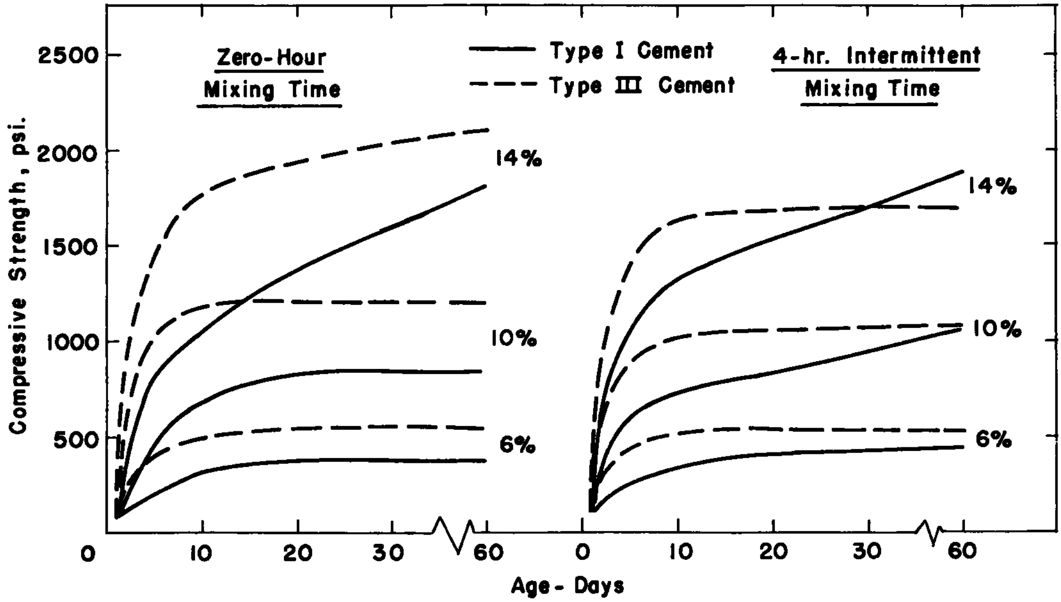


Figure 20. Comparison of compressive strengths obtained with normal (Type 1) cement and high-early-strength (Type 3) cement: Soil 2a-6.

inch, divided by 10, is referred to as the comparative-bearing ratio.

The data in Table 14 indicate that the addition of cement (up to 9 percent) to this fine-grain soil changed its plasticity characteristics by reducing the P.I. from 14 to 0 and greatly increased its strength as indicated by an increase in bearing-ratio values from 2 to 138.

Series 9: Cement-Modified Granular Soils

Relatively small quantities of cement added to granular soils not naturally suitable for base construction will increase their all-round stability and strength. Series 9 is concerned with this field of cement stabilization.

From the data previously discussed on cement-modified fine-grain soils, it appeared that the bearing-ratio test was fairly effective in showing the beneficial influence of the addition of cement to plastic soils which soften when they become wet. Thus this test was used in studying the effect of additions of relatively small quantities of cement to substandard granular materials. In addition, tests on the specimens at different ages and after various cycles of alternate freezing and thawing were made with the soniscope, a device (11) which measures the velocity with which shock pulses travel through the material. Normally, the velocity increases as the strength of the material increases and decreases if deterioration occurs due to frost action or some other cause.

Three granular soils, containing various percentages of silt and clay as listed in Table 15, were used in these tests. Soil 2b was a pit-run sand and gravel containing little silt and clay and might be considered a fairly good base material, except that it was poorly graded in the sand sizes;

TABLE 13
TEST CONSTANTS AND GRADATION OF CEMENT-MODIFIED SOIL
Soil 6e-5, Clay Soil from Illinois

% Cement by Volume	0		2		4		6		8		10	
% Cement by Weight	0		1.93		3.92		5.95		8.04		10.18	
Tests made after . . .	7 dy	60 cy	7 dy	60 cy	7 dy	60 cy	7 dy	60 cy	7 dy	60 cy	7 dy	60 cy
Liquid Limit	49	53	48	49	45	47	45	45	43	45	45	46
Plastic Limit	18	27	23	29	25	34	31	32	30	35	35	36
Plasticity Index	31	26	25	20	20	13	14	13	13	10	10	10
Shrinkage Limit	18	16	20	20	27	24	26	27	30	28	31	37
Coarse Sand, % (2.0-0.25mm.)	1	1	2	2	7	5	21	14	29	18	39	23
Fine Sand, % (0.25-0.05mm.)	6	5	10	5	22	12	27	19	24	19	21	17
Silt, % (0.05-0.005mm.)	59	52	52	51	55	51	40	50	40	42	34	39
Clay, % (less than 0.005mm.)	34	42	36	42	16	32	12	17	7	21	6	21

Specimens compacted to approximately 100 pcf. at 21 percent water and hydrated 7 days, one set pulverized, another set subjected to 60 cycles of alternate freezing and thawing, and the pulverized for test constants and grain size.

Soils 2c and 2d were gravelly plastic soils, fairly good base materials except that they contained considerable plastic clay (particularly Soil 2d) and were subject to softening when they became wet.

In a first group of tests, relatively low percentages of cement were added to Soils 2c and 2d, and test constants were determined for the modified materials after a hydration period of 2 days. The data presented in Table 15 show that the addition of cement greatly reduced the plasticity of the soils. Tests of this type were not made on Soil 2b, since it was nonplastic in its natural state.

TABLE 14

TEST CONSTANTS[a] AND BEARING RATIOS[b] OF CEMENT-MODIFIED SOIL

Soil 4e

% Cement by Volume % Cement by Weight	0 0	3 2.96	6 5.97	9 9.02
Liquid Limit	38	36	34	34
Plastic Limit	24	29	33	N. P.[c]
Plasticity Index	14	7	1	N. P.
Shrinkage Limit	20	21	26	29
Coarse Sand, % (2.0-0.25mm.)	1	1	4	6
Fine Sand, % (0.25-0.05mm.)	15	15	26	28
Silt, % (0.05-0.005mm.)	57	84[d]	70[d]	66[d]
Clay, % (less than 0.005mm.)	27			
Bearing Ratio	2	42	66	138
Bearing Ratio Specimens: Moisture Content, %	16	16	16	16
Density, pcf	98	95	96	99

[a] Test constants and grain size studies made on cement-modified soils hydrated seven days, then pulverized.

[b] Bearing ratio tests made after seven days hydration and four days immersion in water.

[c] Not plastic

[d] Silt and Clay combined

More important, however, than the reduction in plasticity of the soils, is the effect of the cement in increasing their strength. This was investigated by the bearing-ratio test on specimens containing various percentages of cement. Moisture-density tests were made using both the 4-inch-diameter mold (according to AASHO T 134 except that the material retained on a No. 4 sieve was included in the sample) and the 6-inch-diameter mold using equivalent compactive effort. Data obtained with both molds are given in Table 16. Specimens 6 inches in diameter and 4.6 inches high for the bearing-ratio test were molded of mixtures of each of the three soils containing 1½, 3, 4½, 6, and 10 percent of cement by weight. These same specimens were used for soniscope tests. In addition, standard AASHO specimens (4 inches in diameter) were molded for the AASHO wet-dry and freeze-thaw tests.

The specimens for the bearing-ratio

TABLE 15

TEST CONSTANTS AND GRADATION OF GRANULAR SOILS AND TEST CONSTANTS OF CEMENT-MODIFIED SOILS

Soil No.	2b	2c					2d				
% Cem. by Vol. % Cem. by Wt.	0	0	1.9 1.5	3.9 3	5.8 4.5	7.7 6	0	1.9 1.5	3.9 3	5.5 4.5	7.3 6
Liquid Limit	N.P.[a]	23	30	29	29	29	28	34	34	32	33
Plastic Limit	N.P.	13	22	22	24	28	13	25	28	29	32
Plasticity Index	N.P.	10	8	7	5	1	15	9	6	3	1
Plus No. 4 Gravel	25	20					15				
Coarse Sand, % (No. 4-0.25mm.)	67	54					43				
Fine Sand, % (0.25-0.05mm.)	6	7					8				
Silt, % (0.05-0.005mm.)	1	11					16				
Clay, % (less than 0.005)	1	8					18				

[a] Not Plastic

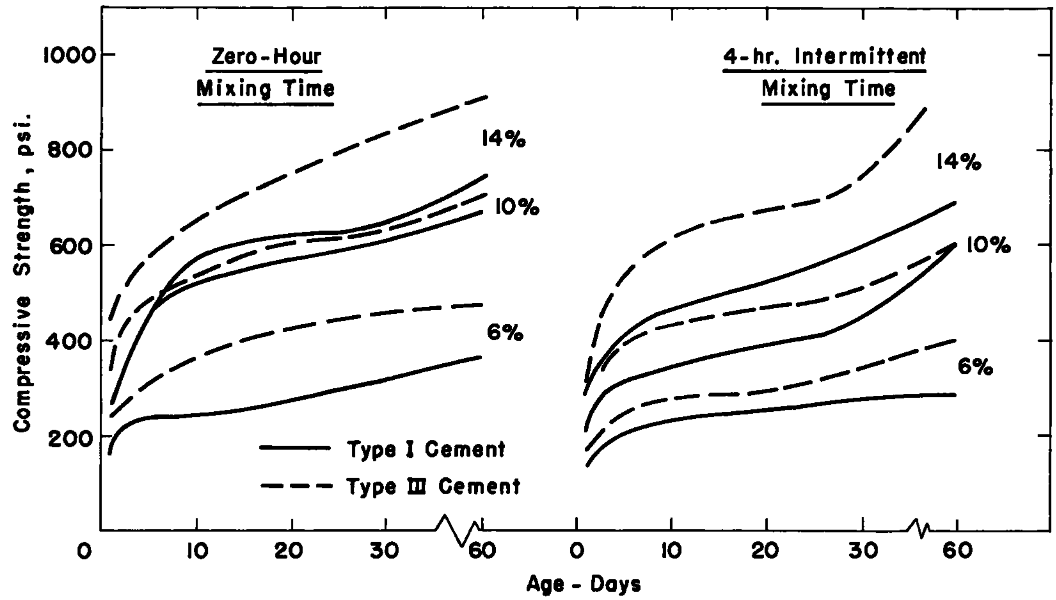


Figure 21. Comparison of compressive strengths obtained with normal (Type 1) cement and high-early-strength (Type 3) cement: Soil 4d.

and soniscope tests were retained in the molds and placed in the moist room. Several specimens of each cement content were molded so that they could be tested after various ages in the moist room and also after various cycles of alternate freezing and thawing. Thus, the tests permitted a study of the effect of cement content and time of curing on the bearing ratio and pulse velocity and, also, a study of the deteriorating effects of freezing and thawing. The bearing-ratio and soniscope specimens for alternate freezing and thawing were cured 21 days in the moist room before the test was started. The freeze-thaw cycle consisted of a period of 16 hours in the freezer at -20 F., followed by a period of 8 hours in the moist room at 73 F. During the test, the specimens were placed on their sides.

TABLE 16
MOISTURE-DENSITY RELATIONSHIPS

Soil No.	4-in. Diam. Mold						6-in. Diam. Mold					
	Cement Content by Weight, %											
	0		3		6		0		3		6	
	D[a]	O[b]	D	O	D	O	D	O	D	O	D	O
2b	133	9.5	132	10	132	10	133	8.5	133	9	134	8.5
2c	130	10	128	10.5	127	10.5	132	9.5	131	10	131	10
2d	126	11	123	12	122	12.5	127	10.5	126	11	124	11.5

[a] D = Maximum Density, pcf.
[b] O = Optimum Moisture Content, %

The cement contents used in this series

TABLE 17
BEARING RATIOS AT DIFFERENT AGES AND AFTER FREEZING AND THAWING
Soil 2b

% Cement by Wt.	As Molded Unsoaked	After Moist Room Cure, Days					After F-T, Cycles		
		7	21	37	87	161	12	48	100
0	36	38	40	41	51		17		
1 1/2	32	164	265	299	336	493	72	24	
3	67	471	578	702	741	806	435	464	463
4 1/2	50	595	788	809	1029	1108	752	743	755
6	40	788	993	1098	1186	1633	963	941	1091
10					1930	4270		1565	1780

overlap those required to produce soil-cement mix according to the criteria accepted for use with AASHO methods T 135 and T 136, the wet-dry test and the freeze-thaw test.

Based upon these tests, the following minimum cement quantities are required with the soils to produce "soil-cement": Soil 2b, 3 percent by weight (4.2 percent by volume); Soil 2c, 4.5 percent by weight (5.8 percent by volume); and Soil 2d, 6 percent by weight (7.3 percent by volume). These cement factors may be kept in mind to differentiate between so-called cement-modified soil and soil-cement.

The bearing-ratio and pulse-velocity data are presented in Tables 17 to 22. Included are the bearing-ratio values for specimens after periods of 7, 21, 37, 87, and 161 days in the moist room and after 12, 48, and 100 cycles of alternate freezeing and thawing. The moist room specimens at ages 37, 87, and 161 days are the same age as the freeze-thaw specimens after 12, 48, and 100 cycles of test.

Soniscope (pulse-velocity) values were obtained for specimens after curing for periods up to 87 days in the moist room and after as many as 48 cycles of freezing and thawing. Some of the specimens with low cement contents deteriorated fairly rapidly, and soniscope readings could not be made during the freeze-thaw test; in several cases the 100-cycle bearing-ratio tests could not be made.

Table 17 gives the bearing-ratio data for Soil 2b. Here it will be seen that the cement-treated soil mixtures, even with relatively low cement contents, had very high values. For instance, the mixture containing 3 percent of cement had a bearing ratio of 471 at age 7 days, and 806 at 161 days. Other mixtures had ratios considerably over 1,000. In making some of these tests, the loading piston was forced into the specimen to a depth of only 0.05 or 0.075 inch. Values in this high range have little significance, other than as indicators of relative hardness and resistance to penetration. Freezing and thawing reduced the bearing ratios for the mixture containing 1½ percent of cement; but mixtures containing 3 percent or more of cement showed no deterioration during the freeze-thaw test, although their rate of strength gain was less than that of specimens continously moist cured.

TABLE 18
PULSE VELOCITIES AT DIFFERENT AGES AND AFTER FREEZING AND THAWING
Soil 2b

% Cement by Wt.	Velocity Through Specimen-100 ft/sec									
	After Moist Room Cure, Days						After F-T, Cycles			
	7	21	37	53	71	87	12	24	36	48
1 1/2	65	73	77	79	83	82				
3	82	91	98	100	102	100	87	89	89	88
4 1/2	96	105	115	115	120	118	104	104	105	104
6	107	116	120	124	125	125	113	116	114	113
10	119	127	129	130	136	135	123	128	129	128

The soniscope data for Soil 2b are presented in Table 18. The pulse velocities did not decrease significantly during the freeze-thaw test, again showing good resistance to deterioration. As might be expected, the pulse velocities for soil-cement mixtures of Soil 2b increased with increased cement content and with time of moist curing. Velocities of more than 10,000 ft. per sec. were common.

Bearing-ratio and soniscope data for Soil 2c are presented in Tables 19 and 20. This soil required higher cement contents than Soil 2b to achieve the same degree of hardness as indicated by bearing-ratio

TABLE 19
BEARING RATIOS AT DIFFERENT AGES AND AFTER FREEZING AND THAWING
Soil 2c

% Cement by Wt.	As Molded Unsoaked	After Moist Room Cure, Days					After F-T, Cycles		
		7	21	37	87	161	12	48	100
0	11	5	3	6	5		3	2	
1 1/2	10	148	230	233	245	378	67	18	
3	15	255	302	316	414	727	131	34	
4 1/2	15	326	387	453	564	901	323	293	358
6	14	459	518	579	742	1048	418	499	507
10					1317	1361		935	829

TABLE 20
PULSE VELOCITIES AT DIFFERENT AGES AND AFTER FREEZING AND THAWING
Soil 2c

% Cement by Wt.	Velocity Through Specimen - 100 ft/sec									
	After Moist Room Cure, Days						After F-T, Cycles			
	7	21	37	53	71	87	12	24	36	48
1 1/2	69	76	85	86	89	86				
3	78	84	90	92	98	96	55			
4 1/2	85	88	97	98	102	102	74	80	82	82
6	87	93	100	102	104	105	87	87	91	91
10	97	101	105	107	111	110	98	97	102	103

and soniscope tests. The data show, however, that cement quantities of 4½ percent by weight or more produced soil-cement mixtures that had good resistance to freezing and thawing. For instance, the 4½-percent mixture showed a bearing-ratio of 323 after 12 cycles of test and 358 after 100 cycles, and pulse velocities for this same mixture were 7,400 ft. per sec. after 12 cycles, and 8,200 ft. per sec. after 48 cycles.

TABLE 21
BEARING RATIOS AT DIFFERENT AGES AND AFTER FREEZING AND THAWING
Soil 2d

% Cement by Wt.	As Molded Unsoaked	After Moist Room Cure, Days					After F-T, Cycles		
		7	21	37	87	161	12	48	100
0	10	7	7	5	6		2	2	
1 1/2	18	94	109	129	128	140	17	7	
3	19	171	240	268	320	396	28	15	
4 1/2	19	226	295	299	355	584	93	31	
6	19	280	367	388	438	738	198	31	
10					709	1144		499	517

Corresponding data for soil 2d are shown in Tables 21 and 22. This soil required more cement than Soil 2c to resist deterioration in the freeze-thaw test. A cement content of 6 percent by weight produced a mixture having good bearing ratios for continuous moist curing, but this same mixture in the freeze-thaw test showed a loss both in bearing ratio and in soniscope velocity. The soil-cement mixture with 10 percent of cement went through 100 cycles of test without apparent deterioration as indicated by bearing-ratio and soniscope tests.

The test data show that relatively small quantities of cement notably increased the overall strength characteristics and resistance to softening from freeze-thaw action of the three substandard granular base materials. Although much value was obtained from cement contents that might be said to produce cement-modified soil, considerably more value was obtained from the slightly greater cement contents required to produce soil-cement mixtures. Both the bearing-ratio and the soniscope tests appeared to be effective in measuring the relative deteriorating effects of alternate freezing and thawing.

SUMMARY AND CONCLUSIONS

The data presented in this report are intended to be helpful to engineers in developing an understanding of many of the physical properties of soil-cement mixtures. In some instances, more mixtures have been tested under each series than were reported in this paper. The data from the other tests corroborate the data presented here. Nevertheless, it is possible that the performance of some may not follow the patterns established here, because of different chemical and physical properties of the soils involved. The comments which follow summarize the information developed from these tests.

TABLE 22
PULSE VELOCITIES AT DIFFERENT AGES AND AFTER FREEZING AND THAWING
Soil 2d

% Cement by Wt.	Velocity Through Specimen - 100 ft/sec									
	After Moist Room Cure, Days						After F-T, Cycles			
	7	21	37	53	71	87	12	24	36	48
1 1/2	66	67	68	68	68	69				
3	77	79	80	80	82	85				
4 1/2	79	81	81	83	88	88	55			
6	82	82	81	84	88	89	70	67	27	
10	89	95	91	96	98	99	85	89	96	94

Series 1 and 2: Moisture and Density

The moisture content of a soil-cement mixture and the density to which the material is compacted have a great influence upon the quality of the product after cement hydration.

For general soil-cement-paving construction, the most-effective moisture content is approximately equal to or slightly above the optimum moisture content indicated by AASHO method T 134; moisture contents below the optimum may produce silty and clayey mixtures of inferior quality. The quality improves as the density increases; thus, mixtures at proper moisture content should be compacted to the highest practical density, at least equal to that obtained by AASHO method T 134.

For special soil-cement construction where very-heavy compactive effort is available, the quality of the mixture may be improved by lowering the moisture content and greatly increasing the density. However, to take advantage of this, special laboratory tests are necessary to establish moisture and density control limits, and careful checks must be made in the field to insure that these control limits are closely maintained.

Series 3: Prolonged Mixing Periods

Prolonged periods of damp mixing impaired somewhat the quality of the mixture. However, with most soils, mixing periods as long as 4 hours were not seriously harmful, provided the mixture was intermittently mixed several times an hour and provided the moisture content at time of compaction was at or slightly above the optimum prevailing at that time. The optimum moisture content generally increases as the length of mixing time increases; therefore, moisture-density tests are necessary toward the end of the damp-mixing period to determine the new optimum prevailing at that time.

Series 4: Clay Balls in the Mix

The quality of silty and clayey soil-cement mixtures was highest when 100 percent of the soil (exclusive of gravel, stones, etc.) was pulverized to pass a No. 4 sieve. However, the quality was not seriously affected by the presence of as much as 30 percent of unpulverized soil, provided the lumps of soil were damp at the time the soil-cement mixture (at optimum moisture content or slightly above) was compacted. If the soil lumps were dry at the time they were compacted in the soil-cement, the quality of the material was seriously impaired.

Series 5: Air-Entraining Cement

Soil-cement mixtures containing air-entraining (Type IA) cement behaved approximately the same as those containing Type I cement. Moisture-density relations, compressive strengths, and wet-dry and freeze-thaw test results were sufficiently similar to show that the cements may be used interchangeably in soil-cement construction.

Series 6: High Cement Contents

The compressive strength and resistance to wetting and drying and freezing and thawing increased as the cement content was increased. Good quality mix was obtained with cement contents generally in the range of 8 percent to 14 percent (depending upon the soil). However, mixtures having unusually high compressive strength (2,000 to 4,000 psi.) and excellent resistance to alternate freezing and thawing and wetting and drying were obtained with relatively high cement contents of, say, 22 percent to 30 percent, by volume.

Series 7: High-Early-Strength Cement

High-early-strength (Type III) cement hardened soil-cement mix at a faster rate than Type I cement. Prolonged mixing periods of several hours did not seriously impair the quality of mix containing Type III cement. Thus, for special construction where high early strength is required, Type III cement may be used.

Series 8: Cement-Modified Fine Grain Soils

The addition of cement in quantities less than those required to produce hardened soil-cement mix reduced the plasticity of clayey soils, thus reducing their tendency to shrink and swell excessively.

Series 9: Cement-Modified Granular Soils versus Granular Soil-Cement

The addition of relatively small quantities of cement to substandard granular materials reduced their plasticity and increased their strength. The addition of slightly greater quantities of cement (producing soil-cement) materially increased strength above those of the cement-modified soils and resulted in materials having considerable resistance to deterioration from alternate freezing and thawing. In this study, bearing-ratio and soniscope tests were effective in measuring the progressive deterioration of specimens in the freezing-and-thawing test.

Acknowledgment

The data reported were accumulated at the Portland Cement Association Laboratories over a period of more than 15 years. Consequently, the author is indebted to a number of engineers who assisted in conducting the test. Much of the work was done by E. G. Robbins and M. S. Abrams. Miles D. Catton, now assistant to the vice president for research and development, was in administrative charge of most of the research and development work on soil-cement mix. The paper was reviewed by Douglas McHenry, director of development, whose comments are gratefully acknowledged.

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Stabilization of Tennessee Gravel and Chert Bases

E.A. WHITEHURST, Director,
Tennessee Highway Research Program, University of Tennessee

This paper reviews laboratory investigations of stabilization of base materials. Most of the tests have been performed on four materials, two cherts and two gravels, in combination with some or all of the following admixtures: portland cement, cutback asphalt (RC-2), emulsified asphalt, road tar (RT-5), hydrated lime, lime and flyash, calcium acrylate, and calcium chloride.

The report presents information in two categories: (1) comparison of the effectiveness of the several admixtures in stabilizing the soils included in the study and (2) comparison of the value of a number of possible methods of test in differentiating between the suitability of the materials, with or without admixtures, for use in base construction.

It is concluded that any of the materials tested may be adequately stabilized by the addition of small percentages of portland cement. Quantities in excess of 4 percent by weight are not recommended. It is also shown that some of the soils may be stabilized to advantage with certain types of asphalt.

The report also concludes that of the various tests employed throughout the study, the triaxial-compression test appears to be the most promising in evaluating the usefulness of coarse granular materials. Attention is directed to a procedure which may evolve into a satisfactory design method. It is pointed out, however, that the full effectiveness of the test and data collected cannot be utilized until these laboratory investigations are supplemented by equally extensive field investigations in the nature of test sections. The construction of such sections is contemplated in the near future.

● FOR nearly 3 years the Tennessee Highway Research Program, a cooperative organization of the Tennessee Department of Highways and Public Works and the University of Tennessee, has been actively engaged in a study of the stabilization of gravel and chert bases. This study was first suggested because there are large areas of Tennessee which do not have available sufficient quantities of good base material. It was hoped that some means might be developed for stabilizing such poorer materials as might be locally available.

As the study has progressed it has more or less divided itself into two phases. One is a study of the changes which occur in the quality of a base material when it is combined with one or more of a number or admixtures. The other is a search for a method of test which adequately defines those properties of the material which determine its usefulness in base construction.

After some preliminary tests on a number of small samples, four soils were selected for use in this study. Two were cherts obtained from Benton County and two were bank gravels obtained from western Tennessee. On the basis of the original samples tested it was expected that one chert and one gravel would be classified as good base materials and one of each classified as a poor base material. When the larger quantities of these materials were obtained, however, approximately 5 tons of each being brought into the laboratory, it was found that there was relatively little difference between the two cherts. These have been identified as Soils 10 and 11, with the poorer chert designated as Soil 10. The two gravels have been identified as Soils 12 and 13 with the poor gravel designated as Soil 12.

The principal properties of the four soils are indicated by the test values shown in Table 1. The grain-size distribution of the two cherts (Samples 10 and 11) and of the two gravels (Samples 12 and 13) are indicated in Figures 1 and 2.

In general, the attempts to stabilize these soils have been made by adding some more or less commercially available admixture to the soils tested. Admixtures included to date have been portland cement, cutback asphalt (RC-2), emulsified asphalt, road tar (RT-5), hydrated lime, lime and flyash, calcium acrylate, and calcium chloride.

All of the admixtures mentioned above, with the exceptions of the lime and flyash and the calcium acrylate, are well known and readily available materials which are widely used in road construction. The two exceptions were included in this study because it was felt that they held special promise. The former is a waste material of lime production. It contains about 47 percent $Ca(OH)_2$, 26 percent $Ca(CO)_3$, and something over 13 percent finely divided carbon. Since combinations of lime and flyash have been successfully used for stabilization of certain types of bases and subgrades in the past, it was felt that this material should be investigated. Its nature, that of a waste product, is such that it would be extremely cheap if it were satisfactory for the use desired.

The second exception, calcium acrylate, is an experimental material produced by the Rohm and Haas Company in Philadelphia, Pennsylvania. It has been and is being extensively investigated by other organizations, notably the Massachusetts Institute of Technology, on behalf of the armed forces. It has been reported to impart to fine-grained soils such properties as tensile strength, flexibility, and resilience. It is rather expensive at the present time, but should it prove satisfactory for base stabilization and its use increase, mass production would doubtless lower the unit cost considerably. It was felt that some of the unusual properties attributed to this material warranted its inclusion in this study.

Several different tests were employed

TABLE I

PHYSICAL PROPERTIES OF SOILS NOS. 10, 11, 12 and 13

	Soil No 10	Soil No 11	Soil No 12	Soil No 13
Liquid Limit (%)	24.6	23 3	32 0	19 8
Plastic Limit (%)	17 2	NP	19 5	NP
Plasticity Index (%)	7 4	NP	12 5	NP
Shrinkage Limit (%)	18 1	20 5	15 6	20 2
Specific Gravity, — No 4 Fraction	2 62	2 63	2 68	2 68
Optimum Moisture Content, — No.4 Fraction (%) (Standard Proctor)	13 4	11 0	11 0	15.0
Maximum Density, — No 4 Fraction (lb/cu. ft.)	116	117	124	109
Absorption, + No. 4 Fraction (%)	4 0	5 6	3 6	4.7

TABLE 2

SUMMARY OF TEST DATA, SOIL 10

		CBR					W & D		F & T		Comp. Str. (psi)	
		After Cure[a]	After Immersion (days)				Cycles Completed	Loss in weight %	Cycles Completed	Loss in weight %	After Cure	After F&T
Admixture	%		2	7	14	28						
None		44	50				0	—	2(max.)	10	61(avg.)	0-10
	1	95					12	15	7	41	179	15
(1)Portland	2	138		148	156	136	12	8	12	52	235	—
Cement	4	225					12	3	12	11	263	402
	1	63					12	14	2	39	68	—
(2)Lime	2	66		36	42	61	12	11	2	23	52	—
	4	70					12	9	0	—	60	41
	1	60					0	—	2	16	89	—
(3)Lime-	2	59		61	61	50	0	—	1	7	—	—
flyash	4	65					0	—	1	11	112	5
	1						0	—	1	8	58	25
(4)Calcium	2						2	13	4	12	113	—
Acrylate	4						4	10	12	10	204	76
	1/2	135	43				0	—	12	41	208	10
(5)Emulsi-	1	135	33				12	4	12	8	201	47
fied Asphalt	2	151	18				12	4	12	2	226	42
	1/2	140	75				12	8	8	5	163	15
(6)Cutback	1	100	76				12	5	12	6	257	17
Asph. (RC-2)	2	85	75				12	6	12	2	178	66
	1/2	130	68				12	10	7	43	158	—
(7)Road Tar	1	112	63				12	11	9	56	139	62
(RT-5)	2	98	70				12	16	12	25	106	12

[a] Cure for no admixture - 5 days at 73 F. and 100% RH
Cure for admixtures 1, 2, and 3 - 5 days at 73 F. and 100% RH plus 2 days immersed
Cure for admixtures 5, 6, and 7 - Storage at 140 F. to constant weight

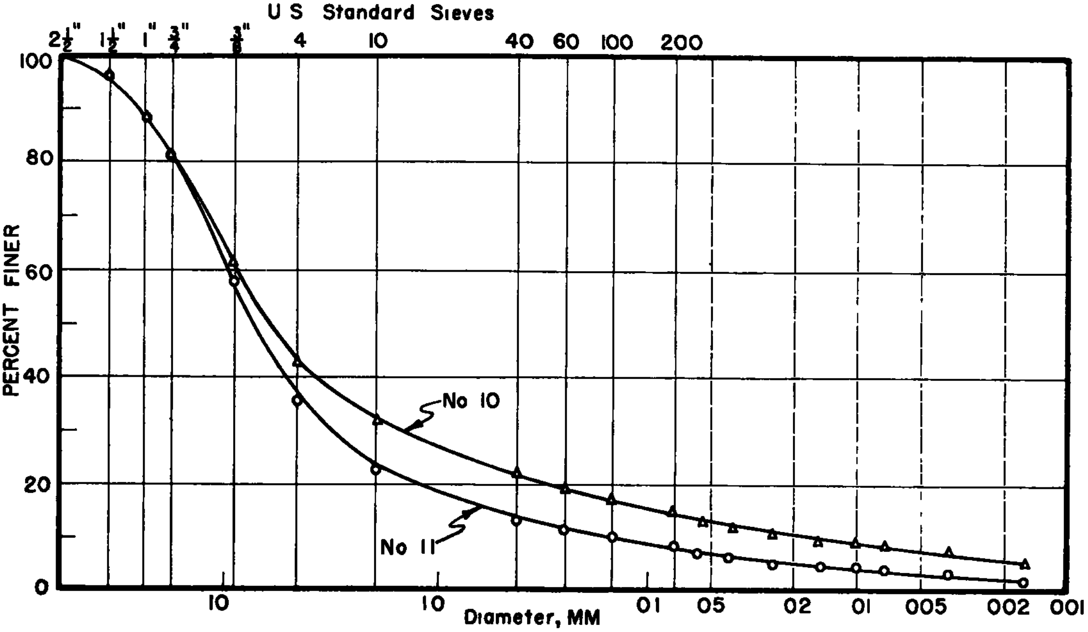

Figure 1. Grain-size distribution for Soils 10 and 11.

to determine the efficiency of the various admixtures when combined with the four soils. During the early stages of the investigation these included the California Bearing Ratio test, wetting-and-drying and freezing-and-thawing tests, and unconfined-compressive-strength tests. CBR tests were performed at the end of a suitable curing period for each specimen and after various lengths of immersion in water. Wetting-and-drying and freezing-and-thawing tests were performed in accordance with ASTM and AASHO specifications for soil-cement testing. Unconfined-compressive-strength tests were made on Proctor specimens at the end of the curing period and after wetting and drying or freezing and thawing. These later tests were made

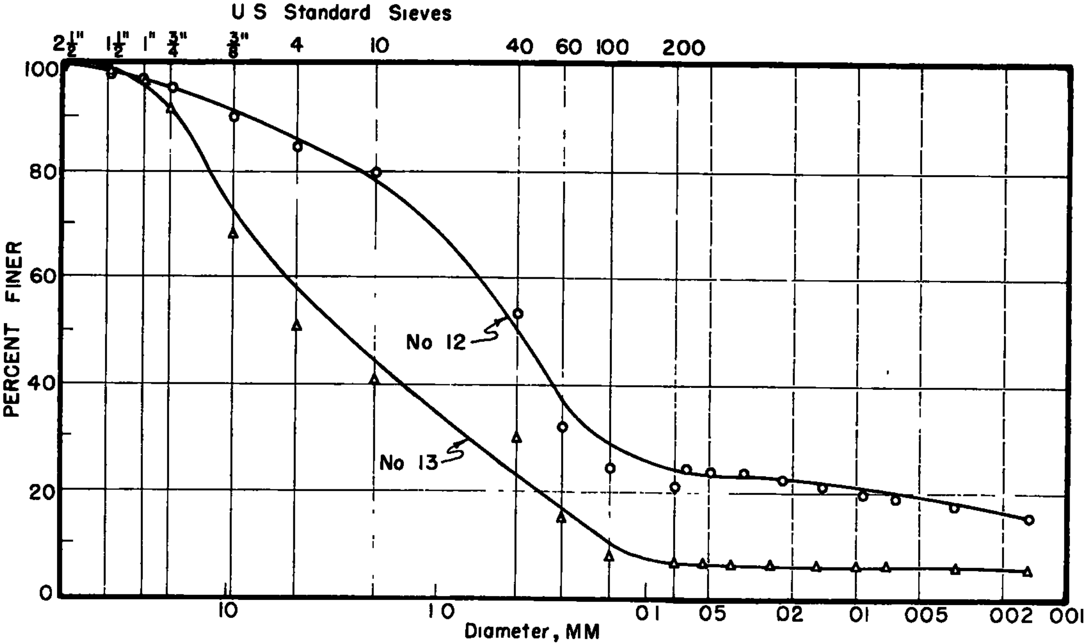

Figure 2. Grain-size distribution for Soils 12 and 13.

TABLE 3
SUMMARY OF TEST DATA, SOIL 11

		W & D		F & T		Comp. Str. (psi)	
Admixture	%	Cycles Completed	Loss in weight, %	Cycles Completed	Loss in weight, %	After Cure	After F & T
		0	–	0	–	21(avg.)	14(avg.)
(1)Portland Cement	1	12	8	12	66	63	–
	2	12	6	12	12	140	–
	4	12	3	12	3	252	388
(2)Lime	1	12	19	1	14	37	8
	2	12	11	1	10	17	11
	4	12	8	1	20	27	–
(3)Emulsified Asphalt	1/2	12	16	12	30	185	–
	1	12	13	12	36	194	63
	2	12	8	12	10	232	103
(4)Cutback Asphalt (RC-2)	1/2	12	29	7	45	284	–
	1	12	30	12	19	205	–
	2	12	15	12	10	143	86
(5)Road Tar (RT-5)	1/2	7	40	6	36	136	–
	1	6	38	6	48	122	–
	2	6	29	12	38	56	37

either after the specimen had undergone a full 12 cycles of the weathering test or at the end of the cycle in which a companion brushed specimen was observed to fail.

The results of these strength tests on specimens at the end of wetting and drying are not conclusive, in as much as the wetting-and-drying cycle ends with the specimens in an over-dried condition. In this condition all specimens, regardless of type or amount of admixture, showed quite high strengths. The similar tests performed at the end of a cycle of freezing and thawing, were quite informative, however, since the specimen in a thawed condition had a high moisture content and represented

TABLE 4
SUMMARY OF TEST DATA, SOIL 12

		CBR					W & D		F & T		Comp. Str. (psi)	
		After Cure[a]	After Immersion (days)				Cycles Completed	Loss in weight %	Cycles Completed	Loss in weight %	After Cure	After F & T
Admixture	%		2	7	14	28						
None							0	–	3(max.)	2	20(avg.)	0-6
(1)Portland Cement	1	143					12	40	12	51	58	21
	2	181		270	232	230	12	18	12	8	125	81
	4	377					12	5	12	4	216	258
(2)Lime	1	75					2	9	6	48	37	2
	2	68		69	81	118	11	9	9	32	47	–
	4	68					5	2	12	46	55	–
(3)Lime-flyash	1						0	–	1	2	24	7
	2						3	59	2	5	–	7
	4						7	83	12	54	65	10
(4)Calcium Acrylate	1						0	–	2	21	20	6
	2						2	6	12	18	46	11
	4						1	2	12	6	75	51
(5)Emulsified Asphalt	1/2	86	27				0	–	3	8	92	–
	1	69	21				3	4	4	21	122	–
	2	42	18				12	27	12	35	93	31
Cutback (6)Asphalt (RC-2)	1/2	80	57				6	18	5	17	68	–
	1	48	57				1	7	8	43	76	–
	2	28	18				2	1	12	20	65	10
(7)Road Tar (RT-5)	1/2	77	28				0	–	1	26	88	–
	1	56	15				6	69	8	52	103	–
	2	28	20				12	58	12	48	60	18

[a] Cure for no admixture - 5 days at 73F and 100% RH
Cure for admixtures 1 and 2 - 5 days at 73F and 100% RH plus 2 days immersed
Cure for admixtures 5, 6 and 7 - Storage at 140F to constant weight

probably the worst condition likely to occur in the field.

Results of the tests outlined above are summarized for the four soils in Tables 2, 3, 4, and 5, respectively. All admixtures except calcium chloride were included with Soils 10 and 12, while only portland cement, lime, emulsified asphalt, cutback asphalt, and road tar were added to Soils 11 and 13. Curing periods used for CBR specimens are indicated on the appropriate tables. For specimens subjected to the weathering tests, those containing portland cement, lime and flyash, or calcium acrylate were cured by storage for one week in a standard moist room at 73 F. and 100 percent relative humidity. Similar specimens containing emulsified or cutback asphalt or road tar were cured at 140 F. until they reached constant weight.

TABLE 5

SUMMARY OF TEST DATA, SOIL 13

		W & D		F & T		Comp. Str. (psi)	
Admixtures	%	Cycles Completed	Loss in weight, %	Cycles Completed	Loss in weight, %	After Cure	After F & T
None		0	–	0	–	3(avg.)	14(avg.)
(1)Portland Cement	1	12	19	12	19	75	39
	2	12	7	12	6	167	150
	4	12	3	12	2	335	348
(2)Lime	1	1	4	1	18	12	4
	2	12	44	1	19	17	–
	4	12	27	1	9	23	2
(3)Emulsified Asphalt	1/2	10	38	12	58	41	10
	1	12	39	12	46	51	14
	2	12	22	12	14	137	18
(4)Cutback Asphalt (RC-2)	1/2	12	44	11	44	58	–
	1	12	44	12	19	78	44
	2	12	36	12	13	130	56
(5)Road Tar (RT-5)	1/2	2	28	1	21	22	20
	1	2	39	1	22	11	4
	2	2	40	8	56	17	7

Examination of the four tables referred to above leads to the following observations:

1. All four soils, when stabilized with 4 percent portland cement by weight and moist cured for one week, develop high values of CBR which are not detrimentally affected by prolonged immersion, strongly resist the action of wetting and drying or freezing and thawing, and develop high unconfined compressive strengths which are not reduced by weathering of 12 cycles duration.

2. Emulsified and cutback (RC-2) asphalts are effective in reducing damage caused by weathering, comparable in this respect when used in quantities of 2 percent by weight with portland-cement stabilization. Compressive strengths of specimens stabilized with these materials, however, are considerably reduced after freezing and thawing, as are CBR values after immersion for relatively short periods. In many cases, however, the reduced values after immersion or freezing and thawing are greater than the original values of the soil alone.

3. Hydrated lime, lime and flyash, calcium acrylate and road tar appear to be inadequate as stabilizing agents for these soils. Limited exceptions may be noted from the tables.

The use of calcium chloride in this study was limited to its inclusion in specimens subjected to both types of weathering tests. It was used in quantities of 1/2, 1, 2, and 4 percent by weight and cured either by one week storage in a standard moist room or one week storage in air in the laboratory where the specimens were originally prepared. All specimens stabilized with calcium chloride acted very much as did the same soil when tested without stabilizing admixtures. Specimens subjected to wetting and drying failed immediately upon the first immersion, and those subjected to freezing and thawing fell apart during the first or second cycle. These results indicate that calcium chloride does not provide any stabilizing action of the nature sought in this investigation. This does not imply that calcium chloride, used as a construction aid, may not result in such improvements as greater and more-uniform densities which will be of lasting benefit to the base.

Perhaps the most-notable result outlined above is the marked increase in durability caused by the addition of cement to all of the soils tested. In an effort to determine whether this would apply to poorer materials, small samples of three additional cherts were obtained. The gradation and clay content of these materials was such that they would not be at all acceptable under present base specifications. Each of the three was stabilized by addition of 4 percent by weight of portland cement and subjected to both wetting-and-drying and freezing-and-thawing tests. Specimens of all three survived the 12 cycles of wetting and drying with weight losses of less than 12 percent. Similar specimens also survived 12 cycles of freezing and thawing. In two cases the losses in weight were less than 15 percent and in the third the loss was approximately 35 percent.

In some areas where cherts and gravels are largely nonexistent, or are of very inferior quality, considerable deposits of sand may be found. It was felt that some advantage, either in performance or economy or both, might accrue from blending these sands with such cherts and gravels as might be obtained. To check this hypothesis samples of two sands were obtained. One was a very fine sand from Madison County and the other a medium coarse concrete sand obtained from Knoxville. Specimens containing combinations of these sands with Soils 10 and 11, the two cherts used, throughout the study, were subjected to weathering tests. Results are shown in Table 6. The sands were designated as Soils 17 and 18, respectively. All specimens containing combinations of sand and chert were stabilized with portland cement.

Comparison of Table 5 and Tables 2 and 3 shows that additions of as much as 20 percent sand to either of the cherts appears to have little influence upon the durability of the materials when stabilized with portland cement. In some cases the accumulative weight losses are slightly greater and in others slightly less when the sand is included.

There is some reason to believe that none of the tests reported thus far are entirely satisfactory for determining the suitability of such materials as those studied in this project for use as base-course materials. As the size of the individual particles in the soil increases, the effect of the restraint of the steel wall of the CBR mold becomes ever more important. At the same time, the ratio of the size of the piston forced into the specimen to the size of an individual particle becomes smaller and the possibility of any one particular particle unduly influencing the test results becomes greater.

The durability tests outlined above appear to have merit in establishing the

TABLE 6

SUMMARY OF DURABILITY TESTS, COMBINATIONS OF CHERTS AND SANDS

		Wetting and Drying		Freezing and Thawing	
Soil	Portland Cement %	Cycles Completed before Failure	Loss in Weight During Indicated Cycles (%)	Cycles Completed before Failure	Loss in Weight During Indicated Cycles (%)
90% No. 10	1	12	22	7	52
10% No. 17	4	12	4	12	4
80% No. 10	1	12	37	12	65
20% No. 17	4	12	4	12	5
90% No. 11	0	0	—	0	—
10% No. 17	1	10	40	10	47
	2	12	15	12	20
	4	12	5	12	5
80% No. 11	0	0	—	0	—
20% No. 17	1	12	35	11	50
	2	12	10	12	14
	4	12	3	12	1
80% No. 11	0	0	—	0	—
20% No. 18	1	12	38	3	46
	2	12	14	12	30
	4	12	3	12	6
70% No. 11	0	0	—	0	—
30% No. 18	1	0	—	3	30
	2	12	16	12	32
	4	12	8	12	3

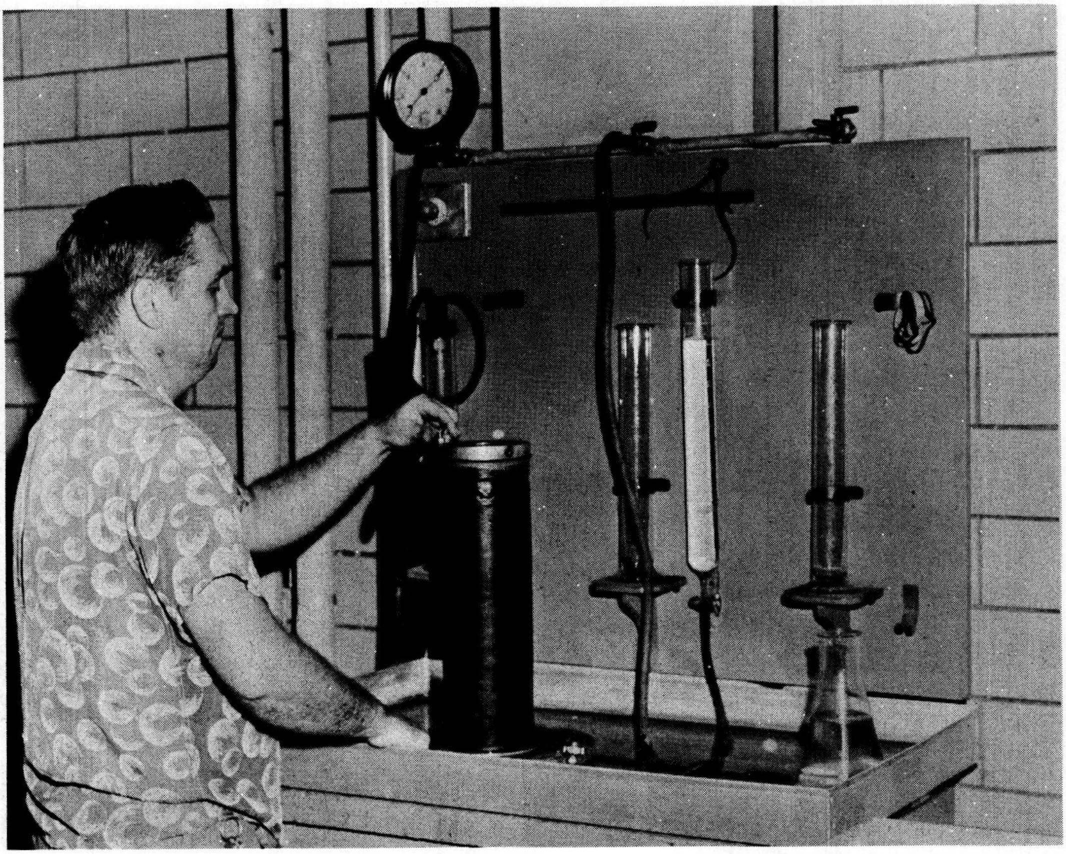

Figure 3. Saturating specimen for triaxial testing.

relative durability of the different soils or stabilized mixes. It may be argued, however, that they are unduly severe for materials used in this climate, since those having known good performance records failed rapidly under either type of exposure. Unconfined-compressive-strength tests made at the end of the curing periods appear to give some information concerning the relative efficiency of admixtures. Unfortunately, since all admixtures do not require the same type of curing conditions, specimens are not in comparable conditions at the time of test, some having a high moisture content and others being oven dry. In an effort to overcome these objections, a test was sought which would measure some basic property of the soil which in turn might indicate its suitability for base use.

The use of a triaxial-compression test was immediately suggested, since results of such tests may be interpreted to give the angle of internal friction and the cohesion of the material, these being the properties which are frequently assumed to control its load-carrying ability. Unfortunately, most existing commercially available triaxial machines were built for testing fine-grained materials such as sands, silts, and clays. The size of the specimen is usually quite small. Since this study was concerned with cherts and gravels, it was felt highly desirable that fractions at least as big as 3/4 inch be included in the test specimen.

After some investigation it was found that the Kansas State Highway Department had built and was using a device suitable for testing granular materials in triaxial compression. A set of drawings for this device was provided by that organization and the instrument was built in the shop of the engineering experiment station at the university.

Specimens used in this test are 5 inches in diameter and 14 inches high. They are molded in nine layers in a split mold, with

Figure 4. Testing specimen in triaxial-compression machine.

each layer receiving 39 blows from a standard Proctor hammer. The restraint bands holding the mold together are then removed, the mold separated, and the specimen lifted out. After suitable curing it is ready to be tested.

The triaxial-compression test chamber consists of four major components, a base plate, a specimen head, a retaining wall, and a chamber head. The chamber head is fitted with a bronze sleeve through which a load piston may be passed.

The specimen to be tested is placed on a pedestal in the center of the base plate. The top of this pedestal contains a porous stone disc having essentially the same diameter as the specimen, i. e. , 5 inches. A neoprene rubber sleeve with an inside diameter of 5 inches and a wall thickness of 0. 03 inch is then worked down over the specimen. This sleeve, 17 inches long, extends down over the base pedestal and up beyond the top of the specimen. The specimen head, containing another porous stone disc, is slipped into the upper end of the sleeve until it rests upon the top of the specimen. Rubber bands are used to keep the sleeve pressed tightly against both the base pedestal and the specimen head.

Except under unusual conditions, all specimens are saturated prior to test. A vacuum is applied to the cylinder head, drawing the air from the specimen and sucking the sleeve against its sides. Water is then admitted to the specimen through the porous stone disc in the base pedestal. This process is shown in Figure 3. The process is continued until 200 cc. of water have been drawn through the specimen. At that point, both the vacuum and water supply valves are closed simultaneously. The amount of water taken by the specimen and the amount of time required by this process are recorded.

The retaining wall, either a seamless steel or a plexiglass tube having an inside diameter of 8 inches, is lowered over the specimen and seated against a gasket in-set in the base plate. The chamber head is seated on top of the retaining wall and bolts connecting it with the base plate securely tightened. The entire assembly, containing the saturated specimen, is lifted into a compression machine and the load piston is lowered through the chamber head until it rests on the specimen head. The moving head of the compression machine is brought into contact with the load piston.

Compressed air at a regulated pressure of 10, 20, 30, or 40 psi. is applied to a reservoir of water, forcing the water into the test chamber where it surrounds the specimen, applying a uniform lateral pressure. A partially filled burette is connected to the valve through which the specimen was saturated and the valve is opened. Any change in volume of the specimen will be reflected in the level of the water in the burette. The specimen is then ready to be tested.

Load readings and readings of water level in the burette are recorded every 30 seconds. The tests are continued until the specimen has failed, as is indicated by progressive falling off of load. Figure 4 shows such a test in progress.

Four similar specimens are made from each soil or each combination of soil and stabilizing admixture. One is tested at each of the lateral pressures indicated above. In the earlier tests only three specimens were made and tested at lateral pressures of 10, 20 and 30 psi. From the data obtained, Mohr's circles of stress may be constructed and the angle of internal friction and cohesion of the material determined.

All four of the basic soils and combinations of these soils with sand were tested as outlined above with several admixtures. Results of these tests are summarized in Table 7. Examination of these data leads to the following observations:

1. The addition of 4 percent portland cement to any of the soils generally increased their angle of internal friction from 9 to 14 deg. and from 30 to 45 psi. Soil 13 is the only exception, with the angle of internal friction increased by only 2 deg. but the cohesion by 66 psi.

2. When specimens stabilized with asphalt are tested in triaxial compression, the asphalt appears to act as a lubricant, generally increasing the cohesion of the material but decreasing its angle of internal friction.

3. The blending of 20 percent of sand with either of the cherts appears to slightly improve the angle of internal friction when stabilized with 4 percent of cement. The cohesion is slightly decreased in some cases. The use of the sand is highly beneficial to Soil 10 when stabilized with emulsified asphalt, largely offsetting the decrease in the angle of internal friction due to the addition of 2 percent asphalt alone to Soil 10.

4. The effect of blending sand with other base materials appears to vary considerably from one material to another and from one admixture to another for any given base material. This suggests that any such blending should be carefully considered and tests performed on the actual materials involved prior to acceptance of the practice.

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TABLE 7
SUMMARY OF ALL TRIAXIAL TEST DATA
(All specimens cured 1 week and saturated prior to test)

	Type Admixture	%	Average angle of internal friction (deg)	Average cohesion (psi)
No. 10	None		43.5	9
	Cement	4	57	40
	Em. asph.	2	38	43
80% No. 10 20% No. 18	None		44	14
	Cement	4	58	53.5
	Em. asph.	2	43	36
No. 11	None		41.5	13.5
	Cement	4	55.5	57.5
	Em. asph.	2	36	53
	RC-2	2	42.5	25.5
	RT-5	5	45	14
80% No. 11 20% No. 18	Cement	4	57.5	47.5
	Em. asph.	2	54	13
	RC-2	2	39	26.5
No. 12	None		36.5	4.5
	Cement	4	51	38
	Em. asph.	2	32	9
	RC-2	2	33	1
80% No. 12 20% No. 18	None		38	10
	Cement	4	49	42.5
	RC-2	2	29	6
No. 13	None		41	6
	Cement	4	43	72.5
	RC-2	2	34	9
80% No. 13 20% No. 18	Cement	4	49.5	44
	RC-2	2	34	6.5

A careful analysis of the data presented in the foregoing shows that the various tests do not always rate the soils in the same order in determining their suitability for base use, as should be the case if all tests were equally valid in evaluating the properties of the various materials. For example, CBR tests performed on Soils 10, 11, and 12 without stabilizing admixtures indicated their usefulness, in decreasing order, to be Nos. 11, 10, and 12. Un-

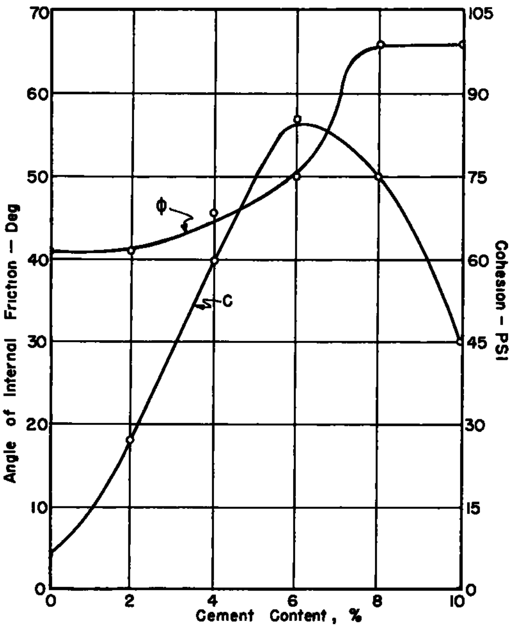

Figure 5. Effect of cement content on triaxial-test results.

confined compression tests performed on the same materials, and also on Soil 13, indicated the value of soils in decreasing order to be Nos. 10, 11, 12, and 13. Triaxial compression tests showed Soil 10 and 11 to be quite similar, followed in decreasing order by Soil 13 and Soil 12. Since specimens made from all soils failed immediately in the wetting-and-drying test and during early cycles in the freezing-and-thawing tests, it is felt that these tests do not effectively differentiate between the materials tested.

When the four basic materials were stabilized with 4 percent of portland cement similar differences between tests were observed. CBR tests on Soils 10, 11, and 12 at the end of the curing period indicated their value in decreasing order to be Soils 12, 11, and 10. Specimens made from all soils performed very well in both wetting-and-drying and freezing-and-thawing tests. When both tests are taken into account, what slight preference exists would indicate the relative value of the soils to be Nos. 13, 11, 12, and 10.

Unconfined-compressive-strength tests at the end of the curing period rated the soils as Nos. 13, 10, 11, and 12, while similar tests after 12 cycles of freezing and thawing showed their relative values to be Nos. 10, 11, 13, and 12. On the basis of the angle of internal friction, as determined from triaxial-compression tests, the soils would be rated Nos. 10, 11, 12, and 13, while on the basis of cohesion they would be rated Nos. 13, 11, 10, and 12. In using the triaxial-compression test for evaluation of such materials, however, it seems proper that both the angle of internal friction and the cohesion should be taken into account. If this is done the soils would probably be rated Nos. 11, 10, 13, and 12.

The tests appear to be in greater conformity with respect to indicating the relative effects of admixtures upon the soils that in differentiating between the soils themselves. For example, CBR tests on Soil 10 indicated the addition of 1/2 percent of emulsified asphalt to be more beneficial than the addition of 1 percent of portland cement, 1 percent of emulsified asphalt to be slightly superior to 2 percent of portland cement, but 2 percent of asphalt to be considerably inferior to 4 percent of portland cement. Results of unconfined-compressive-strength tests on similar combinations showed 1/2 percent of emulsified asphalt to be superior to 1 percent of portland cement, 1 percent of emulsified asphalt to be slightly inferior to 2 percent of portland cement, but 2 percent of asphalt to be notably inferior to 4 percent of portland cement. Results of durability tests on Soil 10 indicate a similar order.

The most-difficult of the tests to perform is the triaxial-compression test. Greater care is required in the preparation of the specimen and in its testing. It is believed, however, that this test is highly representative of conditions existing in a highway base and that, providing information concerning two of the fundamental properties of the base material, its angle of internal friction and its cohesion, it very probably offers more usable information concerning the suitability of the material than do any of the other tests.

Toward the end of the testing program, a limited series of triaxial-compression tests was performed to investigate the effect of percentages of admixture varying over a wider range than that included through the major portion of the study. The results of these tests were extremely interesting and perhaps indicate the possibility of using a technique differing from those employed in the past in determining the optimum percentage of an admixture for use in stabilization.

In the first series of tests, one of the gravels was stabilized with portland cement in quantities of 2, 4, 6, 8 and 10 percent by weight. Each combination was tested in triaxial compression and its angle of internal friction and cohesion determined. The values have been plotted in Figure 5. Lateral pressures of 20, 40, and 60 psi. were used in these tests. It may be observed that the cohesion of the mixture increased rapidly to a maximum at a cement content of 6 percent and decreased beyond that point, apparently indicating that the material had become brittle. The angle of internal friction increased more slowly at low cement factors and appeared to reach a maximum at 8 percent cement content, showing no increase when the cement factor was raised to 10 percent. In supplemental tests where lateral pressures were limited to the 10-, 20-, 30- and 40-psi. levels used in the remainder of the study, the maximum value of angle of friction was reached at, or slightly beyond, 6 percent of cement. This appears to indicate that the stabilized gravel had reached a strength at which the material was no longer influenced by the restraining pressures chosen.

It is generally agreed that road or base materials, with the exception of very rigid ones, such as portland-cement concrete, gain much of their load-supporting-and-distributing ability from the restraint exercised on material immediately beneath the loaded area by adjacent material. It was demonstrated in the early phases of the study that a cylinder of this soil alone, or even in combination with cement, had relatively low ultimate strength when tested in unconfined compression. Under the influence of lateral restraint, however, these ultimate strengths were much increased. The data shown in Figure 5 indicate that if the lateral restraint which a material is capable of developing is known, or can be satisfactorily determined, triaxial tests such as those described above with lateral pressures limited in magnitude to the estimated restraint on the material in place will give a clear indication of the maximum percentage of admixture which may be beneficially employed. In the case of the gravel used in these tests, if the maximum lateral pressure which may be developed in such a base is assumed to be 60 psi., Figure 5 clearly demonstrates that little can be obtained by the use of cement contents in excess of 8 percent. In view of the performance of the cohesion, it seems likely that a cement content in the order of 6 percent by weight would be more nearly optimum.

A similar series of tests were performed on the same material using a cutback asphalt at residual asphalt contents of 1, 2, and 4 percent. Results shown in Figure 6 indicate that the asphalt was acting somewhat as a lubricant with the cohesion of the mixture increasing and the angle of internal friction decreasing as the percentage of asphalt was increased.

Although this project was undertaken with the hope of developing techniques for the construction of low-cost roads from local base materials, it is felt that the results may have considerable significance with respect to construction and reconstruction on the primary road system. With vehicle weights continually increasing, and no apparent end to this trend in sight, it becomes evident that our primary highways will have to be built to standards not previously considered. In many areas, aside from economic considerations, the availability of materials definitely limits the extent to which the thickness of base courses can be increased. The notable improvement in the qualities of the materials included in this study which is brought about by the addition of small percentages of some of the admixtures suggests that the use of such stabilization practices might become a standard prac-

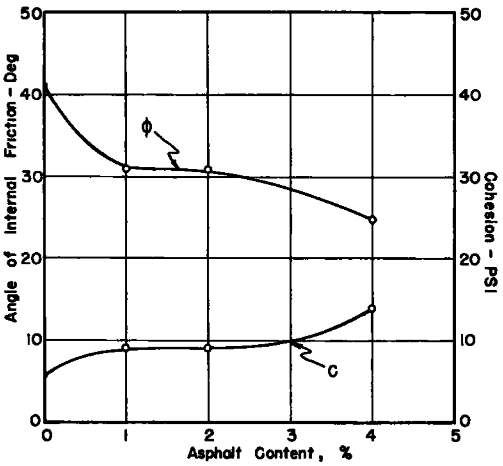

Figure 6. Effect of asphalt content on triaxial-test results.

tice in primary road construction. If the material can be batched and mixed at a central plant, before application to the road bed, there is every reason to believe that small percentages of admixture may be economically and beneficially employed.

It is appreciated that the data reported have all resulted from laboratory investigations. Although it is believed that the angle of internal friction and the cohesion of a material are good measures of that material's load-bearing ability and that changes in these factors may appropriately be used to determine changes in the load-carrying capacity of the material, it must be acknowledged that data presently available are not sufficient to indicate what magnitude of angle of internal friction and cohesion are required in any specific instance. Such data must be collected through actual field experimentation. In this regard, it is anticipated that the Tennessee Department of Highways and Public Works will construct several sections of stabilized base in the near future for the purpose of providing correlation between results of laboratory investigations and actual performance under traffic in the field. In this way it is hoped that a suitable method of highway base design may be evolved.

Discussion

RICHARD H. MILLER, Assistant Professor of Civil Engineering, Clarkson College of Technology — The use of lime and flyash shows very-low values of compressive strength and low resistance to cycles of freezing and thawing. The curing period for these samples consisted of 5 days at 73 F. and 100 percent relative humidity plus 2 days water immersion. It is widely recognized that compositions employing lime do not set up as do hydraulic cements but respond to cycles of high-low humidity as well as presence of CO_2, which slowly forms a recrystallized hydrate and calcium carbonate matrix. It is felt that the tests as presented in this paper might not be indicative of results obtained in field operations using materials such as described in this paper. The curing of the compositions are brought about under entirely different conditions.

The lime-and-flyash mixtures utilize the lime-hardening reaction but, in addition, would be expected to produce a hydraulic set as a result of the pozzolanic reaction between the lime and flyash. This reaction, however, would proceed slowly; if a 7-day cure is contemplated in laboratory testing, it would be necessary to accelerate the reactions, preferably by the use of higher temperatures during the curing period.

This subject has been reported upon in earlier papers[1] presented to the Highway Research Board and reference is also made to Whitehurst's previous paper "Durability Tests on Lime-Stabilized Soils".[2]

Tables of the subject paper show percentages of lime and flyash used of 1, 2, and 4 percent. It is not stated whether these percentages are of lime, flyash, or of a combination of lime and flyash. If the percentages as stated are the total percentages of the combination of lime and flyash used in the mixtures, as they appear to be; then the amounts are far too small and it is not surprising that the results were poor. It is necessary generally to add to the soil 5 percent of lime and 10 to 15 percent of flyash in order to derive a beneficial stabilizing effect. Previous investigations have shown that both the amount of lime and the amount of flyash must be carefully controlled. In particular, the use of too much flyash may actually

TABLE A
PROPERTIES OF HYDRATED LIME AND FLYASH

Chemical Analysis	Hydrated Lime	Flyash
SiO_2	1.0	40.32
Fe_2O_3	0.4	13.39
FeO	0.0	3.95
Al_2O_3	0.2	32.92
CaO	47.8	2.34
MgO	33.8	0.74
Loss on Ignition	16.3	5.79
CO_2	0.8	—
H_2O	0.5	—
Sieve Analysis		
Sieve No.		
60 (total percent retained)	1.0	2.0
100 " " "	2.8	10.1
200 " " "	5.6	21.0
Specific Gravity	2.60	2.20
Dry Rodded Density - lb. per cu. ft.	45	60

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[1] Highway Research Board Proceedings, V. 30, 1950, pp. 489-502, and V. 31, 1952, pp. 511-528, and Bulletin 69, pp 1-28.

[2] Highway Research Board Proceedings, V. 31, pp. 529-540.

produce a mixture which is poorer in some properties than the original soil.

It is not clear, in the paper, that lime and flyash are two distinctly separate materials. The paper states, referring to lime and flyash: "The former is a waste material of lime production." Actually hydrated lime is the major product of the lime industry, while flyash is a fine ash collected when pulverized coal is burned. A detailed analysis of each material is shown in Table A. It would also be helpful, to one not familiar with Tennessee gravel or chert, in evaluating the study, to have available the grain-size curves and AASHO classification of the soils used.

E. A. WHITEHURST, Closure – With reference to Miller's discussion, the author appears to have erred in not more clearly defining the material referred to in the paper as "lime and flyash". It was not intended that this material be identified as a combination of hydrated lime and flyash, with both of which materials the author is generally familiar. The material is, in the words of the company representative who provided it for use in this study, a byproduct and has the chemical characteristics, as reported by the company, shown in Table B.

TABLE B
CHEMICAL PROPERTIES OF
LIME-FLYASH MIXTURE

	%
$Ca(OH)_2$	47.36
$CaCO_3$	25.98
CaO(x)	3.22
SiO_2	8.18
Al_2O_3	0.97
Fe_2O_3	0.42
MgO	0.52
C	13.35

Note: The CaO(x) refers to calcia that is bound to silica and alumina, and which is, hence, unavailable.

It is agreed that the compressive strength of soils stabilized with combinations of hydrated lime and flyash are generally appreciably increased by subjection to prolonged curing. It was the purpose of this study, however, to devise means not only of stabilizing bases for highways in the state highway system, but also roads in the county road system, many of which may not receive the degree of maintenance which is desirable. It was felt, therefore, that the study should be limited to the determination of the effectiveness of various potential stabilizing admixtures after relatively short curing periods.

Although the material under discussion is a byproduct and would probably be relatively inexpensive, no sources are available in or near those areas of Tennessee in which poor base materials are prevalent. Since long hauls would be involved, the use of percentages such as those suggested by Miller would be most uneconomical. The quantities chosen, 1, 2, and 4 percent by weight, were believed to be practical for field use as well as comparable, quantitatively, to the other admixtures investigated.

The physical properties of the soils investigated, together with their grain-size distributions have been provided in the paper as revised for printing.

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