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Soil and Slope Stabilization and Moisture and Density Determination Developments

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A PROBLEM IN HIGHWAY SLOPE STABILITY

Density-Compactive Energy-Calcium Chloride Content Relationships for an Iowa Dolomite

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The effects of calcium chloride content and the amount of compactive effort on the density of a crushed limestone were determined by laboratory experiments. The data were analyzed on the basis of energy cost or savings due to the presence of calcium chloride. The results indicate that the expenditure of compactive energy to obtain a given density is dependent on the calcium chloride content. The results also show that for a given density, an optimum calcium chloride content exists which will produce the density with a minimum compactive effort. The optimum calcium chloride content varies with the density.

● PRESENT DAY theory of soil stabilization is based on mechanical principles either alone or in combination with the addition of chemicals or materials whose principal properties are considered to be of a chemical nature. Regardless of whether the method of stabilization is purely mechanical or chemical and mechanical, compaction principles are always used and generally understood. However, the effects of additive chemicals or materials on the compaction characteristics are not well understood.

Well established mechanical principles leading to mechanical stability are gradation, binding properties of the fine material, and the compaction characteristics of the system. The addition of chemicals to the soil material changes all of these. Gradation is changed because the lower limit of particle size now becomes the size of a molecule or a crystal of the chemical used. The binding properties are changed due to ion exchange and other surface chemical phenomena. The compaction characteristics are also changed due to intermolecular forces in the soil and a change in the liquid used from water to a solution. Pure water is seldom used except in the laboratory.

The investigation described herein was undertaken to better understand the mechanism causing changes in the compaction characteristics of a crushed dolomite when calcium chloride is added. The basis of computation used in the analysis was somewhat different than usually used in density studies. Ordinarily the basis is a cubic foot. Here the basis is weight, because the volume of a mold is a constant, whereas the weight of material forced into a mold is variable. All energy values are expressed as energy per pound of compacted crushed rock plus calcium chloride or per pound of compacted crushed rock.

MATERIALS

The calcium chloride used was a commercial product known as Peladow supplied by the Dow Chemical Company. The material contains a minimum of 94.0 percent calcium chloride and less than one percent water. The magnesium chloride content is less than 0.5 percent and the alkali chloride content (such as sodium chloride) is less than 5.0 percent.

The crushed rock was obtained from Cook's Quarry which is located about 5 mi northeast of Ames, Iowa. The rock is a buff to brown calcitic dolomite of Mississippian age and has physical properties similar to limestone. The material was crushed at the quarry to pass a ¾-in. screen. A partial particle size analysis is given in Table 1.

TABLE 1

SIZE FRACTIONS OF CRUSHED DOLOMITE AND AMOUNTS USED FOR INDIVIDUAL BATCHES

Passing sieve	¾ in.	½ in.	No. 16	Totals
Air-dry wt, gm	85,737	182,835	51,910	319,482
Moisture content, %	nil	0.1495	0.4762	0.1622
Oven-dry wt, gm	85,737	182,562	51,664	319,963
Fraction, %	26.80	57.05	16.15	100.00
Batch composition:				
Oven-dry wt, gm	536.00	1,141.00	323.00	2,000.00
Wt of moisture, gm	nil	1.71	1.54	3.25
Air-dry wt, gm	536.00	1,142.71	324.54	2,003.25

EXPERIMENTAL PROCEDURE AND DATA

Moisture-density relations were determined for batches of dolomite containing 0, 1/2, 1, 1 1/2 and 2 percent calcium chloride (based on the oven-dry weight of the rock) by the AASHO standard method, T-99-57. However, the compactive effort used was 20, 25 or 30 blows per layer, the amount being constant for any one study.

Initial studies gave an indication that the crushed rock suffered degradation when used repeatedly in the density determinations. This was reflected by data which gave erratic density correlations. Henceforth each density determination was conducted with fresh material; material was discarded after being compacted once. All data reported in this paper were derived from fresh material as outlined in the following procedure.

About 1,000 lb of dolomite, crushed to pass a ¾-in. screen, were separated into three size fractions. Each density determination was made with a 2,000-gram sample composed of the appropriate quantities of the size fractions necessary to produce a mixture having the original mechanical analysis. This procedure was followed to reduce errors due to segregation of fine material. The original size fractions and the amounts used in batches are given in Table 1.

Batches were prepared from the indicated quantities of crushed rock. Appropriate amounts of calcium chloride solutions were mixed with the dolomite in a Hobart C-100 kitchen mixer until the material appeared to be uniform. A density test was then conducted according to AASHO designation T-99-57.

Moisture samples were taken from the molds and dried for 24 hr at 105 ± 2 C and were redried for an additional 24 hr to check the initial determination. All checks showed 24 hr drying time to be sufficient.

Data resulting from the experimental work are plotted in Figures 1 and 2 with iso-optimum moistures and iso-density curves plotted as a function of the compactive energy and the calcium chloride content. The compactive energy values are expressed as foot-pounds of potential energy per pound of dry material (rock + $CaCl_2$) and the densities are expressed as pounds of dry material (rock + $CaCl_2$) per cubic foot. The data in Figure 2B are based on a pound of rock only and exclude the $CaCl_2$ content. The optimum moisture values and calcium chloride contents are expressed as a percentage of the dry weight of the soil.

ANALYSIS OF DATA

The study of any process must include an exact accounting of all material and energy that enters of leaves the system. Such an accounting must be accomplished by computing all quantities to some arbitrary basis. The quantities may be added or subtracted provided the basis establishes equivalent amounts of material or energy. The use of mass as the basis for computation has been used in this study inasmuch as compactive energy is absorbed by the mass rather than the volume. The energy is expended in the rearrangement of particles, as a heat loss due to friction and as a force causing fluid flow.

The use of mass as a basis for computations also allows an easier and more readily apparent comparison of values. For instance, using 30 blows per layer and 3 layers in $\frac{1}{30}$ of a cubic foot may produce a density of 120 pcf for one percentage of calcium chloride and 125 pcf for another. Using the cubic foot basis, these two densities result from the same compactive effort, whereas on a pound basis one received 123 foot-pounds per pound of material and the other 118 foot-pounds per pound. This then represents a difference of 5 foot-pounds of energy less per pound of material to attain a higher density, rather than the same compactive effort to attain a higher density.

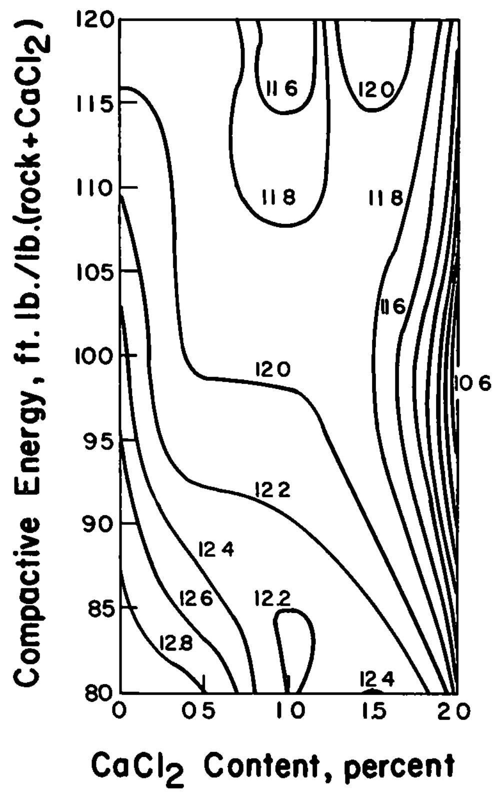

Figure 1. Optimum moisture requirements for the various combinations of compactive energy and calcium chloride content.

The introduction of calcium chloride to the dolomite-water system affects the density that results from any given compactive effort. The variations in density are caused by the physical presence of the calcium chloride and by the chemical influence of the calcium chloride on the system. The amount of energy involved in the chemical influence can be indirectly found because the magnitude of the total effect on energy requirements is known from Figure 2A and the magnitude of the physical effect can be calculated for any amount of calcium chloride added. The difference between these two, therefore, represents the magnitude of the chemical influence.

Consider a cubic foot of dry compacted crushed rock (specific gravity 2.70) weighing 126 pcf. The volume of solid material is 0.750 cu ft and the volume of voids is 0.250 cu ft. The weight of such a cubic foot of material can be increased through partially filling the voids by allowing a solution of calcium chloride to seep into the material and then evaporating the water. Theoretically the void space could be completely filled by hexahydrate calcium chloride thereby increasing the density to 152.2 pcf, the increase in density being accomplished by the physical presence of the calcium chloride. A plot of density versus calcium chloride content for this case is a linear function. The maximum calcium chloride content theoretically possible is limited by the void space available which is dictated by the initial density. In this case the top limit is 20.8 percent hexahydrate calcium chloride or 10.5 percent anhydrous calcium chloride.

Figure 2C shows a series of iso-density lines plotted from the energy value required to produce arbitrarily chosen densities for zero percent calcium chloride. Assuming that the presence of calcium chloride causes only a physical effect (any increase in density is due to added weight of calcium chloride in the voids) and shows no chemical influence, the lines should represent the densities attainable with the same compactive effort that was required to achieve the initial density (0 percent $CaCl_2$).

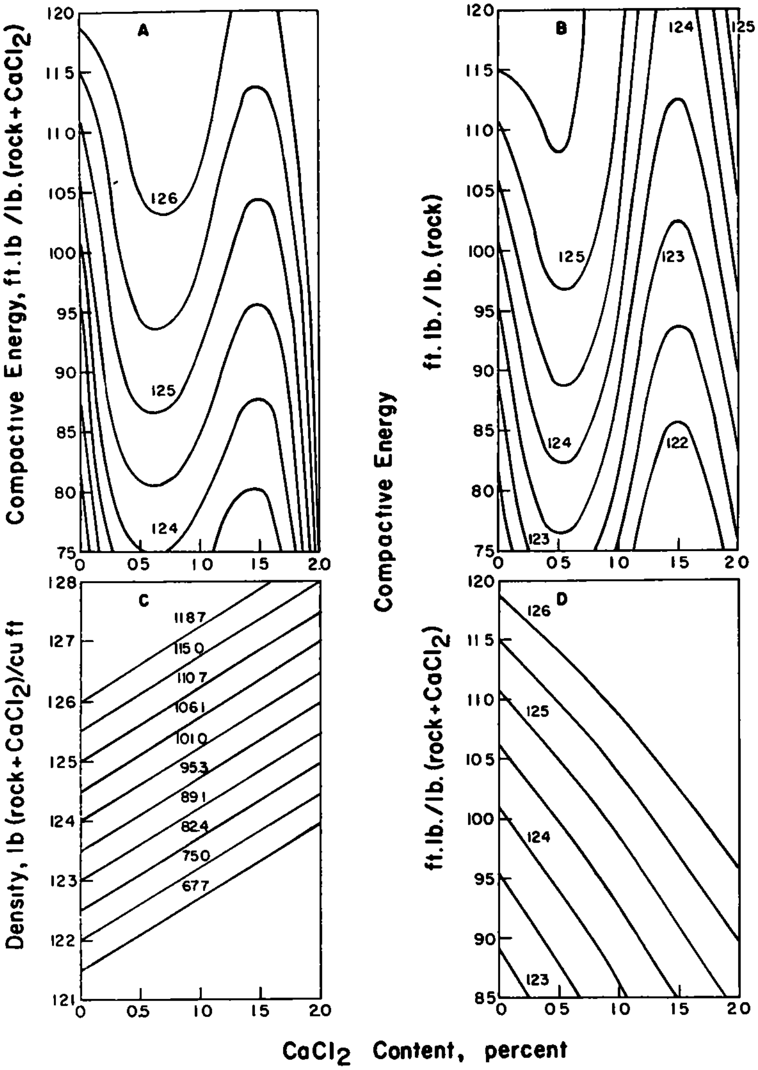

Figure 2. A—Iso-density curves as a function of compactive energy and calcium chloride content. Energy and density values are based on pounds of crushed rock plus calcium chloride. B—Data from Figure 2A plotted with energy and density values based on pounds of crushed rock plus calcium chloride. C—Theoretical iso-energy curves showing the increase in density due to the physical presence of calcium chloride D—Curves from Figure 2C replotted.

The required compactive efforts for initial density may be obtained from Figure 2A and are shown in Figure 2C for each line. Values from Figure 2C can be replotted as an iso-density chart with the same coordinates as used for Figure 2A. This chart is shown in Figure 2D and represents the combinations of compactive energy and calcium chloride content theoretically necessary to achieve a given density if, and only if, the effect due to calcium chloride is purely physical. The differences in the amounts of energy, necessary to produce a given density with a given calcium chloride content, between Figure 2A and Figure 2D then represent the amount of energy saved or the extra energy requirements due to the chemical effects of the calcium chloride. These energy cost-saving curves are shown in Figure 3. The iso-energy cost-saving chart shown at the bottom of Figure 3 was drawn from the upper six graphs of Figure 3.

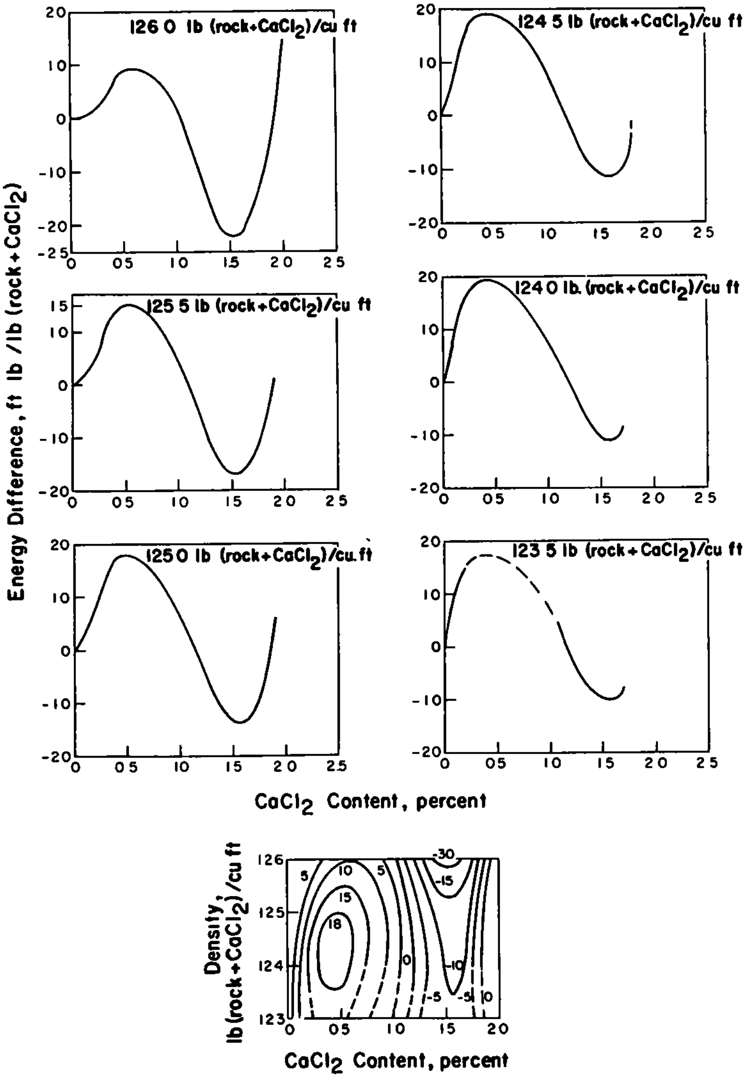

Figure 3. Energy saving or cost due to the chemical influence of calcium chloride. The upper six graphs are for the indicated density and are combined in the lower iso-energy chart. Positive values indicate saving and negative values indicate cost.

The minima occurring near 0.5 percent $CaCl_2$ in Figure 2A show that in order to achieve a specific density with a minimum expenditure of compactive effort, the amount of $CaCl_2$ required varies with the specified density. This does not imply that a combination of compactive effort and $CaCl_2$ content chosen from these minima will be the most economical combination.

Economic analysis of the data is necessarily limited to a relative status by the lack of compaction cost data. Such an analysis can be made from Figure 2D by using a pound of compacted crushed rock for a basis, as follows:

Let E = ft-lb per lb rock,
C = lb $CaCl_2$ per 100 lb rock,
A = cost per lb $CaCl_2$, and
B = cost per ft-lb energy.

The total cost due to compaction needs and the inclusion of calcim chloride is

$$\text{Cost} = BE + (AC/100) \tag{1}$$

Inasmuch as B is unknown

$$\text{Let } R = A/100B \tag{2}$$

Substitute Eq. 2 in Eq. 1 and the total cost becomes

$$\text{Cost} = B\,(E + RC) \tag{3}$$

By rearranging Eq. 3 the total costs are expressed as relative costs

$$\text{Relative cost} = \frac{\text{cost}}{B} = E + RC \tag{4}$$

Eq. 4 relates the variables through the relative ratio, R, which may be assigned any arbitrary value for the purpose of study. Values of E and C must be read from Figure 2B to be commensurate. Curves of Eq. 4 using data from Figure 2B are shown in Figure 4 with the arbitrary values of R indicated for each curve.

DISCUSSION

The iso-density curves of Figure 2A show that the compacted density of the dolomite-calcium chloride system depends on the amount of compactive energy expended and on the amount of calcium chloride in the system. The lowest densities result from the lowest expended energy and the lowest amount of $CaCl_2$; the highest densities result from the highest values of energy and $CaCl_2$ within the ranges studied. Between these limits maxima and minima occur in the family of curves. These maxima and minima are interesting in that they reflect the chemical influence of the $CaCl_2$.

The curves also indicate that the statement "less compactive effort is required to obtain a required density when calcium chloride is incorporated in the mix" does not hold true for all values of $CaCl_2$ content. The statement is certainly upheld by the data between 0 and 1.0 percent but the curves for 125.5 and 126.0 pcf near 1.5 percent show that the same or more effort is required than at zero percent. However, 1.5 percent is generally out of the economic range and would therefore seldom be used.

The changes in energy requirements to achieve a given density are caused by the physical presence and the chemical influence of the $CaCl_2$. The degree of change depends on the amount of $CaCl_2$ contained in the system. As shown in Figure 2D, the physical presence of $CaCl_2$ reduces the energy requirements in a near linear relationship with the amount of $CaCl_2$. The changes in energy requirements due to the chemical influence are of a more complex nature and vary considerably with the amount of $CaCl_2$ present and with the density of the system.

The variations in energy requirements due to the chemical influence are shown in Figure 3. Proper analysis of these curves requires a knowledge of the chemical properties of the $CaCl_2$ solutions that are present during compaction. These properties vary with the molar concentrations of $CaCl_2$. Molar concentrations may be calculated from the optimum moisture data shown in Figure 1. Viscosity, surface tension, lubrication, zeta potential, streaming potential and crystallization are properties of phenomena that are dependent on molar concentrations.

The molar concentration in the ranges studied is nearly a linear function of the $CaCl_2$ content because the molarity is more affected by the amount of $CaCl_2$ than by the amount of water. The optimum moisture contents vary only over the limited range of 12.9 to 10.6 percent, whereas the $CaCl_2$ content varies from 0 to 2.0 percent.

Inasmuch as the viscosity and the surface tension of the solutions are both linear functions of the molarity, the resistance to compaction should increase linearly with $CaCl_2$ content as demonstrated by G. G. Stokes, Osborne Reynolds and many other investigators who have made studies of fluid flow through porous media. The compaction of soil materials is essentially a problem of the forces involved in the flow of fluids through porous media or vice versa because in the process of rearrangement of particles the fluid (water or a solution) must move relative to the solid particles.

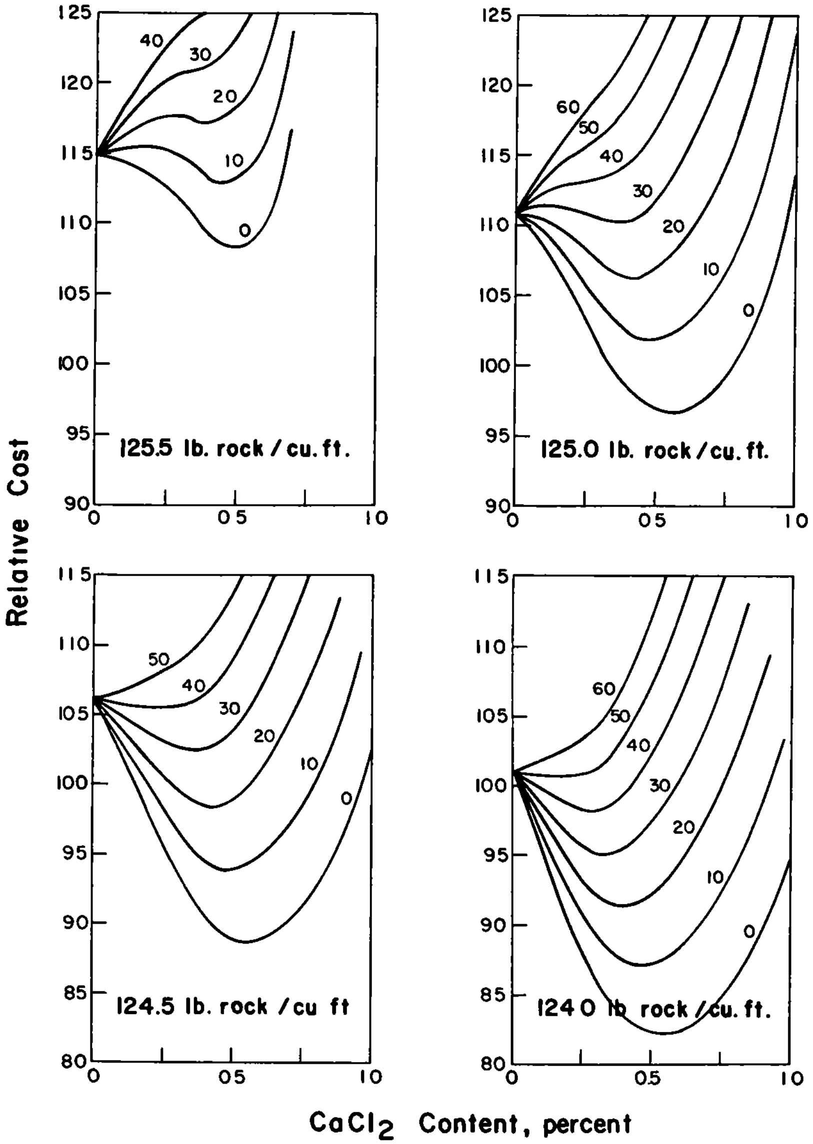

Figure 4. Curves showing relative cost as a function of the calcium chloride content for the density listed. The number identifying each line is the ratio of unit cost per pound of calcium chloride to the unit cost per foot-pound of compactive energy.

Plots of energy versus density, at constant $CaCl_2$ content, show that the energy requirements increase with density and that slopes of the energy-density lines also increase as the $CaCl_2$ content increases.

The quality or the amount of lubrication is a property that has not been described for any liquid to the author's knowledge. Zeta potential and streaming potentials are probably of little influence in the present system because clay minerals and ion exchange are virtually absent.

Preferential absorption of water by the dolomite, which would increase the concentration of the solution, could cause crystallization to occur before compaction was completed if enough water were absorbed to cause the solution to become supersaturated. The hexahydrate would crystallize and the presence of these crystals would increase the resistance to compaction somewhat.

The foregoing discussion points out various factors that are primarily dependent on the molarity of the $CaCl_2$ solutions involved. The total resistance to compaction is in turn a summation of the various contributions of these factors. If all factors were clearly understood, their various contributions could be separated and a summation of these should produce curves such as are shown in Figure 3. This analysis is limited by ignorance of these factors, but it is the author's hope that the discussion will stimulate further investigation of this problem.

A question often posed is whether or not the added cost due to the inclusion of $CaCl_2$ is offset by a reduced cost of compaction. The relative cost curves indicate the added cost is allayed, provided the ratio of the unit cost of $CaCl_2$ to the unit cost of compactive energy is low. A value of R exists, for each density, above which the inclusion of $CaCl_2$ is not economical. This value of R may be defined as a permissible cost ratio, R_p. The value R_p is then the maximum value of R that will permit the economical use of $CaCl_2$. The curves show that R_p decreases with an increase in density from 50 for 124.0 pcf to about 12 for 125.5 pcf. The economical use of $CaCl_2$ is therefore dictated by the density desired.

A general statement as to whether or not the cost of $CaCl_2$ is offset by reduced compaction costs cannot be made until data for many other materials become available and until a permissible cost ratio is determined. Finally, laboratory data and field data should be correlated to form a realistic basis for conclusions.

Stabilization of Fine-Grained Soils with Cutback Asphalt and Secondary Additives

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● GROWING SHORTAGES of quality road-building aggregates in many areas of the United States, coupled with the ever-growing need for roads, undoubtedly will result in greater use of inferior aggregates and soils. Inasmuch as present standards governing the quality of roads will not be downgraded, the means must be found to upgrade these inferior materials. This is evident in the report of Highway Research Board Special Committee on Highway Research Priorities (Special Report 56), which regards improved knowledge of aggregates and soils as a research area of critical importance. This committee estimates that almost one-third of the total $34 million of highway research expenditures over the period of five years should be spent for that purpose.

Because soil stabilization falls in this research area, it can be expected that the stabilization of soils with asphalts will increase in significance and importance.

The stabilization of soils with asphalts (as with any other stabilizing agent) can be defined as a treatment of a natural soil so that, after compaction or consolidation, it will provide a water-resistant and stable structural pavement layer of adequate load-bearing qualities under the anticipated traffic and environmental conditions.

A variety of materials has been used to stabilize natural soils. Depending on the mechanism of stabilization these stabilizers may be subdivided into three general groups:

The first group includes materials which cement the soil particles. These materials, capable of reactions within themselves or with certain soil constituents in presence of water, produce strong interparticle bonds which can support high intensity loads. Typical representatives of this group are portland cement limes, and more recently acidic phosphorus compounds. The soil stabilized with these materials may possess high initial strengths. But, because of inherent nature of the bonds formed, it lacks desirable durability characteristics when exposed to such environmental conditions as drying-wetting or freezing-thawing. Additionally, in the case of fine-grained soils, the quantities of such stabilizers required to produce adequate initial strengths often are economically prohibitive.

A second group of stabilizers could be termed the soil modifiers or conditioners. Cement and lime at low concentrations, calcium or sodium chlorides, and a number of surface active materials are typical of this group. These chemical compounds, because of the surface reactions with the soil minerals (and particularly with fine fractions of soil), change soil texture and structure, thereby altering its physical or engineering properties. Depending on the character of the soil, improvements in mixing, drying, compaction, strength and other wet soil properties can be realized with relatively small quantities of these modifiers. But just as in the case of the cementing agents, soil masses treated with these modifiers are often highly susceptible to the climatic and environmental changes.

The third group embraces the waterproofing agents. Asphalts, certain resinous materials, and coal tars are representative of this group. The basic mechanism of stabilization by these agents is radically different from the two groups previously mentioned. These stabilizers coat individual soil particles or their agglomerates, and thereby prevent or hinder the penetration of water into the stabilized soil layer.

The role of the asphalt film in stabilizing a soil mass depends to a great extent on the properties of the soil. In the case of coarse, non-cohesive soils (such as sands or silts) the asphalt film serves a double purpose. First, it waterproofs the soil mass. Second, it binds the soil particles together, contributing materially to the load-bearing qualities of the stabilized layer. In the case of fine-grained, cohesive soils, on the other hand, waterproofing is the principal role of the asphalt film. Cohesive soil, when compacted at proper water content, possesses high strength. This strength, attributed principally to the cohesive interparticle forces, is highly sensitive to the action of water. Asphalt films distributed throughout the soil protect these bonds and, partially at least, preserve the strength of the soil.

Stabilization of soils with asphalts is probably the oldest road-building process using admixtures as soil stabilizers. With the advances in soil mechanics during the last several decades, long strides have been made toward a better understanding of soil-asphalt systems. Yet, asphalt soil stabilization remains largely an art form rather than science. Knowledge of the basic properties of such systems remains relatively sparse. Little is known, for instance, about determing the composition of the mixture or thicknesses of the stabilized compacted layer to be used. Many test methods, empirical formulas and recipes, although practiced for a long time and suitable for given local conditions often confuse rather than clarify the problem. This applies particularly to the soil-asphalt systems involving cohesive, fine-grained soils.

Many secondary admixtures have been tried with varying degrees of success in soil stabilization with asphalts. Their principal purpose is to condition the soil and improve asphalt adhesion to the mineral surfaces. Again, because of the variety of soils and the many different evaluation methods used, it is difficult to compare results submitted in random reports, often limited in scope.

In view of this, The Asphalt Institute is engaged in soil stabilization studies with the purpose of increasing basic knowledge of soil-asphalt systems and their effective use. The effects of secondary additives on asphalt-stabilized fine-grained soils constitute a partial report of these investigations.

DESCRIPTION AND PREPARATION OF MATERIALS

Soils

Three soils—Massachusetts clayey silt, Maryland sandy clay and Mississippi loess—were used in these investigations. Maryland soil was used in most of the testing because it was readily available. The properties of the three soils are summarized, as follows:

	Mass. Clayey Silt	Md. Sandy Clay	Miss. Loess (1)
Specific gravity	2.74	2.72	2.69
Liquid limit (%)	21.2	31.6	31.6
Plasticity index (P.I.)	5.9	11.8	8.3
Shrinkage limit (%)	16.9	21.0	24.5
pH	5.8	5.9	6.9
Surface area[1] (sq m/gm)	17.5	33.2	54.7
AASHO classification	A 4(6)	A 6(9)	A 4(8)

[1] By glycerol retention.

Figure 1, showing the grain size distribution for the soils, indicates that these materials represented a wide range in gradation. Also, the mineralogical composition varies widely. However, the three major clay minerals were represented by these soils. Maryland soil contains kaolinitic clay minerals, the clay in the Massachusetts soil is composed predominately of illitic clay minerals and the Mississippi loess contains montmorillonitic clay minerals. Figure 1 shows gradations for two samples of Mississippi loess (continuous and broken lines). The majority of tests were made

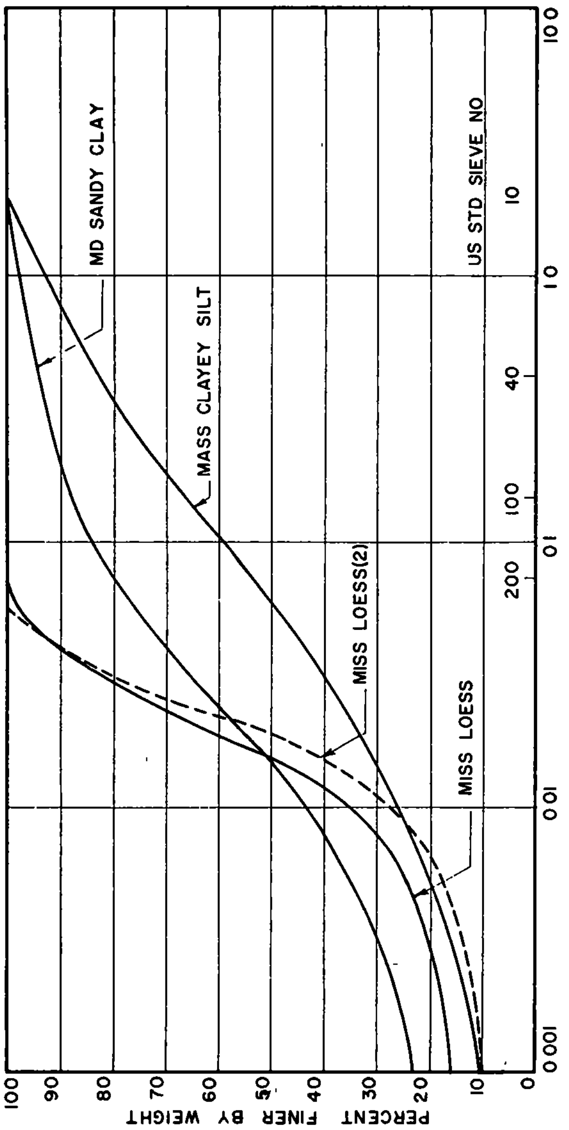

Figure 1. Grain size distribution of soils.

using material represented by the continuous line. Inasmuch as densities varied appreciably for compacted specimens of these two soil samples, gradations of both are shown. Accordingly, both soil samples are identified in the presentation and discussion of the results.

All soils, before testing, were air dried, broken down and sieved through successively smaller size sieves until a soil fraction passing the U. S. Standard Sieve No. 10 was obtained. This fraction was used for the preparation of specimens and testing. To prevent the degradation of the original soil particles minimum crushing or grinding was used.

Asphalt Cutback

Asphalt cutback used in this study was prepared in the laboratory by mixing on a weight basis two parts of 85-100 penetration asphalt cement with one part of light petroleum naptha. These ingredients were placed in a 1-gal glass jar and rotated on ball mill rollers until the solution appeared completely homogeneous. A low boiling range solvent was used to facilitate the evaporation and thereby to reduce or eliminate the effects of the organic solvent on the properties of compacted soil-asphalt mixture.

The properties of the prepared liquid asphalt are as follows:

Specific gravity	0.89
Kinematic viscosity, at 77F (centistokes)	160
Kinematic viscosity, at 140F (centistokes)	70

Distillation characteristics: Temp, (deg. F)	Distillate (% by vol.)(Based on Tot. Dist to 680F)
320	68.8
374	81.7
347	88.3
500	94.3
600	98.7
680	100.0

Residue from distillation to 680F by vol. % of total cutback	61.5
Penetration of residue (77F, 100 gm, 5 sec)	99
Ductility of residue (77F) cm	100+
Spot test on residue	Neg.

Tests made on this liquid asphalt and on the residuum obtained from distillation indicate that, according to Asphalt Institute specifications, this material could be classified as a Rapid Curing (RC-0) asphalt cutback.

Secondary Additives

Following is a list of the secondary additives used in this investigation:

Material	Concentration Used % (based on dry soil)	Effect of Additive on Soil
Hydrated lime (calcitic)	4.0	Cementing and modifying
Normal portland cement (Type III)	3.0	Cementing and modifying
Phosphoric acid (ortho) 84% solution	2.0	Cementing and modifying
Quicklime (calcitic)	3.0	Cementing and modifying
Calcium carbide	3.44	Cementing and modifying
Octodecyl amine (Armeen 18D)	0.3	Modifying and waterproofing
Sodium chloride	3.0	Modifying
Calcium chloride	3.0	Modifying

Commercial grades of these chemical compounds were used in all cases. The concentrations for quicklime and calcium carbide (3.0 and 3.44, respectively) were selected to obtain direct comparison with the effects of hydrated lime. After reaction with water, these additives produced approximately four percent of hydrated lime.

EXPERIMENTAL PROCEDURES

Mixing of Water, Additives and Asphalt with Soils

The air-dried soils were pre-mixed with the varying amounts of water and stored in the polyethylene bags for at least 18 hours before being mixed with stabilizers and compacted in specimens. Depending on the amount of soil required for testing, premixing was done either with a Hobart Model C-100, 10-qt capacity or an LW Lancaster Mueller type 5/8-cu ft capacity laboratory mixer. Water content was always determined before mixing wet soil with the secondary additives or liquid asphalt. Usually, at least five water contents for a given type of compaction were used. Every effort was made to distribute the water contents on both sides of optimum in order to obtain well developed, density vs compaction water content curves.

Quicklime, calcium carbide and octodecyl amine were pre-mixed with liquid asphalt prior to mixing with soil. All other secondary additives were blended in before the liquid asphalt was mixed with the wet soil. Preliminary testing indicated that 1-min mixing in a Hobart mixer equipped with a flat beater was sufficient for the even distribution of the additive.

After mixing the additive, the mixer was stopped and a weighed amount of liquid asphalt was added. The mixing process continued for 1½ min. Then the mixture was scraped from the sides of the mixing boal and additional mixing continued for 1 min. By weighing the mixing bowl and the mixture ingredients before and after mixing, it was possible to determine the amounts of volatiles lost in the mixing process. Approximately 90 percent loss of liquid asphalt solvent occurred during a total 2½ min mixing at room temperature. Tests made on the mixture indicated that only negligible amounts of water were lost during mixing.

Compaction and Stability Determination

Four different compaction and stability determination methods were used in this study. The methods and the pertinent characteristics of these methods are given in Table 1.

In order to obtain comparisons between these different compaction and stability determination methods, some changes in the standard methods were necessary. These changes and the references to the standard test methods are outlined in the following paragraphs.

Specimens compacted by the AASHO T 180 method (see Highway Materials Part III, American Association of State Highway Officials) were tested in unconfined compression. The dimensions of the specimens were not ideally suited for this test. Inasmuch as the trends in the test results approximate closely the results obtained by the other test methods, it is believed that this test was valid.

Marshall compaction and stability determination procedures used in this testing were essentially as described in The Asphalt Institute publication, "Mix Design Methods for Hot-Mix Asphalt, Paving" (Manual Series No. 2). Compaction and testing at room temperatures constituted one deviation from the standard procedures. Modification of the sample mold was another deviation from the standard procedures. A sample mold 2½ in. high equipped with an extension collar was used. Trimming the compacted specimens to that height resulted in the elimination of the height correction for the measured stabilities.

Compaction of the specimens with the mechanical kneading compactor and the determination of stabilometer resistance values were made by the procedures described in the "Materials Manual, Volume I," Department of Public Works, State of California. Deviation was made in the use of the leveling load after kneading compaction. These leveling loads were necessary to obtain parallel faces of specimen. Care was exercised not to obtain additional compaction by these static leveling loads and not to exhude the water from the specimens. A split mold similar to that used in Marshall testing was used to obtain specimens of constant volume.

TABLE 1
PERTINENT CHARACTERISTICS OF COMPACTION AND STABILITY METHODS[1]

Compaction		Specimen Size (in.)		Compactive Effort	Type of Stab. Determination	Loading Rate (in./min)
Method	Type	Diameter	Height			
AASHO T 180	Dynamic-impact and kneading	4.0	4.8	10-lb ham., 18 in. drop., 5 layers, 25 tamps/layer	Unconf. comp. strength	0.24
Marshall	Dynamic-impact	4.0	2.5	10-lb ham., 18 in. drop, 50 tamps on each end of specimen	Semi-confined (Marshall)	2.0
Mechanical kneading compactor	Dynamic-kneading	4.0	2.5	Init. 15-psi ft pressure, 30 tamps, Final 350 psi, 100 tamps	Triaxial (Hveem stabilometer)	0.05
ASTM D 915	Static	2.0	2.0	Total load 6,000 lb maintained for 1 min	Extrusion (punch shear)	1.0

[1] All specimens compacted and tested at room temperature

Compaction and testing of ASTM D 915 test specimens closely followed methods prescribed in American Society for Testing Materials publications (see Procedures for Testing Soils, sponsored by ASTM Committee D-18). The major deviation from these test procedures was in the curing of the test specimens. Curing conditions for all test methods are described in the following section.

Curing and Water Immersion of Compacted Specimen

After compaction, the specimens were removed from the compaction moles and exposed to different curing conditions. Curing was always done in a closed chamber through which an air stream, saturated with water vapor, was passed. The saturation of air with water was obtained by passing the air stream at low velocity through two water scrubbers, and a water layer at the bottom of the curing chamber. Curing the specimens in the water-saturated air, it was believed, permitted further evaporation of organic solvent left after mixing while minimizing or even eliminating the evaporation of water from the compacted specimens. Weight losses of specimens cured under these conditions never exceeded one percent of the total weight of the compacted sample. Curing periods of one and four days were selected as standard in these investigations.

Total immersion of specimens at room temperature was used to determine the effects of water on the properties of compacted mixtures. Specimens were immersed in water after four days of curing in a 100 percent relative humidity chamber. In the case of all immersed specimens, changes in volume and water content after immersion were determined. Curing at 100 percent relative humidity and complete immersion in water, it is believed, represent by far a more severe condition than usually encountered in the field. In this study, four days complete immersion in water was used as a standard immersion time.

RESULTS AND DISCUSSION OF RESULTS

The results and discussion of the results of this study are divided in two parts. The first part is concerned with the basic consideration and description of the effects of asphalt on mixtures of cohesive, fine-grained soils and water. Properties of soil-water mixtures compacted with and without asphalt are compared. In the second part, the effects of a number of secondary additives on cohesive soil-water-asphalt systems are described and compared with wet soil systems treated only with these secondary additives.

Description of Cohesive Soil-Water-Asphalt Systems

Figure 2 shows dry density (soil + residual asphalt) and unconfined compressive strength of three different soils compacted over a range of water or volatiles (water + asphalt solvent) contents, with and without five percent of residual asphalt. It is seen that when these soils are compacted by applying the same compactive efforts a wide range of densities was obtained. Furthermore, it is evident that densities (or the shapes of density-compaction water content curves) are not materially affected by the presence of five percent asphalt. When these compaction efforts are used, densities of compacted specimens of these three different soils are related to grain size distribution of the soils (Fig. 1). Although other factors undoubtedly influence the compaction characteristics of soils, in the case of these three soils, it appears that higher densities were achieved when coarser and better graded materials were used.

The major difference in compaction characteristics of these two systems appears in the water and volatiles content required to attain the maximum compacted densities. Systems containing five percent asphalt required approximately 2½ percent less liquid than the systems without asphalt. This applies for all three soils used in this investigation. In the case of all three soils one part water used for compaction was replaced by two parts of asphalt.

In contrast to the density characteristics, Figure 2 shows that the unconfined compressive strengths of these soils are markedly affected by the presence of five

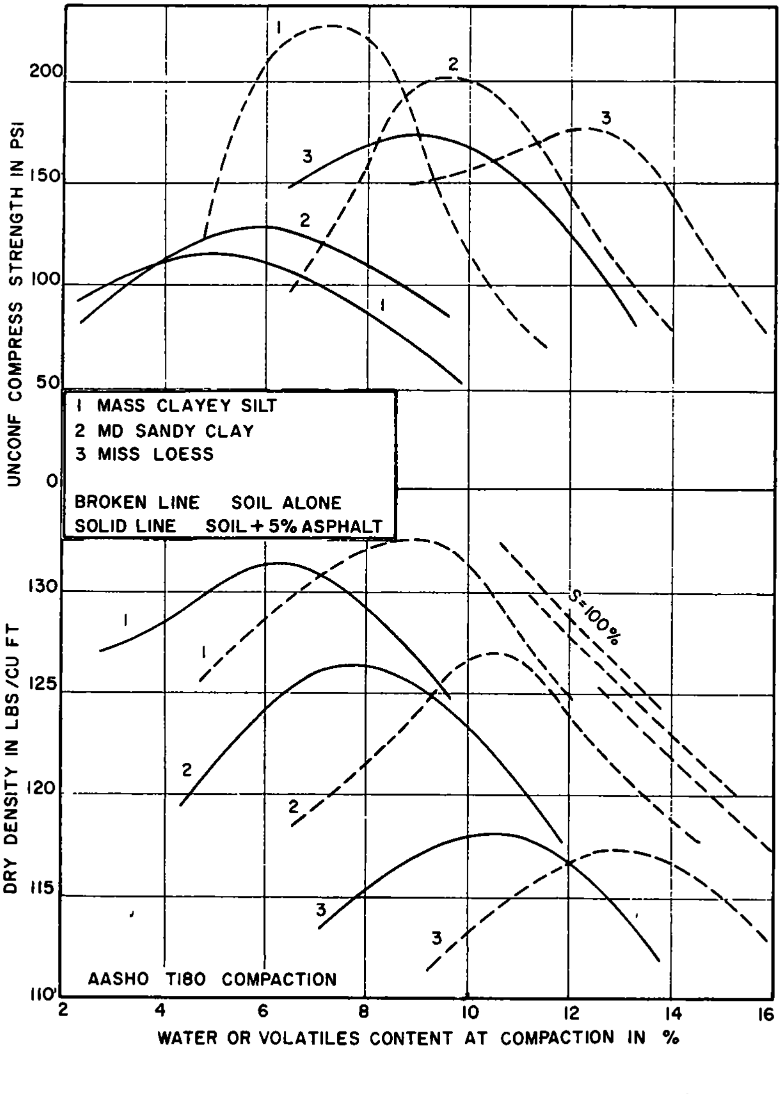

Figure 2. Density and strength of different soils with and without asphalt.

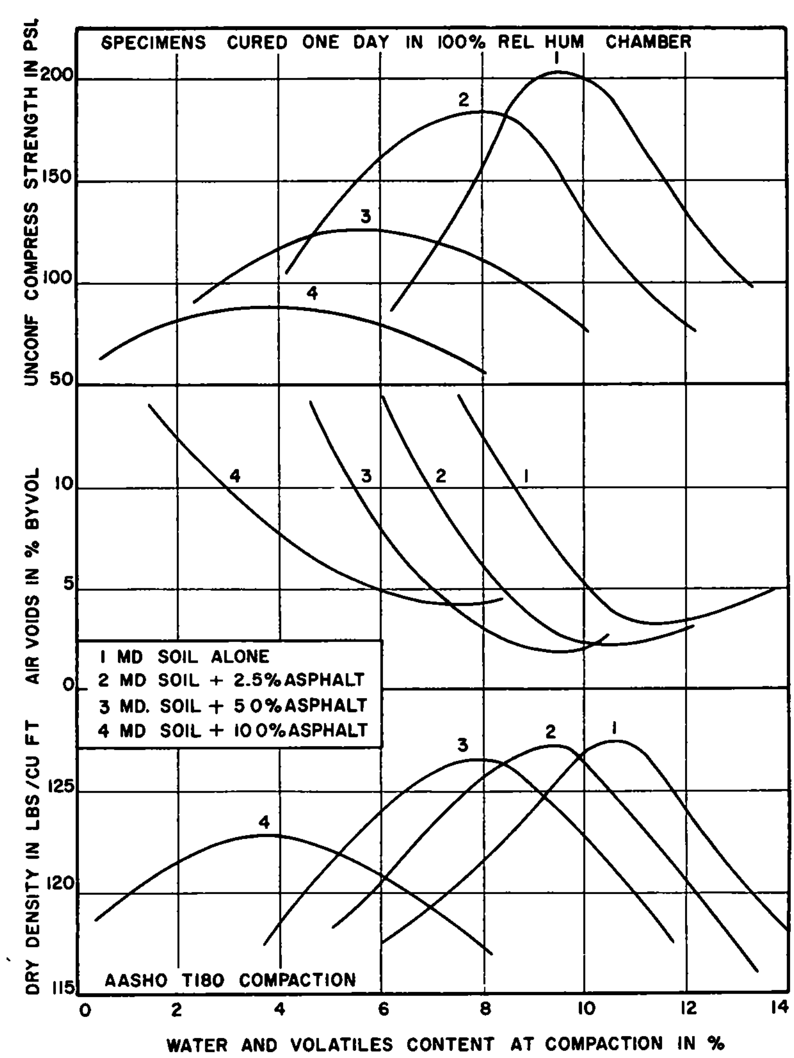

Figure 3. Density, voids and strength of Maryland sandy clay at different asphalt contents.

percent asphalt. In general, asphalt tends to diminish the strength of compacted soil. This loss of strength is more pronounced with more uniformly graded soils. In the case of poorly graded soil, such as Mississippi loess, the strength is virtually unaffected by the presence of five percent asphalt.

This loss of strength in fine-grained cohesive soils may be attributed to the reduction of cohesive interparticle bonds by the asphalt films distributed through the compacted soil mass. Asphalt films, of lower polarity than water, are not attracted to the same extent by the polar clay surfaces.

These films may act as an interparticle lubricant, resulting in a weakening of the compacted mass. Because, in the case of well-graded soil, the number of contact points (and therefore cohesive bonds) is larger, it can be expected that a given amount of asphalt films will proportionately destroy more of such bonds. This, therefore, results in a more pronounced reduction of compressive strength.

Another interesting observation can be made from the comparisons of strength characteristic of the two systems: the curves obtained by plotting strength against compaction water or liquid content are considerably flatter when asphalt is present

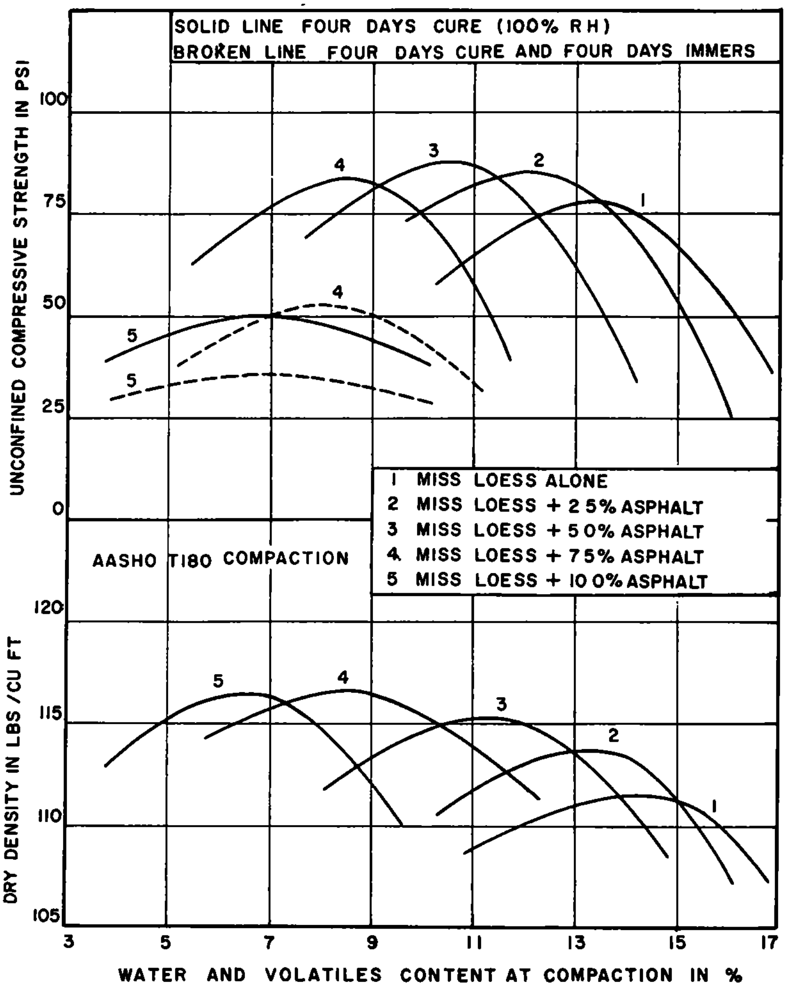

Figure 4. Density and strength of Mississippi loess (2) stabilized with different asphalt contents.

in the mixture. This means that the strengths of such asphalt-containing systems are affected less by the variations in the compaction liquid content. Considering such behavior from a practical viewpoint, it indicates that the mixtures containing asphalt will compact to a layer of more uniform strength even when the water is distributed less uniformly throughout the soil mass (as may often by the case in actual soil stabilization construction).

Figure 3 shows the effects of varying amounts of asphalt on density, air voids and strengths of Maryland soil compacted by the AASHO T 180 compaction method. It is evident that gradually increasing asphalt content in the mixture results in decreasing density of the compacted specimen. It should be noted that mixtures containing higher amounts of asphalt require less compaction liquid to obtain maximum densities. Additionally, it is evident, that when more asphalt is used, replacement of compaction water or volatiles by the asphalt is more pronounced. This means that when more asphalt is used in the soil-water mixtures, drier soil must be used in order to obtain the maximum compaction densities. Undoubtedly, the ratio indicating the substitution of compaction water or volatiles by the asphalt will be affected by factors such as soil properties, viscosity of residual asphalt, compactive efforts used, and others. Nevertheless, the general trend (as indicated by Fig. 3) will prevail, provided that the soil is sufficiently cohesive; therefore, some water is necessary to facilitate the compaction process.

Figure 3 indicates that gradually increasing amounts of asphalt progressively lower the compressive strengths of the specimens. The reasons for such behavior were described previously and it is believed that the results represented in Figure 3 confirm these reasons. It must be kept in mind, however, that such trends may be reversed when non-cohesive or cohesive soils of poor gradation are used. This is illustrated by Figure 4 showing density and strength vs compaction liquid contents of Mississippi loess (2) at various asphalt contents. Maximum dry densities and strengths of this quite poorly graded soil (Fig. 1, broken line) increase even when relatively high amounts of asphalt are used.

It is interesting to note that the maximum strengths for the systems with or without asphalt are obtained at less than optimum compaction liquid content, whether asphalt is present or not (Figs. 3 and 4). This points out the similarity in physical behavior of these two (with and without asphalt) systems.

Figure 3 also shows that a slight decrease, followed by an increase in the volume of air voids in the mixture, is caused by increasing asphalt content. Because these changes are small, it can be said that the influence of asphalt on the mixture air voids is negligible.

Figure 5 shows a more detailed picture of void properties of these mixtures and contains three families of curves, each consisting of four lines. The lower group of

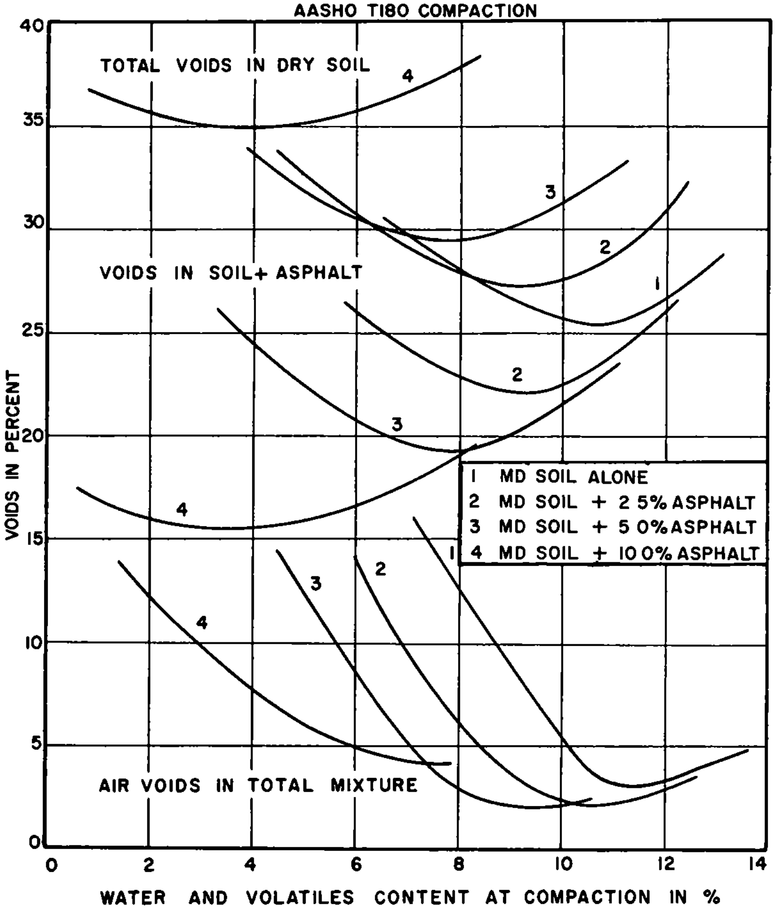

Figure 5. Void properties of Maryland soil and asphalt mixtures.

curves represents the air voids in the mixture (as shown previously in Fig. 3). The middle group of curves shows the voids composed of air and volatile liquids, and the upper curves show the mixture voids composed of air, volatiles and asphalt. These curves indicate that volatile compounds are replaced by the non-volatile asphalt and that the relative distances between the individual soil particles or agglomerates of these particles are gradually increased as progressively more asphalt is added to the mixture. The upper family of curves in Figure 6 indicates that gradual separation of soil particles by increasing amounts of asphalt is also evident when a poorly graded soil such as Mississippi loess (2) is used. It is evident, however, that the separation of soil particles is less pronounced than for a better-graded Maryland soil (Fig. 5).

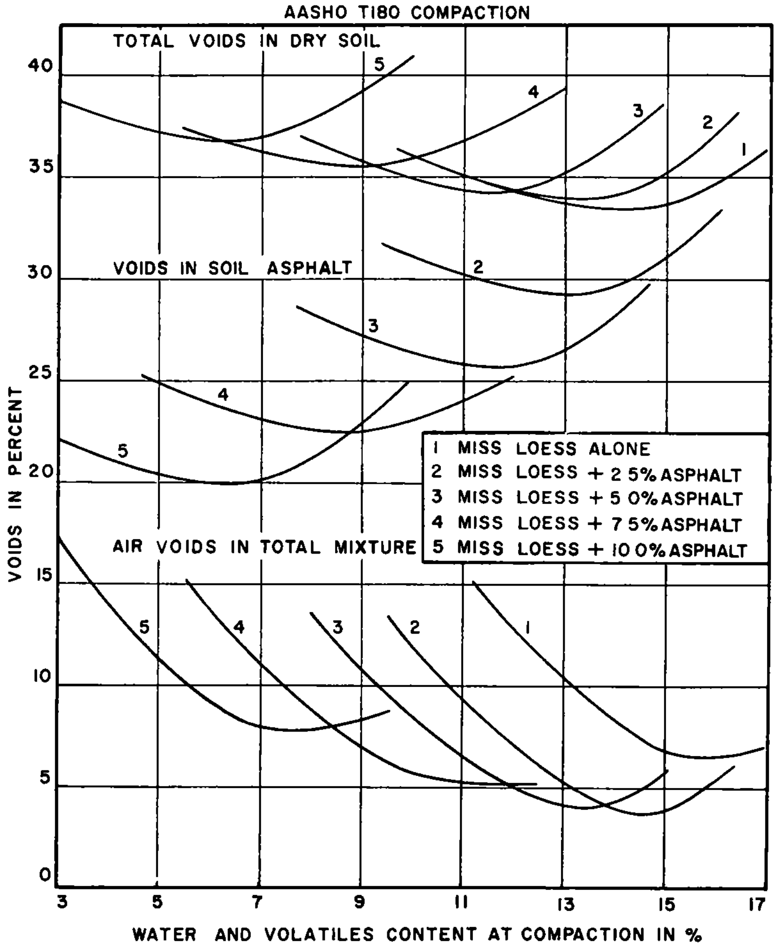

Figure 6. Void properties of Mississippi loess (2) and asphalt mixtures.

In the case of fine-grained water-sensitive soils, the major role of asphalt is to waterproof the compacted soil mass. Figure 7 shows the effects of water immersion on the properties of Maryland soil stabilized with different amounts of asphalt, and Figure 8 shows these properties of three different soils stabilized with five percent asphalt.

In these two figures, three factors are considered in evaluating the waterproofing effects of asphalt: (1) strength characteristics as affected by immersion in water; (2) water absorption; and (3) volumetric changes of specimens when immersed in water.

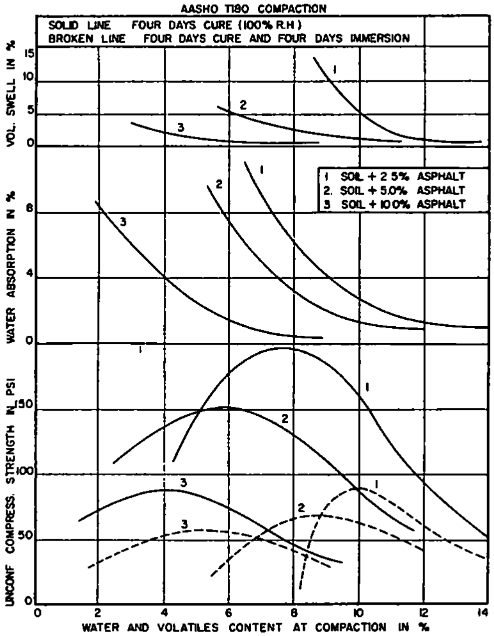

Figure 7. Strength, water absorption and volume swell of Maryland soil stabilized with varying amounts of asphalt.

Before considering the results presented in these two figures, it should be remembered that the numerical values of these properties depend on the size of the specimens (or more exactly, on the relative proportion of their external exposed surfaces and their bulk volume). For small specimens, the values for these properties would be larger than for large specimens.

These figures show that the strengths of immersed specimens, regardless of asphalt content or soil used, diminish more noticeably when compacting water contents are decreased. The loss of strength is sharply reduced by increasing the asphalt contents. Nevertheless, even when ten percent of asphalt is used, the reduction in strength by immersion is evident. (It should be kept in mind that compacted soil-water specimens without asphalt disintegrate rapidly when immersed in water.)

Although no experimental data are presented, it is safe to assume that specimens which retain greater strength during immersion also will behave more favorably when exposed to environmental weathering conditions (that is drying and wetting or freezing and thawing). This is substantiated by the generally lower rate of water absorption or by lower volumetric expansion values of the specimens which decrease less in strength after immersion in water.

Strength data as obtained for the curing conditions used and shown in Figures 7 and 8 indicate that compaction water contents at or somewhat above optimum may be more favorable. At such water contents—although the initial strengths (after curing) are considerably lower than maximum strengths at the lower water contents—a greater measure of the original strength is retained. It should be noted, however, that when a mixture contains more asphalt, the amount of compaction liquid becomes less important, because Figures 3 and 4 indicated that increasing asphalt contents replace progressively greater amounts of compaction liquid.

For the design of cohesive soil-asphalt mixtures, it is believed that the change in strength of the soil during immersion should be considered. The immersed strength could be used as an indication of the load-bearing qualities of the stabilized soil layer, and the change in strength upon immersion would indicate the waterproofing effects of asphalt which in turn would also reflect to the weathering properties of the stabilized layers. Additionally, the use of immersed specimen strength would provide a certain

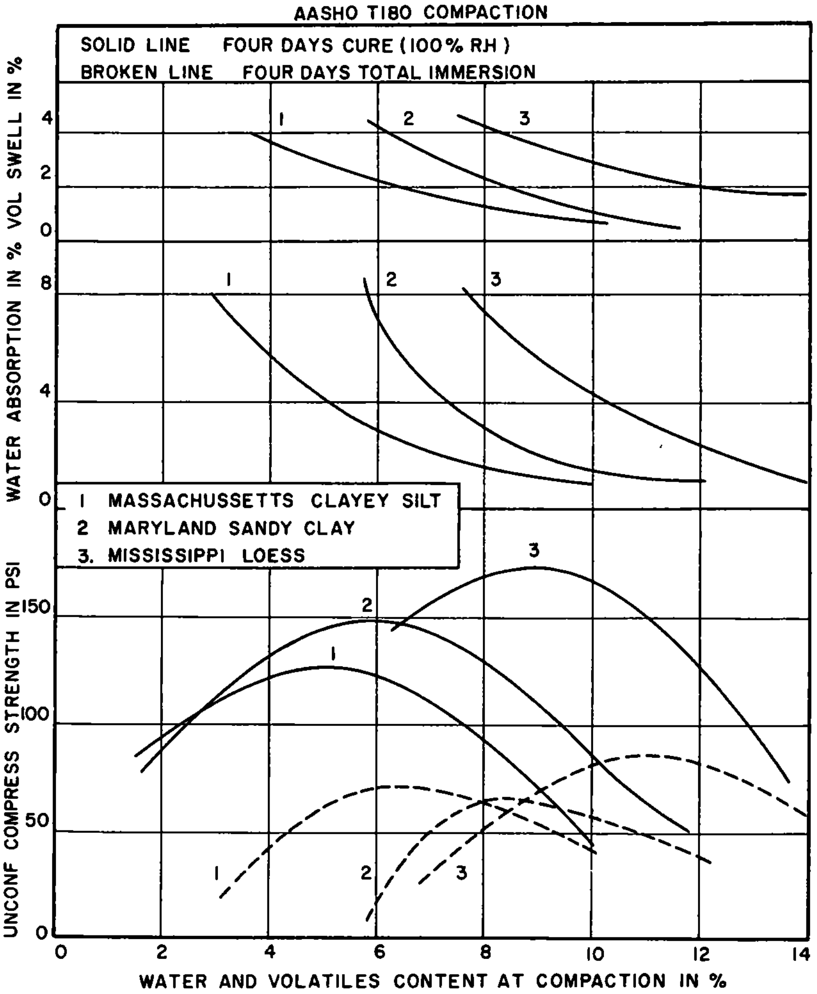

Figure 8. Strength, water absorption and volume swell of different soils stabilized with five percent asphalt.

factor of safety because, in actual use, a stabilized layer of soil is seldom subjected to such a severe condition. Normally, the stabilized soil layers are covered with layers relatively impervious to water (such as asphalt paving or surface treatments) and are provided with adequate drainage.

Comparisons of water absorption, volumetric expansion, and changes in strength of immersed specimens demonstrate the effectiveness of stabilizing the three soils with five percent asphalt (Fig. 8). These values for the Mississippi loess are highest, next highest for Maryland soil, and lowest for Massachusetts soil. It is interesting to note that the magnitude of these values corresponds to glycerol retention or to the specific surface areas for these soils as shown in the table describing the properties of the three soils.

An attempt was made to determine the suitability of different compaction and strength determination methods for testing cohesive fine-grained soil, water and asphalt mixtures. Figures 9 and 10 show density and strength of soil-water mixtures, with and without the asphalt, as determined by four different testing methods commonly used for the testing of asphaltic mixtures. The compaction methods used in this testing differ in two ways—in the manner of applying the compactive loads, and in the

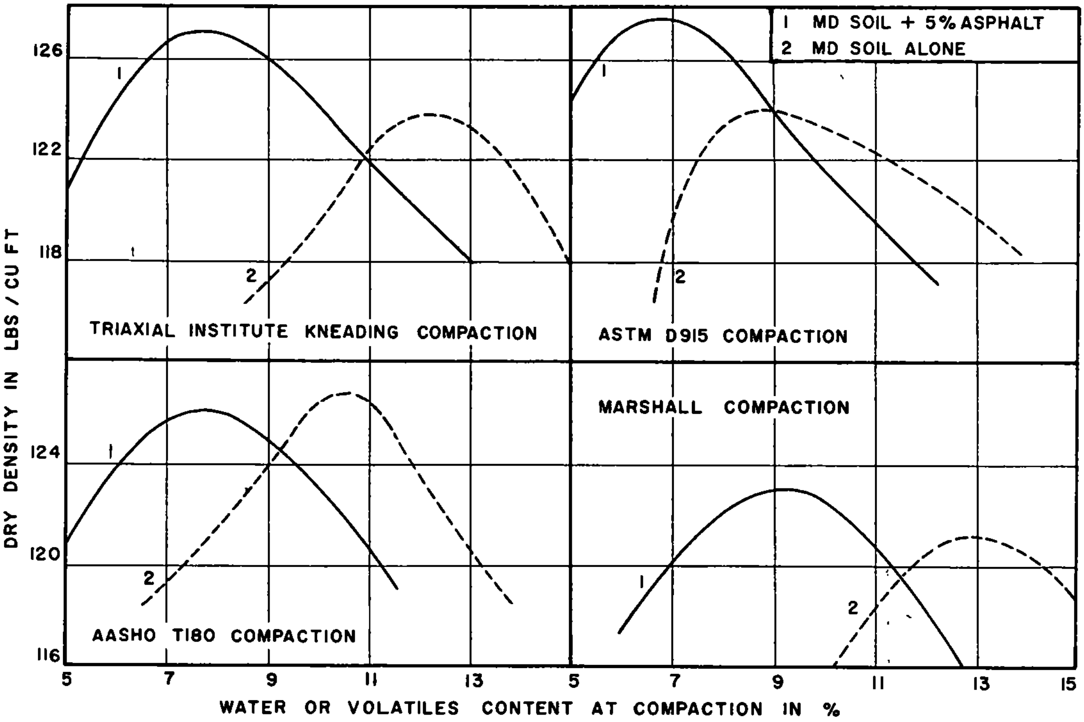

Figure 9. Effect of compaction method on density of stabilized and unstabilized soils.

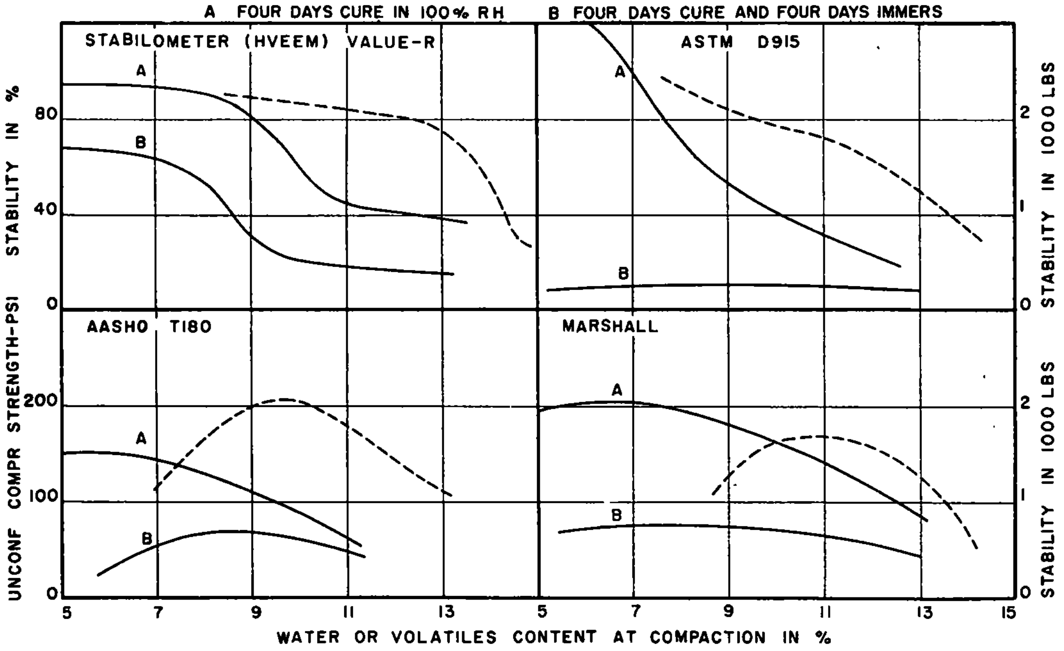

Figure 10. Stability of Maryland soil compacted and tested by different methods (solid line: Maryland soil + 5 percent asphalt; broken line: Maryland soil alone).

amount of compactive energy expended per unit weight or volume of soil mixture. The results (Fig. 9) indicate that all methods except the AASHO T 180 method produced higher maximum densities when the mixture contained asphalt. AASHO T 180 compaction, on the other hand, produced approximately the same specimen densities with and without asphalt. The reasons for this are not too obvious. Probably it can be attributed to the manner in which the compactive loads are applied, rather than to the difference in compactive energy between different compaction methods. During AASHO compaction, the soil mixtures are subjected to the combined efforts of impact and kneading action; other compaction methods lack such combined effects.

Regardless of the compaction method used, optimum water content always is lower when asphalt is present in the mixture. It appears that replacement of compaction water or liquid by asphalt depends somewhat on the density of the compacted specimen. No definite conclusions can be drawn from these data in this respect because the difference in specimen density, as obtained by the different compaction methods, is not sufficiently large.

Figure 10 shows the strengths or stability at different water or volatiles content during compaction and indicates that any of these methods may be used for testing soil-asphalt mixtures. Typical curves for each stability determination method are obtained for cured or immersed specimens. It is, therefore, believed that the choice of method should be based on convenience and expediency until a laboratory test method can be correlated with field performance. In this respect, it appears that the unconfined compressive strength and Marshall stability determinations may be the most convenient of presently available methods.

Additionally, it is apparent that the strength or stability curves for immersed specimens, as developed by these two methods, are sloping downward on the dry side of the optimum compaction liquid content. It is believed that this represents a more complete and realistic picture, because mixtures at such low compaction liquid contents are less stable against water action than the mixtures compacted at optimum liquid content. The two other methods fail to show these effects.

Effect of Secondary Additives on Properties of Asphalt-Stabilized Fine-Grained Soils

The required load-bearing qualities of the individual pavement layer depends not only on the intensity of loading and the strength of other layers, but also on the position and depth of the layer in relation to the other layers.

Very often, as in the case of secondary roads, surface-treated or seal-coated asphalt-stabilized soil constitutes the upper layers of pavement structure. This means that such stabilized soil layers must not only resist weathering but also must be strong enough to support and transmit fairly intense loadings.

In the case of water-sensitive, fine-grained soils, occasions may arise when such soils will not satisfy these conditions even when stabilized with proper amounts of asphalt. Conditions may be encountered where the soil-asphalt layer, although sufficiently waterproof, may not possess adequate strength. Or conversely, it may be strong enough but not sufficiently waterproof.

Earnest efforts were made to improve the waterproofing and load-bearing qualities of asphalt-stabilized cohesive soil mixtures. These efforts invariably concentrated on using materials which would be suitable for such purposes at relatively low concentrations.

In spite of such efforts, little is currently known about the basic factors affecting the behavior of such complicated systems. The purpose of the tests described here was to learn more about these systems and weigh the merits of using a secondary additive with asphalt in the stabilization of fine-grained, water-sensitive soils. In this study, only those secondary additives were used which have been introduced to some extent and with varying degrees of success in actual field applications or field tests.

With some of the more promising additives, all three soils and other tests in addition to the Marshall (for determining compaction and stability) were used. Inasmuch as the results obtained with different soils and test methods developed very similar

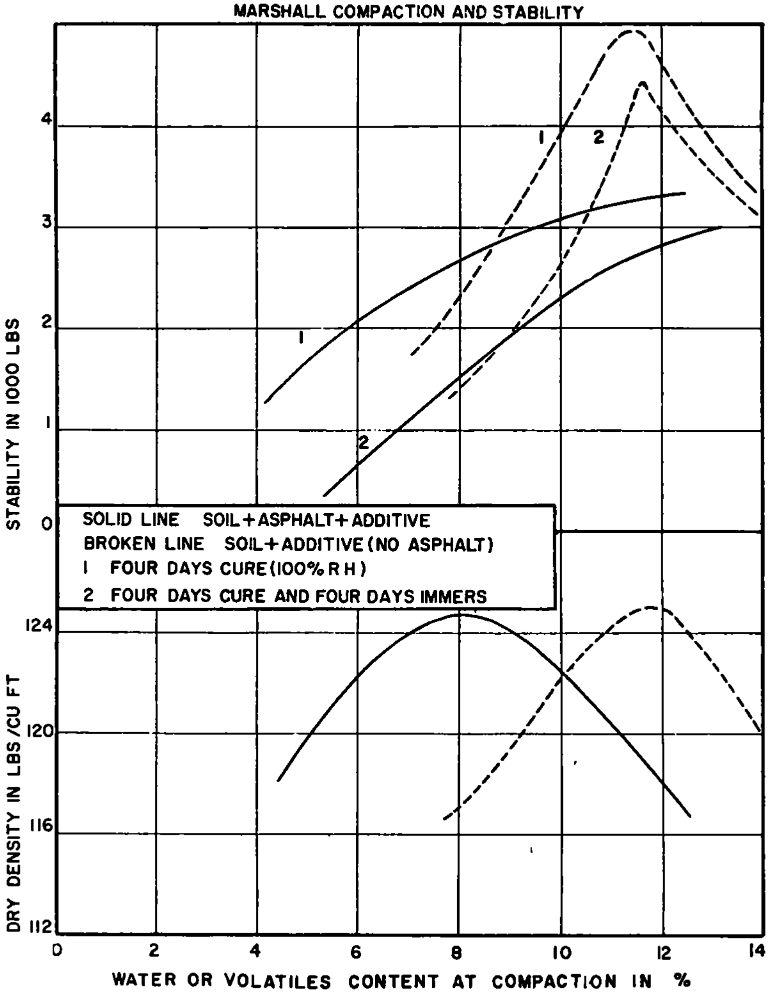

Figure 11. Density and stability of asphalt-stabilized and unstabilized Maryland soil treated with 3 percent portland cement.

trends, the accompanying graphs are limited to data from Marshall test results obtained with the three soils. The effects of secondary additives on soil-water mixtures with and without asphalt were studied. In the case of mixtures without asphalt, specimen density and strength properties are shown only for more effective additives. In all these tests only a single concentration of a given additive was used. Emphasis was placed on determining the effects of the single additive concentration over a range of water contents rather than on the effects of a wide range of additive concentrations. Although the effects of curing time on the properties of soil-additive and soil-additive-asphalt systems were investigated, the main portion of the results shows the properties of these mixtures after four days cure, and after a subsequent four-day complete immersion in water.

Effects of several environmental conditions on specimen properties (such as different immersion times or wetting and drying) also were studied and a part of the results as obtained for the more effective additives are shown in Figures 14 and 15.

Figures 11, 12 and 13 show the effects on Marshall density and stability of Maryland soil treated with normal portland cement, ortho-phosphoric acid and hydrated lime—stabilized and unstabilized with asphalt. Water absorption of specimens also was de-

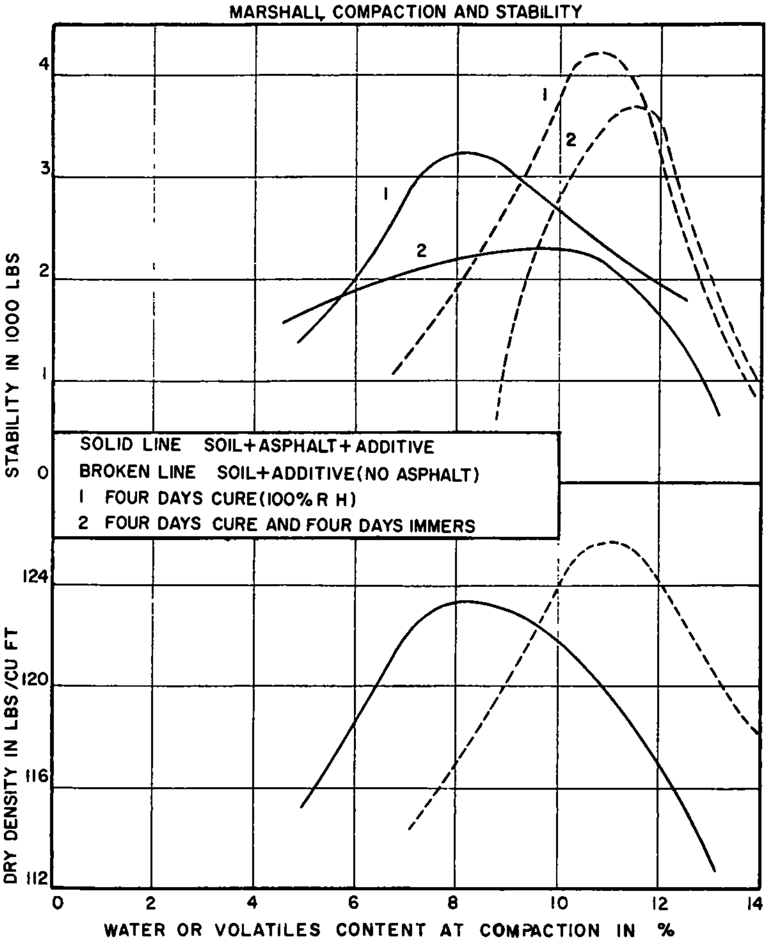

Figure 12. Density and stability of asphalt-stabilized and unstabilized Maryland soil treated with 2 percent H_3PO_4.

termined. Although these data are not shown, it should be noted that the presence of five percent asphalt reduced water absorption values of compacted mixtures by approximately 50 percent. Although in general the volumetric changes were small in the case of all additive-treated, immersed specimens, it was evident that asphalt also tended to reduce the specimen expansion values.

Comparison of Marshall dry densities of soil-water-additive mixtures (broken lines in Figs. 11, 12 and 13) with the densities of soil-water (no additive) systems (shown in the lower right corner of Fig. 9) reveals that different additives affect differently the densities of soil and water mixtures. This can be attributed to the structural or textual changes of soil-water mixtures caused by different additives. For example, soil particles suspended in water are dispersed by the addition of phosphoric acid. On the other hand, the same soil in the presence of hydrated lime exhibits strong aggregation or flocculation tendencies. In the case of normal portland cement, such changes are not as pronounced. These changes in soil particle structure can be seen visually or can be determined by measuring the sedimentation rates or final sedimentation volumes of soil particles suspended in water. Changes caused by different additives also are evident at lower water contents and can be detected by measuring changes in plasticity indices. These differences in soil structure also are reflected in compaction characteristics of different mixtures. It is evident that dispersed systems compact to a higher

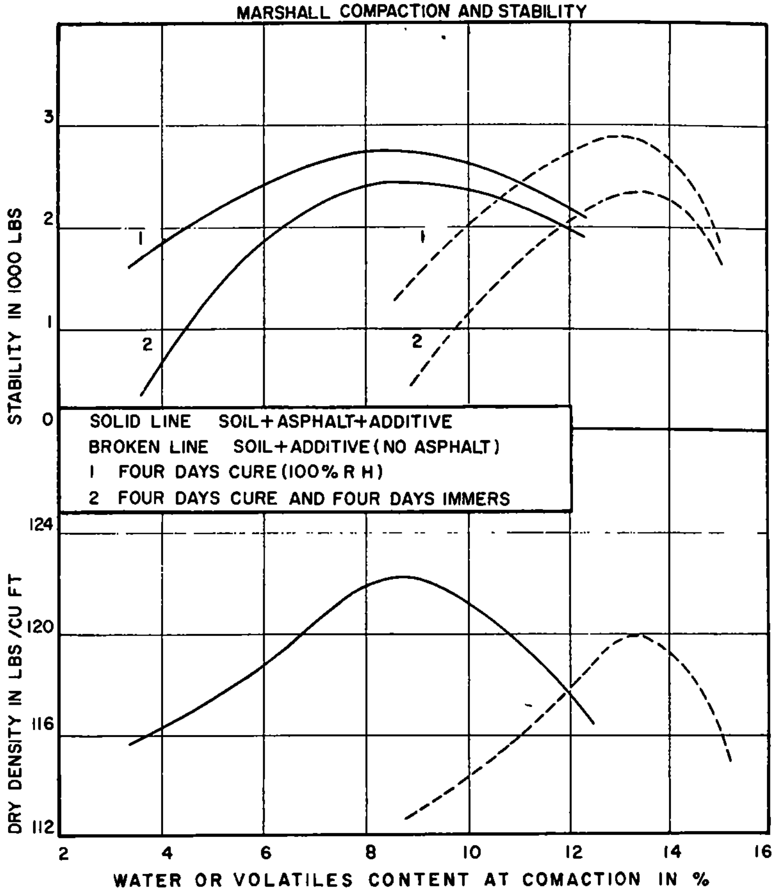

Figure 13. Density and stability of asphalt-stabilized and unstabilized Maryland soil treated with 4 percent hydrated lime.

density than the flocculated systems when an impact compaction method, such as Marshall, is used. These density differences are reduced when, for example, a compaction method such as AASHO T 180, having more kneading action, is used. Evidently, the changes in soil structure as caused by different additives are accompanied by the changes in elastic or plastic behavior of the soil mass.

As shown in Figures 11, 12 and 13, the use of different additives in Maryland soil-water (no asphalt) mixtures results in a difference in maximum specimen density for standard Marshall compaction of approximately 6 pcf. On the other hand, these figures also show that, when five percent asphalt is present in these mixtures, differences in maximum densities are reduced to 2½ pcf. Thus, it appears that asphalt tends to minimize the effects on compaction characteristics caused by different additives. This does not necessarily mean that asphalt promotes a reversal of soil-structure changes caused by different additives. It merely indicates that, particularly in the case of flocculated soil systems, the lubricating effects of asphalt films are more important for densification than the changes in soil structure caused by an additive such as hydrated lime.

Typical reduction of optimum compaction liquid content when asphalt is present in the mixture is also clearly shown in these figures. The magnitude of this partial replacement of compaction water by asphalt varies only slightly for different secondary additives.

The upper graphs of Figures 11, 12 and 13 indicate that approximately three-fold increases in the strengths of immersed specimens can be realized by using different

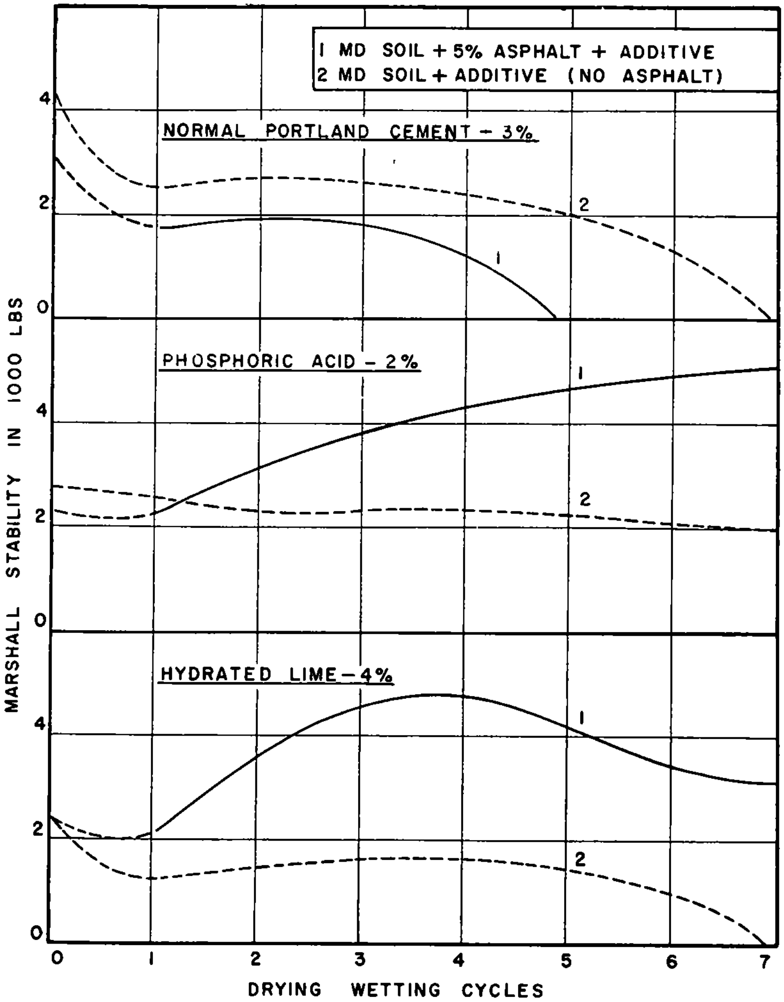

Figure 14. Effect of drying and wetting on stability of asphalt-stabilized and unstabilized Maryland soil treated with different cementing additives.

cementing additives in cohesive soil and asphalt mixtures. These figures also indicate that the additives reduce the water sensitivity of compacted soil-asphalt mixtures. In general, it appears that the reduction in strengths caused by complete immersion in water is lower in the specimens that are compacted at, or higher than, the optimum compaction liquid contents.

Comparison of stability characteristics of soil-additive mixtures reveals that maximum strengths of mixtures without asphalt are often higher than the maximum strengths of mixtures containing five percent asphalt. In the case of phosphoric acid and portland cement these differences appear to be more pronounced than in the case of hydrated lime. Additionally, it is evident that, with the exception of portland cement-asphalt systems, the maximum stabilities occur at the optimum water or liquid contents (that is, also maximum dry densities). The optimum water or liquid contents and maximum densities represent the points on density curves at which the distances between individual soil particles are the shortest. It may be assumed that such a condition is more favorable for the occurrence of cementing reactions between soil constituents and cementing additives. The reasons for the different behavior of portland cement-asphalt mixtures are not entirely clear. Perhaps asphalt retards hydration of cement more pronouncely at optimum compaction liquid content, or perhaps the water content at such a point is not sufficient for complete hydration of the cement.

Another significant difference in strength characteristics of soil-additive systems in the presence of asphalt is the shape of the strength curves of cured and immersed specimens. In the presence of asphalt, these curves flatten out over an extended range of water contents. Conversely, in the absence of asphalt, these curves have pronounced peaks at optimum water or volatiles contents. This, as in the case of soil-water (no additive) systems, suggests the moderating effects of asphalt. Flat stability curves indicate that the compacted mixtures are less susceptible to variations in compaction water or liquid contents. This may represent a considerable asset in construction and performance of such mixtures.

In addition to the density and strength properties and the effects of water on these properties, the influence of the environmental conditions, such as may be encountered during the pavement service, is of great importance for complete evaluation of stabilized soils. Drying and wetting is one environmental condition. Figures 14 and 15 show the effects of drying and wetting on Marshall stability and water absorption of Maryland soil treated with three different cementing additives. In these tests, specimens with and without asphalt were compacted at slightly higher than optimum water or liquid con-

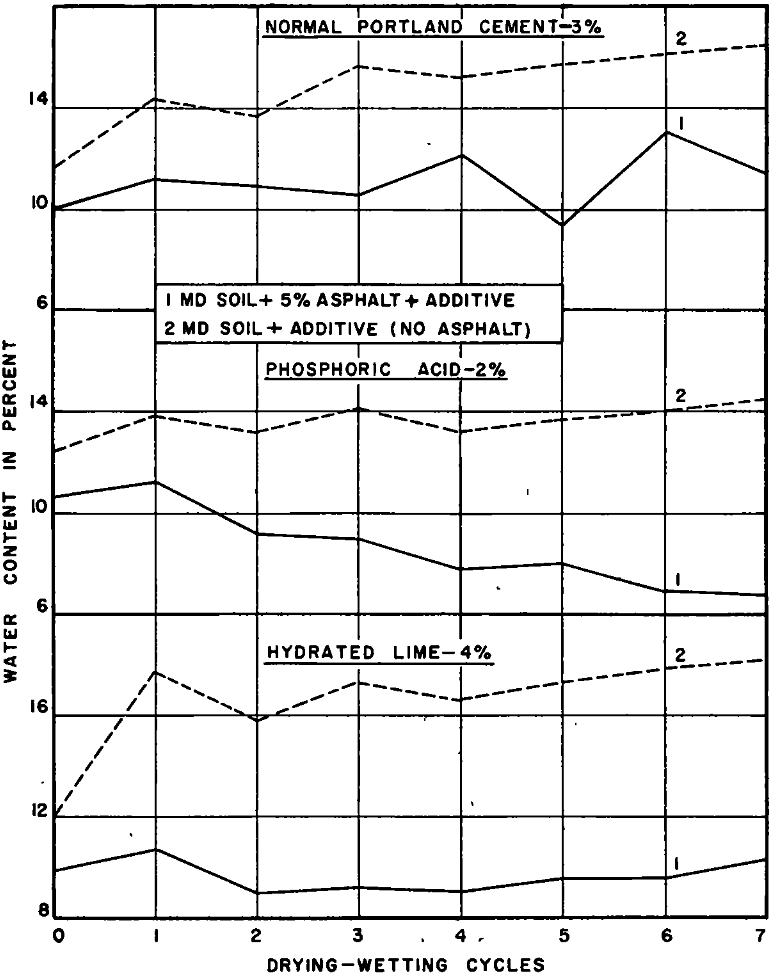

Figure 15. Changes in water content during drying and wetting of asphalt-stabilized and unstabilized Maryland soil treated with different cementing additives.

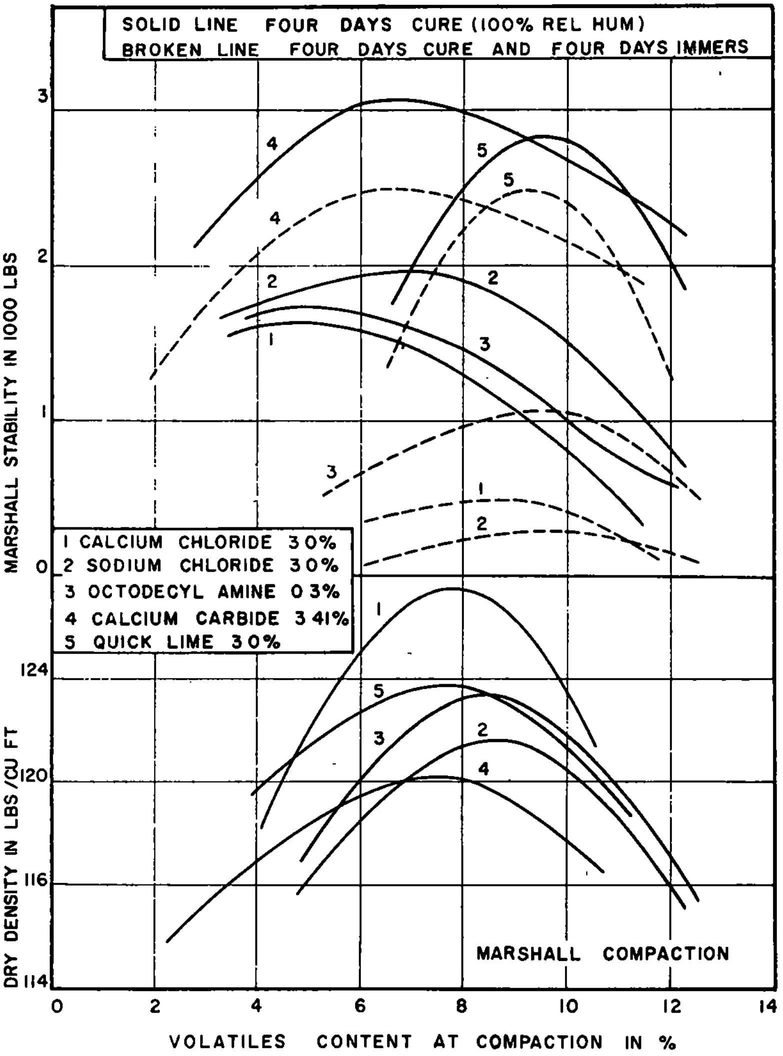

Figure 16. Effect of different additives on Maryland soil stabilized with five percent asphalt.

tent. All specimens were subjected to drying and wetting after curing for four days in 100 percent relative humidity chamber. Four days drying followed by four days complete immersion in water at room temperature constituted one drying-wetting cycle. Stability and water absorption were determined after four days immersion.

The results thus obtained illustrate definite advantages which can be derived from the conjunctive use of asphalt and phosphoric acid or hydrated lime.

Marshall stabilities of phosphoric acid-asphalt specimens actually increased with drying and wetting and after seven such cycles reached the level of 6, 000 lb. Stabilities of specimens treated with phosphoric acid (no asphalt) when wetted and dried did not change appreciably and remained at approximately 2, 000 lb strength level.

Specimens containing hydrated lime and asphalt increased in strength up to three or four cycles and then tended to weaken with subsequent wetting and drying. Specimens containing lime alone disintegrated after seven drying and wetting cycles.

Specimens containing portland cement at the compaction water contents investigated did not benefit by the presence of asphalt. In fact, they became less stable against

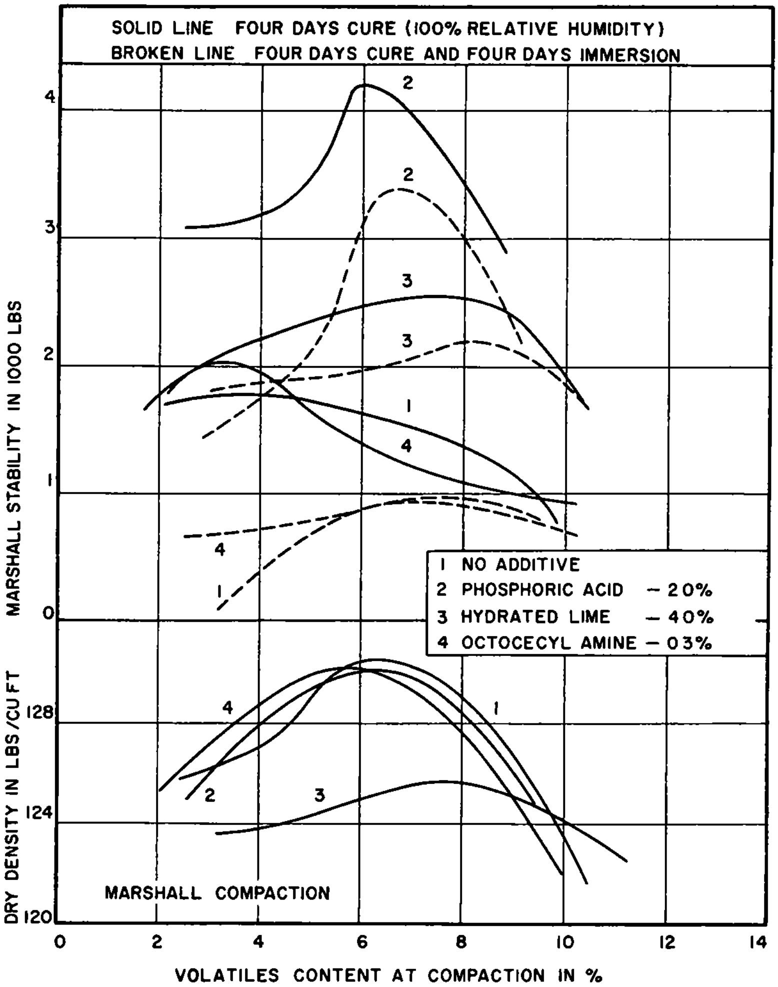

Figure 17. Effect of different additives on Massachusetts soil stabilized with five percent asphalt.

drying-wetting when five percent asphalt was present in the mixture. These specimens were completely destroyed after four cycles, as the specimens containing portland cement alone were destroyed after six or seven drying-wetting cycles.

It is interesting to note that the water absorption (Fig. 15) followed closely the strength changes as affected by the drying-wetting cycles. In all cases, water absorption of specimens containing asphalt was always lower than water absorption of specimens without asphalt.

The findings of this drying-wetting study were substantiated by a small-scale test section constructed on the grounds of The Asphalt Institute. This test section consisted of eight different test panels containing three cementing additives each, with and without asphalt. A somewhat coarser Maryland soil (although similar in mineralogical composition to that used in laboratory tests) was used in the test panels. Determinations of strength and water content, made over a period of several months, indicated that, with the exception of portland cement-asphalt mixtures, panels containing asphalt were considerably more stable against natural weathering elements. Water fluctuations in test panels with asphalt were reduced by approximately 50 percent.

Figure 16 shows the effects of five other additives on Maryland soil stabilized with five percent of residual asphalt. Dry densities and stabilities of cured and immersed

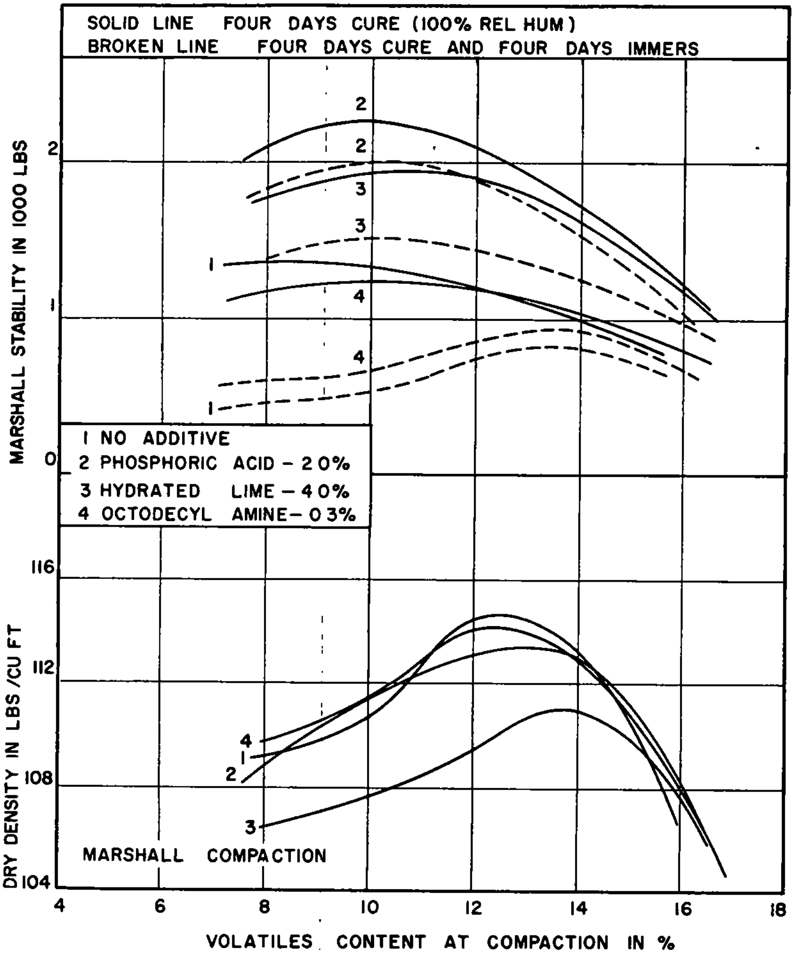

Figure 18. Effect of different additives on Mississippi loess stabilized with five percent asphalt.

specimens compacted by standard Marshall procedures are shown. With the exception of sodium and calcium chlorides, the other three additives were pre-mixed with asphalt cutback before being added to the soil.

It is evident that specimen densities, as affected by different additives, vary over a wide range. It should be noted that the variations in density were much more pronounced when these additives alone (no asphalt) were used with the same soil.

Stability data again indicate that cementitious additives (quicklime and calcium carbide) are far more effective for stabilizing fine-grained soils. These two additives, after reaction with water, produce hydrated lime (calcium hydroxide) which then reacts with the soil constituents.

It should be noted that, for these two additives, the optimum water contents (as shown in the lower half of Fig. 16) are considerably lower than the water contents used in mixing. The reason: some water is consumed in the instantaneous reaction with the additives and some is evaporated due to heat generated by these reactions. Because of the reactions with water, these two additives could conceivably be used advantageously with very wet soils. Compaction of soils treated with these materials should be delayed until the reactions are complete.

Stabilities of immersed specimens indicate that no benefits can be derived from the conjunctive use of asphalt and additives such as calcium or sodium chlorides. In fact, in the case of these two additives, and particularly in the case of sodium chloride, detrimental effects on soil stabilization with asphalt are evident.

Figures 17 and 18 show the effects of a few selected additives on the properties of two other soils; namely, Massachusetts clayey silt and Mississippi loess stabilized with five percent of asphalt. Although other additives were tried with these two soils, only the effects of hydrated lime, ortho-phosphoric acid and octodecyl amine are shown. For comparison, densities and strengths of soils stabilized with five percent asphalt (without additives) are also included (Line 1).

It is evident that the effects of different additives are very similar for all three investigated soils. For example, it is seen that ortho-phosphoric acid and octodecyl amine do not affect the densities of asphalt-stabilized compacted soils, whereas hydrated lime tends to lower these densities. Although the data for the Massachusetts and Mississippi soils are not shown, tests indicated that the presence of asphalt tended to minimize the effects of different additives on the density of compacted specimens, just as in the case of Maryland soil.

From the viewpoint of stabilities, it appears that ortho-phosphoric acid is the most effective additive. Maximum stabilities of more than 4, 000 lb were obtained for Massachusetts soil stabilized with five percent asphalt and treated with two percent of this additive. In the case of Mississippi loess, stabilized in the same manner, maximum stabilities of more than 2, 000 lb are shown. Similar trends are shown for asphalt-stabilized, hydrated lime-treated specimens, although the cured and immersed specimen stabilities are generally lower than for ortho-phosphoric acid. As far as octodecyl amine is concerned, only moderate stability benefits are evident for the concentration levels used. Additionally, it appears that these benefits are somewhat more pronounced when drier than optimum water content soils are mixed and compacted.

SUMMARY AND CONCLUSIONS

On the basis of results presented in this paper the following conclusions are offered:

1. Cohesive soil-water mixtures with and without asphalt respond similarly to compactive loads. Typical compaction curves (plotting density against water or liquid content) are obtained for both systems when a range of compaction water contents is used. Increasing the amount of asphalt in the mixture reduces proportionately the amount of water required to obtain maximum densities of compacted mixtures.
2. Effects of asphalt on density and strength of compacted mixtures appear to depend on the gradation characteristics of soil. With well graded soils, asphalt—even at low concentration levels—tends to lower the values of these properties. In the case of poorly graded soils, a decrease in density and strength becomes evident only at relatively high asphalt contents.
3. The principal function of asphalt in a cohesive soil is to waterproof the consolidated soil mass. This is achieved by coating soil particles, or aggregations of these particles, with water-repelling asphalt films rather than by plugging the void spaces in the soil mass. In more granular soils the asphalt may perform both these functions, and—in addition—it acts as an adhesive between soil particles.
4. Waterproofing effects are improved by the use of more asphalt. The amount of asphalt to be used in a given soil should be governed by the waterproofing effects as reflected in the strength changes of specimens subjected to the action of water. Different soils require different amounts of asphalt for most effective waterproofing.
5. In view of different effects of asphalt on different cohesive, fine-grained soils, each soil should be tested individually to determine the most suitable composition of mixture. Different water and asphalt contents should be tried.
6. Because no correlation between test properties and performance of asphalt-stabilized fine-grained cohesive soil is available at the present time, and because all four testing methods used in this study indicated the same or similar trends in test results, the selection of a laboratory testing method should be based on simplicity and expediency of the method. On this basis it appears that unconfined compressive strength measurement of specimens compacted by AASHO T 180 method and Marshall density and stability test methods are more suitable.

7. Pronounced benefits can be derived from the conjunctive use of asphalt and certain secondary additives in the stabilization of fine-grained soils. Use of asphalt with cementitious additives, such as ortho-phosphoric acid, hydrated lime and normal portland cement, appears to be most suitable for such purposes. In such systems, the additive strengthens and the asphalt waterproofs the compacted soil mass.

8. Asphalt-stabilized, additive-treated fine-grained soil is far stronger and more waterproof than the same soil stabilized with asphalt alone. In general, it is believed that the strengths of soil-asphalt-additive mixtures are more than adequate for use in base courses or, when surface-sealed, for secondary roads.

9. With the exception of portland cement, the systems containing asphalt and cementitious additive respond to severe drying-wetting conditions more favorably than the similar soil mixtures with the additive alone. At compaction water contents around optimum these systems actually increase in strength when repeatedly wetted and dried, as the systems without asphalt tend to disintegrate.

10. Asphalt may tend to retard the additive-soil reactions, but the results indicate that soil-additive systems containing asphalt are less sensitive to variations in compaction water contents. Such moderating effects of asphalt could be useful during construction operations. It may mean that use of asphalt would permit more time for mixing and compaction of soil-additive mixtures and also would minimize the detrimental effects of inadequate distribution of mixture components.

11. The effects of additives which modify the soil structure or are intended to enhance the waterproofing qualities of asphalt are far less pronounced than the effects of cementitious additives. In some cases, some of these additives actually may be detrimental to stabilization of soil with asphalt.

12. Careful laboratory testing is a prerequisite to determining the proper asphalt, additive and water contents to be used with a given cohesive soil. Provided the compaction characteristics of soil are known, on the basis of information presented in this paper, the number of specimens and tests required to determine a satisfactory mixture composition could be appreciably decreased.

Discussion

MANUEL MATEOS and CARLOS de SOUSA PINTO*, Research Associate and Graduate Assistant, respectively, Soils Research Laboratory, Engineering Experimental Station, Iowa State University, Ames — This discussion presents the results of a limited investigation on the use of lime as an auxiliary additive for improving the stabilization of soils with cutback asphalts. The testing procedure used differed from that described in the paper, and it is felt that the data obtained present additional information on the subject of asphalt stabilization.

The soil was a Kansan-age glacial till from southwestern Iowa. Characteristics of the soil are given in Table 2. The lime was commercial calcitic hydrated, $Ca(OH)_2$, from U. S. Gypsum Company. The asphaltic materials were MC-0 and MC-2 cutbacks from Texaco, Inc.

The amounts of cutback asphalt were calculated as a percentage of the weight of the oven-dry soil. The amounts of fluids were determined by oven drying and samples at 110 C; they include the weight of water plus volatiles.

Tests specimens were prepared from batches mixed in a Hobart C-100 kitchen mixer at the lower speed. The required amount of soil and lime was first machine mixed for 1 min, after which water was added and machine mixed for 2 min. The asphaltic material was poured into the bowl and premixed by hand. The cutback MC-0 was used at room temperature; MC-2 was previously heated to 150 F. Next, the materials were machine mixed for 1 min, the sides of the bowl hand scraped, and the materials mixed again for 1 min.

Immediately after mixing, 2-in. diameter by 2-in. high specimens were molded to near standard Proctor density (2). The specimens were cured as indicated in Table 3, then tested in unconfined compression with a load travel rate of 0.1 in. per minute.

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TABLE 2

PROPERTIES OF SOIL USED

Textural:	
Sand (2 to 0. 074 mm)	32. 7%
Silt (0. 74 to 0. 005 mm)	30. 8%
Clay (less than 0. 005 mm)	36. 5%
Consistency limits:	
Liquid limit	42%
Plastic limit	15%
Plasticity index	27
Chemical:	
Organic matter	0. 1%
Cation exchange cap.	20 me/100g
A. A. S. H. O. Classification	A-7-6(13)

The most favorable type of curing for compacted soil-asphalt mixtures is air drying to permit evaporation of moisture and other volatile products. For soil-lime mixtures it is best to retain the moisture, essential to the formation of cementitious reaction compounds. To properly evaluate the stability of a mixture after the required period of curing, it should be submitted to unfavorable conditions which simulate what may occur in the field. One of the most unfavorable conditions that may affect the stability of a stabilized soil base or subbase is water saturation.

The soil-lime-cutback combinations tested and the results obtained are presented in Table 3. The mixing water added was suggested by a previous investigation made with the same soil (3).

It should be mentioned here that during the process of mixing 6 percent MC-2 cutback with the soil, without lime, at a moisture content of 16 percent, slightly below the optimum for maximum density, the materials formed an unyielding paste and mixing was discontinued. In previous tests a similar mixture gave such high resistance to mixing that it broke the mixer (3). When lime was added, even in the smallest amounts, the mixing process proceeded normally. These difficulties were not experienced with MC-0 cutback. This may be explained by the fact that MC-0 has a lower viscosity than MC-2. It is also possible that MC-2 lost its heat-induced fluidity when it came in contact with the unheated soil.

Taking into account that only one soil was used in a limited number of mixtures, the observation of mixtures and the results obtained suggest the following comments:

1. With respect to the mixing: The addition of small amounts of lime improves the mixing of asphaltic materials with cohesive soils. In the case of MC-2 cutback, it was impossible to mix it with the moist clay soils, but previous addition of 1. 5 or 3 percent lime to the soil made it possible to obtain a uniform mixture with the asphalt. Visual observations show that uniform mixtures were obtained when lime was used as additive, regardless of the quantity of lime used. With 6 percent MC-0, a good mix was obtained with and without lime as an additive.

2. With respect to strength: It was observed that only the specimens of mixtures with MC-2 stood one day immersion after 7 days of air curing. However, the specimens that withstood immersion did not retain much strength, the maximum being 87 psi for the mixture with 3 percent lime and 10 percent MC-2. Seven-day air cured specimens of mixtures with MC-0 failed during immersion. On the other hand, soil-lime specimens containing as little as 2 percent lime showed about 90 psi after 7 days moist curing and one day immersion. This may indicate that lime alone is more effective for soil stabilization than cutback asphalts with or without lime additives. There

TABLE 3

DATA AND RESULTS OBTAINED[a] WITH SOIL-LIME-ASPHALT MIXTURES

Mixture No	Lime	Asphalt		Dry Density (pcf)	Fluids Content (%)			Unconfined Compr Str (psi)	
		%	Type		At Molding	After 7 Days Air Curing	After Immersion in Water	7 Days Air Cured[b]	7 Days Moist Cured[b]
1	3	6	MC-2	105.0	14.8	2.8	14.9	30	ND
2	3	6	MC-2	103.2	16.2	2.6	20.0	20	ND
3	3	6	MC-2	102.5	18.4	2.8	15.0	60	ND
4	1.5	8	MC-2	105.8	15.6	2.5	20.0	15	ND
5	3	8	MC-2	105.3	16.0	2.5	10.6	60	ND
6	3	10	MC-2	102.0	16.8	3.0	11.5	87	ND
7	3	6	MC-0	105.2	17.2	2.0	ND	0	ND
8	1	6	MC-0	104.4	17.5	1.8	ND	0	ND
9	0	6	MC-0	104.7	17.5	1.8	ND	0	ND
10	6	0	None	102.2	15.0	ND	ND	ND	88
11	6	3	MC-2	102.0	15.0	ND	ND	ND	64
12	2	0	None	107.7	18.0	ND	ND	ND	99

[a] ND=not determined [b] Plus one day immersion

was also come correlation between the fluids content of the mixtures, after immersion, and the decrease in immersed strength. This shows the beneficial effects of the waterproofing characteristics of asphaltic materials.

A comparison of strengths for mixtures 1, 2 and 3 shows that higher strengths were obtained when the amount of fluids was 18.4 percent, which is greater than the optimum for maximum density. This does not correspond with previous findings using a different test to evaluate stability (3).

Additions of small amounts of cutback asphalt to soil-lime mixtures (compare mixtures 10 and 11) may decrease strength by interference with the formation of the cementitious compounds. The consequent reduction of strength apparently is not compensated for by the beneficial effects, if any, of the asphalt.

The following conclusions are based on the observations and test results obtained in this investigation:

1. Lime can be used in asphalt stabilization of cohesive soils as a mixing aid.
2. However, the stabilization of Kansan till with cutback asphalts does not appear to be promising, even when the soil is treated with lime to facilitate mixing. The same strengths can be obtained with small amount of lime at a lower cost than using cutback asphalt.
3. The addition of small amounts of cutback asphalt to clayey soil-lime mixtures to improve stabilization does not appear promising with conventional methods of mixing. It is possible that lime pretreatment of soils might be promising in connection with techniques of mixing using foamed asphalt (1, 4).

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CLOSURE, V. P. Puzinauskas and B. F. Kallas— The authors welcome the comments by Messrs. Mateos and de Sousa Pinto. These comments were generated by their laboratory studies involving a single, highly plastic soil stabilized with lime and asphalt separately and, also, by both stabilizers conjunctively.

Their tests indicate that in regard to strength of stabilized compacted soil, lime alone may be a more effective stabilizer for the fine-grained, plastic soil than the asphalt alone. This agrees with the authors' findings, even when soils less plastic than the Kansan age glacial till were used. However, the discussers' hypothesis that lime alone may be a better stabilizer than asphalt-lime mixture does not agree with the authors' findings. Laboratory studies and outdoor test panels have indicated that soil-water mixtures strengthened by the lime treatment and waterproofed by asphalt films are stronger, or at least equal in strength, and more durable against weathering effects produced by environmental conditions.

Perhaps the basis for the disagreement lies in the discussers' assumption that drying is the most favorable type of curing for compacted soil-asphalt mixtures. Although this is true for granular, nonplastic soils, it is not necessarily true in the case of fine-grained, plastic soils. The validity of this point is illustrated by the curves submitted in the paper. It is seen that, because of greater water absorption, soil-asphalt mixtures compacted at lower than optimum water contents weaken considerably more than the same mixtures compacted at optimum or higher than optimum water contents.

Similar effects could be expected with dried, plastic soil-asphalt mixtures even when compacted at optimum water conditions. This is substantiated by some unpublished test results obtained in the authors' laboratory. Such effects appear to be more pronounced when insufficient amounts of asphalt are used to stabilize mixtures of fine-grained soils and water.

Furthermore, the authors believe that the comparison of dry-cured, soil-asphalt-lime with wet-cured soil-lime systems is not warranted. We agree with the discussers that whenever lime is present in the stabilized soil system, regardless of whether asphalt is present or not, wet-cure is necessary to achieve the optimum beneficial effects of this additive. The tests indicate that asphalt films in the compacted mixture reduce the rate of water evaporation, thereby allowing more time for the occurrence of the cementitious, pozzolanic reactions of lime.

To reiterate, although strength is an important property of the stabilized soil, it does not necessarily reflect the durability of weathering characteristics of the stabilized compacted soil mass. The authors' studies indicate, at least as far as the effects of wetting-drying are concerned, that soils stabilized conjunctively with lime and asphalt or with phosphoric acid and asphalt behave far more favorably than the same soil treated with these cementitious additives alone.

Resistance of Soil-Cement Exposed to Sulfates

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●SOIL-CEMENT has gained widespread acceptance as a stabilized base for both concrete and asphalt pavements. Many miles of highway construction, particularly in the arid west, pass through areas containing "alkali" soils. The destructive action of sulfates on portland cement concrete is well known, and extensive research has developed sulfate-resistant cement and sulfate-resistant concrete structures. Soil-cement also contains portlant cement, but the resistance of soil-cement to sulfates has not been established.

The Portland Cement Association supported a research project at Utah State University during the past several years to establish the durability of soil-cement exposed to sulfate salts, and also the durability of soil-cement made with sulfate soils. The first part of the investigation was a study of the durability of soil-cement specimens, fabricated from soils containing negligible amounts of sulfate salts, when exposed to a solution of sodium and magnesium sulfate in the laboratory and to sulfate soils in the field. Laboratory specimens were stored in a 2 percent solution of sulfate salts while companion field specimens were buried in sulfate soil south of Huntington, Utah, adjacent to the location of Interstate 70.

In the second part of the investigation, soil-cement specimens were made with the same fine-grained and coarse-grained soils, but various percentages of sodium and magnesium sulfate were mixed with the soil-cement at the time of fabrication. The first part of the investigation studied the resistance of soil-cement to sulfate attack from outside sources, whereas the second part of the investigation was designed to determine the durability of soil-cement with sulfate salts mixed integrally.

MATERIALS

Portland Cements

Experience with portland cement concrete indicates that the durability of concrete when exposed to sulfates, depends to a large extent on cement composition and the richness of the mix. Nine different portland cements were used in this investigation. These included three different types, Type I, Type II, and Type V, with three different brands of cement of each type. The cements were secured primarily from the Rocky Mountain area with the exception of two Type V cements, which came from the West Coast. The chemical analysis and the calculated compound composition of each cement are given in Table 1.

Soils

Soils used to fabricate soil-cement specimens included a coarse-grained soil and also a fine-grained soil. Both of these soils were obtained from the drainage area of the Blacksmith Fork River in Cache Valley, Utah. Material retained on the No. 4 sieve was removed from both soils.

Approximately 12 percent of the total coarse-grained soil sample passed the No. 200 sieve, whereas 90 percent of the fine-grained soil passed this sieve size. The grading analysis of both soils is given in Table 2. Both soils were tested for soluble sulfates. The fine-grained soil contained 0.027 percent soluble sulfates, whereas the coarse-grained soil contained 0.014 percent of these salts.

TABLE 1

CHEMICAL ANALYSIS AND CALCULATED COMPOUND COMPOSITION OF CEMENTS

PCA Lot No	Cement Brand	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	SO_3	Na_2O	K_2O	Tot Alk as Na_2O	Loss on Ign	Ins Res	Free CaO	Fineness Blaine (sq cm per g)	Calculated Compound Composition, %*			
															C_3S	C_2S	C_3A	C_4AF
								TYPE I										
19691	I	21 05	5 63	2 75	62 70	2 76	2 65	0 17	0 67	0 61	1 70	0 40	0 76	3,550	45 9	25 8	10 3	8 4
19693	E	21 58	5 67	2 29	64 16	1 83	1 79	0 16	1 02	0 83	1 52	0 17	1 56	3,025	50 6	23 8	11 2	7 0
19695	M	22 06	5 64	2 80	64 00	1 05	1 69	0 13	0 80	0 66	1 67	0 15	1 02	2,970	46 1	28 6	10 2	8 5
								TYPE II										
19692	I	22 35	4 82	3 45	62 80	2 94	2 01	0 23	0 46	0 53	1 13	0 28	0 69	3,190	42 7	31 9	6 9	10 5
19694	E	22 83	4 77	2 83	64 54	1 56	1 91	0 09	0 64	0 51	1 08	0 09	0 70	2,945	47 6	29 6	7 9	8 6
19696	M	21.83	5 07	4 54	63 91	0 87	1 87	0 10	0 68	0 55	1 25	0 19	0 64	3,100	48 3	26 2	5 8	13 8
								TYPE V										
19697	M	23 44	3 64	3 43	64 24	0 86	1 73	0 10	0 45	0 40	1 96	0 20	1 06	3,450	49 0	30 3	3 8	10 4
19698	R	23 82	3 25	3 21	63 30	2 08	2 09	0 06	0 54	0 42	1.66	0 06	0 76	3,270	44 2	35 0	3 2	9 8
19699	V	24 34	3 18	2 82	62 76	3 53	1 63	0 10	0 30	0 30	1 26	0 08	0 62	3,170	40 4	39 4	3 7	8 6

*Not corrected for free CaO

TABLE 2

GRADING OF COARSE- AND FINE-GRAINED SOIL

Soil	Sieve Size	% Passing
Coarse-grained	No. 4	100
	No. 10	81
	No. 40	41
	No. 200	12
Fine-grained	No. 4	100
	No. 10	100
	No. 40	98
	No. 200	90

Sulfate Salts

The 2 percent solution of sulfates, in which laboratory specimens were stored, was made up of 75 percent sodium sulfate by weight and 25 percent magnesium sulfate. An anhydrous sodium sulfate, Na_2SO_4, and a dried powder of magnesium sulfate, $MgSO_4$, were used. The equivalent SO_4 content of the solution was 1.42 percent by weight.

Samples of the soil in which the field specimens were buried were tested for sulfate content. Specimens were buried in a fine silty sand which was found to contain 1.77 percent SO_4. A second sample from a slightly different location in the same area contained 1.46 percent SO_4 by weight.

PROCEDURES AND METHODS

Standard tests of the American Society for Testing Materials, and the American Association of State Highway Officials, along with the recommendations and publications of the Portland Cement Association were used in establishing the proper testing methods and procedures. The following standard tests were used in this investigation:

Test	ASTM Designation	AASHO Designation
Moisture-density relationships of soil-cement mixtures	D558-57	T-134-57
Freezing and thawing of compacted soil-cement mixtures	D560-57	T-136-57
Wetting and drying of compacted soil-cement mixtures	D559-57	T-135-57
Optimum cement content of compacted soil-cement	Portland Cement Association Manual	

In establishing the optimum cement content for soil-cement, the Portland Cement Association recommends testing in freezing and thawing and wetting and drying at a medium cement content (6 percent by weight of soil for coarse-grained soils and 10 percent cement by weight of soil for fine-grained soils). The optimum cement content should be sufficient, in each case, so the loss will not exceed the allowable loss suggested by AASHO (not more than 14 percent for coarse-grained soil, Type A-I, and not more than 10 percent for fine-grained soils, Type A-4).

Freezing and thawing and wetting and drying tests made with the soils used in this investigation showed that at a medium cement content the losses were only approximately one-half the allowable loss. Another set of specimens were made in which the cement content was reduced 2 percent in each case. It was found at these cement contents that the losses in freezing and thawing and wetting and drying were approximately twice that suggested by AASHO. It was decided, therefore, that the medium cement content for the coarse-grained soil should be 6 percent cement by weight of soil. The medium cement content for the fine-grained soil should be 10 percent.

In order to establish the influence of richness of mix, cement contents lower than medium and higher than medium were used with each cement. Coarse-grained soil cement specimens were made with 3, 6, and 10 percent by weight of soil. Fine-grained soil-cement specimens were made with 6, 10, and 14 percent portland cement by weight of soil.

A total of 54 different soil-cement mixtures were made in the first part of the investigation. Specimens from each of the 54 mixtures were stored in sulfate solutions in the laboratory and duplicate specimens were buried in sulfate soils near Huntington, Utah. All test specimens were molded at the predetermined optimum moisture content and at maximum density (Fig. 1). If any specimen did not fall within the range of maximum density, ± 3 pcf, or within optimum moisture, ± 1 percent, the specimen was discarded and another specimen molded.

All field specimens were cured in the laboratory for a minimum of 28 days before being buried in the field. Because of the long distance to the area where the field specimens were buried, it was necessary to bury them at one time, and a uniform curing period before being placed in the field could not be maintained. Field specimens were buried approximately 1 ft deep along a bank of sulfate soil close to Seven-Mile Creek south of Huntington, Utah. Photographs of this location and of soil-cement specimens after being buried are shown in Figures 2 and 3.

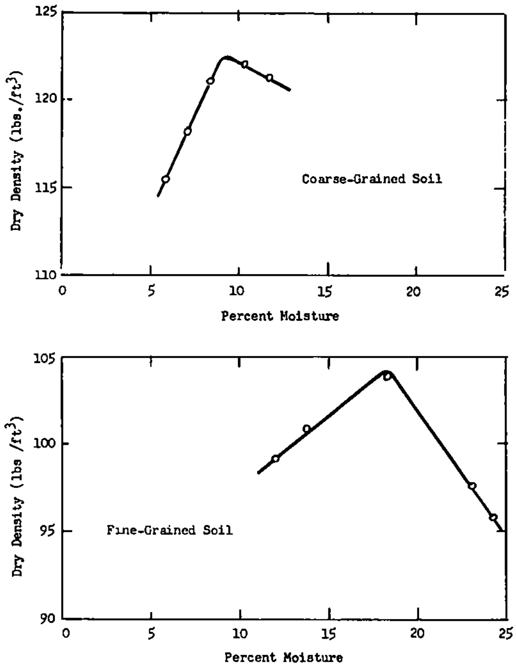

Figure 1. Moisture-density relationships.

In the second part of the investigation, soluble salts of sodium and magnesium sulfate were mixed directly in the soil-cement. The salts were first dissolved in mixing water and then added in amounts of 0, 0.5, 1, 2, and 3 percent by weight of the soil. Sulfate salts were combined in a proportion of 75 percent sodium sulfate and 25 percent magnesium sulfate. Higher concentrations of sulfate salts resulted in supersaturated solutions at lower temperatures. This necessitated heating the water before the entire amount of sulfate salt could be put into solution.

Tests used to measure the influence of various concentrations of sulfate salts in soil-cement included the compressive strength, freezing and thawing durability, and wetting and drying durability. The

Figure 2. Location of field specimens. Note white sulfate salts on surface.

Figure 3. Specimens uncovered after 1 year.

compressive strength was measured on 4-in. standard compaction cylinders after curing for 7 days, 28 days, 3 months, and 1 year in a moist cabinet. Standard ASTM tests and AASHO tests were used to measure the durability in wetting and drying and freezing and thawing.

DISCUSSION OF TEST RESULTS

First Phase

The primary purpose of the first part of the investigation was to determine the resistance of soil-cement specimens to sulfate attack under different exposure conditions. A photographic record, taken at different time intervals, was considered the best method of demonstrating progress of the attack of sulfate salts. Although this method does not give quantitative test results, it does record many aspects of deterioration which cannot be recorded by laboratory tests. The following procedure was used in making this record. The soil-cement specimens, both in the laboratory and in the field, were brushed and photographed at 6-month intervals. Progressive deterioration of typical soil-cement specimens are shown in Figures 4 to 12. These photographs are arranged so that all specimens fabricated with a typical brand of cement are included in one plate. The variables shown on each plate include laboratory and field specimens of three different cement contents made with both fine- and coarse-grained soils.

Photographs of sulfate deterioration indicate that sulfate attack on soil-cement varies with cement content, type of cement used, type of soil, and also type of exposure. Specimens containing low cement contents generally deteriorated from the outer surface of the specimen. The surface would become soft and flake off as the sulfate salts penetrated the specimen. The specimens containing high cement contents, however, became harder when exposed to sulfate solutions or buried in sulfate soils. The durability and strength of some specimens increased with sulfate exposure. As the inner part of other specimens expanded, cracks would appear and large pieces of the specimen would break off. Some specimens split along the compaction plane formed during molding. The hardening of specimens was more noticeable in laboratory specimens than in field specimens.

Specimens containing fine-grained soils deteriorated more rapidly in field exposure than in the laboratory. Specimens containing coarse-grained soils and low cement contents also deteriorated rapidly under field exposure. The difference between field and laboratory temperatures may account for these differences. The field specimens were not only required to resist the attack of sulfate salts, but in all probability were also subjected to an unknown number of cycles of freezing and thawing.

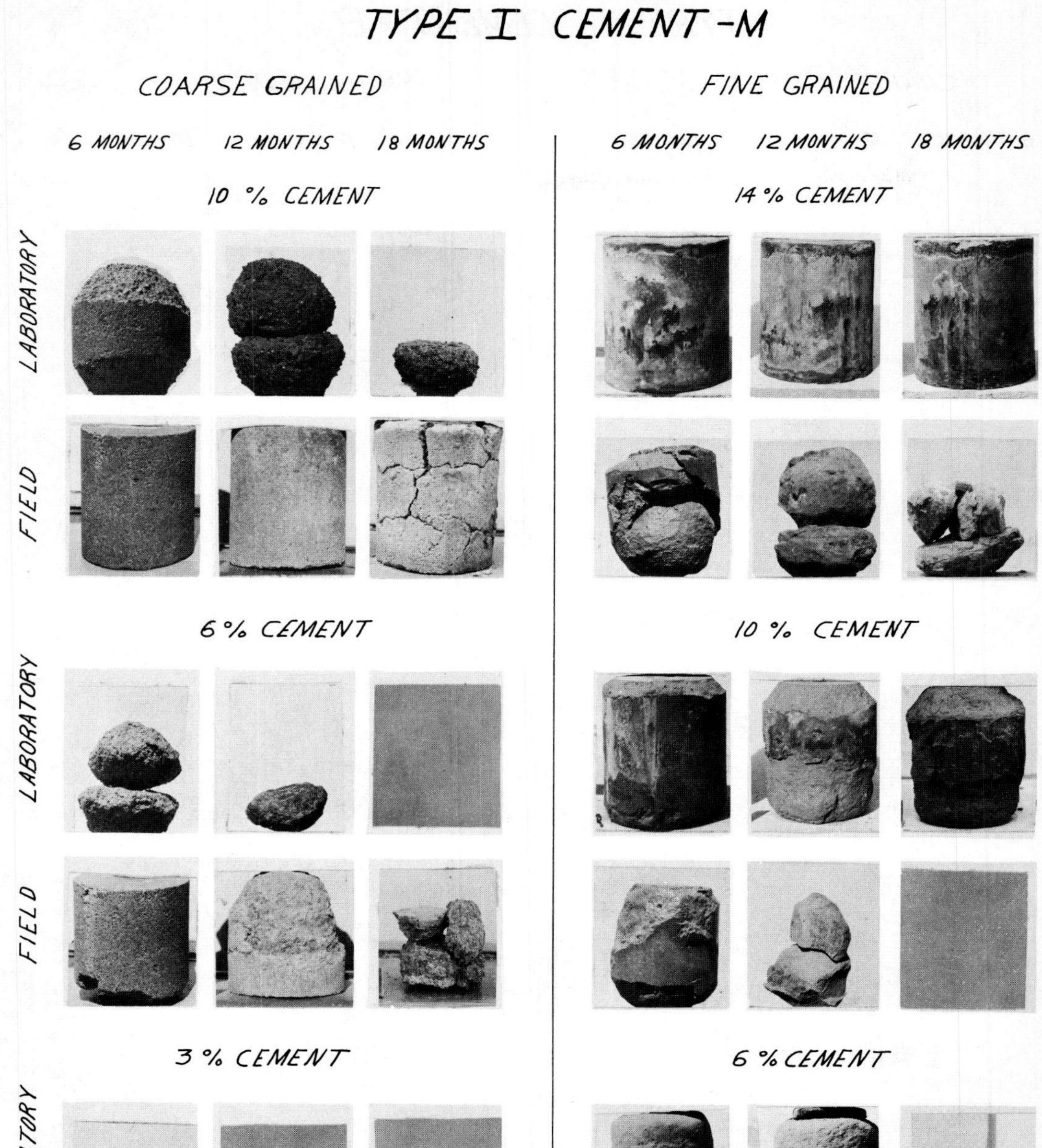
TYPE I CEMENT -M
COARSE GRAINED
FINE GRAINED
6 MONTHS
12 MONTHS
18 MONTHS
6 MONTHS
12 MONTHS
18 MONTHS
10 % CEMENT
14 % CEMENT
LABORATORY
FIELD
6 % CEMENT
10 % CEMENT
LABORATORY
FIELD
3 % CEMENT
6 % CEMENT
LABORATORY
FIELD

Figure 4.

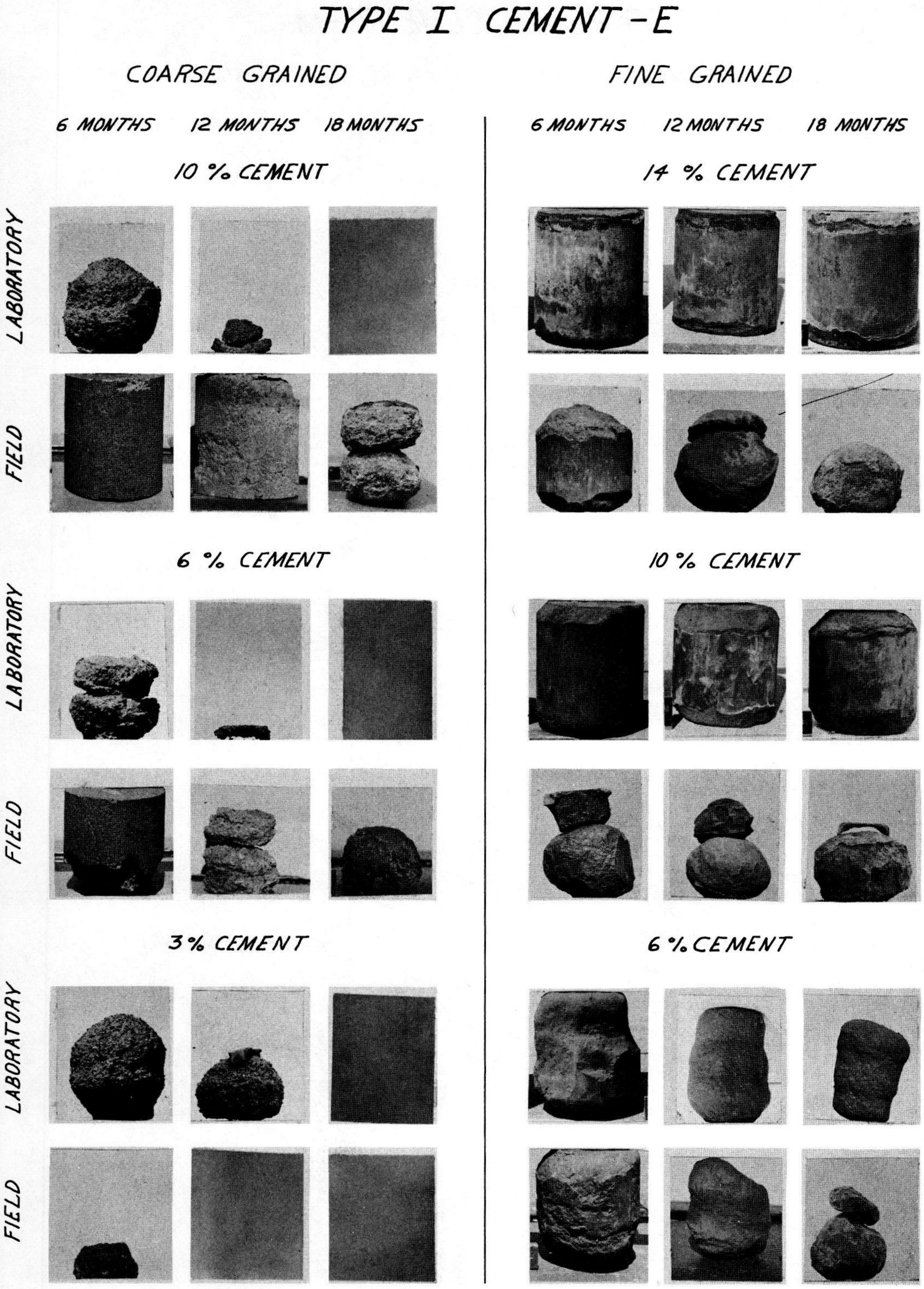
TYPE I CEMENT - E
COARSE GRAINED
FINE GRAINED
6 MONTHS
12 MONTHS
18 MONTHS
6 MONTHS
12 MONTHS
18 MONTHS
10 % CEMENT
14 % CEMENT
LABORATORY
FIELD
6 % CEMENT
10 % CEMENT
LABORATORY
FIELD
3 % CEMENT
6 % CEMENT
LABORATORY
FIELD

Figure 5.

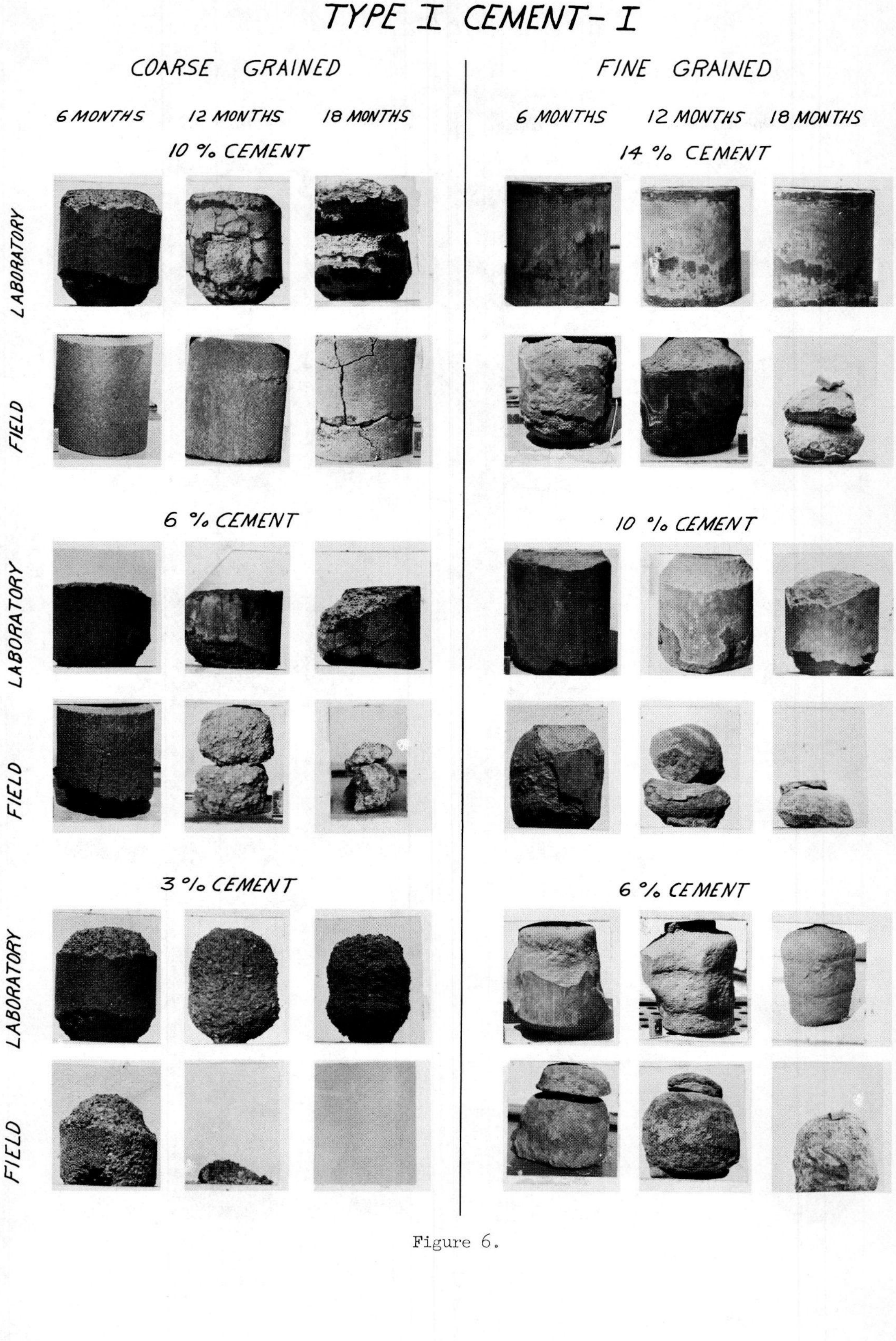
TYPE I CEMENT- I
COARSE GRAINED
FINE GRAINED
6 MONTHS
12 MONTHS
18 MONTHS
6 MONTHS
12 MONTHS
18 MONTHS
10 % CEMENT
14 % CEMENT
LABORATORY
FIELD
6 % CEMENT
10 % CEMENT
LABORATORY
FIELD
3 % CEMENT
6 % CEMENT
LABORATORY
FIELD

Figure 6.

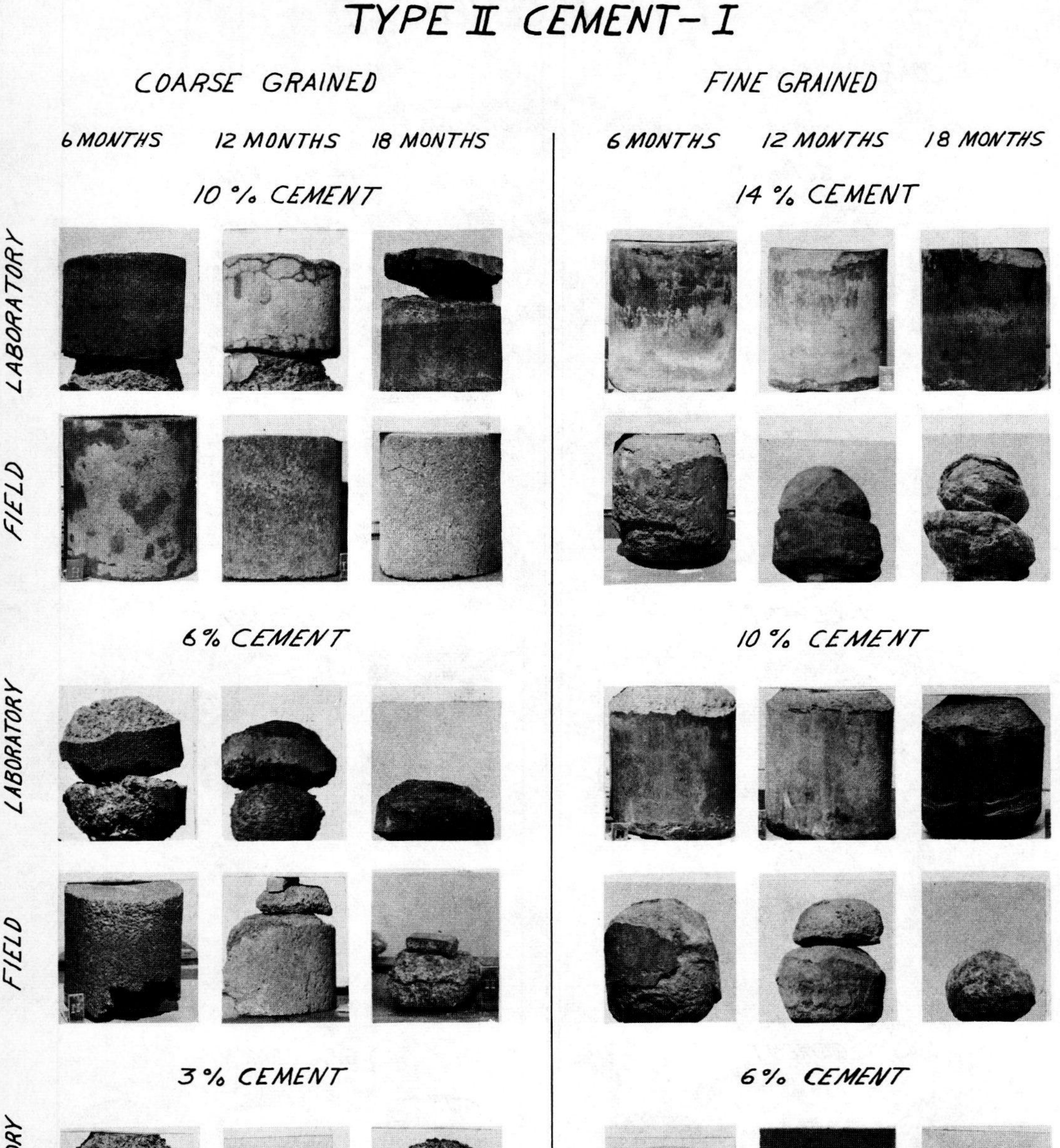
TYPE II CEMENT-I
COARSE GRAINED
FINE GRAINED
6 MONTHS
12 MONTHS
18 MONTHS
6 MONTHS
12 MONTHS
18 MONTHS
10 % CEMENT
14 % CEMENT
LABORATORY
FIELD
6% CEMENT
10 % CEMENT
LABORATORY
FIELD
3 % CEMENT
6 % CEMENT
LABORATORY
FIELD

Figure 7.

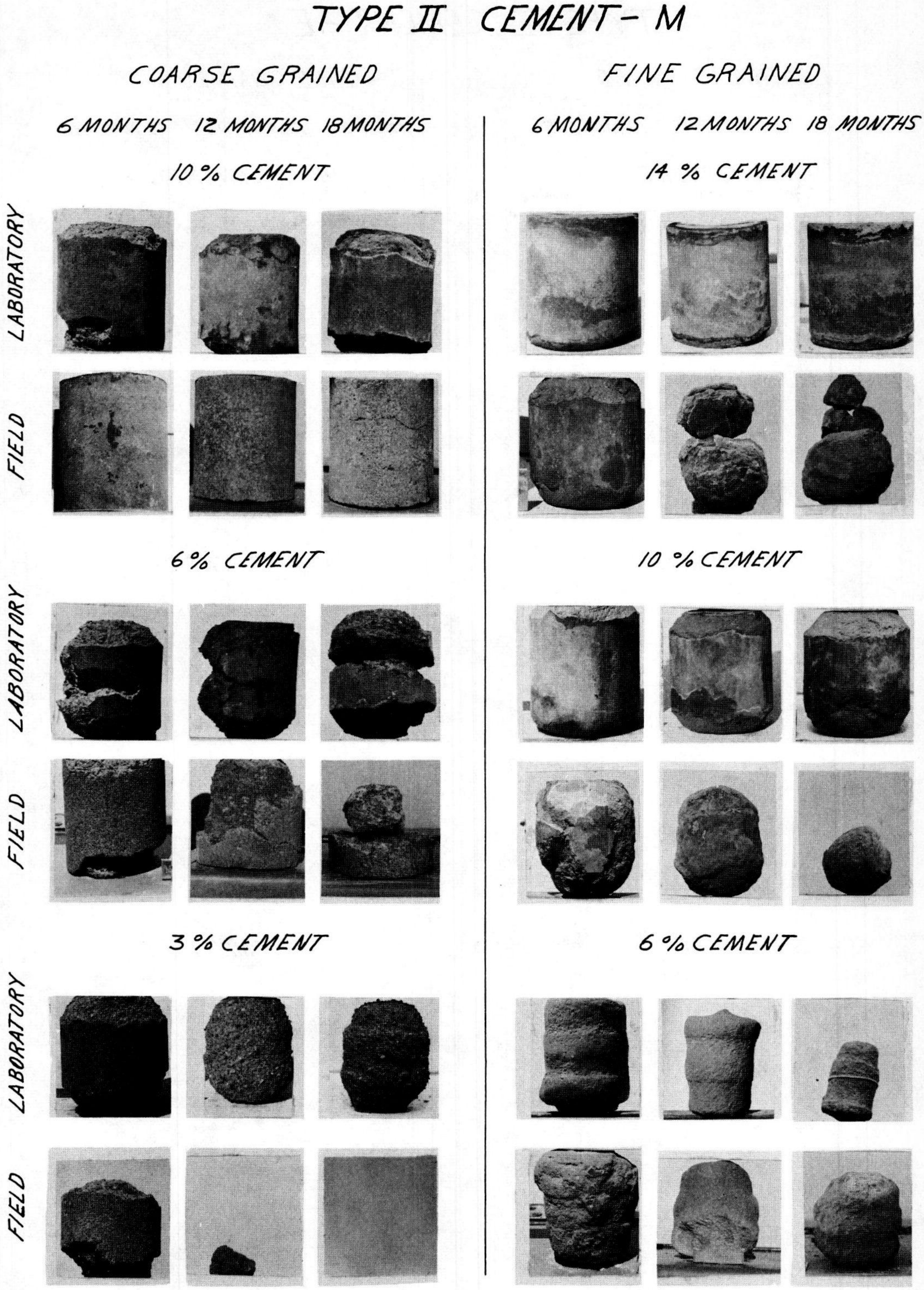
TYPE II CEMENT- M
COARSE GRAINED
FINE GRAINED
6 MONTHS
12 MONTHS
18 MONTHS
6 MONTHS
12 MONTHS
18 MONTHS
10 % CEMENT
14 % CEMENT
LABORATORY
FIELD
6 % CEMENT
10 % CEMENT
LABORATORY
FIELD
3 % CEMENT
6 % CEMENT
LABORATORY
FIELD

Figure 8.

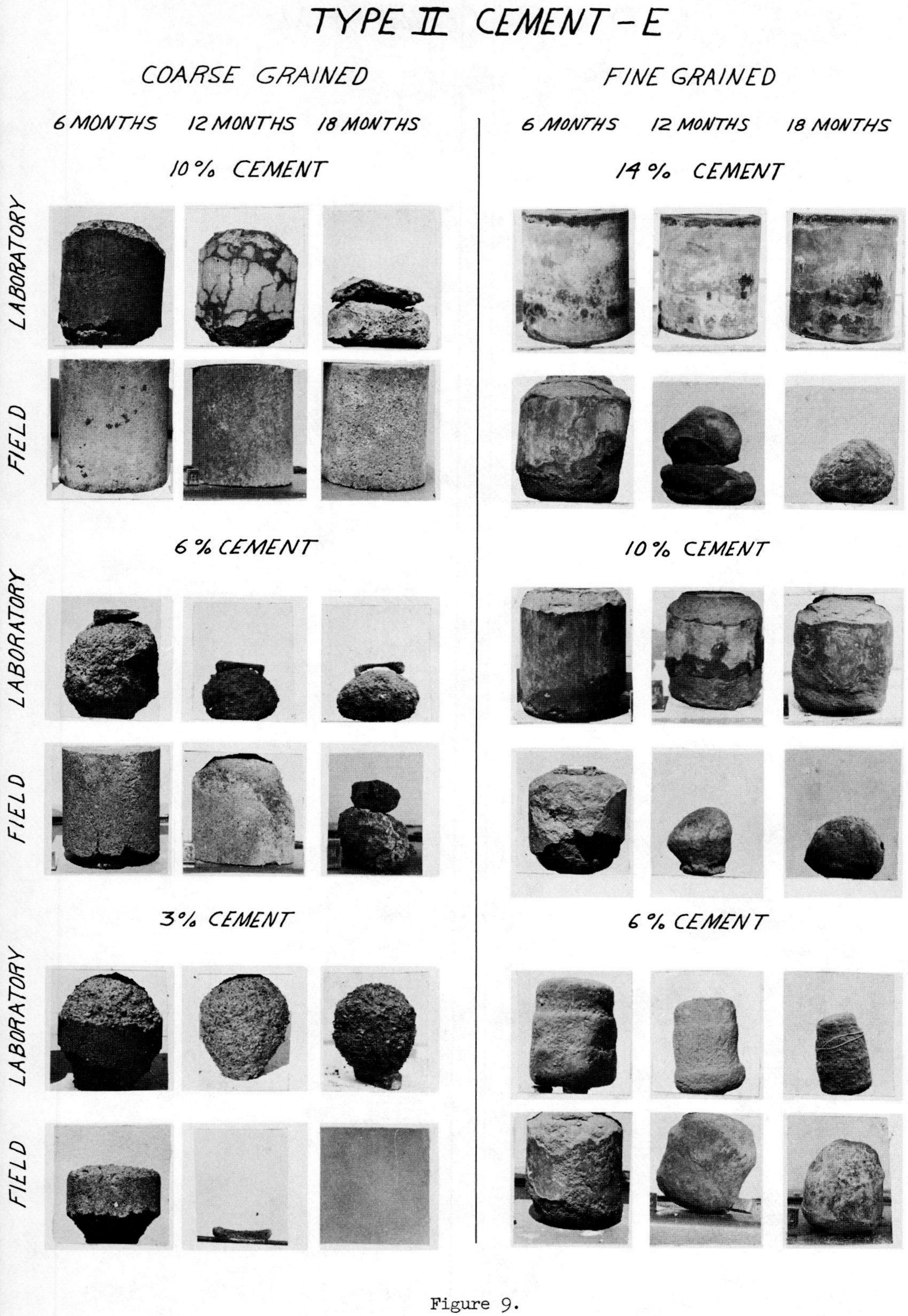
TYPE II CEMENT - E
COARSE GRAINED
FINE GRAINED
6 MONTHS
12 MONTHS
18 MONTHS
6 MONTHS
12 MONTHS
18 MONTHS
10% CEMENT
14% CEMENT
LABORATORY
FIELD
6% CEMENT
10% CEMENT
LABORATORY
FIELD
3% CEMENT
6% CEMENT
LABORATORY
FIELD

Figure 9.

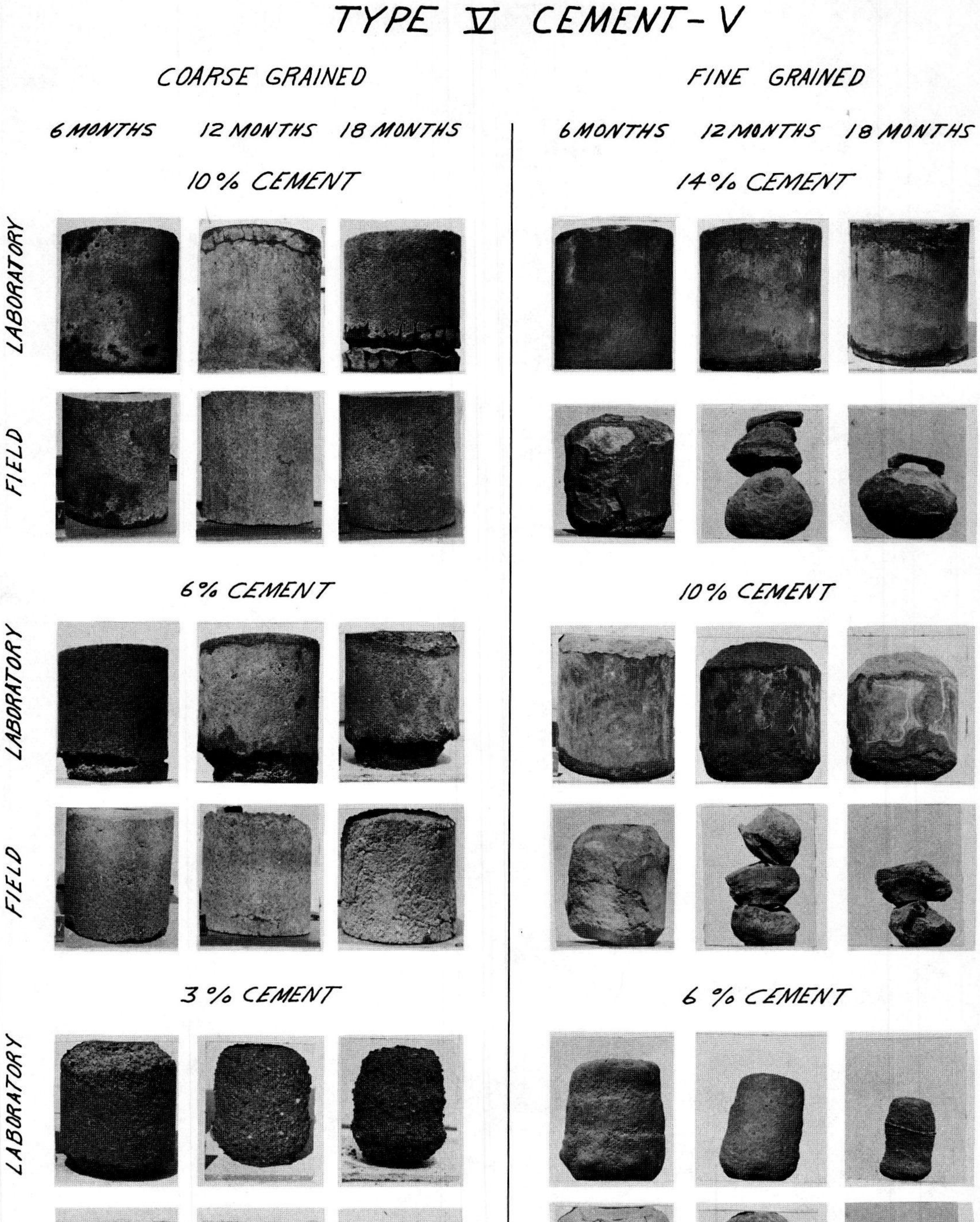
TYPE V CEMENT-V
COARSE GRAINED
FINE GRAINED
6 MONTHS
12 MONTHS
18 MONTHS
6 MONTHS
12 MONTHS
18 MONTHS
10% CEMENT
14% CEMENT
LABORATORY
FIELD
6% CEMENT
10% CEMENT
LABORATORY
FIELD
3% CEMENT
6% CEMENT
LABORATORY
FIELD

Figure 10.

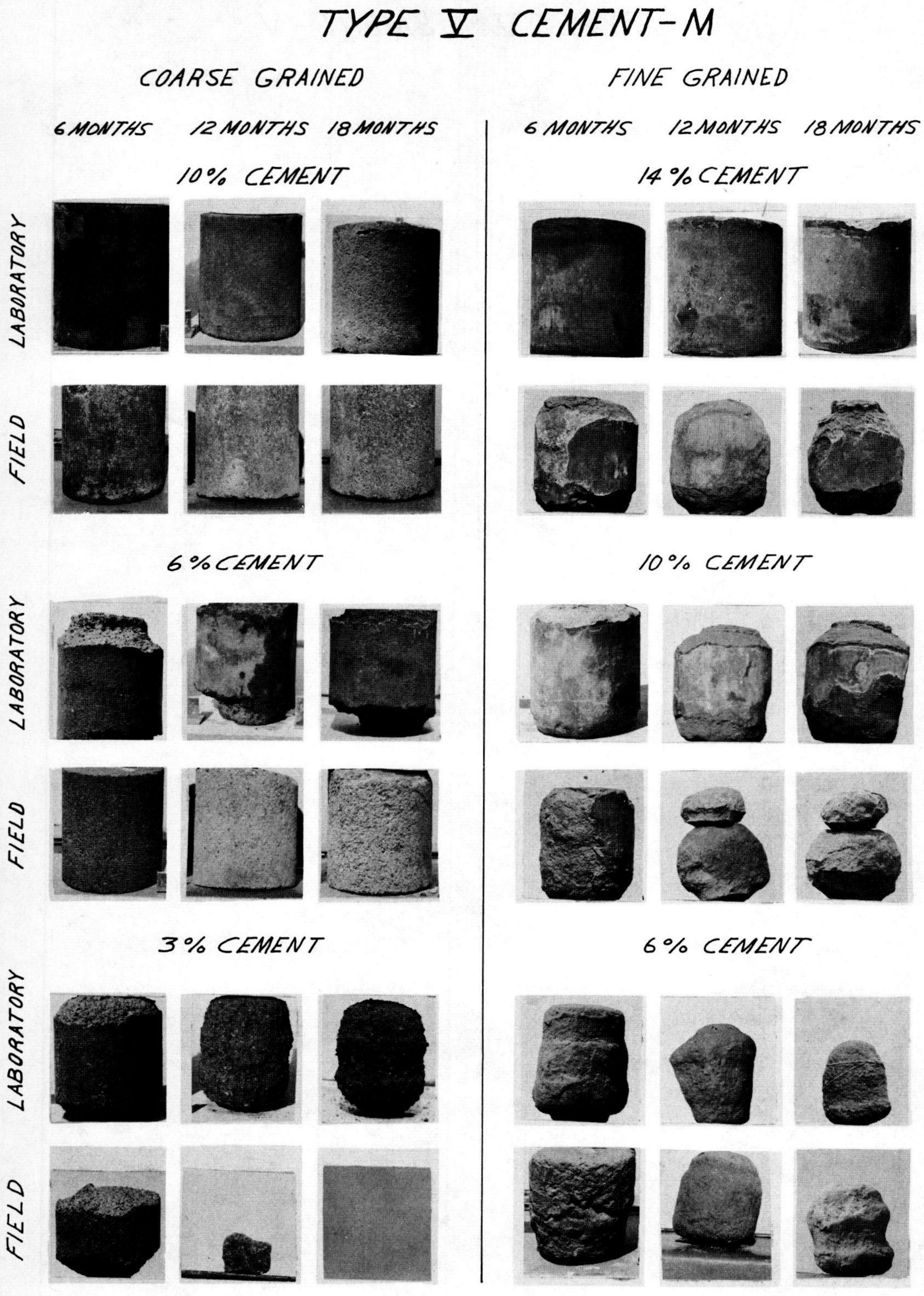
TYPE V CEMENT-M
COARSE GRAINED
FINE GRAINED
6 MONTHS
12 MONTHS
18 MONTHS
6 MONTHS
12 MONTHS
18 MONTHS
10% CEMENT
14% CEMENT
LABORATORY
FIELD
6% CEMENT
10% CEMENT
LABORATORY
FIELD
3% CEMENT
6% CEMENT
LABORATORY
FIELD

Figure 11.

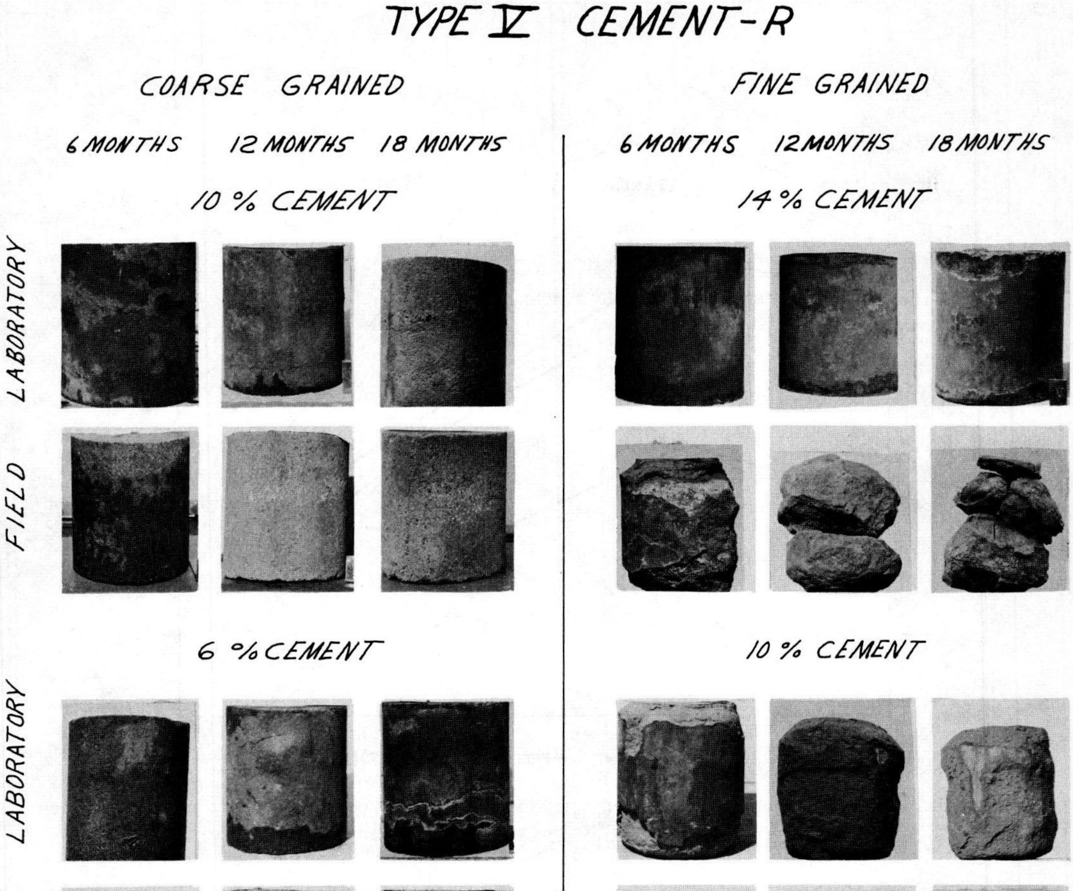
TYPE V CEMENT-R
COARSE GRAINED
FINE GRAINED
6 MONTHS
12 MONTHS
18 MONTHS
6 MONTHS
12 MONTHS
18 MONTHS
10 % CEMENT
14 % CEMENT
LABORATORY
FIELD
6 % CEMENT
10 % CEMENT
LABORATORY
FIELD
3 % CEMENT
6 % CEMENT
LABORATORY
FIELD

Figure 12.

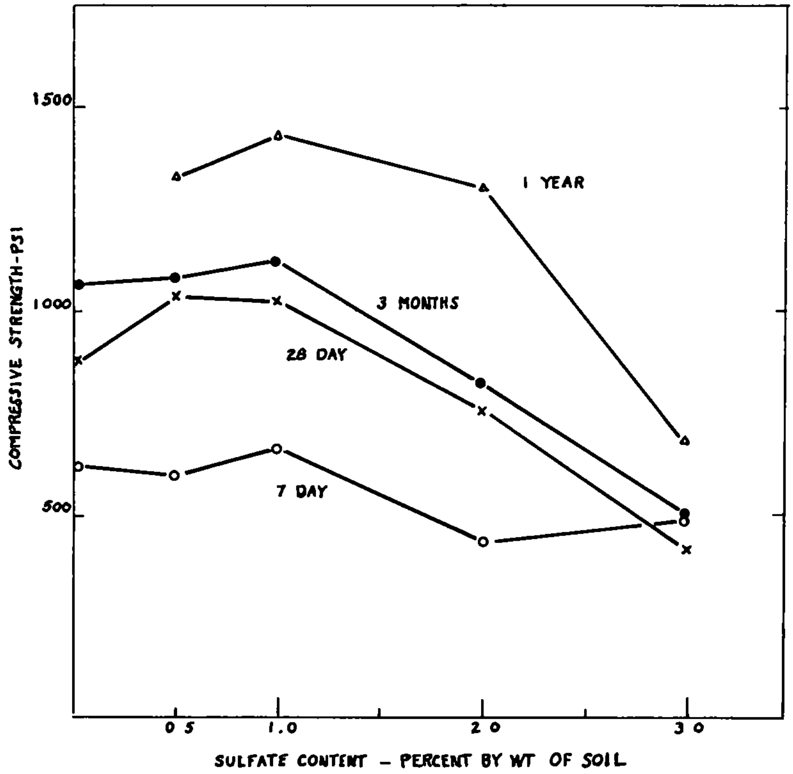

Figure 13. Influence of sulfate salts on the compressive strength of coarse-grained. soil-cement, 6 percent cement.

Soil-cement specimens studied in the first part of the investigation corroborate conclusively the original assumption that soil-cement deteriorates when exposed to sulfate salts. The fact that soil-cement is less dense than concrete permits sulfate salts to penetrate more rapidly.

The effect of cement content and the compound composition of the cement is also apparent in the photographic record of the soil-cement specimens. Soil-cements containing high cement contents resist the attack of sulfate salts much better than soil-cements containing low cement contents. Type V cements and, to a lesser extent, Type II cements are more resistant to the attack of sulfate than Type I cement. Soil-cement specimens containing Type I cement and a high cement content were more resistant to the attack of sulfate than Type V soil-cement with a low cement content.

Second Phase

The results of tests made in the second phase of the investigation in which sulfate salts were added directly to the soil-cement mixture at the time of fabrication are given in Tables 3 and 4.

In some cases the soils containing the higher salt concentrations deteriorated in less than 12 cycles of freezing and thawing. The number of cycles required for 100 percent loss in freezing and thawing are in parenthesis. Table 3 includes all tests made with coarse-grained soils. Table 4 gives test results with fine-grained soils. All soil-cements used in this investigation contained a medium cement content of 10.0 percent for the fine-grained soil and 6.0 percent for the coarse-grained soil.

The influence of various concentrations of sulfate salts on the compressive strength of coarse-grained soil-cement is shown in Figure 13. It is indicated in Figure 13 that small amounts of sulfate salts in soil-cement may improve the compressive strength of soil-cement cylinders. Sulfates added in amounts up to 1 percent caused no decrease in compressive strength in any of the specimens tested. As the amount of sodium and

TABLE 3

COMPRESSIVE STRENGTH AND DURABILITY TEST RESULTS—COARSE GRAINED SOIL

Cement			Percent Sulfate Salts by Wt. of Soil	Compressive Strength				Durability Losses	
Brand	Type	%		7 Day	28 Day	3 Mo	1 Yr	Freeze and Thaw	Wet and Dry
I	I	6	0	640	920	1,280	----	7.04	4.4
	I	6	0.5	710	990	900	1,270	3.8	---
	I	6	1.0	570	1,105	1,020	1,390	10.8	4.0
	I	6	2.0	420	505	565	1,110	100 (10)	5.8
	I	6	3.0	520	310	210	80	100 (6)	5.6
I	II	6	0	571	860	1,080	----	----	---
	II	6	0.5	560	892	1,010	1,230	5.5	---
	II	6	1.0	520	865	1,060	1,380	11.6	2.3
	II	6	2.0	420	605	805	1,410	9.8	1.8
	II	6	3.0	485	400	565	820	100 (6)	6.5
E	I	6	0	748	1,000	1,100	----	----	---
	I	6	0.5	720	1,140	1,105	1,360	3.15	---
	I	6	1.0	825	890	1,220	1,570	3.3	2.1
	I	6	2.0	535	560	650	1,110	100 (10)	3.5
	I	6	3.0	590	0	0	0	100 (5)	4.3
E	II	6	0	707	970	1,300	----	3.60	3.7
	II	6	0.5	705	1,135	1,305	1,305	3.70	---
	II	6	1.0	870	1,230	1,260	1,510	3.0	2.3
	II	6	2.0	465	790	910	1,410	4.6	1.5
	II	6	3.0	580	500	415	745	100 (8)	2.4
M	I	6	0	563	890	1,180	----	----	---
	I	6	0.5	560	970	1,020	1,420	3.15	---
	I	6	1.0	860	1,110	1,160	1,470	3.4	2.3
	I	6	2.0	340	840	635	1,380	100 (10)	2.6
	I	6	3.0	440	290	360	475	100 (5)	7.2
M	II	6	0	650	960	1,210	----	----	---
	II	6	0.5	630	1,040	1,090	1,510	3.28	---
	II	6	1.0	815	1,260	1,200	1,530	2.9	1.8
	II	6	2.0	630	970	955	1,400	2.7	1.5
	II	6	3.0	560	650	685	1,390	100 (7)	3.2
M	V	6	0	628	840	1,100	----	8.58	5.2
	V	6	0.5	535	1,110	1,070	1,500	4.23	---
	V	6	1.0	510	1,080	1,195	1,420	3.0	2.0
	V	6	2.0	475	900	1,080	1,340	2.7	1.5
	V	6	3.0	475	645	805	1,280	100 (9)	2.8
R	V	6	0	660	820	1,230	----	----	---
	V	6	0.5	520	1,000	1,225	1,130	3.8	---
	V	6	1.0	535	840	955	1,370	3.0	1.6
	V	6	2.0	575	770	865	1,280	5.0	1.6
	V	6	3.0	305	570	650	1,030	100 (9)	2.4
V	V	6	0	437	740	1,050	----	----	---
	V	6	0.5	460	1,040	1,030	1,370	3.15	---
	V	6	1.0	520	965	1,035	1,340	5.0	1.9
	V	6	2.0	515	835	970	1,410	3.2	1.6
	V	6	3.0	455	640	750	430	100 (7)	3.2

Number in parenthesis denotes number of cycles to complete disintegration.

TABLE 4

COMPRESSIVE STRENGTH AND DURABILITY TEST RESULTS—FINE-GRAINED SOIL

Cement			Percent Sulfate Salts by Wt. of Soil	Compressive Strength				Durability Losses	
Brand	Type	%		7 Day	28 Days	3 Mo	1 Yr	Freeze and Thaw	Wet and Dry
I	I	10	0	600	920	----	-----	----	3.57
	I	10	0.5	760	950	910	1,090	36.6	3.53
	I	10	1.0	685	980	1,000	1,230	35.4	4.42
	I	10	2.0	540	805	940	900	57.8	5.53
	I	10	3.0	545	725	790	920	61.5	6.16
I	II	10	0	560	820	---	----	----	----
	II	10	0.5	670	870	860	1,430	42.4	4.13
	II	10	1.0	590	1,060	1,040	1,250	40.7	4.67
	II	10	2.0	520	620	860	830	58.8	5.02
	II	10	3.0	445	635	875	820	67.0	6.32
E	I	10	0	575	890	---	----	----	----
	I	10	0.5	700	1,150	990	1,280	35.0	4.58
	I	10	1.0	615	1,220	1,020	1,250	58.5	4.70
	I	10	2.0	575	820	970	930	65.4	4.72
	I	10	3.0	525	735	965	890	73.0	6.18
E	II	10	0	570	835	---	----	----	2.78
	II	10	0.5	725	955	970	1,440	24.4	3.38
	II	10	1.0	680	1,260	1,150	1,410	36.5	5.16
	II	10	2.0	580	750	1,010	930	51.8	5.88
	II	10	3.0	515	685	940	900	56.4	4.71
M	I	10	0	705	790	---	----	----	----
	I	10	0.5	630	850	900	1,270	55.3	2.95
	I	10	1.0	655	1,160	870	1,270	41.3	8.45
	I	10	2.0	520	780	960	900	55.7	5.30
	I	10	3.0	540	620	925	830	65.7	5.16
M	II	10	0	555	910	---	----	----	----
	II	10	0.5	675	955	950	1,190	51.3	4.58
	II	10	1.0	655	1,200	1,130	1,340	41.0	4.42
	II	10	2.0	520	700	990	760	44.5	4.72
	II	10	3.0	560	725	980	830	51.5	5.32
M	V	10	0	565	890	---	----	----	----
	V	10	0.5	645	955	840	1,310	29.9	3.68
	V	10	1.0	655	1,195	1,030	1,240	31.8	4.41
	V	10	2.0	520	765	1,060	920	40.4	5.90
	V	10	3.0	500	780	1,060	715	49.6	4.87
R	V	10	0	605	885	---	----	----	----
	V	10	0.5	695	970	820	1,200	25.2	4.00
	V	10	1.0	575	960	1,070	1,020	29.7	4.72
	V	10	2.0	520	830	1,060	940	45.5	4.71
	V	10	3.0	550	730	935	780	50.4	5.01
V	V	10	0	500	835	---	----	----	----
	V	10	0.5	645	940	740	1,030	30.1	4.44
	V	10	1.0	605	1,035	930	1,180	35.4	4.43
	V	10	2.0	540	805	1,070	880	47.5	6.20
	V	10	3.0	460	780	910	750	73.0	5.30

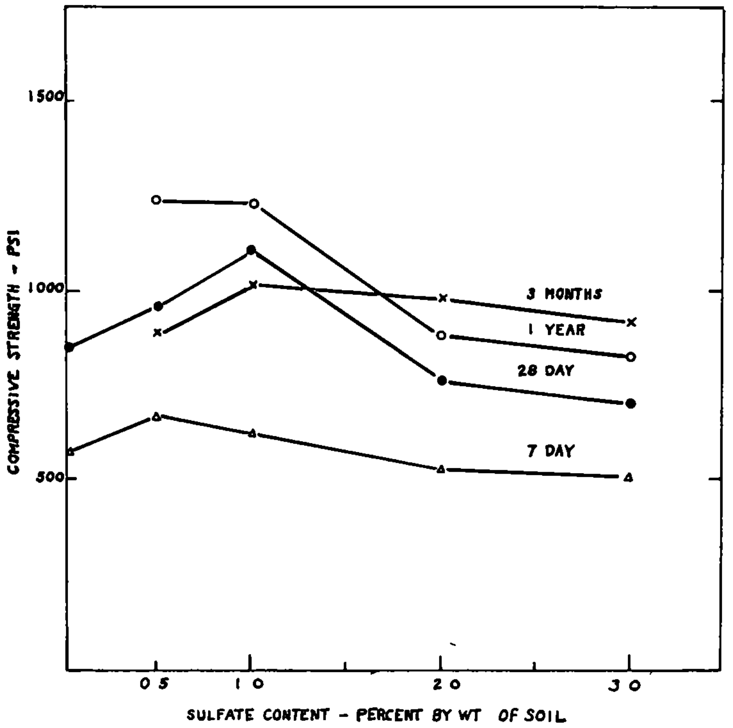

Figure 14. Influence of sulfate salts on the compressive strength of fine-grained soil-cement, 10 percent cement.

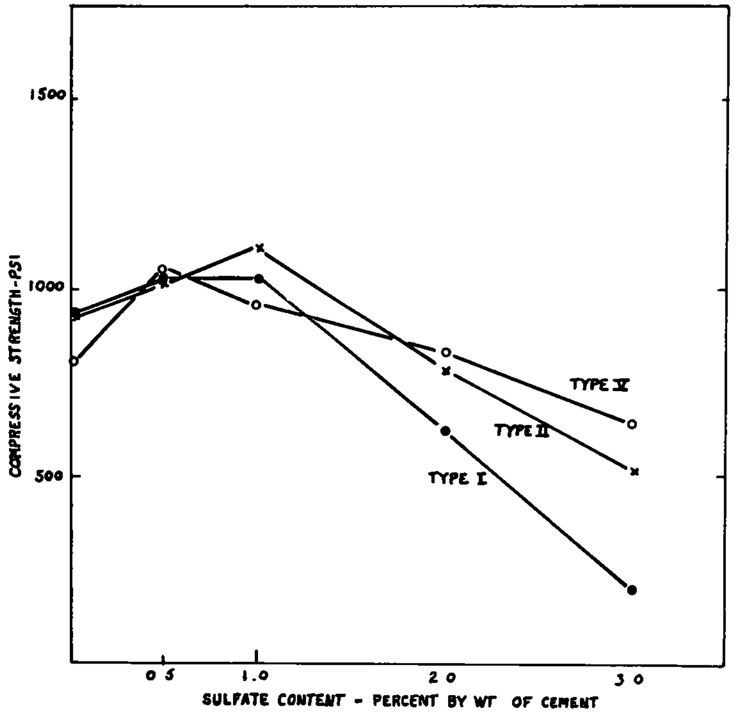

Figure 15. Influence of the type of cement on the compressive strength of coarse-grained soil-cement containing sulfate salts.

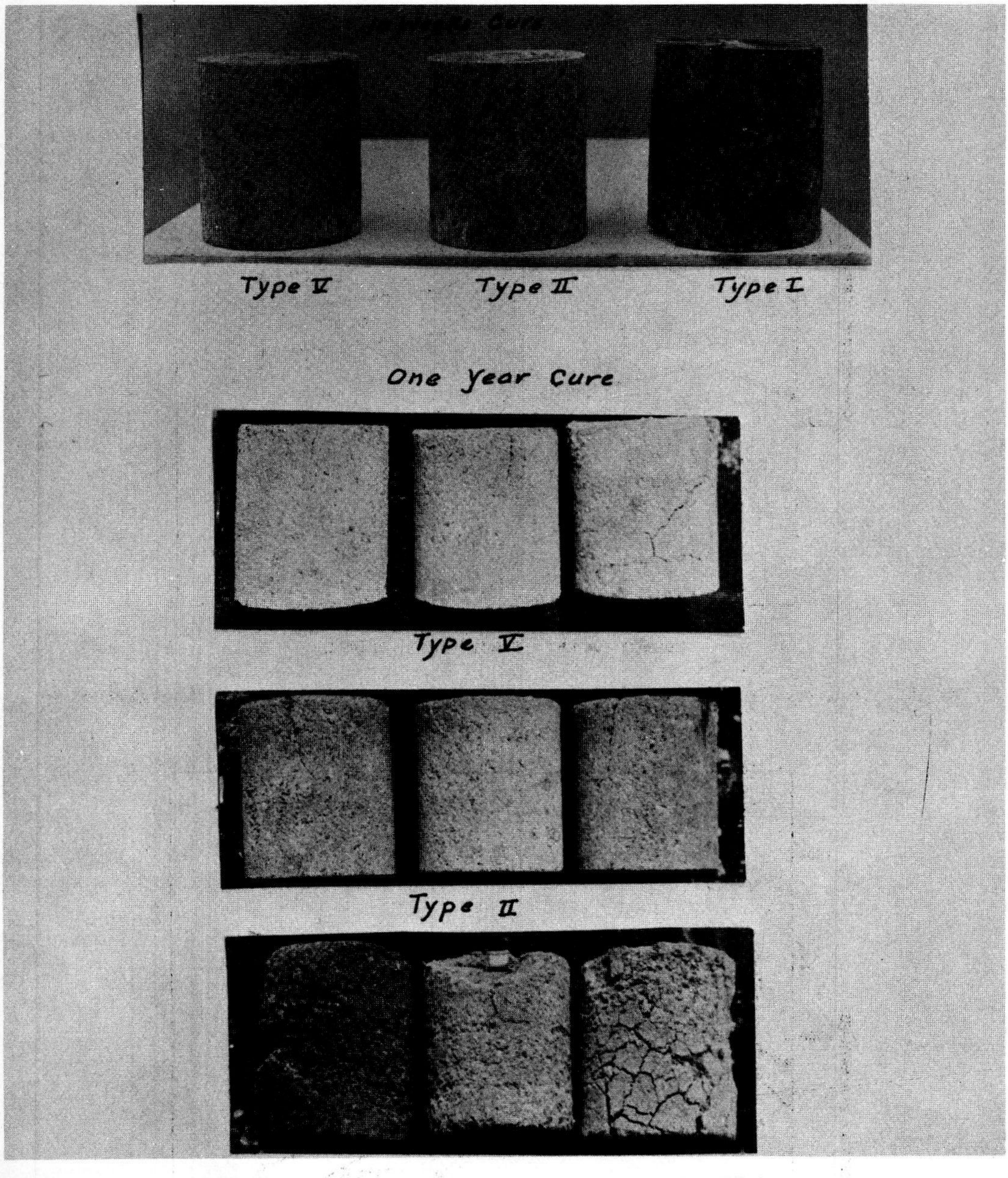

Figure 16. Specimens containing 3 percent sulfate.

magnesium sulfate increased to 2 and 3 percent, there was a definite decrease in compressive strength, particularly at later ages. The test points shown in Figure 13 are the averages for nine different cements including all three types.

Results of fine-grained soil-cements are shown in Figure 14. In this case, a decrease in compressive strength with excessive concentrations of sulfates in the soil was not as pronounced as with the coarse-grained soil. Even with 3 percent sulfate by weight of soil, the decrease in strength was not pronounced. Figure 14 also indicates that the presence of sodium and magnesium sulfate up to 1 percent by weight of soil improves the compressive strength of the soil-cement specimen.

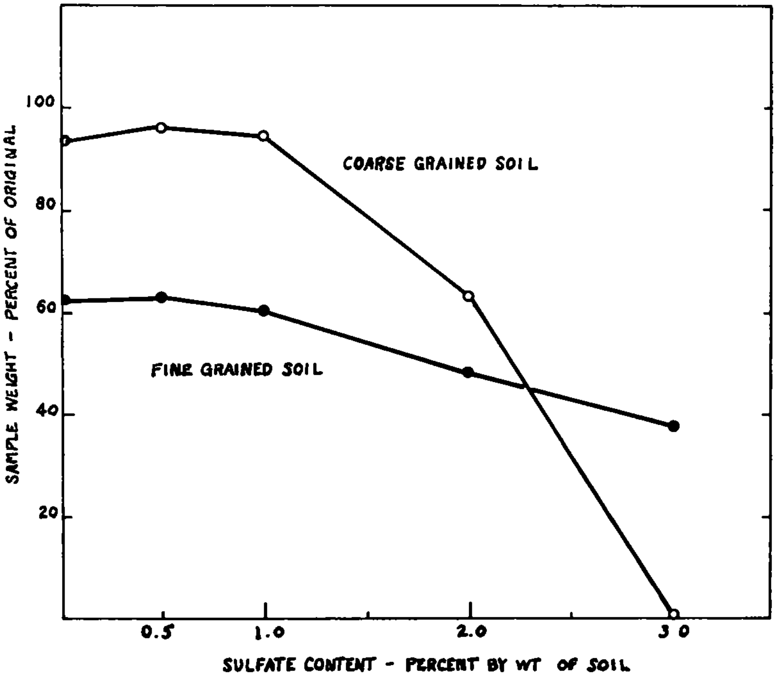

Figure 17. Deterioration of soil-cement specimens after 12 cycles of freezing and thawing.

The influence of the type of cement on the compressive strength of coarse-grained soil-cement containing various amounts of sulfate salts is indicated in Figure 15. Results from three Type I, three Type II, and three Type V cements were averaged in each case to obtain the points on this figure. Figure 15 also shows an increase in compressive strength with all three types with sulfate contents up to 1 percent by weight of soil. Apparently the type of cement had little influence when the concentration of sulfate salts was low. There was a definite difference between Type I, Type II, and Type V cements with high concentrations of sulfate. Type I cement deteriorated very noticeably with 3 percent sulfate, whereas Type II and Type V cements showed less deterioration as measured by the compressive strength. The influence of the type of cement is shown more dramatically in the photographs (Fig. 16). These photographs show specimens containing 3 percent sulfate after 10 weeks curing in a moist cabinet and after 1 year moist curing. The type of cement has a very definite bearing on the expansion and cracking of the soil-cement specimens.

There was also a hardening of some soil-cement specimens containing sulfate salts similar to that found in the first part of the investigation. The specimens became hard and would expand and crack. Cracked specimens indicated compressive strength as shown in Tables 2 and 3. Only one Type I cement at 3 percent sulfate and none of the Type II and Type V soil-cement specimens showed complete loss of compressive strength strength. Yet, the photographs (Figure 16) indicate excessive cracking and deterioration.

The durability of coarse-grained soil-cement with various percentages of sulfate salts added at the time of fabrication is indicated in Figure 17. Specimens containing less than 1 percent sulfate salts showed no loss of durability when subjected to freezing and thawing. As the amount of sulfate increased, however, complete deterioration was experienced in this test. This provides further indication that small amounts of sulfate salts mixed with soil-cement may improve its quality.

CONCLUSIONS

1. Soil-cement is subject to attack of sulfate salts much in the same manner as concrete. Deterioration in soil-cement is more rapid than concrete specimens primarily because soil-cement is less dense and sulfate solutions penetrate at a more rapid rate.
2. Soil-cement specimens of high cement contents are more resistant to the attack of sulfate than soil-cement of low cement content.
3. Soil-cement specimens fabricated with Type V and Type II cements are more resistant to the attack of sulfate salts than salts than soil-cement specimens fabricated with Type I cement.
4. Cement content of richness of mix is apparently more important in producing a sulfate resistant soil-cement than the type of cement used.
5. Soil-cement specimens made with fine-grained soils deteriorate more rapidly under field exposure (subjected to alternate cycles of freezing and thawing in addition to the attack of sulfate salts.)
6. Coarse-grained soil-cement specimens at low cement content deteriorated more rapidly when exposed to alternate cycles of freezing and thawing under field exposure than when exposed to sulfate salt solutions at laboratory temperatures.
7. Soil-cement specimens made with coarse-grained soil, high cement content, and sulfate resisting cements produced the most resistant soil-cement to the attack of sulfate salts.
8. Soil-cement specimens of high cement content crack and expand on deterioration but do not revert back to the original characteristics of the soil. Many of the fine-grained soil-cement specimens broke up into small pieces of cemented soil. It can be assumed that although sulfate salts attack soil-cement, stability of a highway base may not be destroyed by this action.
9. Small amounts of sulfate salts mixed with soil-cement at the time of fabrication increase the compressive strength and the resistance of the specimen to the action of freezing and thawing. Sulfate salts mixed with coarse-grained soil have a greater influence on strength and durability than when mixed with fine-grained soil.
10. High concentrations of sulfate salts mixed with soil-cement at the time of fabrication will cause cracking, expansion, and reduction in compressive strength and reduction in durability in soil-cement specimens.
11. Soil-cement specimens tested in the first phase of the investigation were subjected to high concentrations of sulfate salts which caused deterioration of many of these specimens. Results obtained in the second phase of the investigation, however, would indicate that if these same specimens had been exposed to low or moderate concentrations of sulfate salts, their durability and strength may not have been impaired. This leads to the conclusion that sulfate salts cause deterioration of soil-cement only when they are used in the soil-cement mixture in high concentrations, above 1 percent by weight of the soil, or when soil-cement specimens are placed in sulfate soils having high concentrations of sulfate salts.
12. Possible sulfate problems encountered in the construction of soil-cement bases on highways should first be investigated as to the concentration of sulfates encountered, and second, the cement content which might be required to withstand the attack of the sulfate salts.

Improvement of Soil-Cement with Chemical Additives

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The search for chemical additives to improve the properties of soil-cement has been carried out intensively at the Massachusetts Institute of Technology Soil Stabilization Laboratory during the past few years. Previous papers have described the effectiveness of a group of alkali metal compounds in improving the properties of a spectrum of cement-stabilized soils of widely different origins, degrees of fineness, and mineralogical and chemical compositions. A general pattern of behavior was established whereby the most effective additive type and concentration for a particular soil type could be designated.

This paper summarizes the most recent results obtained from further detailed study on the use of alkali additives in soil-cement. Salient results presented include the following:

1. Study of the long-term effects of immersion in sulfate solution on four cement-stabilized soils indicates that sodium additives materially improve the resistance of soil-cement to possible sulfate attack. The four soils studied included a clean sand, a clay, a sand containing organic matter, and a silty clay with high soluble salt content.
2. Calcium and magnesium sulfates, in addition to sodium sulfate, are found to be very effective in increasing the strength of organic sand-cement.
3. Attempts to find a general formulation of sodium additives for all soil types by combining sodium hydroxide and sodium sulfate at various molar ratios have not been successful.
4. Pretreatment of heavy clays with small quantities of polyvalent metal salts and salts of organic cations improves their response to cement-additive stabilization by reducing the expansion of the montmorillonitic soils in water immersion.
5. Study of the effects of soda-to-silica ratio in sodium silicate when used as an additive to cement-stabilized silt indicates that the silicates of high soda content are very effective in strength improvement.

● RESEARCH on improving the properties of soil-cement by the use of chemical additives has been carried out intensively at the Massachusetts Institute of Technology Soil Stabilization Laboratory during the past few years. Two of the primary objectives of the investigation are (1) to increase the effectiveness of portland cement as a soil stabilizer so as to reduce the quantity of cement required to treat responsive soils and (2) to find trace chemicals that will enhance the effectiveness of cement as a stabilizer for "problem" soils; i.e., those that cannot be stabilized economically with cement alone. Previous papers by the authors (1, 2) have described the unique effectiveness of a group of metal alkali compounds in improving the properties of a spectrum of cement-stabilized soils of widely different geological and geographical origins and widely different physical, mineralogical, and chemical compositions. A general pattern of behavior was established whereby the most effective additives for a particular soil-type of known composition could be designated with an element of assurance. Based on the soils studied, at that time were the following general findings:

1. Sodium hydroxide is effective in improving strength of all soils with low to moderate amounts of organic matter.
2. Sodium salts of weak acids are not effective in heavy clays.
3. Sodium sulfate is uniquely effective on sandy soil containing organic matter.
4. The effectiveness of sodium compounds decreases with increasing plasticity and/or organic matter content of the soil.

This paper describes the most recent results obtained from further laboratory studies on the use of alkali additives in soil-cement.

The research summarized herein covers the following topics:

1. Examination of the long-term effects of sulfate on the stability of cement-stabilized soils with and without additives.
2. Evaluation of other sulfate compounds (calcium and magnesium) as additives to organic sand-cement.
3. Investigation of the possibility of a general formulation of additives for all soil-types.
4. Examination of the use of secondary additives to improve the effectiveness of sodium hydroxide in cement-clays.
5. Examination of the effect of soda-to-silica ratio in sodium silicate as an additive to soil-cement.

MATERIALS AND PROCEDURES

Materials

Soils. The seven soils employed in the five studies reported in this paper were selected from the large number of soils investigated previously. Their response to the treatment of alkali sodium additives and portland cement has been established (2, 3).

Four soils were chosen for the examination of sulfate attack on soil-cement: a sand - Wisconsin Sand 2 (1057), a clay - Iraq Clay 2 (1067), a silty clay with high soluble salt content - Iraq Silty Clay (1068), and a sand containing organic matter - Wisconsin Sand 1 (1056).

The response to cement stabilization of the organic sand (Wisconsin Sand 1) was further examined by incorporating one of several sulfate compounds other than sodium sulfate as an additive. Study of the effect of soda-to-silica ratio in sodium silicate when used as an additive was carried out with New Hampshire Silt, the soil least complex in composition and most responsive to treatment.

Two heavy clays, Texas Clay 2 (1059) and Vicksburg Buckshot Clay (VBC), were selected for evaluation of secondary additives to improve their response to cement-sodium additive treatment.

The three soils selected for investigation of the possibility of a general formulation of sodium additives (i.e., mixtures of sodium hydroxide and sodium sulfate) for all soil types were New Hampshire Silt, Wisconsin Sand 1, and Texas Clay 2.

The properties of all seven soils tested are summarized in Table 1.

Cement and Additives. Type 1 portland cement was used: Table 2 summarizes its properties. For most of the tests, 5 percent (on dry soil weight) was used.

Table 3 lists the additives employed in the investigation. Selection was based on observations from earlier studies. In addition to the primary additives, several polyvalent metal salts and salts of organic cations were investigated as a means for reducing the lattice expansion of heavy clays on immersion. These compounds are listed as secondary additives.

TABLE 1

PROPERTIES OF SOILS INVESTIGATED

	New Hamp. Silt	Vicksburg Buckshot Clay	Texas Clay 2	Iraq Clay 2	Iraq Silty Clay	Wis. Sand 1	Wis. Sand 2
	NHS	VBD	TCI	Ircl	IrSC	WS 1	WS 2
	NHS	VBC	1059	1067	1068	1056	1057
Textural composition[a] (% by wt)							
Sand, 0.06 to 2mm	3	4	3	17	13	82	85
Silt, 0 002 to 0.06mm	90	61	28	46	62	18	15
Clay, < 0.002mm	7	35	69	37	25	0	0
Physical property							
Liquid limit (%)	28	60	71	39	31	---	---
Plastic limit (%)	20	28	42	20	20	---	---
Plasticity index (%)	8	32	29	19	11	N P.	N. P.
Specific gravity, $20^\circ c/20^\circ c$	2.72	2.67	2.71	2.72	2 80	2.64	2.64
Max. dry density[b] (pcf)	99 5	103.0	94.0	106.3	110 5	102.0	103.0
Optimum water content (%)	19 9	22.0	25.8	19 5	16.6	14.2	12.5
Classification							
Unified	ML	CH	OH	CL	CL-ML	SM	SM
Bur. Pub Roads	Silty Loam	Clay	Clay	Clay	Silty Clay	Sand	Sand
Hwy Res. Board	A-4(8)	A-7-5(19)	A-7-5(20)	A-6(12)	A-6(9)	A-2-4	A-2-4
Chemical Property[c]							
Organic matter (% by wt)	0 4	1.1	2.9	0 9	1.3	1.9	0.2
pH	5.4	4.6	7.3	7.5	7.2	6.7	6.2
Carbonates (% by wt)	---	---	---	27	50	---	---
Total soluble salts m eg NaCL/100gm	---	0.3	1.6	1.7	73.0	0.1	0.1
Cation ex. capacity m eg/100 gm	3	30	27	20	16	16	10
Glycol retention (mg/gm)	6	65	93	45	88	32	24
Mineralogical composition[c]							
Clay composition (% by wt)	10	50	65	45	30	0	0
Illite montmorillonoid clay chlorite	1 0 0	1·1 0	3 2.5 1	1 1 1	1·2 1	---	---
Chlorite, nonclay (%)	---	---	---	15	15	---	---
Calcite (%)	---	---	---	30	50	---	---
Free iron oxide (%)	1.0	1 9	2.0	1.0	1.3	---	---

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[a] Massachusetts Institute of Technology Soil Classification.

[b] Harvard Miniature Compaction, 40-lb tamper, 3 layers, 25 blows per layer.

[c] For minus 74 micron fraction.

TABLE 2

COMPOSITION[1] OF CEMENT USED

Composition	% by Weight
Silica, SiO_2	19.78
Aluminum oxide, Al_2O_3	5.54
Ferric oxide, Fe_2O_3	3.45
Calcium oxide, CaO	62.59
Magnesium oxide, MgO	3.90
Sulfuric anhydride, SO_3	2.25
Sodium oxide, Na_2O	0.25
Potassium oxide, K_2O	0.71
Manganese oxide, Mn_2O_3	0.07
Insoluble residue	1.30
Loss on ignition	0.08
Specific surface (Blaine)	3270 sq cm/gm

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[1] Analyzed by Analytical Laboratories, Portland Cement Association.

TABLE 3

CHEMICAL ADDITIVES TESTED

Additive	Formula	Source
Primary additive:		
Sodium hydroxide	NaOH	Reagent grade
Sodium sulfate	Na_2SO_4	Reagent grade
Sodium orthosilicate	Na_4SiO_4	Diamond alkali Co
Sodium silicate, grade 50 (43.5% solid)	Na $NA_2Si_2O_5xH_2O$	Diamond alkali Co.
Sodium silicate, grade 40 (37.5% solid)	$NA_2Si_{3.22}O_{7.44}$ x H_2O	Diamond Alkali Co.
Sodium metasilicate	Na_2SiO_3 $9H_2O$	Reagent grade
Magnesium sulfate	Mg SO_4	Reagent grade
Calcium sulfate anhydrite	$CaSO_4$	Reagent grade
Gypsum	$CaSO_4$ $2H_2O$	Reagent grade
Secondary additives:		
Barium chloride	$BaCl_2$	Reagent grade
Ferric chloride	$FeCl_3$	Reagent grade
n - Octylamine	CH_3 $(CH_2)_6$ CH_2NH_2	Sharples Chemical Co.
Arquad 2 HT	Di-hydrogenated tallow dimethyl ammonium chloride	Armour & Co.
Arquad 12	Lauryl trimethyl ammonium chloride	Armour & Co.

Procedure

Strength Tests. All air-dried soils were pulverized and screened through a No. 10 sieve. Each batch of soil was first hand-mixed with half of the molding water and with the secondary additive (when used), and the equilibrated for 24 hours. (This step of equilibration was omitted for the two sandy soils.) A solution or slurry of chemical and cement was mixed into the soil. After equilibration, mixing was completed in a finger-blade mechanical mixer. (Mixing time was 7 minutes for the two Iraq soils and 5 minutes for all others.)

Specimens were molded by two-end static compaction in a Harvard miniature-size mold to constant density. The molding water content and density corresponded approximately to the optimum moisture content and maximum density of the untreated soil-cement or soil.

All specimens were cured under approximately 100 percent relative humidity and room temperature for various periods of time. Specimens were then immersed in distilled water for one day prior to testing to failure by unconfined compression. Weights and dimensions of the specimens were measured both after curing and after immersion.

Sulfate Attack Study. Specimens were prepared as if for the usual strength test and cured for 7 days. Half the samples were then immersed in distilled water and the other half in saturated calcium sulfate solution with excess solid calcium sulfate for various lengths of time and then tested in unconfined compression.

RESULTS AND DISCUSSION

Effect of Sodium Additives on the Resistance of Soil-Cement to Sulfate Attack

This study was aimed at evaluating the effects of sodium additives on the resistance of soil-cement to sulfate attack, inasmuch as several investigators (4, 5) had reported that sulfates are generally as detrimental to soil-cement as to concrete.

Two sets of samples from each of the four soils selected for study were prepared. One set was immersed in saturated calcium sulfate solution and the other in distilled water to study the long-term effects from up to one year of immersion. Figures 1 through 4 compare the strength, density, and water-content changes for each type of soil-cement sample, with or without additives, immersed in water or in sulfate solution.

Sand - Soil WS 2 (1057). The left half of Figure 1 shows that the soil cement samples (with ten percent cement, no additive) immersed in sulfate solution absorbed considerably more water and suffered more swelling (as shown by dry density decrease) than those immersed in pure water. The continuous increase in strength and the relatively small volume change of samples immersed in water indicate that prolonged immersion does not have any detrimental effect on soil-cement. The large and continuous decrease in the sulfate-immersion strength after 28 days, however, along with large volume expansion, clearly indicates the detrimental effects of the sulfate.

On the other hand, the right half of Figure 1 shows that the sulfate-immersion strength of the sodium metasilicate-treated samples (with 7 percent cement) continued to increase up to 90 days of immersion, with very little swelling and water pickup. The drop in strength after 90 days indicates that the sodium metasilicate in this sand-cement greatly delays or reduces the deteriorating effects of the sulfate; in other words, the additive prolongs the life of soil-cement. After 1 year of immersion in the sulfate solution, the strength was still higher than the early strength of the untreated soil-cement.

Clay - Soil IrC 2 (1067). The differences in the behavior of the clay-cement with and without the additive, when immersed in either water or sulfate solution, were not as pronounced as in that of the sand-cement. Figure 2 shows that sulfate has no adverse effect on the strength development of either soil-cement or soil-cement-sodium hydroxide systems. However, the soil-cement (no additive) swelled slightly more when immersed in sulfate solution than in water, while the reverse was true for soil-cement-sodium hydroxide samples. Furthermore, strengths of additive-treated soil-cement were higher than untreated at all immersion ages and in both solutions.

Soils with High Soluble Salt Content - Soil IrSC (1068). Results of Iraq Silty Clay with 10 percent cement, with and without 1.0 Normal sodium hydroxide, are shown in Figure 3. This soil, due to its high salt content, showed considerable water absorption during curing and loss of dry weight during immersion, as was described previously (2).

The general behavior of this soil after prolonged immersion in either water or sulfate solution was similar to that described for the Iraq Clay 2. Sulfate did not a detrimental effect on the soil-cement with or without additive up to 1 year of immersion.

Sand Containing Organic Matter - Soil WS 1 (1056). The results obtained with Wisconsin Sand 1, shown in Figure 4, are extremely interesting. The sulfate, rather than being detrimental to the soil-cement, appears to be beneficial. The strengths of soil-cement (no additive) samples immersed in sulfate solution were much higher than those immersed in water. The strengths after 28 days immersion in sulfate solution were the same for soil-cement with or without additive (10 percent cement plus 1.0 Normal sodium sulfate with additive and 16 percent cement without additive), while the strength after 1-day immersion was very low in the case of untreated soil-cement compared to the sulfate-treated samples.

Effect of Magnesium and Calcium Sulfate on the Strength of Organic Sand WS 1 (1056)-Cement

Earlier test results (2) had shown clearly that the poor response of the Wisconsin Sand 1 to cement and alkali additive treatment (except sodium sulfate) was due solely to the presence of the organic matter in the sand. The addition of sulfate ions appeared to depress the reactivity of the organic components. Also, as noted in the preceding section (on Figure 4), cement WS 1 immersed in saturated calcium sulfate solution was found to develop much higher strengths than that immersed in water. Hence, it was logical to examine other sulfate compounds in addition to sodium sulfate as additives to organic sand-cement. Sulfate compounds included in this investigation were anhydrous calcium

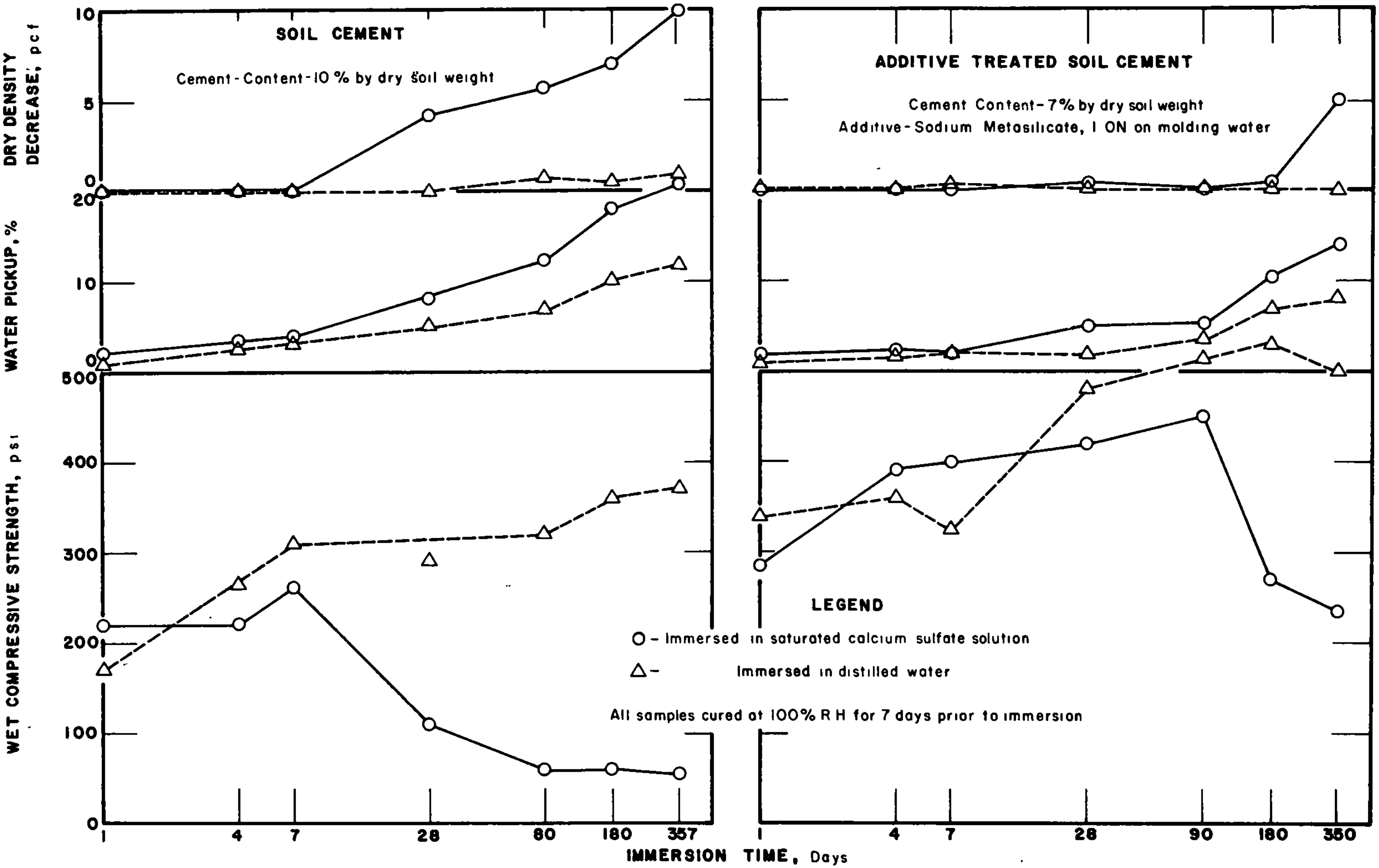

Figure 1. Strength, density, and water-content changes of cement-stabilized Wisconsin sand 2 (1057) immersed in water and in sulfate solution.

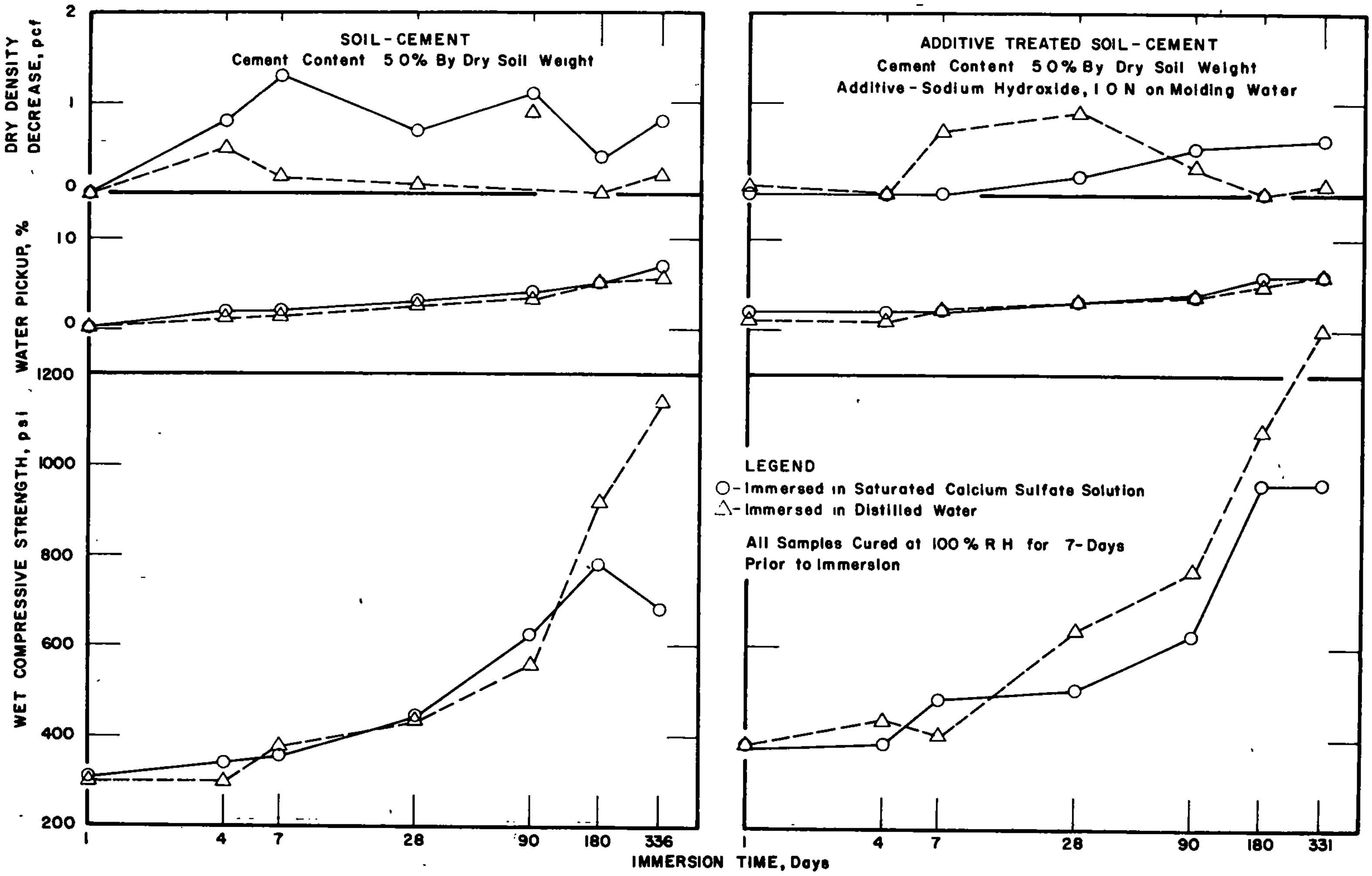

Figure 2. Strength, density, and water-content changes of cement-stabilized Iraq clay 2 (1067) immersed in water and in sulfate solution.

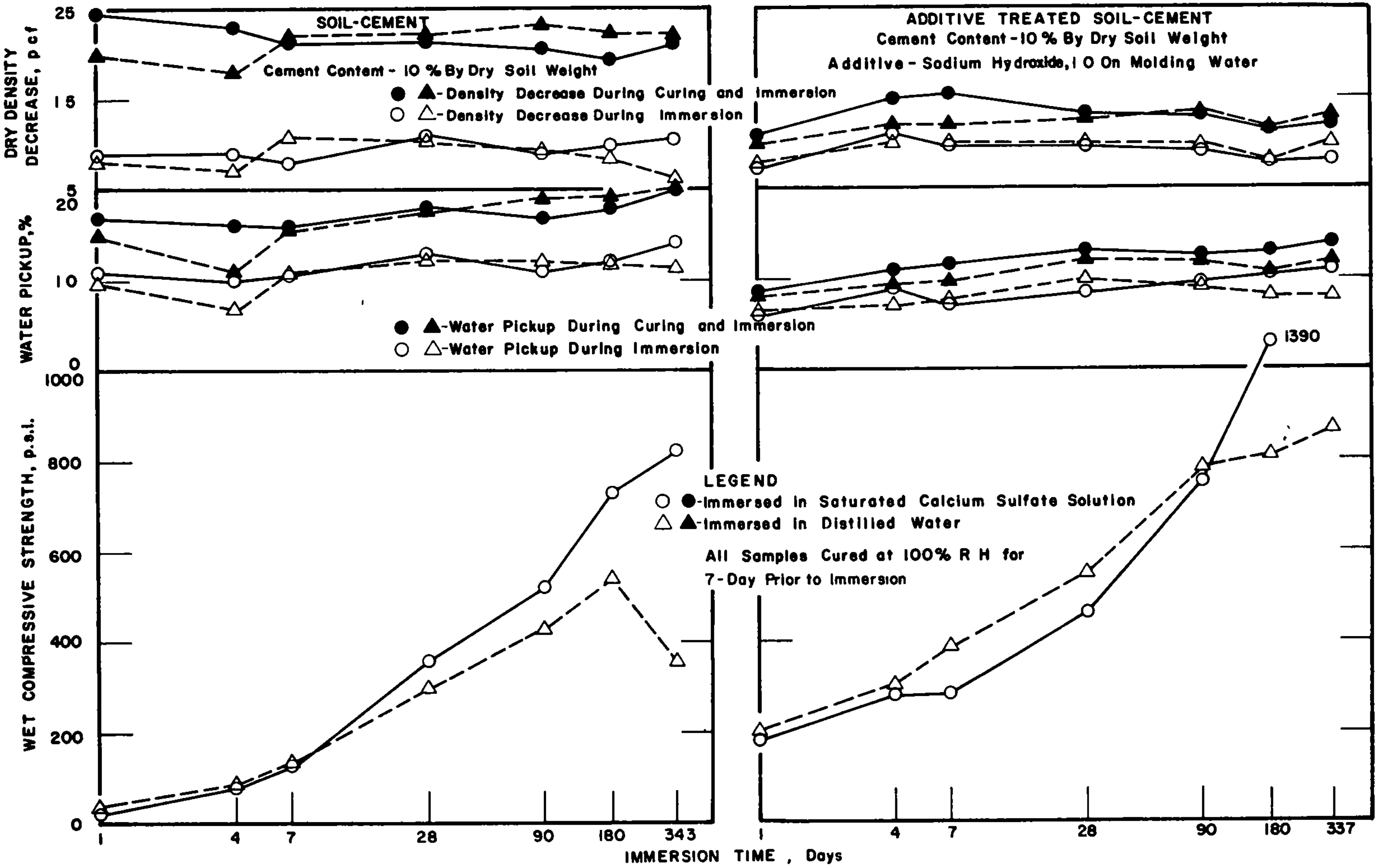

Figure 3. Strength, density, and water-content changes of cement-stabilized Iraq silty clay (1068) immersed in water and in sulfate solution.

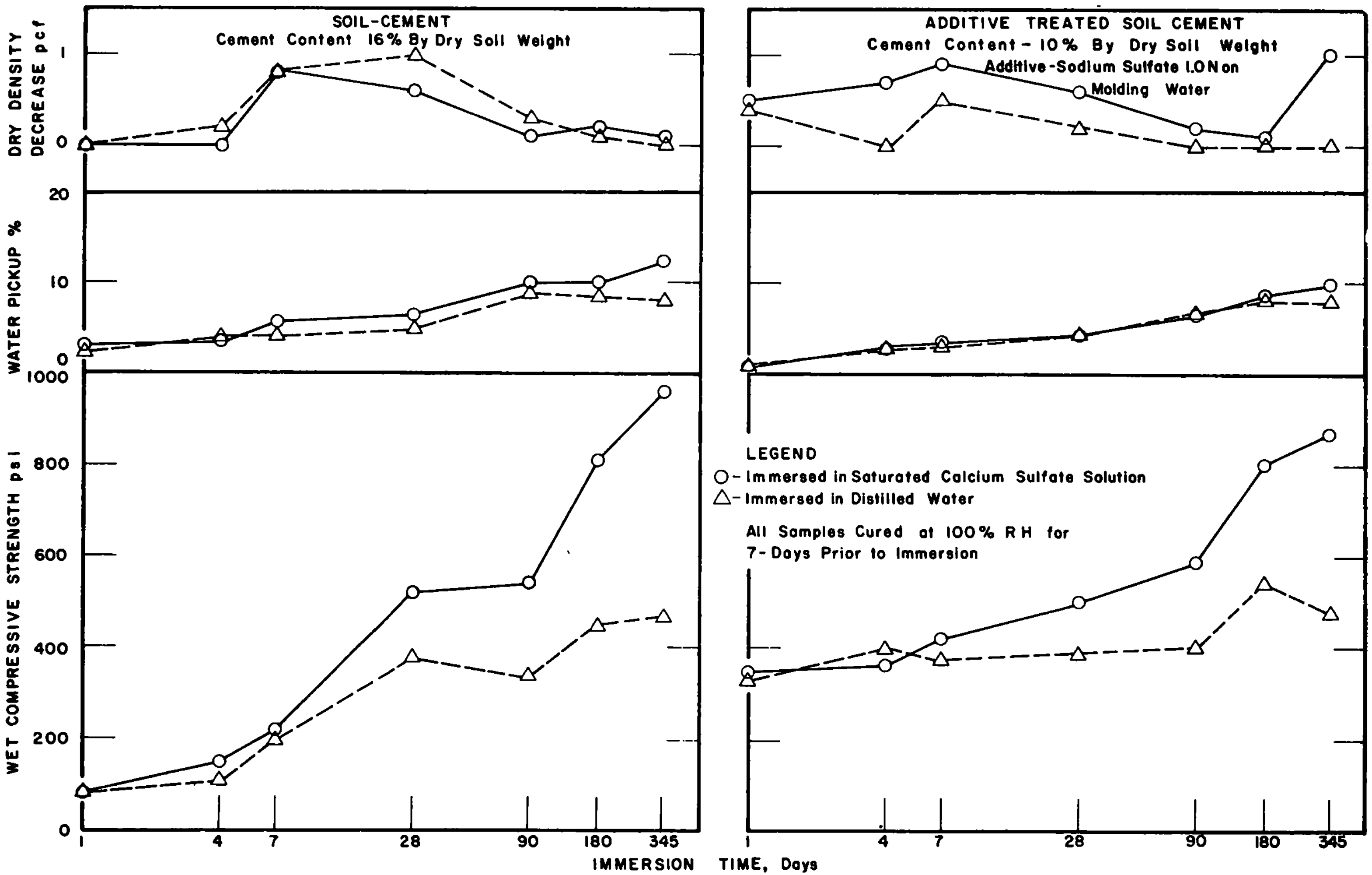

Figure 4. Strength, density, and water-content changes of cement-stabilized Wisconsin sand 1 (1056) immersed in water and in sulfate solution.

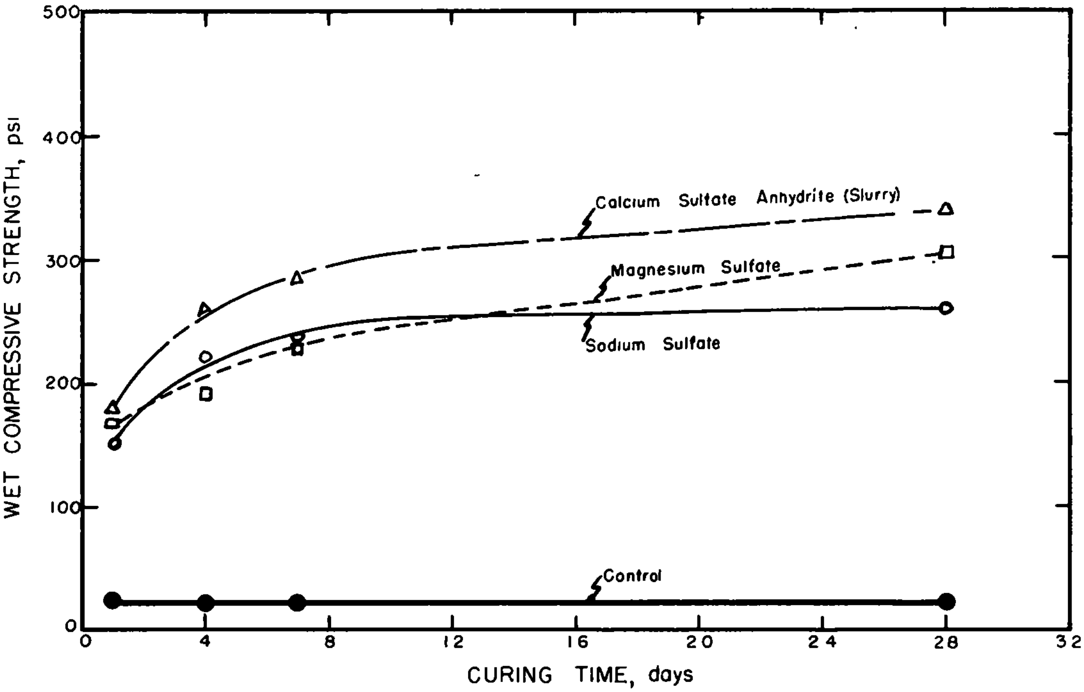

Figure 5. Effect of three sulfate compounds at 0.5 N concentration on the strength of Wisconsin sand 1 (1056) with 10 percent cement.

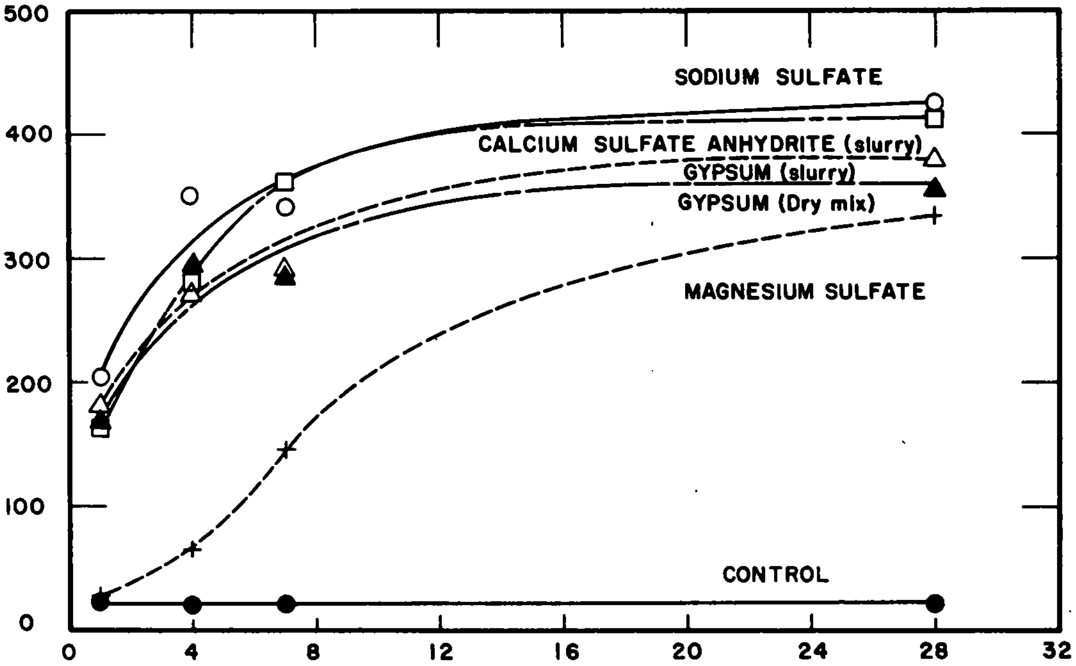

Figure 6. Effect of four sulfate compounds (1.0 N) on the strength of Wisconsin sand 1 (1056) with 10 percent cement

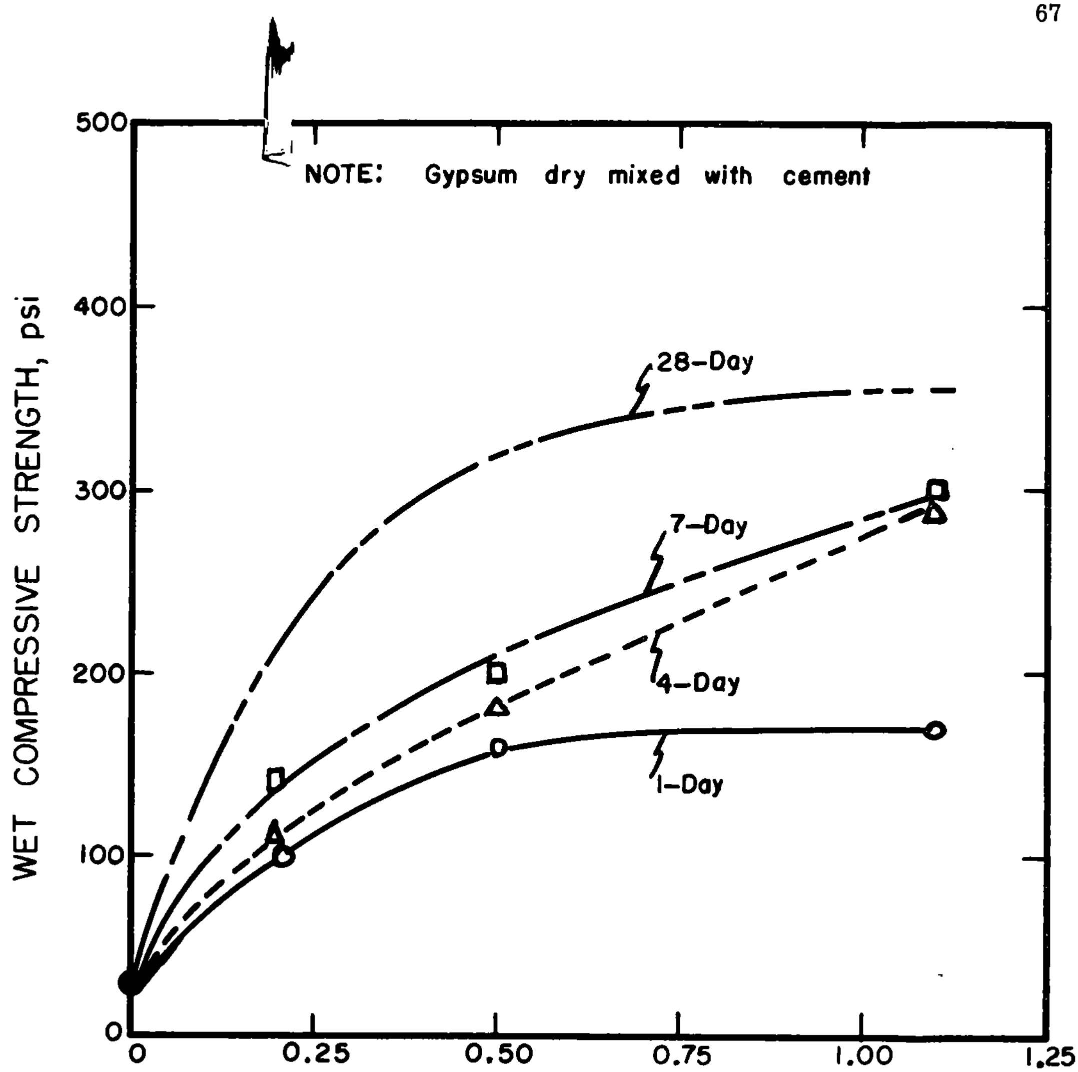

Figure 7. Effect of concentration of gypsum on strength of Wisconsin sand 1 (1056) with 10 percent cement.

sulfate, calcium sulfate hydrate (gypsum), and magnesium sulfate. As the two calcium sulfates are insoluble in water, incorporation as a water slurry and as a dry powder mixed with the cement were both investigated.

At 0.5 Normal concentration, as shown in Figure 5, both magnesium sulfate and a hydrous calcium sulfate (slurry) produced higher strength than sodium sulfate, particularly after 28 days of cure. Furthermore, calcium sulfate was more effective than magnesium sulfate. At higher additive concentration, i.e., 1.0 Normal (Figure 6), the anhydrous calcium sulfate gave about the same results as sodium sulfate, and gypsum slurry gave a somewhat lower 28-day strength. The magnesium sulfate was not only the least effective (28-day strength, about 25 percent lower than sodium sulfate) but it also retarded strength development considerably.

With both anhydrous calcium sulfate and gypsum, the dry-mix process produced slightly lower strength than the slurry, but both processes were very effective. From the economic standpoint, it is rather significant that gypsum dry-mixed with cement is an effective additive to organic sand-cement, inasmuch as several cement manufacturers

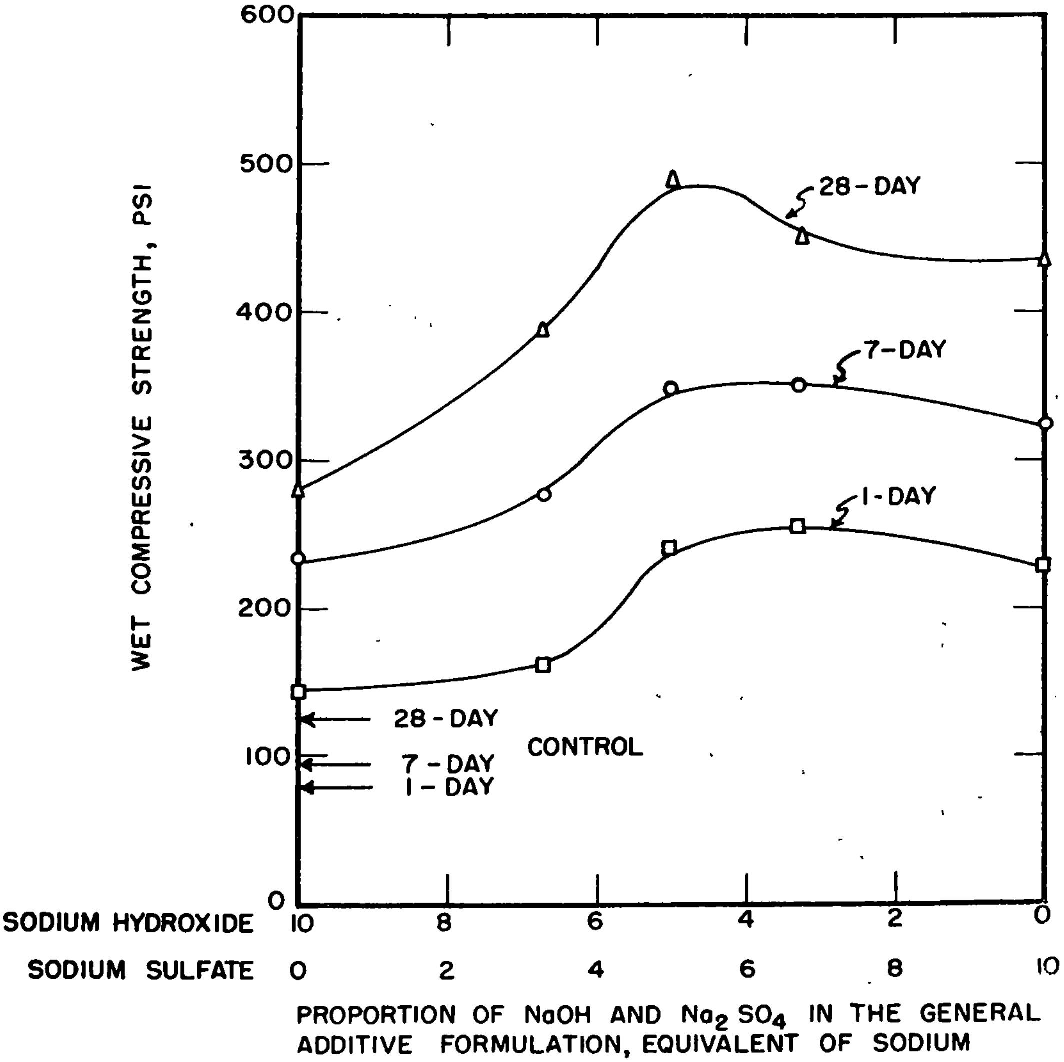

Figure 8. Effect of general additive formulation on strength of cement—New Hampshire silt (NHS); cement content = 5.0% on dry soil weight, additive concentration = 1.0 N sodium in molding water.

produce high-gypsum cement. Gypsum was further investigated by using various percentages of gypsum dry-mixed with the cement. Figure 7 shows that the strength of gypsum-cement WS 1 increased with gypsum content (at the same cement level) up to 1 percent.

Effectiveness of Mixtures of Sodium Hydroxide and Sodium Sulfate as a Possible General Additive Formulation for All Soil Types

Previous results (3) have shown that the effectiveness of a particular additive to soil-cement is largely dependent on the soil in question. In a summary by Lambe, Michaels, and Moh (2), sodium hydroxide was reported as the only beneficial additive to clay-cement; on the other hand, sodium sulfate was reported uniquely effective on sandy soil containing organic matter. Therefore, it appeared desirable to investigate the possibility of a general additive formulation for all soil types.

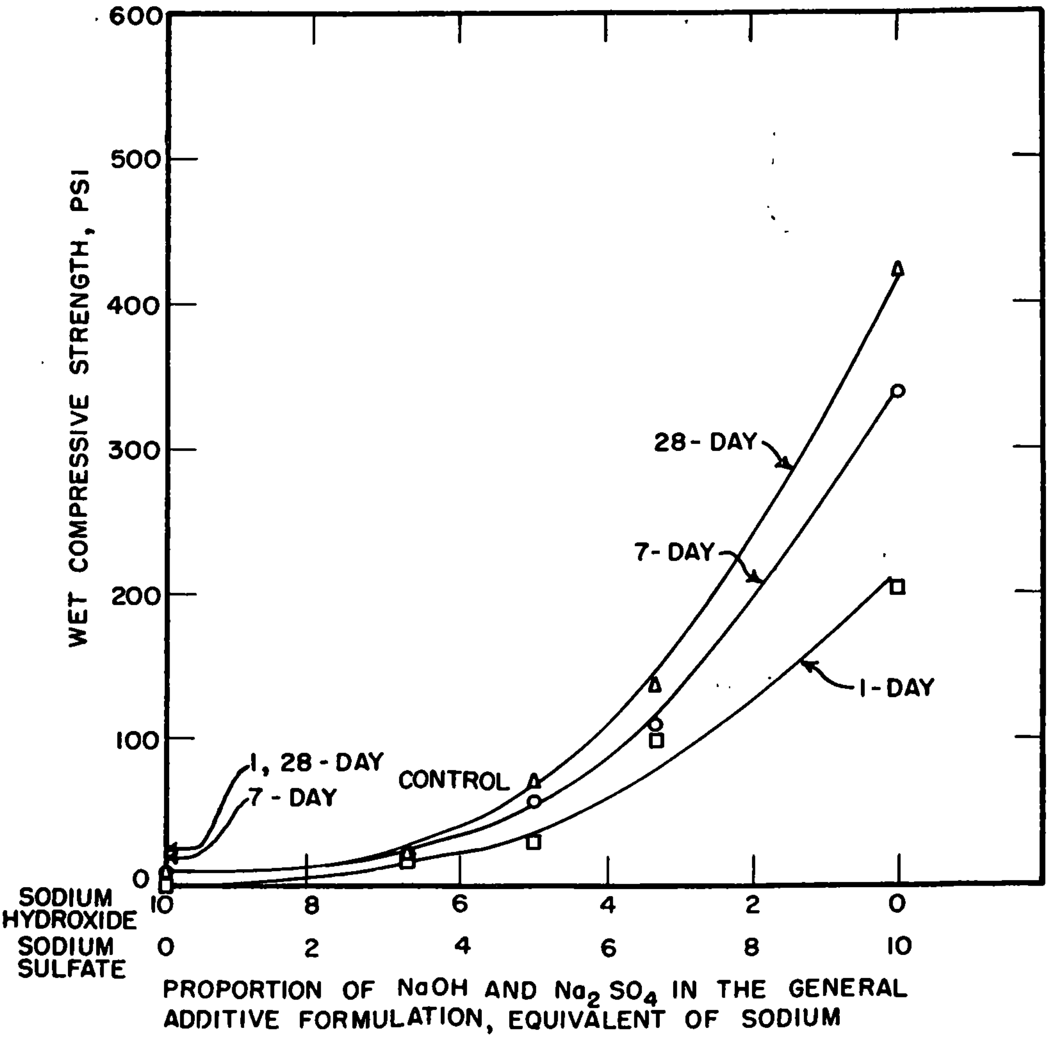

Figure 9. Effect of general additive formulation on strength of cement—Wisconsin sand 1 (1056); cement content = 10.0 percent on dry soil weight, additive concentration = 1.0 N sodium in molding water.

Because sodium hydroxide and sodium sulfate appeared to have contradictory effects on the properties of clay-cement and organic sand-cement, combinations of these two compounds in various proportions were chosen as a possible general additive formulation. The three soils selected, TC 2 (1059), WS 1 (1056), and NHS, represent three types of soils with distinctly different mineralogical compositions, chemical properties, and established responses to additive treatment. The results of this investigation are shown in Figures 8, 9, and 10 for NHS, WS 1, and TC 2, respectively.

The first general observation from these data is that this attempt to find a general additive formulation for all soil types was unsuccessful. The effectiveness of the additive mixture was no greater than that of the active component of the mixture. In other words, the effectiveness of this combination in WS 1 cement increased as the ratio of NaOH to Na_2SO_4 decreased, while the reverse was true for TC 2 cement.

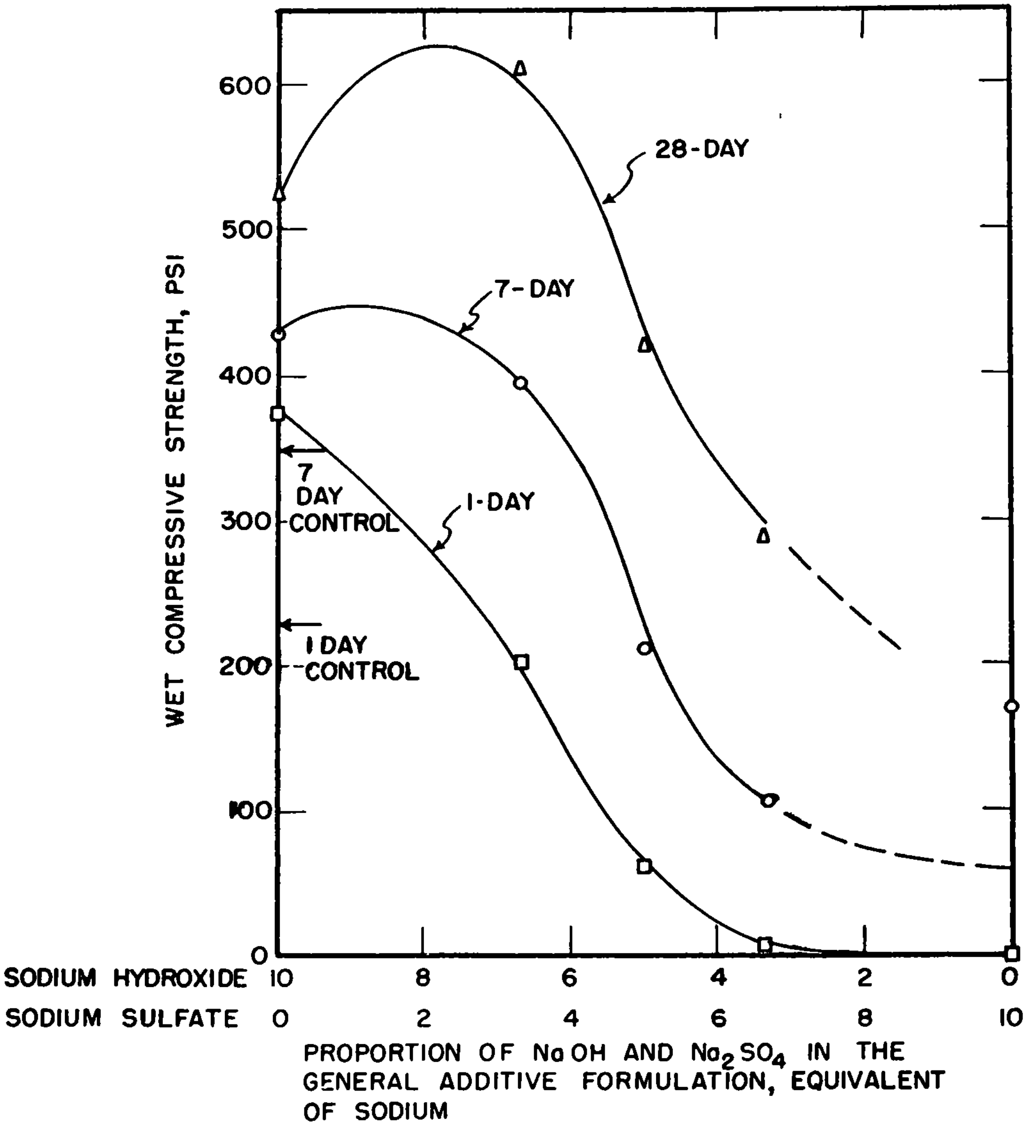

Figure 10. Effect of general additive formulation on strength of cement—Texas clay 2; cement content = 10.0 percent on dry soil weight, additive concentration = 1.0 N sodium in molding water.

Silt - Soil NHS (5 Percent Cement). At the same additive contentration (1.0 Normal of sodium in molding water), the strength of additive-treated specimens increased with a decreasing ratio of hydroxide to sulfate in the additive and achieved optimum effect at a ratio of about one, as shown in Figure 8.

As discussed elsewhere by the authors (2), by adding sodium sulfate to soil-cement the rate of formation of soluble silicate is slower and the calcium solubility in the pore fluid is less suppressed than when sodium hydroxide is added. The processes of formation of soluble silicate and precipitation of calcium silicate gel are gradual and more or less simultaneous. On the other hand, when hydroxide is the additive, there is rapid formation of soluble silicate but delayed gelation by calcium due to the high pH of the

TABLE 4

EFFECT OF SEQUENCE OF ADDITION OF GENERAL ADDITIVE FORMULATION (MIXTURE OF SODIUM HYDROXIDE AND SODIUM SULFATE) ON THE STRENGTH OF CEMENT - WISCONSIN SAND 1

Cement Content = 10.0% on dry soil wt

Total Additive Concentration (N)[a]	Ratio[b] of Na OH to Na_2SO_4 in Additive	Curing Days	Wet Compressive Strength (psi)	Method of Adding Additive
Control	---	1	25±5	
		4	20±0	
		7	19±1	
		28	23±2	
1.0	1 : 1	1	28±1	Together[c]
		4	45±5	
		7	57±6	
		28	71±2	
		1	29±5	Sulfate pretreatment[d]
		4	88±8	
		7	88±10	
		28	95±5	
1.5	1 : 2	1	12±1	Together[c]
		4	14±0	
		7	16±1	
		28	16±1	
1.5	1 : 2	1	176±6	Sulfate pretreatment[d]
		4	267±3	
		7	310±2	
		28	310±50	

[a] Normality of sodium in molding water.

[b] Equivalent basis.

[c] NaOH and Na_2SO_4 added mixed solution.

[d] Soil treated with sodium sulfate solution for 24 hours, cement and sodium hydroxide solution.

system. The fact that there is an optimum ratio of hydroxide to sulfate in this silt-cement clearly indicates that there must be a proper balance between the rate of attack of silica by the caustic and the rate of silicate precipitation by calcium in order to obtain both rapid cure and high ultimate strength.

Sand Containing Organic Matter - Soil WS 1 (10 Percent Cement). Figure 9 shows that the effectiveness of the additive increased with a decreasing hydroxide-to-sulfate ratio. It is rather interesting to note that the sequence of addition of reagents has an important effect on the effectiveness of the combined additive; Table 4 illustrates this importance. The first method was to add the combined additive solution as in all other cases; the second method was to pretreat the soil with sulfate solution, then add the cement and hydroxide. The final hydroxide-to-sulfate ratio was kept the same as in the first series. The beneficial effect of pretreating the soil with sulfate was particularly obvious when the additive concentration was increased from 1.0 to 1.5 Normal. These results further indicate that the unique effectiveness of sodium sulfate is due to depressing the reactivity of the organic components in the soil.

Clay - Soil TC 2 (10 Percent Cement). Figure 10 shows that there appears to be an optimum hydroxide-to-sulfate ratio, (about 4 to 1) that gives the greatest increase in strength to the clay-cement. The need of a small portion of sodium sulfate in the additive probably can be attributed to the presence of 3 percent organic matter in this soil.

Use of Secondary Additives to Sodium Hydroxide-Treated Cement-Clays

It has been observed (3) that significant volume changes occur during the curing and subsequent immersion for samples of cement-clays treated with sodium additives. The amount of volume change depends on the curing time and cement content: the shorter the curing time, the more the expansion; and the higher the cement content, the less the volume change. It is believed that the observed expansion on immersion and the attendant deterioration of samples with consequent low strength and ineffectiveness of sodium additives result primarily from partial conversion of the montmorillonoid components of the clays into the highly swelling sodium form. If the montmorillonoids could be converted to a less hydratable form, i.e., rendered less water-sensitive, or be waterproofed while still retaining the beneficial action of sodium hydroxide, higher strengths could be expected. Three types of chemicals were selected as secondary additives in addition to sodium hydroxide. They were polyvalent metal salts, octalamine, and cationic organic compounds, as listed in Table 3. The two clays, TC 2 and VBC, were pretreated with the secondary additive and equilibrated for 24 hours prior to the addition of cement and sodium hydroxide.

Table 5 summarizes the effect of those beneficial secondary additives in improving the properties of the two caustic-treated cement-clays. Figure 11 shows the effect of secondary additives on the strength development of soil TC 2-cement-caustic-mixtures.

For soil TC 2 stabilized with 5 percent cement, pretreatment with 0.5 percent Arquad 12, 0.1 percent Arquad 2HT, or 0.1 percent ferric chloride nearly doubled the effectiveness of sodium hydroxide on this clay-cement, producing strength higher than that with 10 percent cement; while 0.5 percent of 1.0 percent n-octylamine, and 1.0 percent Arquad 12 increased the strength of the clay-cement with 1.0 Normal caustic about 50 percent. However, none of the secondary additives tested was effective in TC 2 stabilized with 10 percent cement and 1.0 Normal caustic.

TABLE 5

EFFECT OF BENEFICIAL SECONDARY ADDITIVES[a] ON STRENGTH OF SODIUM HYDROXIDE-TREATED CEMENT-CLAYS

Soil	Cement Content (%)[b]	Sodium Hydroxide Content (N)[c]	Secondary Additive	Secondary Additive Concentration (%)[b]	1-Day Cure		28-Day Cure	
					Immersed Compressive Strength (psi)	Strength Ratio Treated to Untreated	Immersed Compressive Strength (psi)	Strength Ratio Treated to Untreated
Texas	5	---	---	---	172	---	184	---
Clay 2	5	1.0	---	---	81	0.47	220	1.20
	5	1.0	Ferric chloride	0.10	235	1.37	390	2.12
	5	1.0	Ferric chloride	1.00	81	0.47	228	1.24
	5	1.0	n-Octylamine	0.50	186	1.08	362	1.97
	5	1.0	n-Octylamine	1.00	255	1.48	335	1.82
	5	1.0	Arquad 2 HT	0.10	100	0.58	390	2.12
	5	1.0	Arquad 12	0.50	208	1.21	423	2.30
	5	1.0	Arquad 12	1.00	293	1.70	364	1.98
	10	---	---	---	229	---	315	---
	10	1.0	---	---	376	1.64	525	1.67
	10	1.0	n-Octylamine	1.00	257	1.13	558	1.77
Vicksburg	5	---	---	---	45	---	107	---
Buckshot	5	1.0	---	---	148	3.29	208	1.94
Clay	5	1.0	Arquad 12	1.00	147	3.27	260	2.43

[a] Soil pretreated with secondary additive prior to addition of cement and sodium hydroxide.

[b] Percent on dry weight of soil

[c] Normality of sodium in molding water.

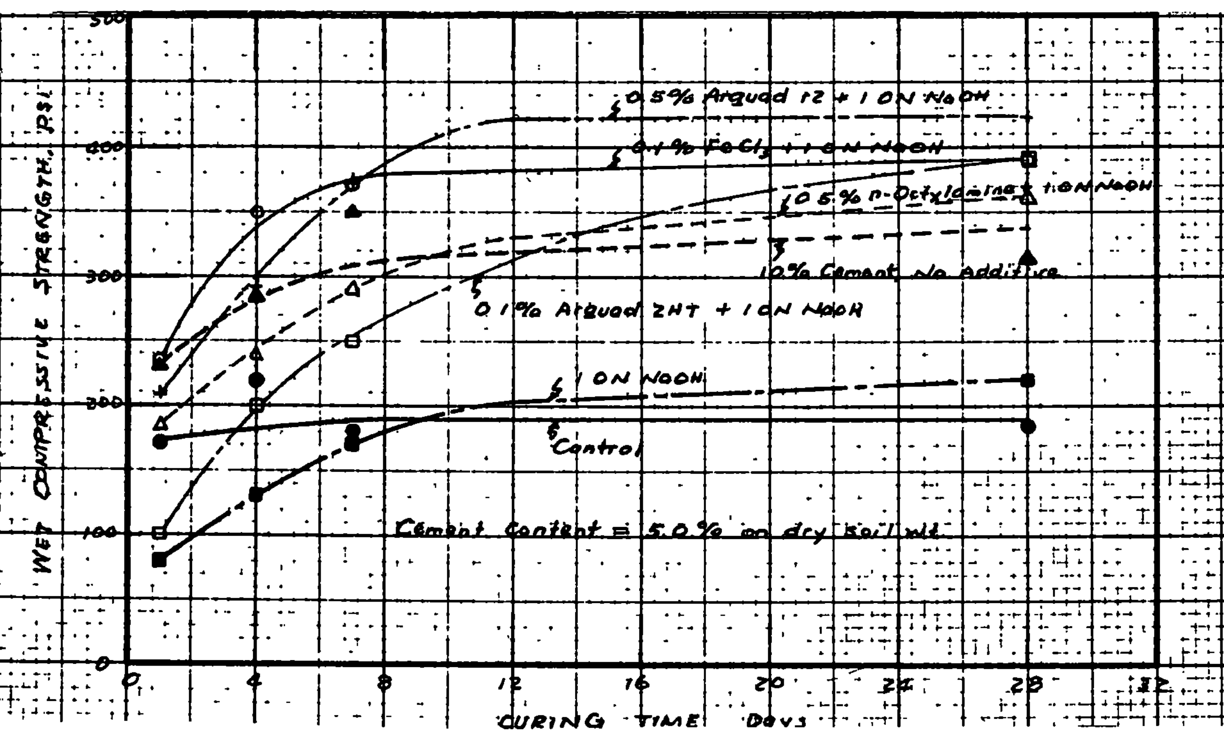

Figure 11. Effect of secondary additives on strength development of Texas clay-cement NaOH—treated.

Except with 1.0 percent Arquad 12, all of the secondary additives have an adverse effect on the strength of caustic-treated VBC-cement; although the measured strengths were higher than the control, they were lower than those obtained with caustic treatment alone. Nevertheless, it was observed that all these secondary additives reduced water pickup and swelling of the samples during immersion, as compared to the control and the caustic-treated samples.

It is noteworthy that the two heavy clays responded to cement and additive treatment to quite different degrees, although in the same direction. A comparison of these two clays shows Soil TC 2 has higher clay content, more organic matter, much higher pH, more soluble salts, and higher glycol retention, but slightly lower plasticity and exchange capacity than VBC. All these differences in properties indicate that Soil TC 2 should be less responsive to cement-additive stabilization. Two possible explanations for the inferior response of VBC are (1) the soil is too acidic or (2) the organic matter though less in quantity, is of a more reactive form than that in Soil TC 2. Furthermore, a more detailed mineralogical analysis was conducted, and the results indicate that in VBC the montmorillonoids are mostly montmorillonites, but in TC 2 they are mostly non-montmorillonites. This difference may be the main factor responsible for the different behavior of the two clays.

Effect of Soda-to-Silica Ratio in Sodium Silicate as Additive on the Strength of Cement-Stabilized New Hampshire Silt

Because the rate of strength development of additive-treated soil-cement depends on the ratio of the alkali silicates to dissolved calcium and because the ultimate strength is a function of the total amount of cementitious material formed, it was reasoned that sodium silicate should be effective in accelerating the cure rate as well as improving the final strength. Sodium metasilicate has been shown to be one of the most effective additives for New Hampshire silt stabilized with 5 percent cement. Besides providing reactive silicates, sodium silicate also raises the pH of the soil-cement mixture and attacks the soil constituents. Therefore, it could be reasoned that the ratio of soda to silica in sodium silicates should have an important effect on the strength development of soil-cement.

Three sodium silicates with soda-to-silica ratios (Na_2SiO_2) varying from 2:1 to 1:3.22 were examined; the results are shown in Table 6 and Figure 12.

TABLE 6

EFFECT OF SODA-TO-SILICA RATIO IN SODIUM SILICATE AS ADDITIVE TO NEW HAMPSHIRE-CEMENT

(Cement Content = 5.0 Percent on dry soil wt)

Additive	Ratio of Na^2O to SiO_2	Additive Concentration		Curing Days	Wet Compressive Strength (psi)
		(N)[a]	(%)[b]		
Control				1	80±0
				7	95±0
				28	125±5
Sodium orthosilicate	1:0.5 (2:1)	0.50	0.54	1	175±0
				7	200±0
				28	352±8
		1.00	1.03	1	217±15
				7	286±14
				28	491±51
Sodium metasilicate	1:1	0.50	0.60	1	130±5
				7	202±6
				28	305±35
		1.00	1.33	1	135±15
				7	218±7
				28	344±45
Grade 50 silicate	1:2	0.51	1.00	1	123±28
				7	420±10
				28	553±3
		1.00	1.98	1	0[c]
				7	462±0
				28	626±24
Grade 40 silicate	1:3.22	0.40	1.00	1	290±2
				7	386±14
				28	530±35
		1.00	2.80	1	0
				7	40±20[d]
				28	607±23

[a] Normality of sodium in molding water.

[b] Percent of solid on dry soil weight.

[c] Specimens disintegrated upon immersion.

[d] Specimens partially disintegrated in water.

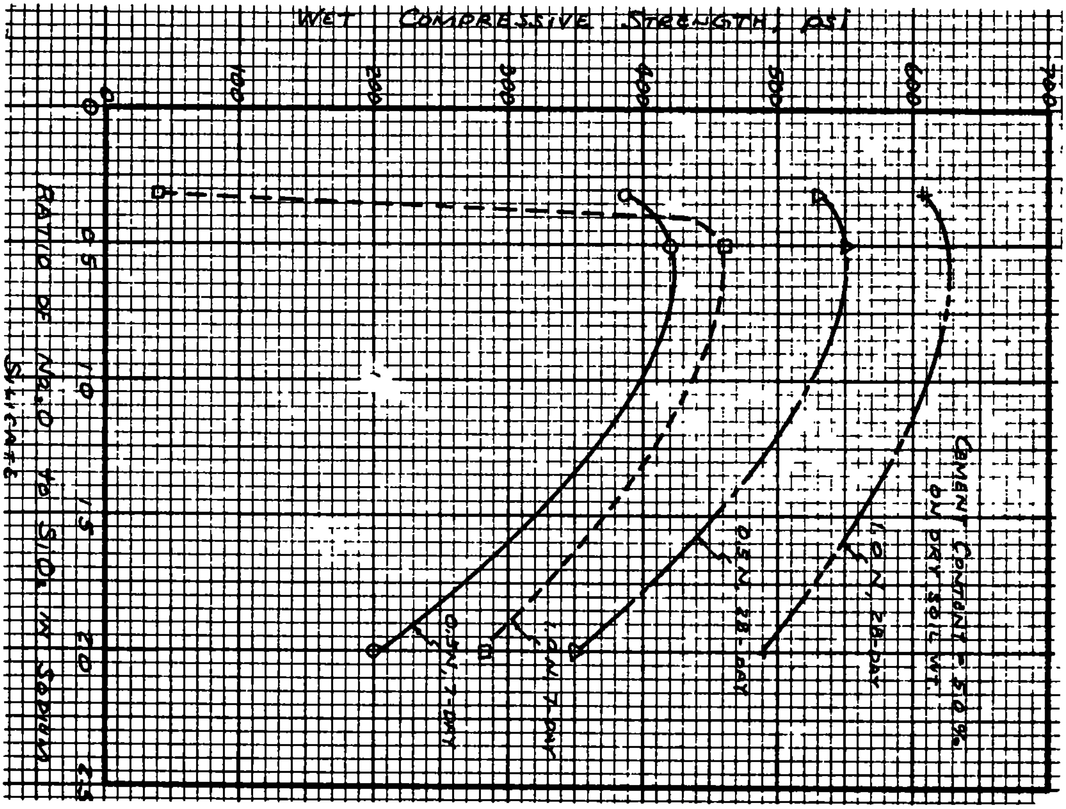

Figure 12. Effect of soda-to-silica ratio in sodium silicate as additive to New Hampshire silt-cement.

Careful examination of the results obtained with the three silicates suggests that, if either the 7-day or 28-day strength is used as a quantitative evaluation of additive effectiveness, there is an optimum soda-to-silica ratio for the most effective sodium silicate additive. This ratio appears to be 1:2 at both 0.5 and 1.0 Normal concentrations. At a given sodium concentration, the higher soda-to-silica-ratio silicate gave higher early strength but lower ultimate strength. The relatively smaller amount of silica added in orthosilicate could probably be readily precipitated by the available calcium. With increasing silica, the rate of diffusion of calcium is not sufficiently fast to precipitate all the alkali silicate in a short period of time. In other words, around the soil particles there are free soluble alkali silicates that have no cementing power. Therefore, a delay of strength development might be expected. On the other hand, the ultimate strength depends on the total quantity of cementitious material, which in turn is a function of the total amount of silicates and calcium available. Higher strength would thus be produced by the lower soda-to-silica silicate (Grade 40). However, the ultimate strength is also limited by the total calcium available, which depends on the cement content; if the amount of silicate added is too high, there is not enough calcium either to precipitate it or to form a low-calcium silicate hydrate, and consequently lower strength would occur. The results with specimens treated with 1.0 Normal Grade 40 illustrate this trend. Therefore, it can be reasonably predicted that the optimum soda-to-silica ratio would move toward a lower value if the cement content were increased.

The results with metasilicate do not fit into the pattern of these three silicates. This may be due to the difference in structure between metasilicate and the other silicates.

CONCLUSIONS

The laboratory test results presented in this paper further indicate the beneficial effects of chemical additives for improving the engineering properties of soil-cement. Detailed conclusions are as follows:

1. Sand-cement (no additive) deteriorates when in contact with sulfate solution; cement stabilized clays are susceptible to sulfate attack only after long periods of contact with concentrated sulfate solution.
2. Sodium additives considerably increase the resistance of all types of soil-cement to sulfate attack.
3. Sulfate compounds are uniquely effective in improving the strength of cement-stabilized sandy soils containing organic matter. At 0.5 Normal concentration, magnesium sulfate and calcium sulfate anhydrite are more effective than sodium sulfate; while at 1.0 Normal concentration, the order of effectiveness of the sulfate compounds is sodium, calcium anhydrite, gypsum, and magnesium.
4. Attempts to find a general formulation of sodium additive for all soil types have not been successful. Results obtained with the general formulations, e.g., combination of sodium hydroxide and sodium sulfate in various proportions, are in the same trend as each chemical used individually.
5. A molar ratio of 1:1 of sodium hydroxide to sodium sulfate appears to be the optimum for increasing strength of NHS with 5 percent cement, and a ratio of 4:1 gives highest strength improvement to TC 2 with 10 percent cement. The strength of WS-1 cement decreases with increased molar ratio of the general additive formulation.
6. The effectiveness of sodium hydroxide in clay-cement can be materially improved by pretreating the heavy clays with secondary additives.
7. Pretreatment with 0.5 or 1.0 percent Arquad 12, 0.1 percent Arquad 2HT, 0.1 percent ferric chloride, or 0.5 or 1.0 percent n-octylamine increases the effectiveness of sodium hydroxide on soil TC 2 stabilized with 5 percent cement, producing strengths higher than that 10 percent cement.
8. The strengths of VBC stabilized with 5 percent cement and 1.0 Normal caustic was not materially improved by use of the secondary additives tested.
9. A soda-to-silica ratio (Na_2O/SiO_2) of 1:2 in sodium silicate (at both 1.0 and 0.5 Normal) appears to be optimum for improving strength of New Hampshire silt with 5 percent cement. Higher soda-to-silica-ratio silicate tends to give higher early strength but lower ultimate strength, while lower ratio silicate retards the cementing process.

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Moisture Content Determination by the Calcium Carbide Gas Pressure Method

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This paper is concerned with a moisture testing instrument that employs a chemical in the rapid determination of moisture content in soils. A brief description of the moisture tester is given along with the recommended procedure for its use. A testing program that investigated temperature build-up in the moisture tester is described. Calibration curves of direct moisture tester readings versus oven-dry moisture contents and a description of the procedure used in checking the validity of the curves are presented. Some of the possible application of the moisture tester for determination of moisture contents at site locations and field laboratory locations are listed.

● A QUICK, ACCURATE METHOD of determining moisture contents in soil materials has long been one of the goals of soil engineers. Researchers have devoted much effort in developing gravimetric, chemical, electrical, nuclear, penetrometer, tension, and thermal methods for the determination of the moisture content of soils (1). Among these efforts, a chemical method using calcium carbide as a reagent has shown great promise.

The principle involved in this chemical method for moisture determination is that a given quantity of moisture will react with calcium carbide to produce a specific volume of gas (acetylene). The reaction is as follows: $CaC_2 + 2H_2O = Ca(OH)_2 + C_2H_2$.

A device developed on this principle in England confines the gas produced from the reaction in a pressure vessel. A gauge located in the base of the vessel is calibrated to read the gas pressure in percent moisture based on the wet weight of the sample.

Although the device has been used in England for several years in connection with moisture determinations of foundry sand, it has only been used by highway engineers in the United States for about two years. This paper describes the results of a study by the Bureau of Public Roads to determine the accuracy and usefulness in highway soil testing of a commercially manufactured "Moisture Tester."

APPARATUS AND TESTING PROCEDURE

The moisture tester used in this study is a hollow aluminum vessel having a pressure gauge on one end and a cap with a clamping arrangement on the other. The manufacturer supplies two sizes of moisture testers, one for a 6-gm sample and the other for a 26-gm sample. Only the larger device, (Figure 1) having approximate external dimensions of 14 x 16 in. and a weight of 3.7 lb was used in this study. The moisture tester is equipped with a carrying case, a tared scale for weighing the sample to be tested, a small scoop for measuring the calcium carbide, and a table to convert the percent moisture to the dry weight basis used in soils work. Figure 2 shows the complete apparatus.

Preliminary tests using the procedure recommended by the manufacturer indicated that some changes in procedure were desirable. Consequently, a variety of soil samples passing the No. 4 sieve, and ranging from A-2 through A-7, were used in a procedure study. In order to arrive at a procedure that would give moisture contents as close as possible to those determined by oven-drying, the soil samples were tested at varying moisture contents in the moisture tester and by the standard oven-dry proce-

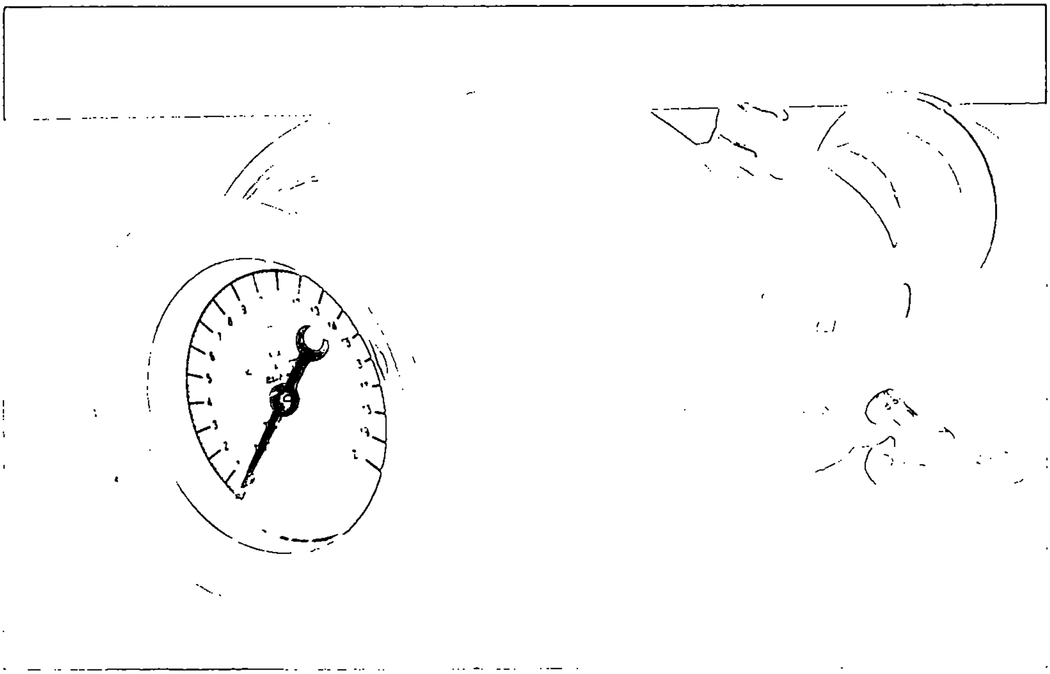

Figure 1. Moisture tester.

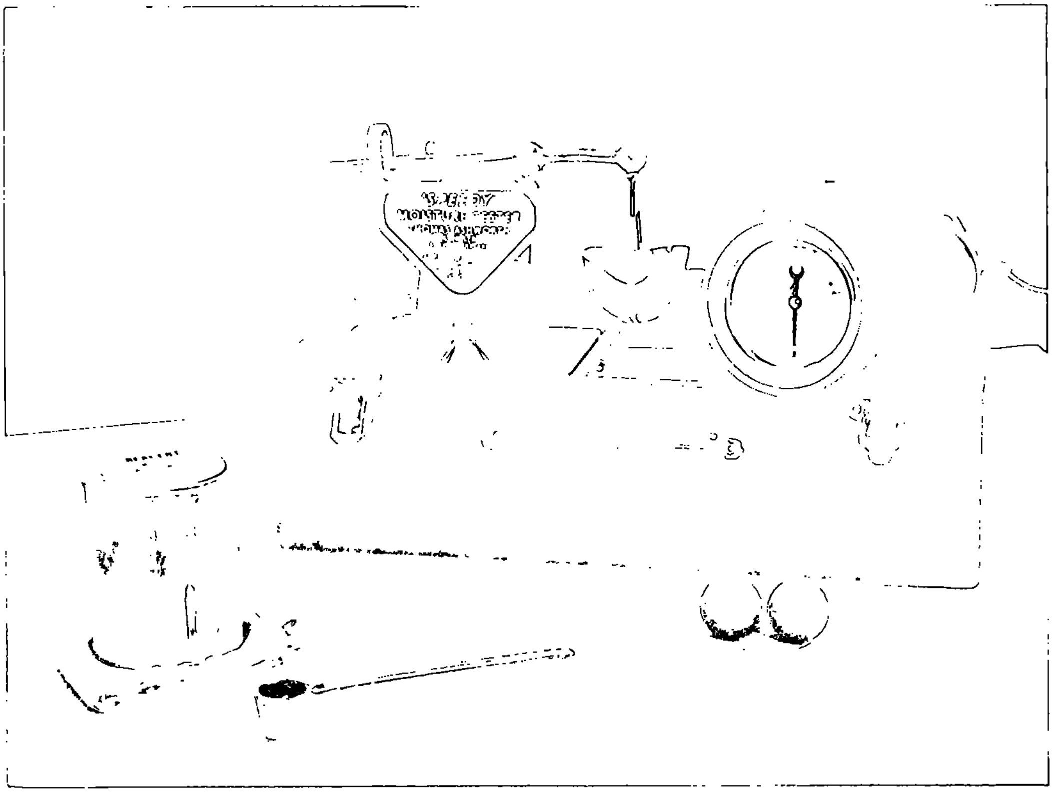

Figure 2. Moisture tester with accessories.

dure, and tests using varying combinations of shaking time, amount of reagent, and types (shape, size, and weight) of pulverizers to break up clay lumps were performed. The most effective pulverization of clay lumps was obtained by placing two 1¼ in. steel balls, weighing 0.4 lb each, in the moisture tester with the soil sample and calcium carbide. The procedure outlined in the Appendix was eventually adopted for use of the moisture tester.

TEMPERATURE STUDY

The chemical reaction of the calcium carbide with the moisture in some soils caused a rapid rise in temperature in the moisture tester, hence it was decided to study this thermal effect more fully. An experimental program was carried out to determine if a correlation could be made between temperature and moisture tester readings. The characteristics of the 11 selected soils, ranging from A-2-4(0) to A-7-5(20), are given in Table 1.

A thermometer was taped to the outer surface of the moisture tester at about its midpoint. Although the thermometer was insulated from external temperatures, it was recognized that the temperature recorded was not the actual temperature inside the moisture tester. However, for purposes of determining whether a correlation existed between temperature and moisture tester readings, it was not considered essential to know the actual inside temperature of the tester. After adding water to the soil, the moisture content was determined by (1) the standard oven-dry method, and (2) the moisture tester, using the procedure in the Appendix, except that readings were taken at 1-min intervals. When a soil was tested in the moisture tester, the temperature prior to mixing the soil and the calcium carbide was recorded. The soil and chemical were then mixed and readings of the pressure gauge and thermometer were recorded at 1-min intervals until the temperature returned within 5 degrees F of the initial temperature. Moisture was added to the selected soil in increments of about 2 percentage points until the moisture content was past the optimum moisture content.

TABLE 1

PHYSICAL CHARACTERISTICS OF SOILS USED IN TEMPERATURE STUDY

Soil Classification	Liquid Limit	Plasticity Index	Optimum Moisture Content[a] (%)	Range of Moisture Contents Tested[b] (%)
A-2-4(0)	24	6	14[c]	8.8 - 16.9
A-3(0)	NP	NP	9[c]	4.3 - 15.2
A-4(2)	30	6	17[c]	11.8 - 22.0
A-4(2)	20	1	10	5.8 - 15.2
A-4(8)	26	3	15	9.6 - 19.8
A-4(3)	35	9	15	11.1 - 22.3
A-5(8)	41	8	20	14.1 - 25.5
A-7-5(6)	42	11	20	15.1 - 28.0
A-7-5(14)	52	18	23	18.4 - 29.1
A-7-5(19)	66	27	27	23.8 - 34.4
A-7-5(20)	82	47	33[c]	29.4 - 41.9

[a] AASHO Designation T 99-57, Method A.

[b] Moisture contents determined by standard oven-dry method.

[c] Estimated optimum moisture content.

Two successive tests were performed with the moisture tester for each increment of moisture tested. The readings were often identical or within 0.1 percentage point of each other, and most were within 0.5 percentage point of each other. The range of moisture contents tested for each soil is shown in Table 1. If the moisture contents of the soils exceeded the limit of pressure gauge, half-size samples (13 gm) were used and the percentage indicated on the gauge was then doubled. There were 1,598 recordings of moisture tester readings and temperature for the selected soils in the temperature study.

The moisture tester readings were converted to a dry-weight basis using the conversion chart supplied by the manufacturer. Diagrams of moisture content versus temperature were plotted for 1-min intervals until the temperature returned to within about 5 deg of its initial value. The oven-dry moisture content was also plotted as a vertical line on each diagram. Some typical diagrams are shown in Figure 3. The curves fall to the right of the oven-dry moisture content line, indicating that the moisture contents determined by the moisture tester were greater than by oven-drying. The greatest moisture content values were at the 1-min moisture tester observations. Those moisture content determinations made from the moisture tester observations at

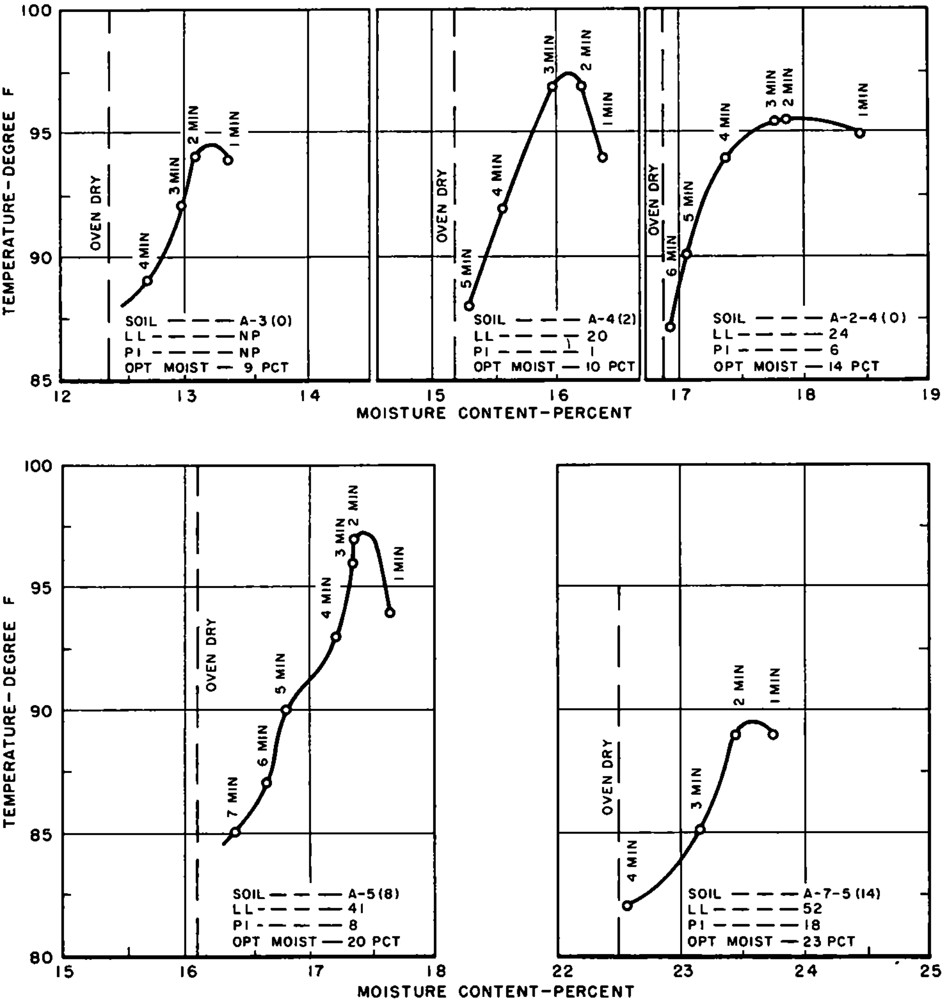

Figure 3. Moisture content by moisture tester versus temperature.

4, 5, and 6 min more nearly approach the oven-dry moisture content of the soil than those at lesser time intervals. It was anticipated that a relation might exist between the peak points of the curves and the oven-dry moisture contents; however, no such relation could be determined.

CALIBRATION CURVES

The moisture content data from this temperature-moisture program were used in developing two calibration curves of direct moisture tester readings versus oven-dry moisture contents. Tests covering a range of moisture contents from 4.3 to 41.9 percent were used in developing the calibration curves in Figures 4 and 5. The direct moisture tester readings at 1 min in Figure 4, and 3 min in Figure 5 are plotted versus oven-dry moisture contents. The 1-min readings were selected for a calibration curve since the first recordings of the pressure gauge of the moisture tester were taken at 1 min. A calibration curve was developed for the 3-min readings since this time is not considered to be excessive for running tests in the field and is adequate for the reaction of the calcium carbide with moisture to occur in clay soils. Either 13-gm or 26-gm samples were used in the development of the curves.

In order to establish the validity of the calibration curves and to compare the deviations using the 1- and 3-min curves, 6 additional soil samples (Table 2) were selected for the moisture testing. Each sample was tested at moisture contents bracketing the optimum (AASHO Designation T 99). The oven-dry moisture content was determined for each moisture increment added to the 6 soils. Using the direct moisture tester readings at 1 and 3 min and the 1- and 3-min calibration curves (Figures 4 and 5), a predicted oven-dry moisture content was determined. These predicted values, along

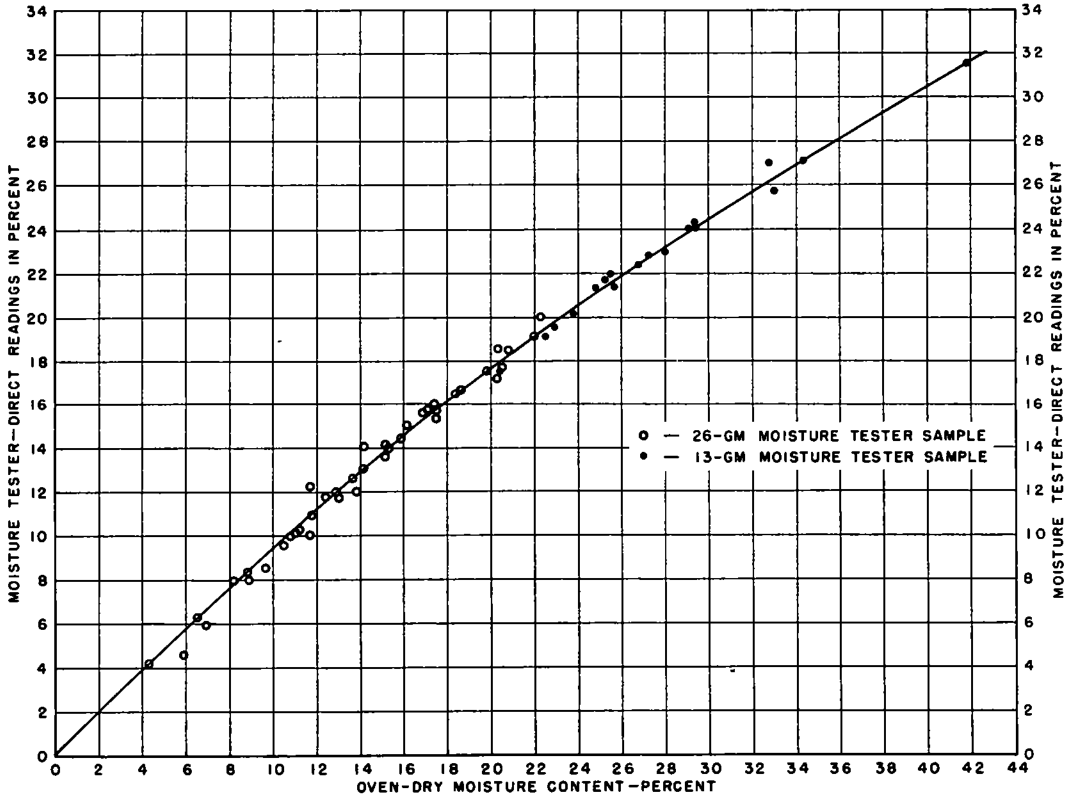

Figure 4. Calibration curve for 1-min moisture tester reading.

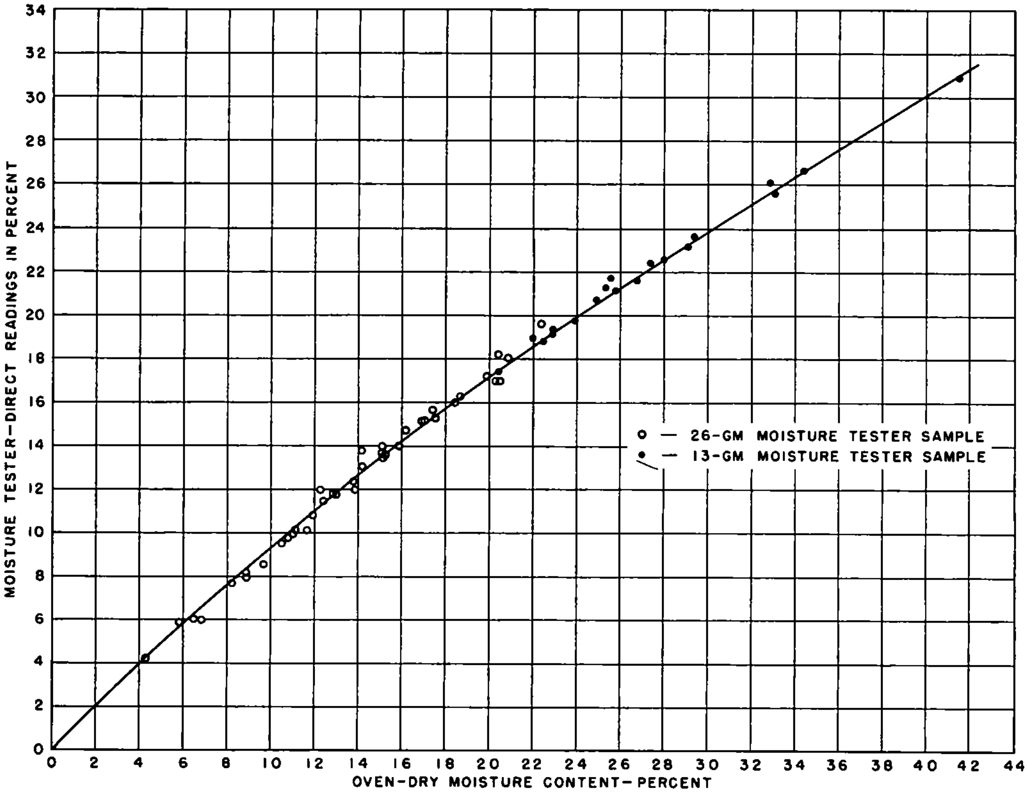

Figure 5. Calibration curve for 3-min moisture tester reading.

TABLE 2

COMPARISON OF MOISTURE CONTENTS BY MOISTURE TESTER AND OVEN-DRYING

AASHO Soil Classification	Moisture Content by					Moisture-Content Difference[a]	
	Dial Reading on Moisture Tester		Calibration Curves		Oven-drying (%)	1 Min (Percentage Point)	1 Min (Percentage Point)
	1 Min (%)	3 Min (%)	1 Min (%)	3 Min (%)			
A-2-4(0)	5.0	5.0	5.2	5.1	5.3	0.1	0.2
	10.7	10.5	11.4	11.3	11.2	-0.2	-0.1
	13.8	13.8	15.1	15.4	15.7	0.6	0.3
A-4(5)	8.9	8.7	9.5	9.3	9.1	-0.4	-0.2
	12.5	12.3	13.5	13.6	13.5	0.0	-0.1
	18.0	17.6	20.5	20.6	20.1	-0.4	-0.5
A-6(10)	11.8	11.6	12.6	12.6	12.8	0.2	0.2
	15.2	14.9	16.7	16.8	16.5	-0.2	-0.3
	19.2	18.7	22.0	22.1	21.4	-0.6	-0.7
A-7-6(9)	14.4	14.0	15.7	15.6	15.6	-0.1	0.0
	18.9	18.3	21.6	21.5	20.4	-1.2	-1.1
	22.4	21.9	26.7	26.8	26.7	0.0	-0.1
A-7-5(3)	19.6	19.4	22.6	23.1	24.5	1.9	1.4
	23.0	22.6	27.7	28.1	28.9	1.2	0.8
	27.0	26.4	34.2	34.0	35.0	0.8	1.0
A-7-5(20)	22.4	22.2	26.8	27.5	29.0	2.2	1.5
	25.2	24.8	31.2	31.4	32.4	1.2	1.0
	29.6	28.8	38.5	38.0	37.9	-0.6	-0.1
				Average deviation		0.7	0.5

[a] Deviation of moisture content by moisture tester (calibration curve) from oven-dry moisture content

with the oven-dry values and the deviation between the two sets of values, are shown in Table 2. The moisture contents obtained from the 3-min readings tend to be more accurate than those for 1-min readings. For example, the A-7-5(20) soil had the greatest deviations from true moisture content, but the deviation of moisture-tester from oven-dry moisture content was 2.2 percentage points for the 1-min reading and only 1.5 for the 3-min reading. The average deviation using the 1-min calibration curve in Figure 4 is 0.7 percentage point, while the average deviation using the 3-min calibration curve in Figure 5 is 0.5 percentage point. Although there is only a 0.2 percentage point difference in average deviations between the 1- and 3-min readings, it is recommended that the moisture tester readings be taken at 3 min, particularly for clay samples, (1) to insure complete reaction of the calcium carbide with the moisture, and (2) to achieve greater accuracy.

CONCLUSIONS AND APPLICATIONS

The moisture tester has been in use in the Soils Laboratory of the Bureau of Public Roads for over a year. It has proven to be a sturdy, dependable, and reasonably accurate instrument. Above all, it has proven to be fast and easy to operate. It is believed that by using the procedure outlined in the Appendix, an inexperienced operator will become proficient in a short time. By using the calibration curve in Figure 5, one can convert direct moisture tester readings to a dry-weight basis and obtain a moisture content that agrees closely with that obtained by oven-drying. Inasmuch as the calibration curve was developed for the specific moisture tester device, it is not known whether the curve is applicable to each tester supplied by the manufacturer. Consequently, a State highway department or other agency that purchases the moisture tester should either (1) perform check tests with each device to determine that the calibration curve in Figure 5 is applicable, or (2) develop a calibration curve for the specific moisture tester and local soils.

The calibration curve could be used in the production of a direct reading dial that would (1) expedite moisture content determinations, and (2) eliminate errors in reading the calibration curve.

There are many applications of the moisture tester in highway engineering. The moisture tester is well suited for the control of materials and construction practices where reasonable accuracy and rapid operations are required. The moisture tester can also be used as a quick check for field laboratory tests involving moisture content determinations. Some of the possible applications of the moisture tester are as follows:

Site locations

1. In-place density tests.
2. Auger and split-spoon samples.
3. Sands used in concrete mixtures.
4. Proper moisture content for earthwork.

Field laboratory locations

1. Hygroscopic moisture.
2. Development of compaction curves.
3. Low-value liquid and plastic limits.

Thirty-four State highway departments have reported that they have made tests with the calcium carbide moisture tester and several are using the testers in construction control. This interest by the State highway departments indicates that the calcium carbide moisture tester is rapidly being accepted by the construction industry and the engineering profession.

REFERENCES

1. Shaw, M.D., and Arble, W.E., "Bibliography on Methods for Determining Soil Moisture." Penn. State Univ., College of Eng. and Arch., Eng. Res. Bull. B-78 (1959).

APPENDIX

Procedure for Use of 26-gm Moisture Tester

1. Place three measures (approximately 22 gm) of calcium carbide and two 1¼ in. steel balls in the large chamber of the moisture tester.
2. Using the tared scale, weight a 26-gm sample of soil.[a]
3. Place the soil sample in the cap; then with pressure vessel in a horizontal position, insert the cap in the pressure vessel and tighten clamp to seal cap to unit.
4. Raise moisture tester to a vertical position so the contents from the cap fall into the pressure vessel.
5. Holding the moisture tester horizontally, rotate the device for 10 sec so that the steel balls are put into orbit around the inside circumference and then rest 20 sec. Repeat shake-rest cycle for a total of 3 min. Do not allow the steel balls to fall against and damage either the cap or orifice leading to the dial.
6. Read pressure gauge of the moisture tester and determine moisture content on a dry-weight basis from calibration curve.

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[a] If the moisture content of the 26-gm soil sample exceeds the limit of the pressure gauge, a half-size sample can be used; the percentage indicated on the gauge is then doubled. Since the sample to be tested is relatively small (13 or 26 gm), care must taken to obtain a representative soil sample.

Developments in Radioisotope Measurement of Soil Moisture Content and Density

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The paper describes research and development work carried out on the technique of determining the moisture content of soil by neutron irradiation and density by back-scattering of gamma rays. A new instrument is described that employs a single radium-berryllium source giving both neutrons and gamma rays. The source is used in a single probe for surface work or in a single depth probe for measurements at depth. Some new features in the apparatus include a means of controlling the effective depth to which density is measured and the use of special reflecting devices to increase the sensitivity in moisture determinations.

Extensive studies of the volume and depth of soil measured and of the problem of achieving accurate calibration of the instrument over the practical range of moisture content and density are described. Examples of different fields of application of the instrument are given.

●EFFECTIVE CONTROL of the compaction of soil or stone layers is an important factor in the construction of roads, airfields, and other types of foundations for civil engineering structures. This control involves the measurement of in-place densities and moisture contents. Of various tests commonly used for making these determinations, the sand-replacement density test and the moisture content determination by oven drying are perhaps the most generally used.

Work on a nondestructive technique by which the density and moisture content of soil can be tested in situ has been in progress for a number of years, principally in the U.S.A. and, to some extent, in Great Britain and some continental countries (1, 2, 3, 4, 5, 6).

The method that has shown most promise and on which most work has been done has used the radiation back-scatter technique. In this technique, soil density is determined by the amount of gamma radiation back-scattered and soil moisture is determined by the thermalization and scattering back of neutrons. Density measurements require radioactive sources emitting gamma radiation and moisture measurements require high energy or fast neutron radiation. In the instrument described in this paper, it has been found satisfactory and extremely convenient to use a single radium-beryllium source, emitting both gamma rays and fast neutrons.

Although considerable work had already been done on laboratory instruments overseas and, at least two commercial instruments were already available, it was felt that an instrument could be developed in South Africa that would include certain desirable and important features for field work, as well as general improvements in the instrumentation of the equipment, particularly in the reduction of electric power requirements with its consequent saving in weight. Accordingly, a development program was started, both in designing and field testing the necessary portable electronic counting equipment, as well as in developing field techniques for the use of the apparatus. The instrument now developed consists of a transistorized, battery-operated scaler and two types of combination probe: (a) a surface probe, and (b) a depth probe that can be lowered down a 2-in. (internal) diameter pipe to depths up to 100 ft (Figure 1).

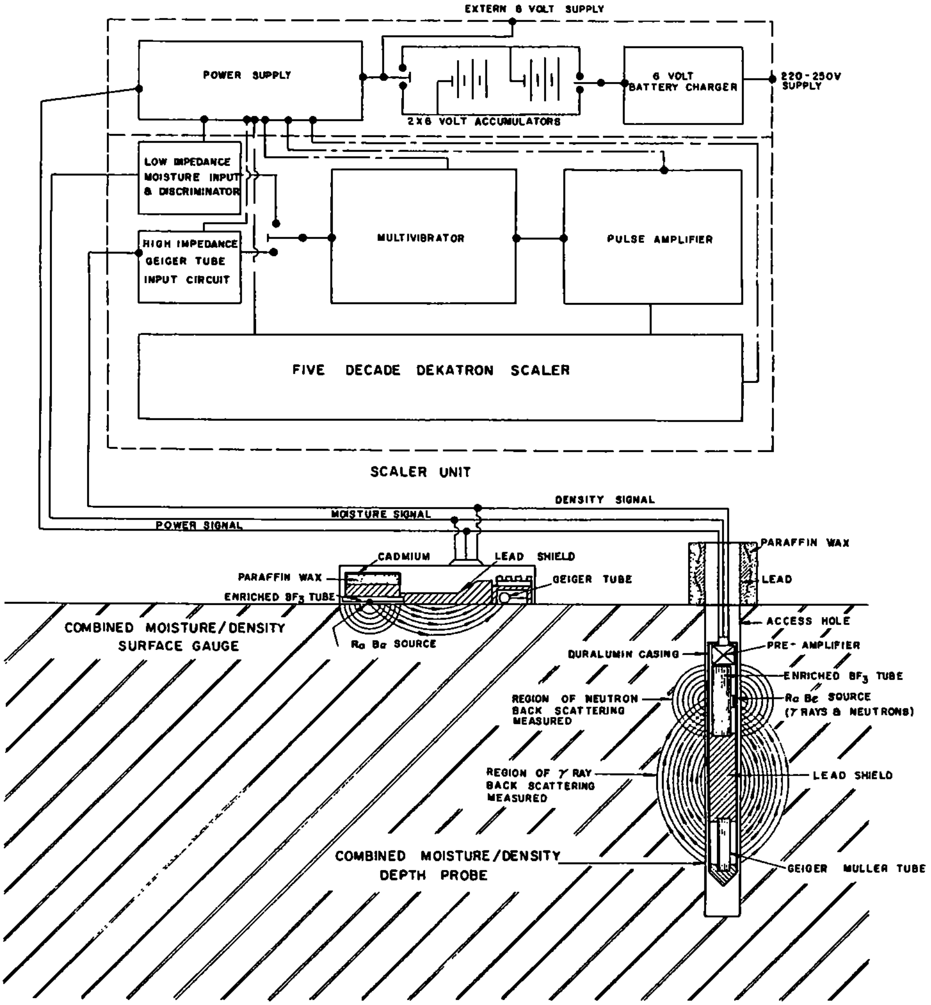


Figure 1. Diagrammatic sketch of moisture density apparatus.

PRINCIPLE OF OPERATION

The basic physical principles involved in this method are probably now so well known as to require only a brief mention here.

The surface probe contains at the bottom the radioactive source and suitable detector tubes protected from direct radiation by lead screening. When the probe is placed on the soil surface the gamma radiation from the source is emitted in all directions into the soil, where it is either absorbed or scattered back towards the Geiger-Müller detector tube by the Compton scattering process. The measured amount of radiation is indicated on a suitable counting unit. For the detector geometry employed, the quantity of back-scattered gamma radiation is inversely proportional to the total soil density (including water) in the range of about 70 to 170 pcf and is virtually independent of soil composition.

The fast neutrons from the source are also emitted into the soil in all directions and undergo collision with the nuclei of the different atoms present. If a nucleus is of the same order of mass as the neutron, the neutron undergoes an inelastic collision, loses a large proportion of its energy to the nucleus, and becomes a slow neutron. If, on the other hand, the nucleus is very much heavier than the neutron, collision is elastic; the neutron loses very little energy and remains a fast neutron. Inasmuch as the hydrogen nucleus has nearly the same mass as the neutron, hydrogen is the most effective element in slowing down or moderating the neutrons, whereas the nuclei of the other materials present in soil are generally much heavier than the neutron and so have comparatively very little moderating effect. After repeated collisions with hydrogen atoms, a proportion of the neutrons that have been slowed down are scattered back towards the probe. These are then detected by boron trifluoride tubes, which are sensitive only to slow neutrons, and the neutron count is again indicated on the counting unit. The count rate (i.e., the number of neutron pulses counted by the scaler per unit time) is then proportional to the number of hydrogen atoms in the soil and hence proportional to the moisture content of the soil. Organic matter also contains hydrogen, but this material is normally present in soils used in road construction only in very small amounts.

GENERAL DESCRIPTION OF INSTRUMENT

The general arrangement of the equipment is shown in Figure 1. Figure 2(a) shows a view of the equipment connected for use on the road. Figure 2(b) shows a set of replaceable plug-in units that make up the scaler circuits.

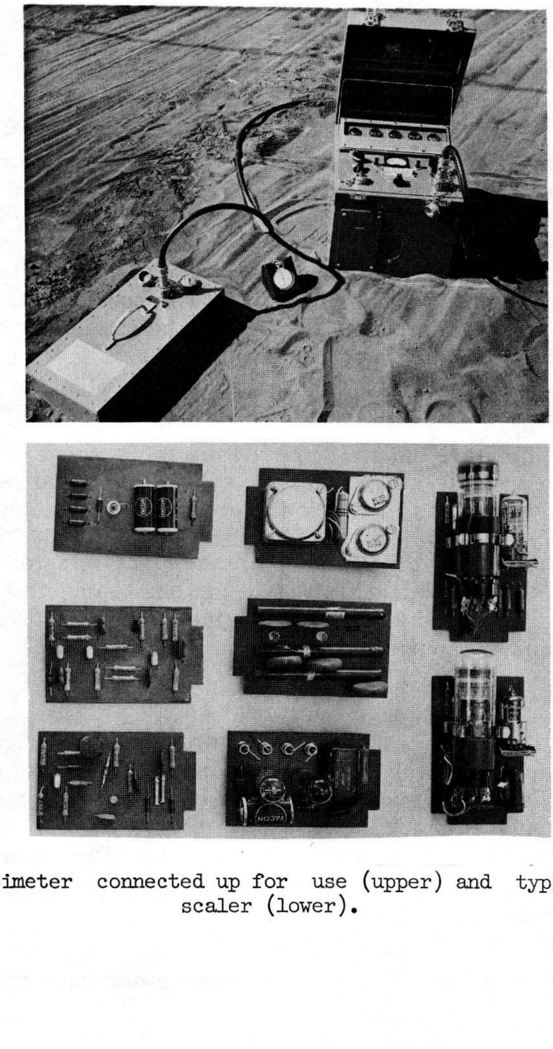


Figure 2. Hidrodensimeter connected up for use (upper) and typical plug-in units of scaler (lower).

Probe and Cable

The probe contains a 10 millicurie radium-beryllium source in a holder situated at one end. Screened for direct transmission from the source by 9½ in. of lead is the detector tube for gamma rays, an Anton, type 106C, halogen-quenched Geiger-Müller tube, operating at about 900 volts. Provision was also made to slide the Geiger-Müller tube horizontally so as to vary its distance from the source. In the "full-in" position referred to in this paper, the source-detector distance was 9½ in. In the "full-out" position this distance is increased to 13 in. At the "mid" position the distance is 11¼ in.

Also mounted in the probe are two boron trifluoride tubes (12EB40), copper-walled, operating at about 1800 volts. These two tubes lie at an angle, with the source between them.

Output pulses from the BF_3 detector tubes are fed to a four-stage transistorized pre-amplifier giving an overall gain of 150, with low impedance output into the probe socket. The signals from the Geiger-Müller detector tube are fed directly to the probe socket. A six-core cable, each core separately screened, carried all supply voltages and signals between the probe and scaler. The cable is terminated at each end with "Plessey" plugs mating into standard sockets. The six cores carry the following:

1. Moisture signal input from pre-amplifier in probe to scaler.
2. Up to 1, 000 volts for the Geiger-Müller tube.
3. Negative 6 volts for pre-amplifier supply.
4. Earth.
5. Earth.
6. Up to 2, 000 volts for the boron trifluoride tubes.

Reference Standard for Surface Probe

The surface probe has been designed with heavy top and side screening to protect the operator from stray radiation. The only emergent radiation is downwards. A special container has been designed for storage of the probe when not in use. In this container the probe rests on a 10-in. thickness of hard paraffin wax that absorbs neutrons. A block of lead is positioned in the wax under the source to absorb the gamma radiation. The wax also serves to provide reference counts for both "density" and "moisture." If the whole instrument remains in good working order, these reference counts will remain constant.

Scaler

The "density" and "moisture" signals are fed via two separate input circuits, the density signals through a single-stage transistor to the moisture/density selector switch and start/stop buttons and thence to the grid of the first dekatron-triggering stage, a hard valve (type 6AK 5). The moisture signals are fed via a single-stage transistor and multi-vibrator to the variable voltage discriminator, and hence via the moisture/density selector switch and start/stop buttons to the first dekatron stage. The discriminator control can be adjusted under the dummy knob on the control panel to avoid unwanted gamma-ray input.

Dekatron Counting Circuits

The first-stage dekatron uses a hard valve, a 6AK 5, as its triggering stage, the resolution time being approximately 33 microseconds. The output of this stage is fed to the trigger tube of the second stage. The remaining three stages are fed in a similar way, the total count displayed being 99, 999.

Power supply for the counting circuits is derived from the dekatron drive unit in the power supply section.

Power Supply Unit

This unit consists of two P. N. P. power transistors driving a pot core into saturation at a frequency of 1 kcs, producing a square wave of 20 volts amplitude peak to peak with an input of 6 volts D.C. A second transformer, also ferroxpot core, is fed from the 1-kcs transformer, the secondary being tapped to provide the necessary voltages for the complete equipment.

Battery Supply

The total power consumption of the equipment is only 9 watts. The battery used is a 6-volt Venner silver-zinc type, of capacity 15 ampere-hours. Due to the very flat discharge characteristics, at the normal current consumption of 1.5 amps, a fully charged battery should give a total operating time of about 10 hours. The battery voltage can be checked with the meter on the control panel by depressing the test button immediately to the left of the meter. Similarly, the high voltages for both the density and the moisture circuits can be checked by depressing the test button immediately to the right of the meter.

If the built-in battery voltage is low, a 6-volt external battery can be used by connecting into the socket on the scaler and switching the main control to the appropriate position.

Certain troubles have been experienced with the battery and voltage regulation and further work is in hand on this.

Charging Arrangements

For charging the internal battery from A.C. mains, a built-in battery charger is provided. The charger has a selector switch for 110, 200, 220, and 250 volts supply and regulates the current initially at 0.75 amp. This is reduced as the battery reaches the fully charged state. Charging is done by connecting the cable into the socket (as for the external battery) and switching the main control to the "charge" position.

The Depth Probe

The arrangement in the depth probe differs from that in the surface probe in that the Geiger-Müller tube lies along the axis of the probe (see Figure 1) and the single BF_3 tube lies along the probe axis with the source located against the center of the active portion. The change-over is made sinply by unplugging the cable from the surface probe and plugging the depth probe cable into the counting unit.

Counting Time

The emission of gamma rays and neutrons from the radioactive source is not absolutely constant with time but has a random distribution. Experience has shown that a minimum of 10,000 counts on the scaler is required in any particular measurement to ensure that the error from this source does not exceed 1 percent. Under average test conditions on roads, a count for 1 minute is usually sufficient to exceed the figure of 10,000. When necessary, however, the count can be taken for 2 minutes if the count rate is significantly below 10,000 per min.

INVESTIGATIONS INTO THE EFFECTIVE VOLUME AND DEPTH MEASURED

For the initial investigations into the effective volumes and depths of the "zones of influence" of the gamma and neutron radiation, a mold was required that would satisfy the requirements for depth determinations with the surface probe and also those for volume determinations with the depth probe.

Various tests were conducted to find (a) the best type of materials to be used, (b) the amount of compaction required, (c) the homogeneity of the mixture, and (d) its ability to be handled often. Finally, after repeated tests on a 6-in. diameter standard mold, the following materials were selected as components of the mixtures: river sand, blast-furnace slag, vermiculite, steel filings, paraffin wax, and tar. From these tests it was decided to make up five compositions by varying the proportions of the component materials to give the densities and simulated moisture contents shown in Table 1. The mixtures were prepared hot, and the wax and tar served as a binder to give a firm coherent material after compaction in the special ring molds to be described.

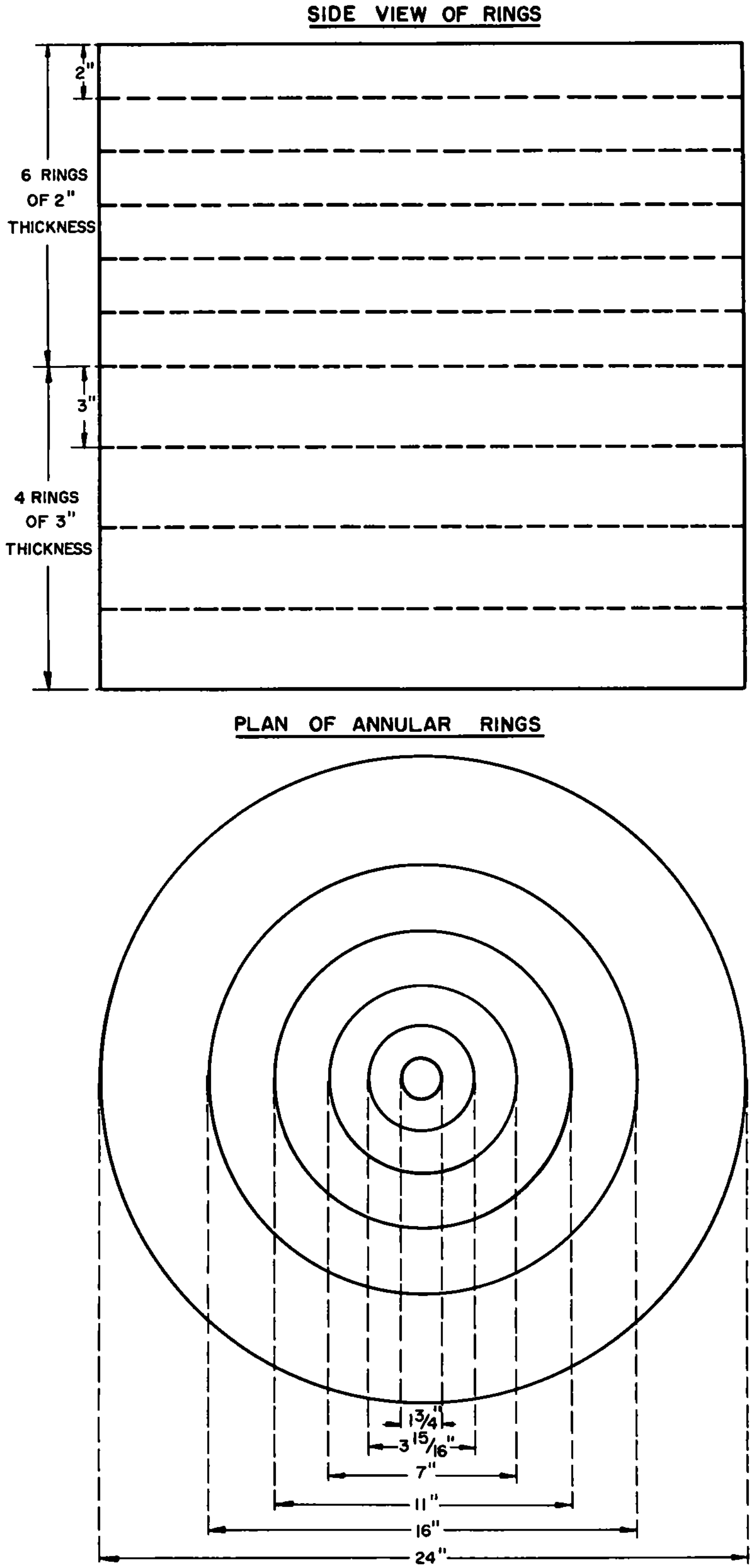


Figure 3. Annular ring molds.

TABLE 1

Mold	Density (pcf)	Simulated Moisture Content (% dry wt)
1	78.2	30
2	86.4	23
3	106.7	16
4	125.4	9
5	145.2	4

Consideration was then given to the mold construction, and it was decided that each composition should be made up in annular rings reaching a final diameter of 24 in., with a centre core of 1¾-in. diameter that could be removed when using the depth probe. In all, 5 annular rings were made having diameters of 1¾ to $3^{15}/_{16}$ in, $3^{15}/^{16}$ to 7 in., 7 to 11 in., 11 to 16 in., and 16 to 24 in. Each completed mold also had to be 24 in. high and was made up of 4 rings of 3-in. thickness and 6 rings of 2-in. thickness. The final densities and simulated moisture contents were calculated from the weights and volumes of the molds and the percentages by weight of the wax and tar used. The exact hydrogen content of the wax and the tar was determined by chemical analysis. Figure 3 shows the details of a mold.

Depth Investigation Using Surface Probe

It was first established that, for the surface probe to give its maximum count rate, the full 24-in. diameter mold was necessary (because the count rate fell after removing the outer ring). A 2-in. thick section (24 in. in diameter) was placed on the table and the surface probe positioned on the center of the mold. Readings of counts per minute were then taken for both moisture content and density, the distance between source and detector being varied for the density measurements in equal steps, a count being taken for each step. This section of the mold was then replaced by a 3-in. thick section and the tests repeated. Further thicknesses were then tested, increasing an inch at a time (i.e., 4 in., 5 in., 6 in., etc.) until it was observed that no change in count rate occurred with increase in mold thickness. This complete procedure was then repeated over the five compositions of varying density and moisture content. From the results obtained, curves were plotted to establish the following relationships:

(a) count rate against moisture content (Figure 4),
(b) count rate against wet density in pcf for various source-detector distances (Figure 5),
(c) counts per minute against mold thickness (Figure 6), and
(d) effective depth of measurement against wet density (Figure 7).

From these tests it was found that the effective depth measured by the radiation was a function of both the source-detector distance and the density. From the moisture investigations it was found that the effective depth measured was independent of both the density and the moisture content and that it remains constant at 4¼ in.

Volume Investigation Using the Depth Probe

For this work the full mold thickness of 24-in. was used throughout, the diameter being varied by adding annular rings as required. Initially the probe was lowered into the hole in the center of the mold and readings were taken at preset depths to ascertain the effect of the probe being used near the surface of the material. This was done for all positions of source-detector distance. Subsequent volume investigations were done with the radioactive source at the center of the mold, i.e., 12 in. from the top surface.

The depth probe was then lowered into the hole, using only the first annulus (i.e., $3^{15}/_{16}$-in. diam.) and counts per minute were taken for moisture and density. This procedure was then repeated, adding the annular rings until the full diameter was reached. Again this was done for the five compositions and curves were drawn to establish the following relationships:

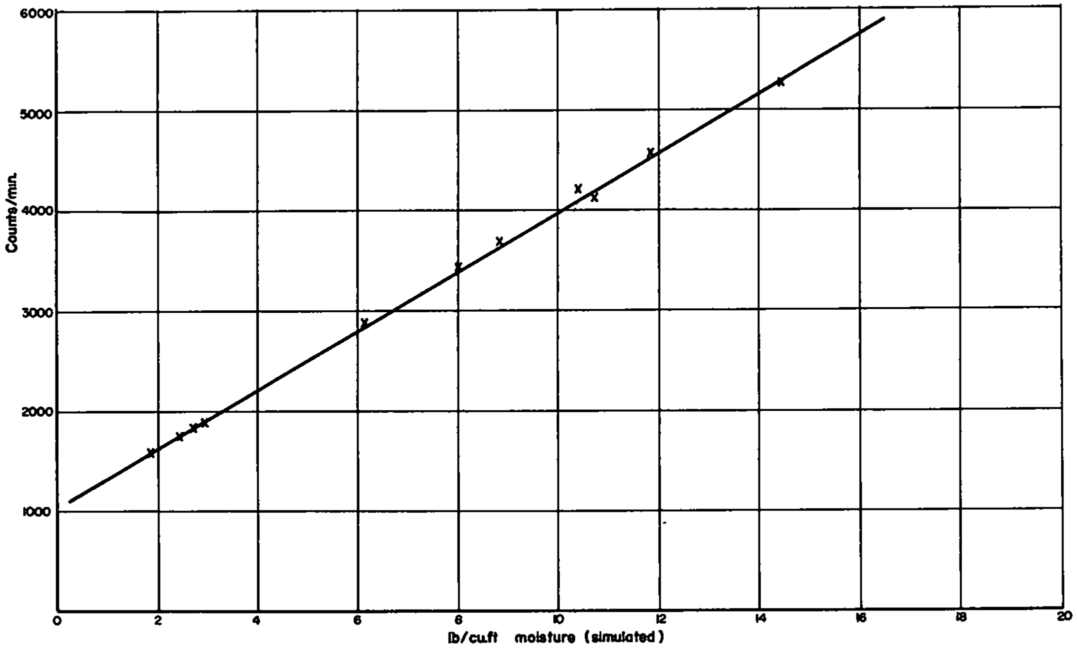


Figure 4. Relationship between moisture content and counts per minute.

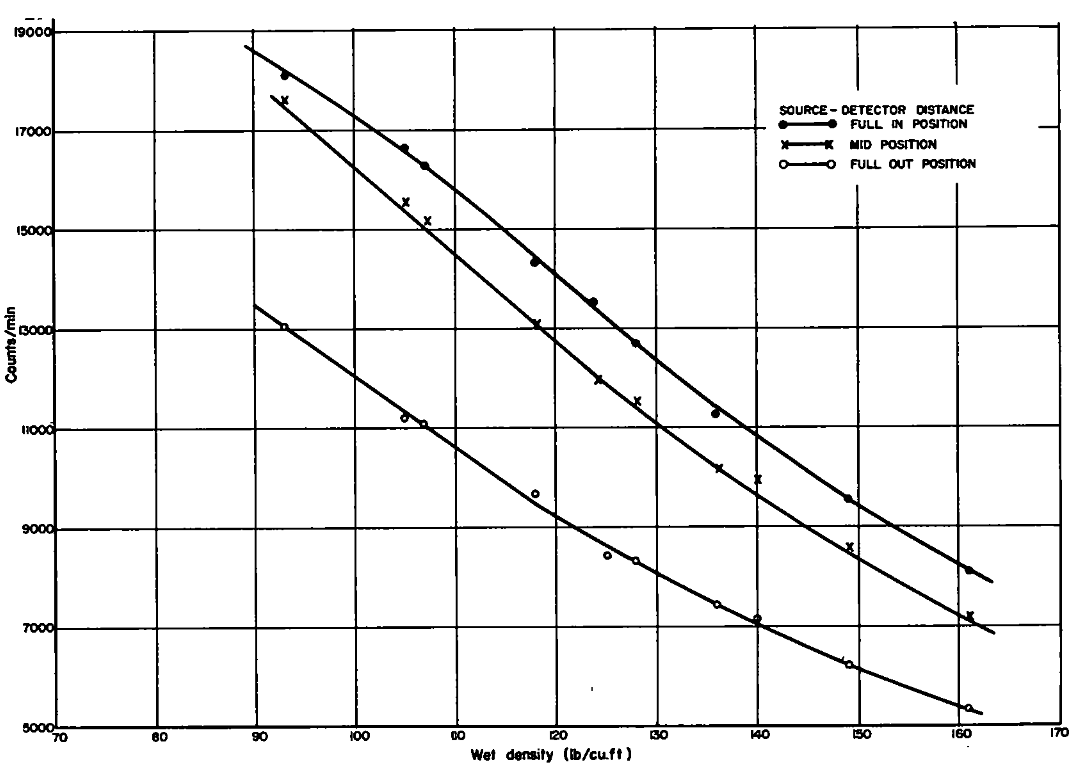


Figure 5. Relationship between wet density and counts per minute.

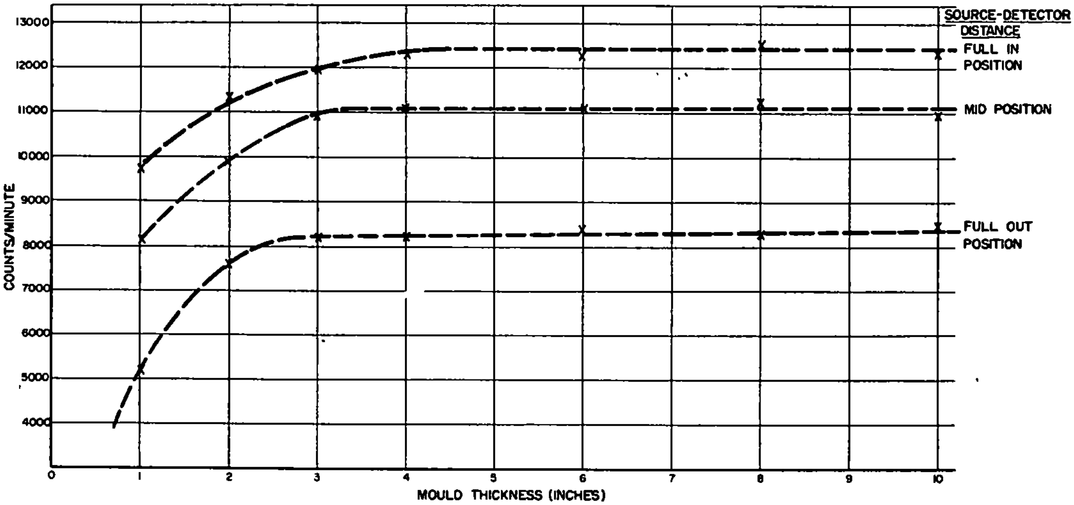


Figure 6. Counts per minute vs mold thickness for a density of 129.6 pcf.

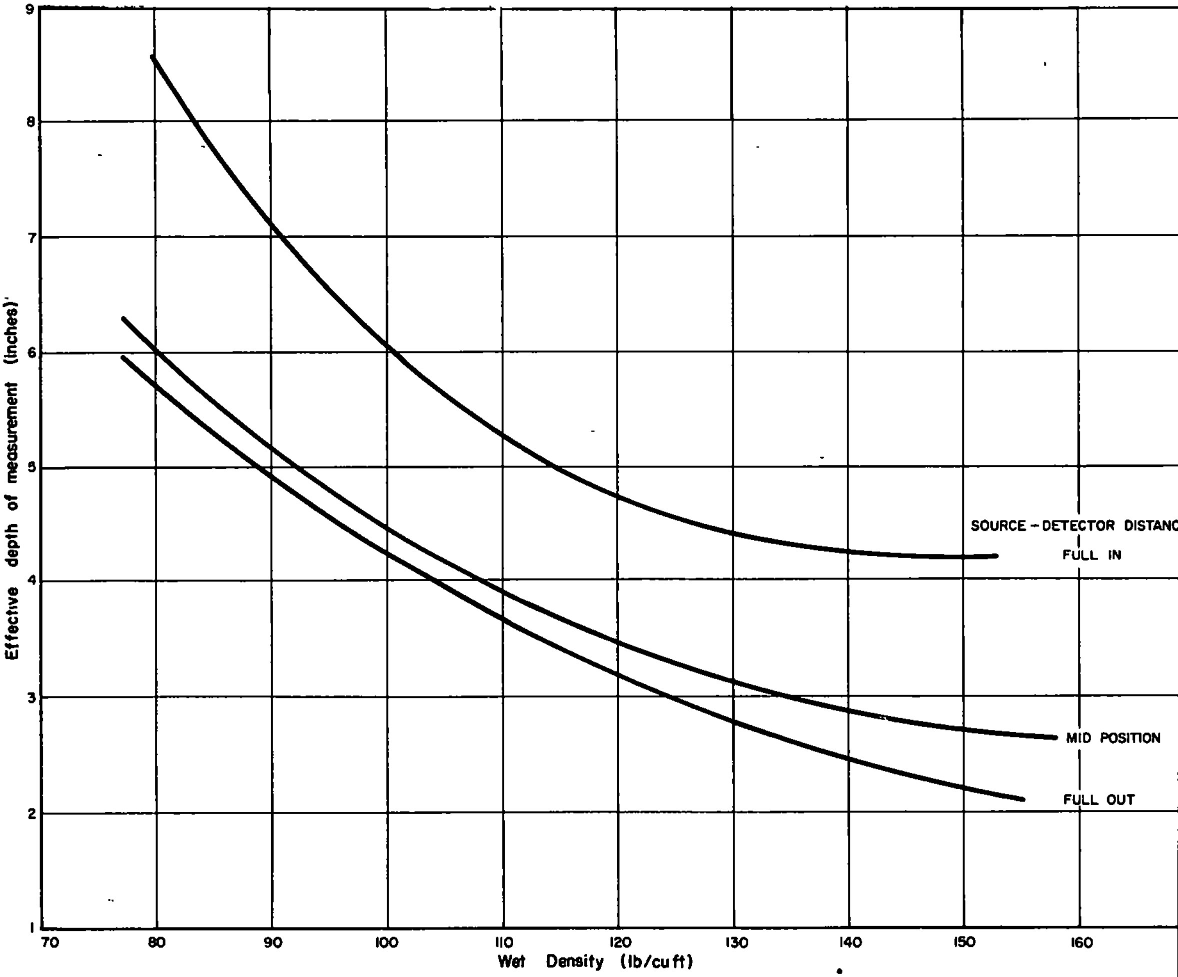


Figure 7. Relationship between effective depth of measurement and wet density.

(a) count rate vs moisture content on full diameter molds (Figure 8)
(b) count rate vs density on full diameter molds (Figure 9)
(c) counts per minute vs mold diameter (Figure 10, an example at one density), and
(d) effective radiation diameter vs wet density (Figure 11).

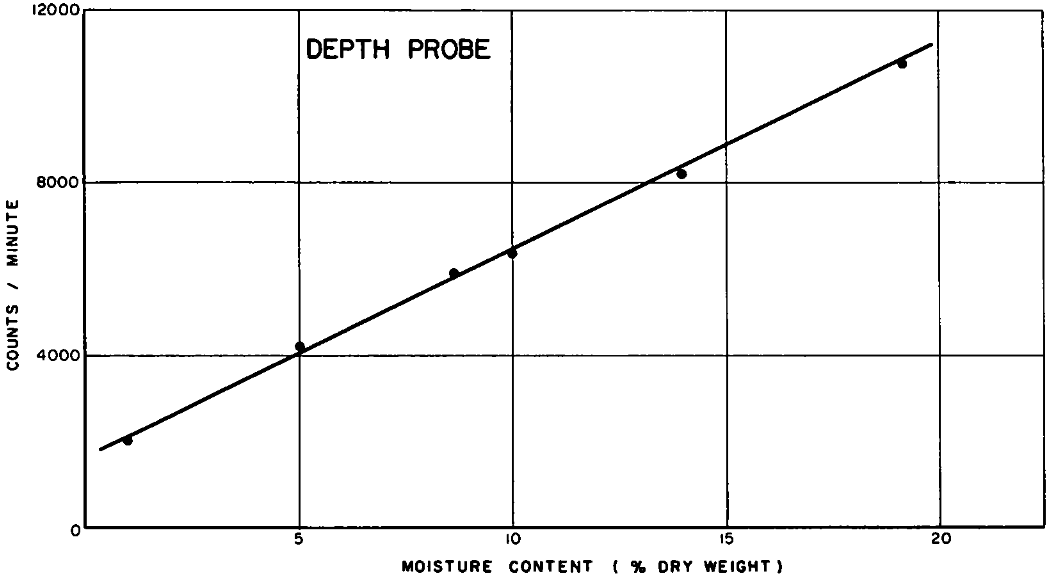


Figure 8. Counts per minute vs moisture content at one density.

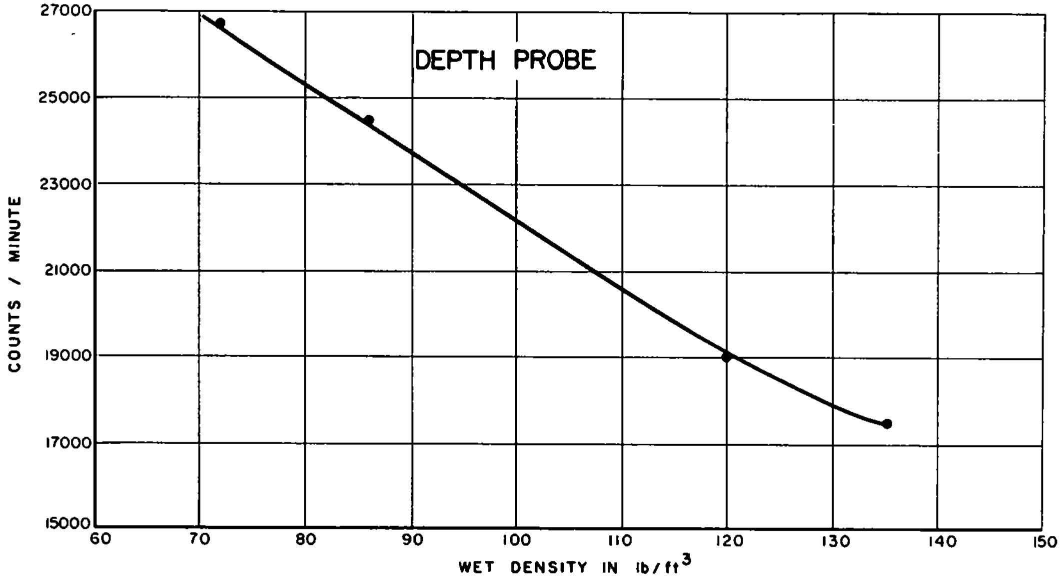


Figure 9. Counts per minute vs density.

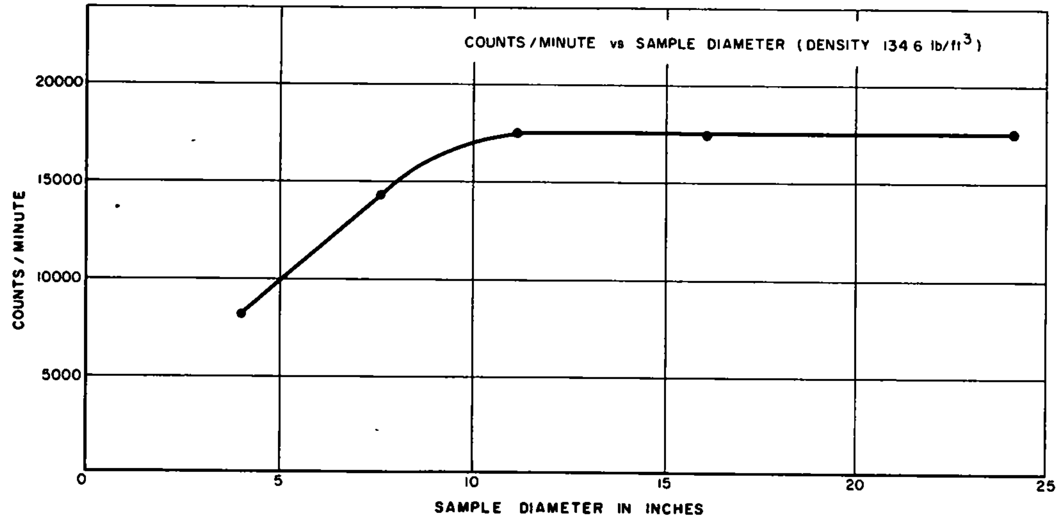


Figure 10. Counts per minute vs sample diameter at one density with the depth probe.

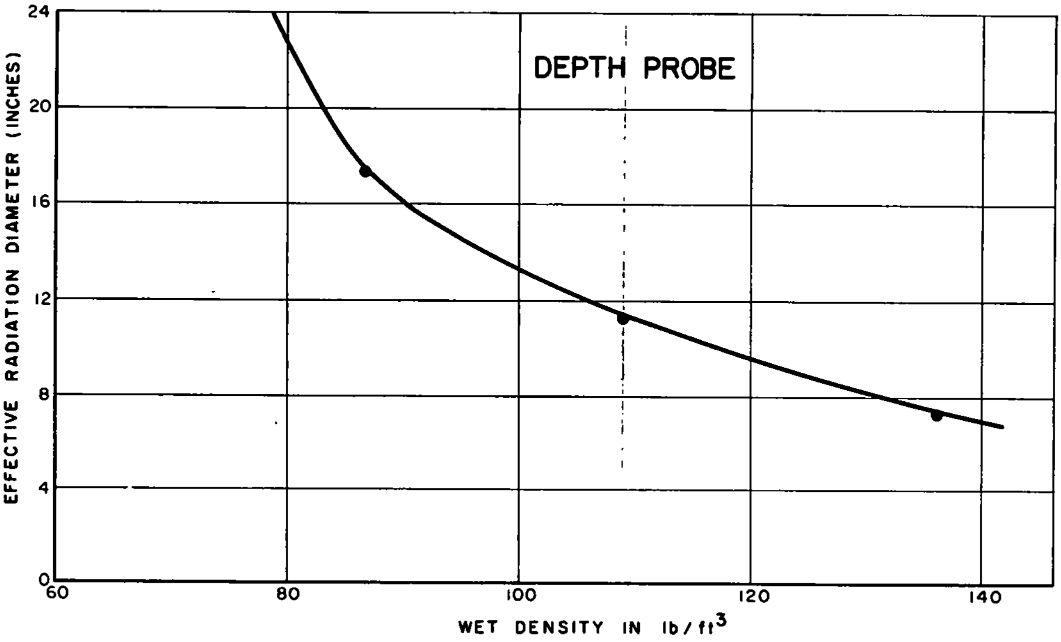


Figure 11. Effective radiation diameter vs wet density.

density measurement with the depth probe varies from about 7 in. at 140 pcf up to about 22 in. at a density of 80 pcf. For moisture content measurements, it was found that the effective volume measured by the depth probe was not influenced by either the moisture or the density.

In general, however, with both the surface probe and the depth probe the count rate for moisture measurement at a constant moisture content (percent moisture on dry weight) increased as density increased because at increasing density more water is enclosed by the fixed "influence volume." This implied the need for a family of moisture calibration curves over a range of densities. It was then found that a single moisture calibration curve can be used, independent of density, if the moisture content is expressed as pcf of water instead of the usual percent moisture on dry weight. This procedure also simplifies calculation of the results of measurements with the instrument. The gamma ray count gives wet density (cp cf) direct from the calibration curve, the neutron count gives water content (pcf) direct from the calibration curve and dry density is then given by the difference of these two quantities. The moisture (percent on dry weight) is then obtained by dividing the pcf of water by the dry density pcf).

FIRST SET OF STANDARD MOLDS

On completion of these investigations, it was decided to build a laboratory set of standard molds, again with a working range of density and simulated moisture content. The dimensions of these molds were to be based on results obtained with the annular ring molds for both surface and depth probes. The same type of composition was used as for the annular ring molds. Aluminum boxes were constructed of $^3/_{16}$-in. sheet into which the mixtures were to be compacted. In the center of each box a horizontal aluminum access tube of 2-in. outside diameter was welded to accomodate the depth probe. These molds were found to be more suitable in drawing up calibration curves for field work, inasmuch as the annular ring molds suffered from several disadvantages: small air gaps between the annular rings, slight changes in density between rings of the same size due to compaction difficulties, and superficial damage to the surfaces from repeated handling.

CALIBRATION OF SURFACE PROBE ON SOILS

Following the laboratory work, investigations with the surface probe were then carried out using four different types of soils, again to establish the effect of soil density on the effective depth of measurement and also to study the effects of different types of road building materials on the count rates obtained with the Hidrodensimeter.

A steel container, measuring 3 ft x 3 ft x 1 ft deep, was constructed for compaction of the material and lines were accurately scribed at 2-in. intervals around the sides for layer investigations. Six wet densities ranging from 70 to 145 pcf were selected. At each wet density, the moisture contents were varied from 2 to 20 percent on dry weight. Compaction was carried out in 2-in. layers and counts per minute for density and moisture were taken at each step for all three positions of source-detector distance until the count rates became constant with increasing depth of material. After each series of tests had been completed, sand-replacement tests were done and samples for oven drying taken to check Hidrodensimeter results. The results of this investigation, with its large number of combinations of density and moisture content, were used to establish the practical calibration curves. Figure 12 shows the density calibration curve obtained on the soils, with the points from the block molds included. Figure 13 shows the moisture calibration curve, also including block mold points. The effective depths of measurement found at the various densities, source-detector distances, and moisture contents verified the earlier results obtained on the annular ring molds. In fact, the results obtained with the four soils chosen did not effectively change the calibration curves already found (see Figures 12 and 13).

Referring to Figure 7, it will be seen that some control of depth of measurement is provided by varying the source-detector distance. At the high end of the practical range of wet density (150 pcf) the depth of measurement can be varied from 2½ to 4¼ in. At the low end of density (e.g., 100 pcf) the depth of measurement can be adjusted from 4¼ to 6 in.

At present, a limitation on this range of adjustment is created by the physical size of the lead screen between the source and the Geiger-Müller tube. This screen governs the closest spacing of the source and detector and hence the greatest depth measured. Further work is in progress to find more efficient screening materials to enable this spacing to be reduced so as to increase the depth of measurement in the high density materials and, therefore, give the operator an increased degree of control over the depth of measurement.

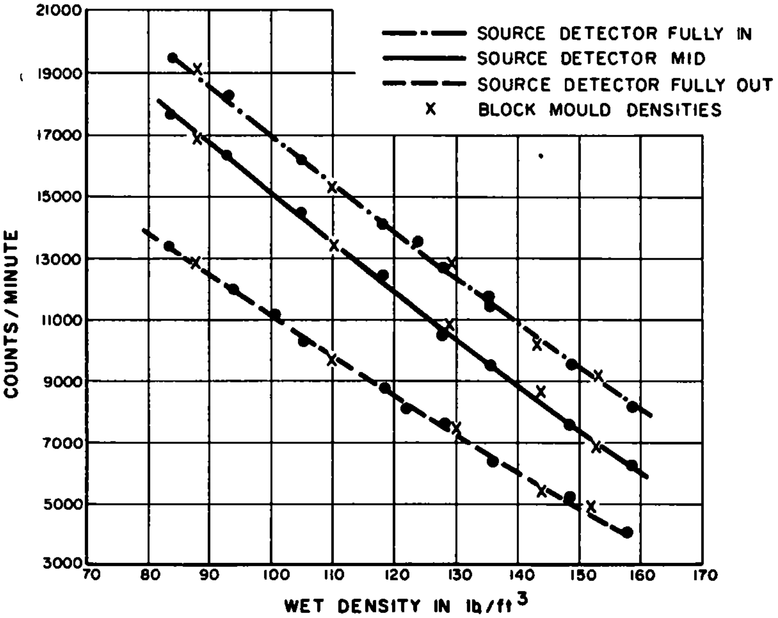


Figure 12. Density calibration curve.

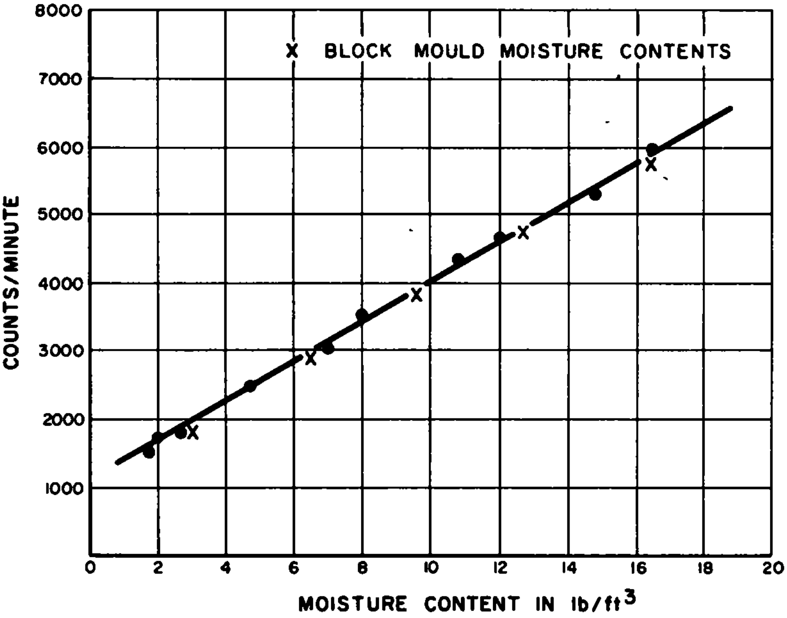


Figure 13. Moisture calibration curve.

Effect of Clay Content on Moisture Measurement

Holmes in Australia reported (7) that clay content of soils influenced the moisture count rate obtained by this method. Some preliminary work was done to examine this effect. Three materials were chosen: a heavy black clay, a medium clay, and a sandy soil of P.I. below 4. Each material was compacted into a large steel tray at three different moisture contents (the densities and moisture contents being verified by sand-replacement tests and oven drying). Readings were taken with the Hidrodensimeter surface probe and curves plotted from results obtained (Figure 14). From these results it was found that the type of material had no effect on the density count rate, but that the clay materials and sandy soil gave separate "moisture" curves. Figure 14 shows counts per minute against moisture content (percent dry weight) by oven drying.

The data obtained from the sandy soil agreed with the normal calibration curve to within ± 1 percent. Significant departures from the calibration curve were found with the two clays however. This was due to the bound hydrogen in the clay mineral structure giving a high background count. Materials normally used in constructing road foundation layers are nevertheless severely limited in clay content and experience in the field has shown this effect is negligible.

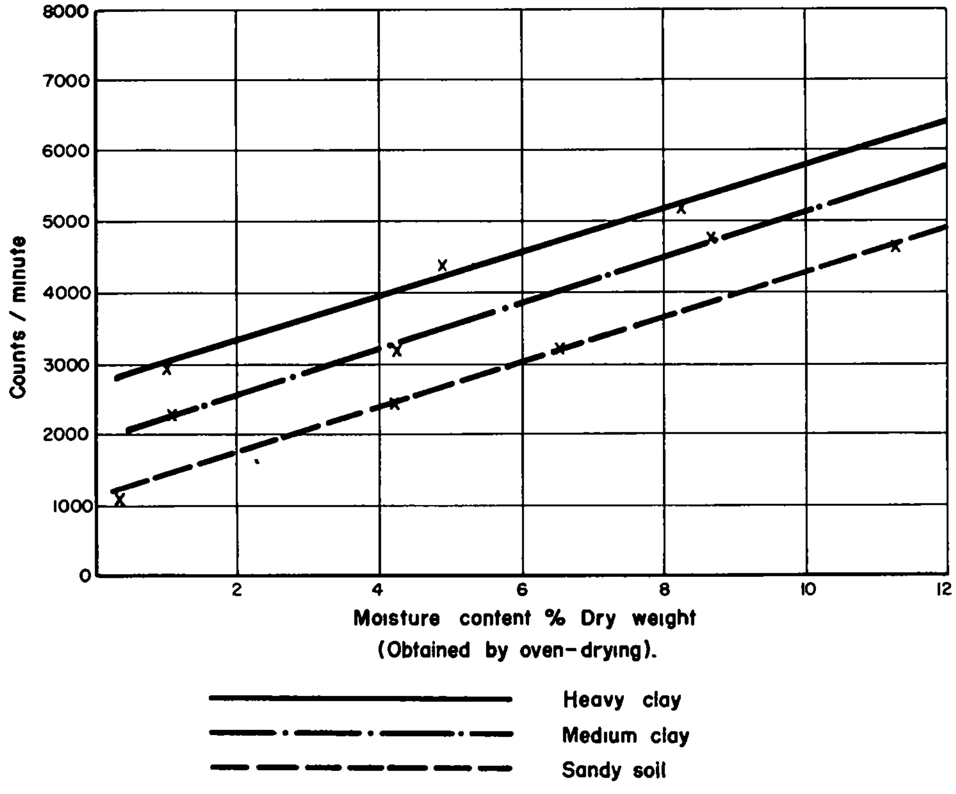


Figure 14. Effect of the clay content on moisture measurements using the Hidrodensimeter.

FIELD TRIALS WITH THE HIDRODENSIMETER

The next step after the laboratory calibration work was to take the instrument into the field and use it on road construction jobs alongside the normal sand-replacement method. This was done in all four Provinces of the Union of South Africa and a range of types of material was covered. The tour also gave an opportunity to demonstrate the speed of measurement in the field. Only the surface probe was used, as this provides the type of measurement of most interest to engineers in controlling compaction of soil layers.

To compare Hidrodensimeter results with those from the normal method, several series of tests were taken on a range of construction jobs with teams running sand-replacement tests alongside the operator with the Hidrodensimeter. Although it is commonly accepted that the sand-replacement test is itself liable to error, it was nevertheless felt that, if a good correlation between the two methods was found, the Hidrodensimeter could be used in the field with at least the same confidence as the present methods.

The results of about 95 tests obtained by both methods are discussed later. The detailed data is given in Table 2. These tests covered densities from about 100 to 150 pcf and moisture contents from about 1 to 12 percent. Each test consisted of a measurement with the Hidrodensimeter surface probe, followed by the sand-replacement method on precisely the same spot. The time required to make a determination at one point of both moisture content and density with the Hidrodensimeter is about three minutes. On the crusher-run materials, with their high density, the effective depth of measurement was about 4 in., while on most other materials tested the densities were lower and the effective depth of measurement was about 6 in. The sand-replacement tests were, in general, done to the same depths.

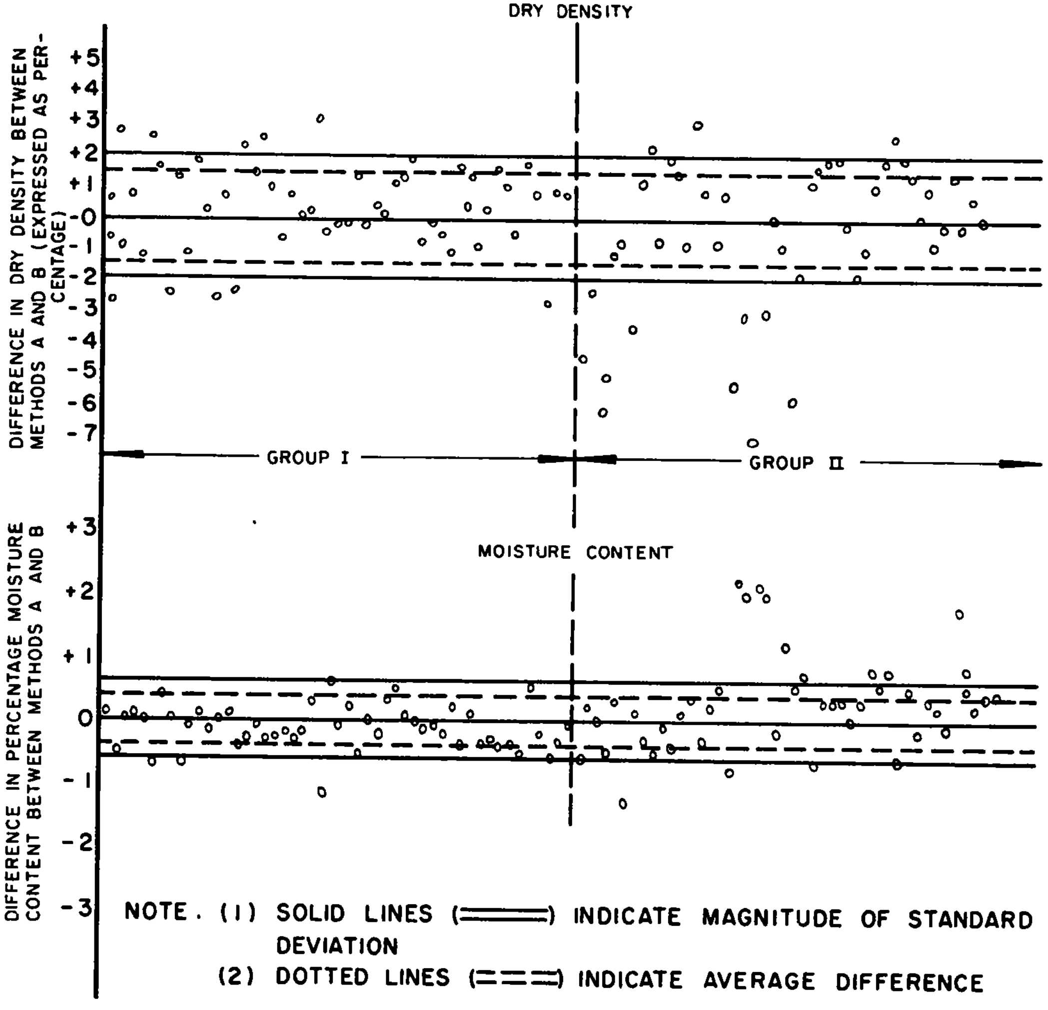


Figure 15. Graphical presentation of differences in results by the two methods.

TABLE 2

COMPARATIVE DATA OBTAINED WITH HIDRODENSIMETER AND SAND-REPLACEMENT METHODS

	Group I				Group II				
Type of Soil	Hidrodensimeter[a]		Sand Replacement[b]		Hidrodensimeter[a]		Sand Replacement[c]		Type of Soil
	Dry Dens. (pcf)	Moist. Cont. (%)	Dry Dens. (pcf)	Moist. Cont. (%)	Dry Dens. (pcf)	Moist. Cont. (%)	Dry Dens. (pcf)	Moist. Cont. (%)	
Crusher Run (Primed surface)	147.1	1.6	146.2	1.7	125.1	6.2	125.2	5.6	
	146.4	2.2	150.4	1.7	116.0	4.9	111.0	5.1	Decomposed granite
	148.2	1.8	146.9	1.8	113.7	7.3	111.0	7.3	
	144.1	1.7	145.1	1.8	112.0	7.5	106.5	7.0	
	145.8	1.1	147.3	1.1	112.2	7.9	111.0	8.2	
	147.0	2.2	150.9	1.5	112.2	10.1	111.3	8.8	
	142.5	2.4	144.8	2.8	111.8	8.3	108.0	8.75	
	147.7	1.1	144.2	1.1	135.8	3.5	137.3	3.2	Subbase Granite and Sandstone
	141.0	2.1	142.8	1.4	134.0	3.0	137.0	2.5	
	145.2	2.1	143.5	2.0	138.9	2.8	138.8	2.7	
	149.7	1.1	152.4	1.2	143.1	3.4	145.8	3.0	
	144.5	1.8	144.9	1.6	131.2	2.7	133.0	2.8	
	145.8	1.1	142.2	1.1	131.4	3.4	130.2	3.7	
	143.8	2.3	144.8	2.4	133.8	3.0	137.9	2.7	
	145.7	0.9	142.3	0.5	125.2	3.1	126.2	3.3	
	149.8	2.4	153.3	2.1	126.0	5.5	125.0	6.0	Base Crusher-run Sandstone
	143.6	2.6	145.7	2.5	148.0	3.1	149.1	2.3	
	144.5	1.9	148.2	1.6	119.0	6.1	113.0	8.3	
Base Course	133.0	9.9	133.1	9.7	117.9	6.3	114.2	8.3	
	134.7	9.8	135.0	10.1	117.8	6.9	110.0	9.0	
	129.4	11.8	133.5	10.7	119.4	5.6	115.8	7.6	
	136.0	8.3	135.4	8.9	115.8	5.9	115.8	5.7	
	127.8	10.8	127.6	10.7	115.9	7.8	114.9	9.0	
	135.0	10.1	134.9	10.3	119.1	6.7	112.6	7.2	
	142.0	8.1	143.8	7.6	96.2	12.7	94.4	13.4	Decomposed granite
	137.8	9.6	137.5	9.6	99.8	9.4	101.0	8.7	
	133.0	9.6	133.4	9.4	101.3	8.9	102.8	9.2	
Sandy soil	115.7	7.6	115.8	7.9	99.2	11.1	101.0	11.4	
	102.1	11.1	103.2	11.6	101.9	10.3	103.9	10.6	
	107.7	12.2	110.1	12.3	102.2	10.6	102.0	10.6	
	112.1	11.5	114.3	11.5	109.0	8.7	107.0	9.0	
	109.4	10.4	108.6	10.3	102.4	9.9	101.4	10.7	
	112.9	9.1	112.8	9.0	104.1	9.9	105.1	10.5	
	107.0	12.2	106.5	12.0	108.6	9.8	108.8	10.6	Medium clay
	123.0	10.9	121.7	11.1	115.3	8.2	118.3	7.6	
	119.5	12.0	121.4	11.6	101.5	10.1	103.5	10.6	
	116.2	11.9	116.6	12.0	108.0	10.8	109.5	10.6	
	121.9	12.3	120.8	11.9	113.6	8.4	113.6	8.7	
	120.4	12.0	120.6	11.7	113.0	7.4		7.1	
	118.9	11.9	120.7	11.5	134.2	3.3	135.4	3.5	Crusher run base course
	113.4	12.2	114.5	11.8	136.0	3.8	134.9	3.7	
5-in. gravel layer	130.3	4.2	131.6	3.9	136.4	3.5	136.0	5.3	
	130.4	3.3	129.5	3.1	131.3	4.5	133.0	5.0	
	132.6	2.7	133.6	2.4	137.9	4.4	137.6	4.6	
Natural shoulder material	111.0	7.9	110.5	7.4	137.7	4.4	138.5	4.8	
	116.5	8.1	118.5	8.6	135.0	4.4	135.0	4.8	
	122.3	7.0	123.2	6.8					
	111.4	7.9	108.5	7.3					
	109.1	9.5	110.0	9.1					
	109.2	9.7	110.0	9.6					

---

[a] Results obtained by National Institute for Road Research.

[b] Results obtained by experienced teams.

[c] Results obtained by less experienced teams.

Because the results of the sand-replacement method are, in general, subject to a significant but unknown error, they do not really provide a satisfactory reference against which to compare the Hidrodensimeter results (8). This point must be borne in mind in considering the results discussed herein. As a first attempt to compare the two sets of results, the differences between the individual results obtained by the two methods were plotted for all the determinations relative to a line signifying zero difference. In other words, differences between the test results by the two methods were examined, considering either set as reference. Figure 15 shows the differences in dry density (expressed as a percentage) and the differences in percent moisture content.

Three quantities were calculated from these results: the mean arithmetical difference (A), the mean of the modulus of the difference (B), and the standard deviation of the differences (C). These are given in Table 3 and shown graphically in Figure 15.

TABLE 3

QUANTITIES A, B, AND C FOR RESULTS OBTAINED IN FIELD TESTS

Results	Differences in Dry Density (%)			Differences in Moisture Content (%)		
	A	B	C	A	B	C
Group I	+0.4	1.3	1.4	+0.13	0.28	0.36
Group II	-0.5	1.7	2.4	-0.3	0.6	0.88
All	-0.05	1.44	2.0	+0.08	0.4	0.62

In Table 3, the results in Group I were obtained by field teams having considerable experience in using the sand-replacement test method, while the Group II results were obtained by less experienced teams. It will be seen in Figure 15 that the Group I differences show only a moderate and consistent scatter, while in Group II a number of the differences are well outside the standard deviation. This is reflected in the figures given in Table 3. Because the standard deviations for Group II are about twice those for Group I, it was concluded that most of the differences between individual results from the two methods could be ascribed to operational errors in the sand-replacement test.

Further analysis of the data was carried out, using more conventional statistical methods, to compare the results given by the two methods. Two points were examined:

1. The correlation coefficient between the two sets of results. The dry density results were grouped into data obtained on crusher-run bases and on other bases. For crusher-run material a value of the correlation coefficient (r) of 0.96 (26 observations) were obtained. The 95 percent confidence limits for r were 0.91 and 0.98. For other bases a value for r of 0.95 was found (69 observations). The 95 percent confidence limits were 0.92 and 0.97. Regression equations were calculated for both the crusher-run and the other data and it was found that the slopes of the two lines did not differ significantly. For all 95 results taken together a value of r of 0.98 was obtained with 95 percent confidence limits of 0.97 and 0.99. For moisture content the correlation coefficient for all 95 results was 0.83 with 95 percent confidence limits of 0.76 and 0.88.
2. The significance of the differences in the results by the two methods. The "Wilcoxon matched-pairs signed ranks" test was then carried out. No significant difference in the results obtained by the two test methods was found (probability level a = 0.05, two-tailed test). This applied to both the density and moisture content data.

When these correlation tests were carried out, no consideration was given to the possibility of density gradients occurring in the layers measured, a factor that only came to light from later field work. Because, however, the effective volume of soil measured by the radiation method differs from the volume involved in the sand-replacement method, it becomes necessary to take this factor into account when making comparative measurements by the two methods on layers through which a density gradient exists. A procedure for the identification of density gradients and for carrying out density tests in such cases was worked out.

DETERMINATION OF THE WET DENSITY OF A SOIL LAYER WHEN A DENSITY GRADIENT EXISTS

To find whether a density gradient exists in the compacted soil layer being tested, use is made of the fact that, by changing the source-detector distance on the surface probe, the effective depth of measurement of density can be varied.

Density determinations are made at the same spot with the source-detector distance in the "full-in" position and then in the "full-out" position. If there is no difference in the densities read off from the respective calibration curves, no density gradient exists and further determinations on the same material can be made with the source-detector distance in the "full-in" position. Under these conditions, the results obtained with the Hidrodensimeter can be compared directly with those found by the sand-replacement test. It is advisable to repeat this procedure whenever there is a change in type of soil or compaction plant being used.

If, on the other hand, the two densities differ, a density gradient exists and it is normally found, that the density increases with an increase in source-detector distance (i.e., with a decrease in the effective depth of measurement). In other words, the density is usually greatest near the top of the layer.

The following procedure should then be used. Let the wet densities obtained with the detector in the "full-in" and "full-out" positions be respectively d_1 and d_2 as read off the appropriate calibration curves, and let the corresponding effective depths of measurement be h_1 and h_2, as found from the curves shown in Figure 7. Plot a curve of wet density (d) against $^3/_8$ times the effective depth (h) as shown in Figure 16 and draw a straight line through the points. This line represents the density gradient in the soil layer.

The average density as determined with the Hidrodensimeter for any depth h' in between h_1 and h_2 can now be read off opposite $^3/_8$ h' (i.e., d'). Similarly, the average

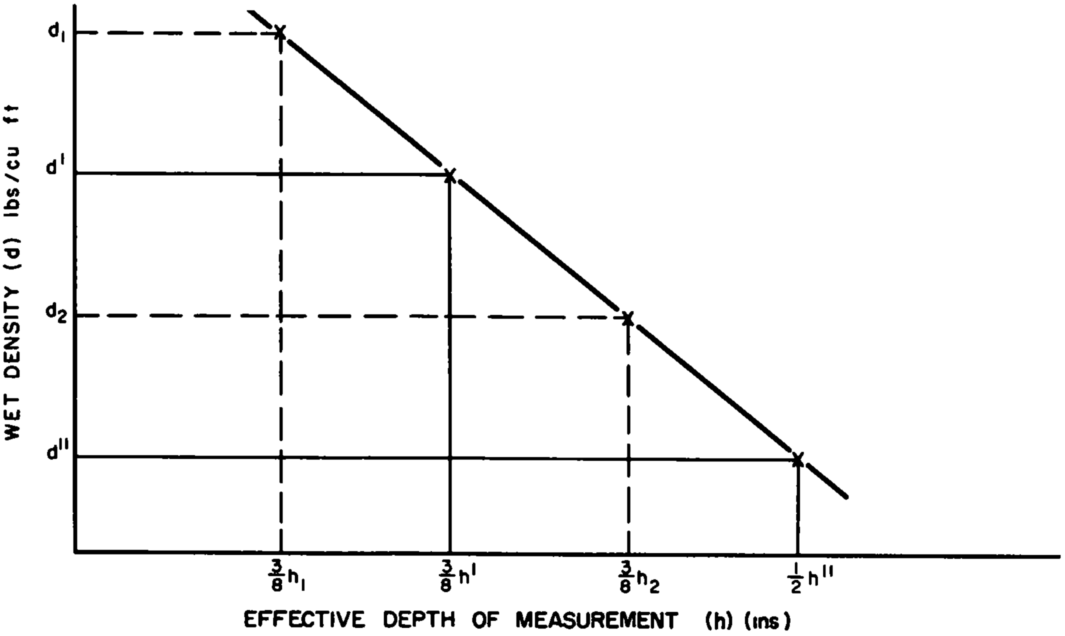


Figure 16. Procedure for dealing with density gradient.

density as determined by a sand-replacement test at a depth h" can be read off opposite ½ times h" (i.e., d"). When the depths h' and h" are the same, there may still be a small difference between the densities as determined by the two methods due to the different shapes of the volumes measured in each case.

It is again advisable to repeat this procedure whenever there is a change in soil type or in the compaction equipment being used.

FURTHER DENSITY CHECKS AND INVESTIGATION OF EFFECTIVE VOLUMES MEASURED

In addition to the systematic calibration work already described in this paper, it was felt desirable to make one or two spot checks on the absolute accuracy of the instrument by using materials of known density, and also to determine more closely the effective shape and volume of the "zone of influence" of the radiation. This work is not yet completed, so only a few preliminary results can be given here.

Three concrete blocks measuring 2½ ft square and 1 ft thick were made up with approximate densities of 157; 135, and 104 pcf. The lower densities were obtainend by using "no fines" concrete and an ash aggregate. After using the Hidrodensimeter to test completely both top and bottom surfaces for density, 5-in. diameter holes were drilled in each block at selected spots and the average density of the extracted core was determined. The cores were then cut into slices and densities were determined for each slice.

The first block, made from a conventional dense concrete mix, was found to be reasonably uniform in density and suitable for the purpose. Unfortunately, the second block of "no fines" concrete and the third block containing an ash aggregate were not sufficiently uniform in density to provide useful checks for the Hidrodensimeter. The results obtained with the first block are given in Table 4, which shows the minimum and maximum densities found anywhere on the top face and again on the bottom face using the Hidrodensimeter, the respective mean densities and effective depth of measurement, and the density results obtained directly from weight and volume of the core and its slices. Although

TABLE 4

COMPARATIVE MEASUREMENTS ON A CONCRETE BLOCK

Surface of Block	Hidrodensimeter Readings (pcf)				Densities Found on Cores (pcf)							
	Minimum Density Measured	Maximum Density Measured	Average Density for Block	Effective Depth of Measurement (in.)	Average Density for 12-in. Core	Densities on Individual Slices					Mean for Top 4 in.	Mean for Bottom 4 in.
						0-2 in. (Top Slice)	2-4 in.	4-8 in.	8-10 in.	10-12 in. (Bottom Slice)		
Top	154.2	157.0	155.6	4.2	156.2	154.1	154.9	161.4	152.2	156.0	154.5	-
Bottom	156.4	159.2	157.8	4.1	-	-	-	-	-	-	-	154.1

some variation in density occurred over both faces, the agreement is reasonably good. Work along these lines however, is, being continued.

By moving the surface probe near to the edges of this square block and by observing the distances to the edge at which the count rate just began to fall off, it was possible to define fairly closely the shape in plan of the effective "zone of influence" of the gamma radiation. This is shown in Figure 17. As would be expected, this is a distorted or egg-shaped ellipse due to the finite length of the gamma-ray detector tube.

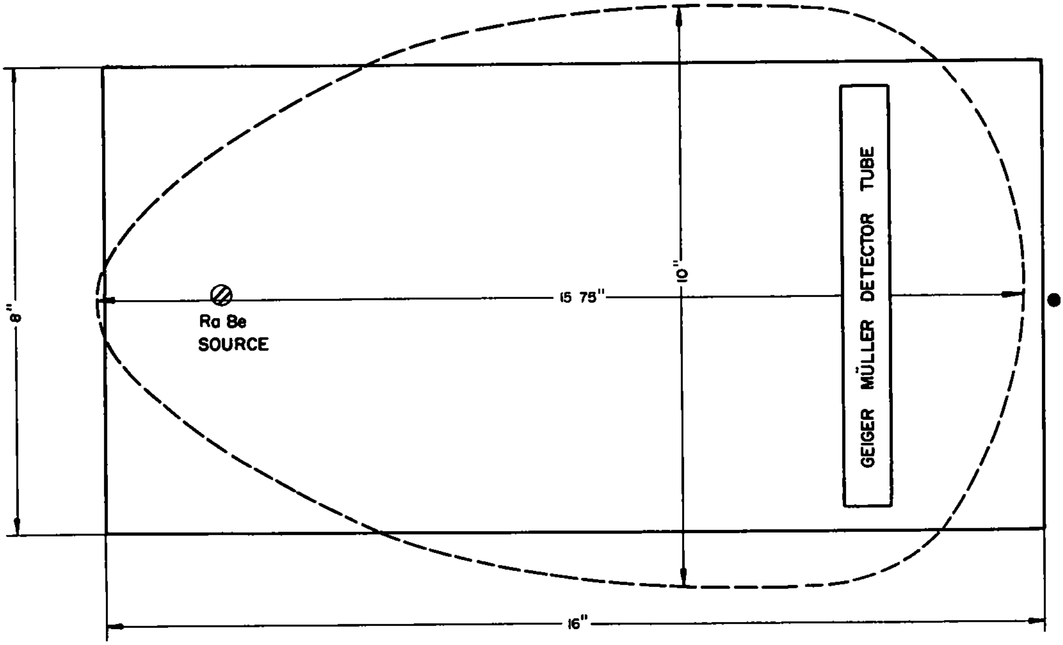


Figure 17. Diagram of surface radiation sphere around probe.

A new system of molds is at present being constructed to investigate accurately the volumes and depth measured and also to study in more detail the effects of density gradients in soil layers. Seven separate sets of molds are being made, each mold consisting of 12 separate slabs, 1 in. thick and 2½ ft square. Again the densities will be varied throughout the working range from 75 to 150 pcf and simulated moisture contents from 2 to 16 percent of dry weight. New types of special materials are being used to give very accurate dimensions and finished surfaces highly resistant to repeated handling. From this work it is hoped that the true shapes of the "zones of influence" can be determined and the effect of different types of density gradient studied.

FACTORY CALIBRATION OF COMMERCIAL INSTRUMENTS

It was considered necessary, in view of the fact that the Hidrodensimeter was to be manufactured commercially, to have a set of standard molds made that would be compacted to known densities and simulated moisture contents. Twenty-three of these molds, each a 3-ft cube, were constructed on the premises of the manufacturers, the wet densities ranging from 80 to 160 pcf and simulated moisture contents from 1 to 15 percent dry weight. The molds were made of granular materials, including sand, vermiculite, and powdered iron ore, the simulated moisture contents being introduced by addition of a hard paraffin wax and tar, which also served to bind the mixtures together.

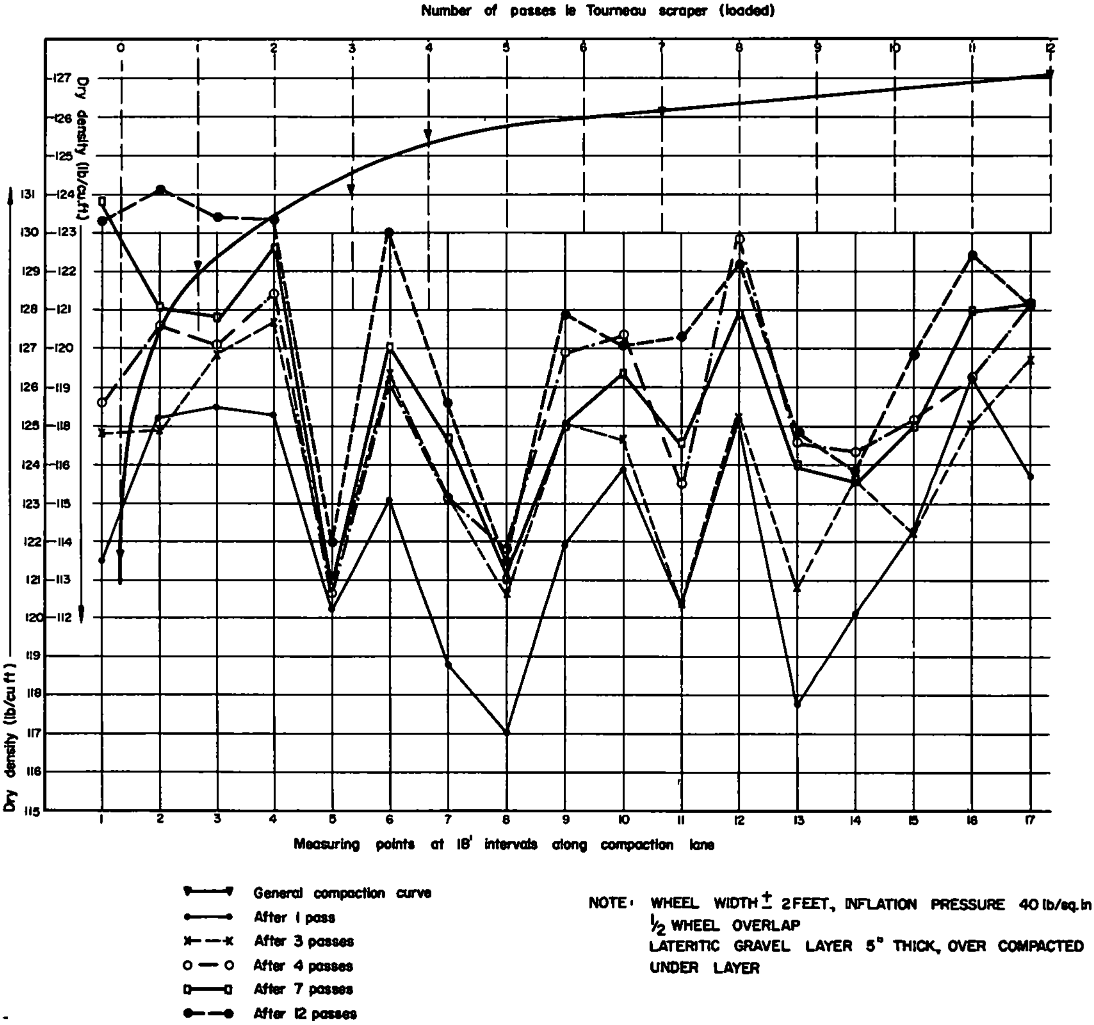
FIELD APPLICATIONS

More recently, the Hidrodensimeter has been used in the field on a number of road construction jobs in various parts of the country to demonstrate its application in compaction control or to assist in the solution of particular compaction problems that have arisen. A few of the more interesting points that have come to light in this work may be mentioned.

Density Checking

In one case a 3-mi length of new crushed-stone base had to be checked quickly before being used for a major surfacing experiment. In two days the whole length was scanned with the instrument and only one short "low spot" was shown up. The limits of this spot were accurately defined. In all, some 250 density measurements were made.

The value of obtaining a more general picture of the compaction of a particular length, rather than density values from only one or two check points, has been emphasized. In practice, layers are rarely uniform over a few 100-ft lengths, even under well-controlled conditions. As an example Figure 18 shows the results of a compaction study



NOTE: WHEEL WIDTH ± 2 FEET, INFLATION PRESSURE 40 lb/sq.in
½ WHEEL OVERLAP
LATERITIC GRAVEL LAYER 5" THICK, OVER COMPACTED UNDER LAYER

Figure 18. Compaction study with loaded le Tourneau scraper.

using the Hidrodensimeter along a compaction lane in nominally uniform material (a lateritic gravel subbase). After 12 passes there was still a range of density from 121.5 to 130 pcf at different points.

Density gradients

The Hidrodensimeter has indicated in compacted layers the presence of density gradients not previously suspected when using the sand-replacement method. As an example, Figure 19 shows results obtained on a 6-in subbase layer of natural course gravel of P. I. about 7. The trouble was found to be due to wrong moisture contents during compaction. It was, in fact, demonstrated on this job that "on-site" studies with this instrument can give valuable information on the applicability of laboratory compaction data to field conditions, particularly concerning the choice of optimum moisture content for compaction in the field.

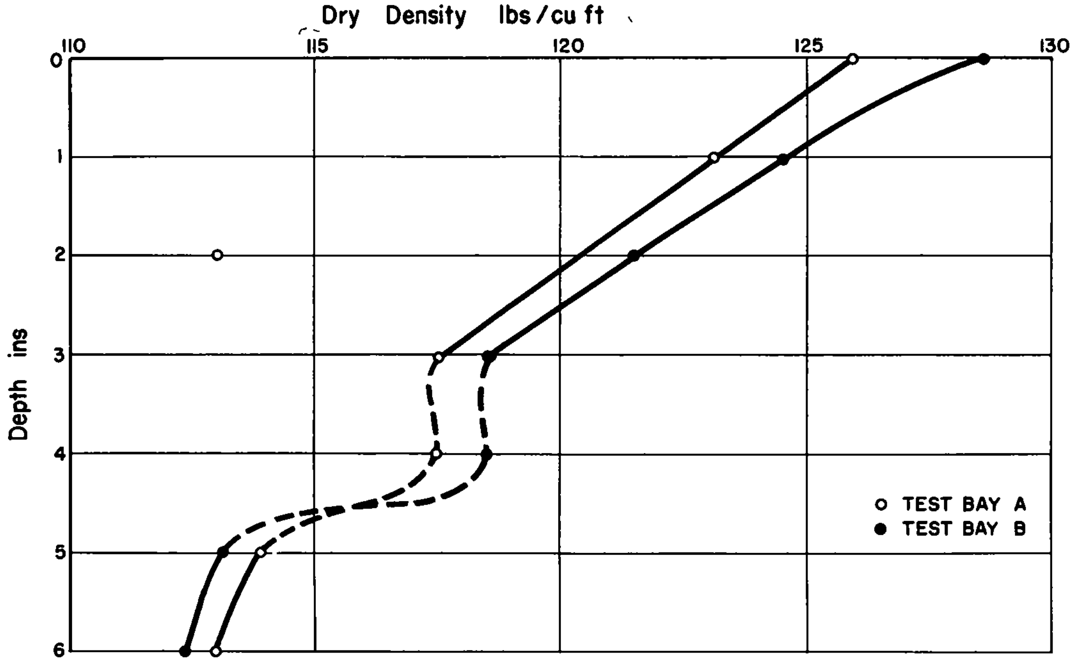


Figure 19. Relationship between dry density and depth on a length compacted with a 3.3-ton vibrating roller.

The whole approach to the probelm of specifying the density and control of density of compacted foundation layers needs some fresh thinking. A more thorough statistical approach that recognizes the inherent variations present would appear to be called for. The radioisotope method with its high speed of measurement provides the engineer with a new tool with which such an approach can become feasible. However, further discussion of this point is beyond the scope of the present paper.

Performance of Compaction Equipment

The instrument has been used on several jobs to assist road authorities with the problem of finding the best type or combination of compaction equipment to achieve required compaction in the most economic manner. As an example, Figures 20 and 21 show results from a study in the Orange Free State to find the optimum plant usage for compacting a cement-stabilized silty-sand layer, a type of material commonly used in that Province. Figure 20 shows density against number of passes of a 50-ton

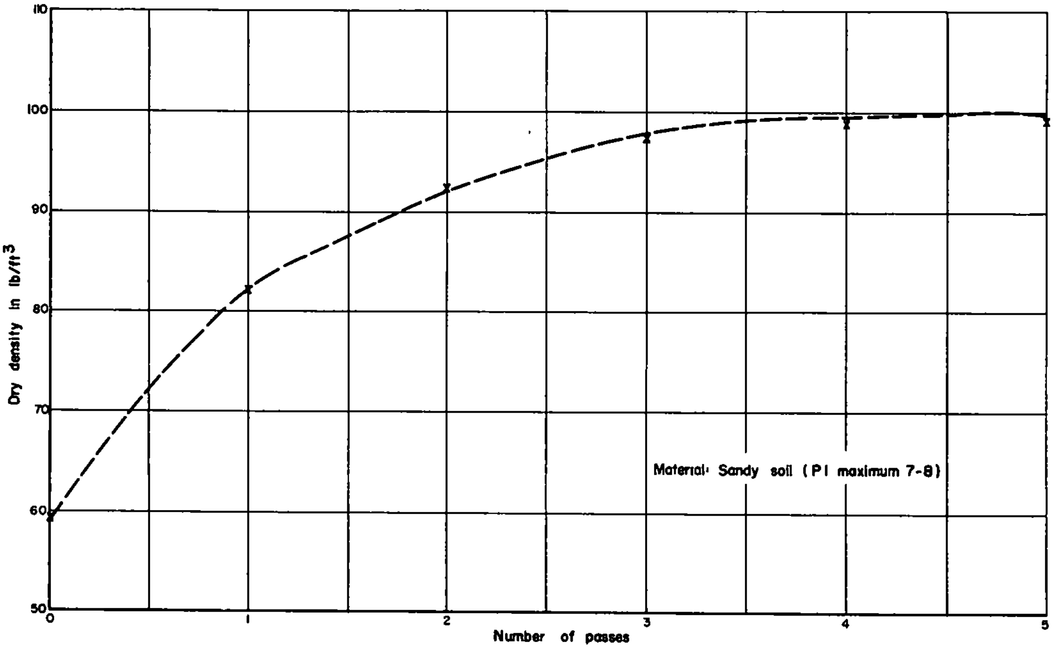


Figure 20. Dry density vs number of passes with 50-ton pneumatic roller, Heilbron-Petrus Steyn (O.F.S.).

pneumatic compactor. Several different compactors were available on this job and a study was made in two or three days to find the best combination of equipment and number of passes to achieve the required density. An example of the type of results obtained is given in Figure 21.

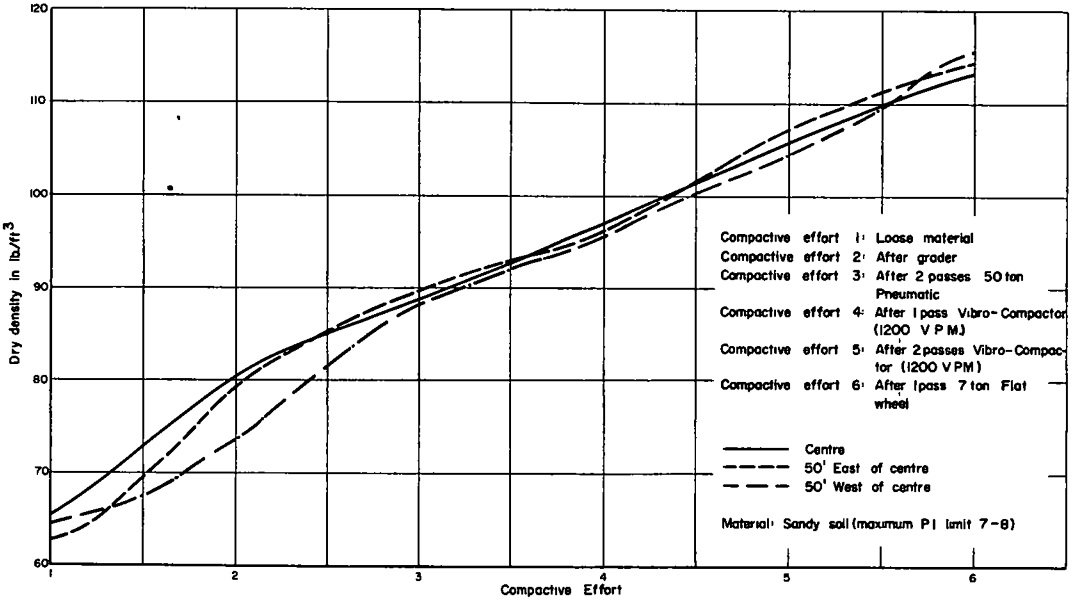


Figure 21. Compactive effort vs dry density, Heilbron-Petrus Steyn (O.F.S.).

Other Applications

Other applications of the instrument include density and moisture studies at depth (using the depth probe) on earth dams, large fills, and building foundations. In Natal, the depth probe was used successfully to establish density/depth profiles in river beds for investigations into scour depth following heavy floods. In these investigations the depth probe was specially waterproofed for working under water down to a 100-ft depth. In the National Institute for Road Research, the depth probe is also being used to follow long-term trends in the moisture profile under roads to certain selected sites where permanent aluminum access tubes (2 in. in diameter) have been sunk beneath the road surface. Agricultural authorities have also experimented with the moisture probes.

CONCLUSIONS

In conclusion, the more obvious advantages of the radioisotope method can be given:

1. The test is nondestructive, i.e., for all normal purposes using the surface probe.
2. Results for moisture content and density at a single point can be obtained in 2 to 3 minutes.
3. The method is accurate and easy to perform, minimizing the human error.
4. Apart from routine density control, its speed of working gives it a number of other important uses, particularly in studying compaction problems on site.
5. It provides a method of density control that is sometimes difficult with the sand-replacement method, e.g., on crushed stone bases and concretes.
6. It can be used under adverse weather conditions.

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Influence of Chlorides and Hydroxides of Calcium and Sodium on Consistency Limits of a Fat Clay

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Data are presented to show the relative effects of various quantities of sodium chloride, calcium chloride, sodium hydroxide, and calcium hydroxide on the liquid limit, plastic limit, shrinkage limit, and pH value of a highly plastic fat clay. Values were observed immediately after testing and after periods of 1, 4, and 18 months. The mechanisms by which the additions of salts may influence consistency properties of clays that are discussed include (a) the physical presence of the additive, (b) the effect on surface tension, (c) changes in viscosity in the aqueous phase, (d) promotion of cation exchange, (e) effect of quantities of additive on flocculation and hydration, and (f) effect on soil bacteria. Conclusions drawn from the data compare the essential differences between the results obtained with the different admixtures.

● THIS STUDY was undertaken in connection with research (1) on the effect of lime on the properties of fat clay.

Much has been written about the role of cation exchange in clays with regard to stabilization. Often the stabilizing effect of lime has been attributed to this phenomenon, despite the large quantities of lime (and thus of Ca^{++}) required to bring about the stabilization and apart from the fact that the stabilized soils were largely calcium saturated prior to additions of lime.

The effects of various amounts of the chlorides and hydroxides of calcium and sodium on the consistency limits of a fat soil is determined so as to compare the influence of the cation and pH.

PROCEDURES

The soil was air dried and pulverized to minus 40. The chemicals (from ½ to 15 percent by weight) were added in solution with enough water to bring the mixture to a moisture content of 66 percent (the liquid limit of the natural soil). The lime was freshly prepared from CaO. The solutions (suspensions in the case of lime) were well mixed in the soil and stored in sealed jars. Tests were run immediately after mixing, after 1 month, after 4 or 6 months, and after 18 or 24 months (without prior air drying).

THE SOIL PROPERTIES

The soil tested (from the Valley of Jezreel) is well known in Israel, being highly plastic (LL 65-80, PI 43-55, SL 8-10), and exhibits severe swelling properties. The cation exchange capacity is 46.6 me per 100 gm (Ca = 32.6, Mg = 11.7, Na = 1.3, K = 1.0). There is less than ½ me of soluble salts per 100 gm soil (as determined from a 5:1 extract). The pH of the soil is 7.5. Clay mineral analyses (X-ray diffraction and D.T.A.) revealed that the dominant mineral was dioctahedral smectite and that very little kandite was present. The soil is a silty clay that contains 10 percent calcite (of all particle sizes) by weight and 3 percent organic matter. It has a free swell of 110 percent.

RESULTS AND DISCUSSION

The results are reported in Figures 1 and 2. Among the many mechanisms by which the additions of salts might influence the rheological-consistency properties of clays are the following:

1. The physical presence of the additive thus increasing the effective volume of the soil water by acting as a filler.
2. The effect on surface tension.
3. Changes in viscosity of the aqueous phase.
4. Promoting cation exchange and thus changes in the double layer, hydration and changes in interparticle forces.
5. The effect of larger quantities of electrolyte on the double layer from the point of view of flocculation and hydration.
6. pH and its effect on the double layer, on the clay mineral itself, and on organic matter.
7. Effect on soil bacteria and subsequently on the soil organic matter.
8. Solubility relationships in the soil water.
9. Effect on oriented water.

Because of the complexity of the reactions and the relatively low state of knowledge, it would be very difficult to ascribe with any certainty variations in soil properties as due to any one or more of these factors. However, suggestions as to the nature of the cause of the variations are presented for whatever they may be worth.

LIQUID LIMIT (LL)

It is evident that additions of NaCl and $CaCl_2$ affect the soil essentially in a very similar manner, except that the additions of NaCl lower the LL more. The LL expressed on a basis of soil only (that is, discounting the salt) is hardly affected by large additions of $CaCl_2$. If one should consider the salt acting as an inert filler in the water, it is evident that the soil interparticle spacings at the liquid limit have not necessarily been altered. If so, it should be expected that NaCl would cause lower LL's than $CaCl_2$ due to its larger volume both when considered on the basis of equivalent quantities and similar proportions by weight. This is so (more obviously) for the tests run after 18 months.

There appears to be in the case of $CaCl_2$ a tendency for the LL to decrease until 4 months, after which a reversal takes place that is very probably parallel to the increase in LL of the raw soil. (It is interesting to note that the LL of the raw soil originally 66, rose to 75 within a period of 18 months of storage at a high water content and, as a result of air drying, subsequently fell to 69). The changes with time in the case of NaCl additions are not as regular.

The influence of the alkalies is far more striking. Additions of lime cause the LL (tested immediately after mixing) to drop, the smaller additions being of greater importance. Within a month, the mixtures containing 4 or more percent lime (expressed on the basis of CaO) had hardened and were exuding a gas reminiscent of NH_4 and with an alkali reaction. The LL rose considerably, but the 24 months' determination shows a tendency for a reversal of the trend for the smaller effective additions. The initial decrease in LL may be ascribed partly to the influence of lime acting as in inert materials and partly to the influence of high pH on the dispersion and reduction of strength of silicate pastes. The latter has been described by Weyl (2). This is supported by the facts that the significant drop in LL is brought about by up to 4 percent lime (in terms of CaO) and that with further additions a decrease occurs of the order of magnitude of what might be caused by additions of inert material.

The subsequent increase could be ascribed to increased interparticle attraction, but a simpler and probably more correct explanation would be that, as a result of the hardening, the mixtures acquired a new texture. Thus the clay may be regarded as being in the form of small, porous aggregates whose size and number is probably a function of the degree of pulverization of the sample.

Whether the LL of a cemented clay rises or falls is dependent on, among other factors, the arrangement and density of the soil particles at the time of the cementation.

Thus the clay particles that were permitted to harden at a high density will be much less porous than those that have hardened at a low density. The portion of the moisture content at LL that is due to water in the voids of the individual aggregates varies, and so accordingly should the LL. Whether the aggregates cause a decrease or increase depends on the porosity of the aggregates, their size, and the LL of the material in the completely broken-up state.

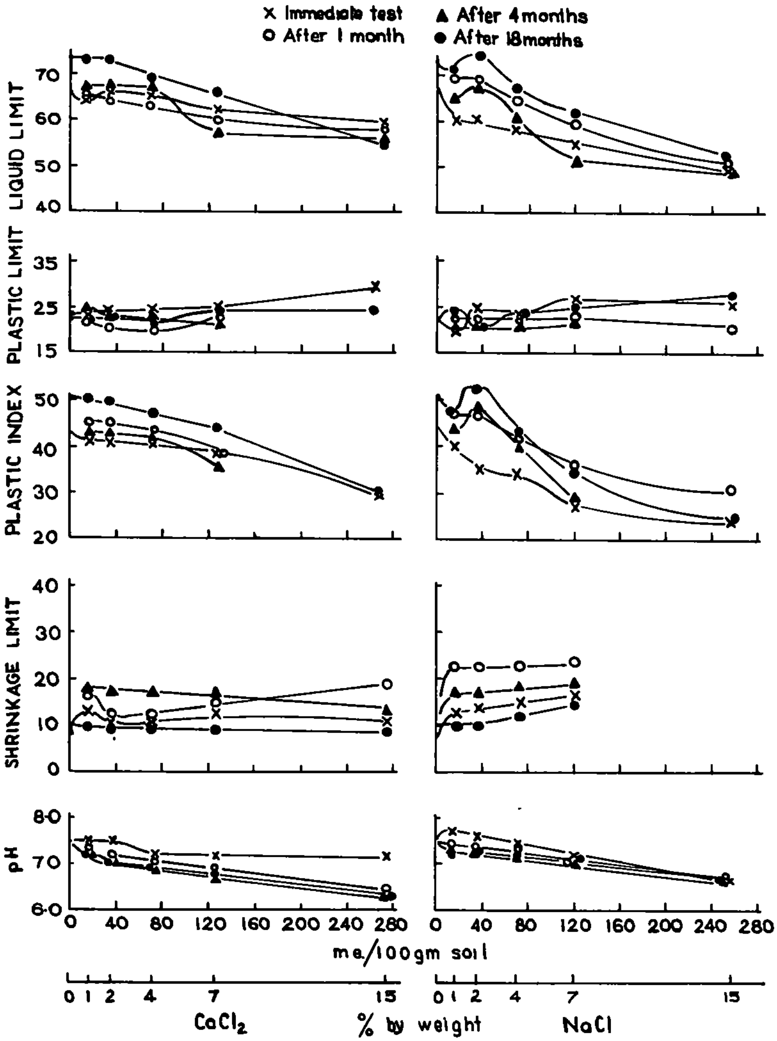

Figure 1. Changes with time in consistency limits, SL, and pH due to additions of $CaCl_2$ and NaCl.

The mixtures of soil-NaOH stiffened but did not harden and, as in the case of the soil with larger amounts of lime exuded upon addition of the NaOH a gas reminiscent of NH_4. The color of the soil darkened considerably and the free liquid (upon placing the soil paste in water) was black. This is probably due at least partially to the destructive reaction of NaOH on the organic matter.

The increase in LL at small additions of NaOH is in keeping with Na being absorbed in the exchange complex. This effect decreases with time, due probably to a strong

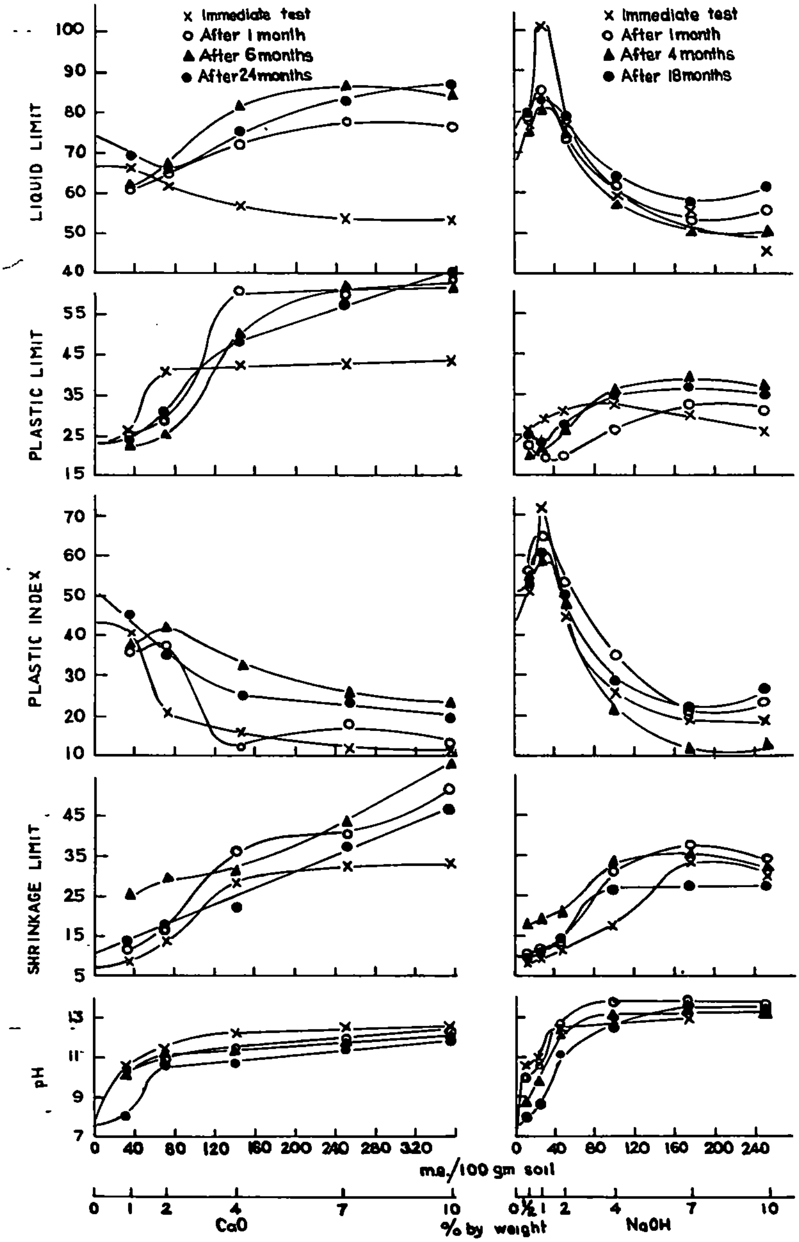

Figure 2. Changes with time in consistency limits, SL, and pH due to additions of CaO and NaOH.

countereffect (of high pH) that at higher quantities of added NaOH tends to reduce the LL. The destruction of humus might tend to increase the LL by causing a decrease in the degree of aggregation. It is not felt that the subsequent release of NH_4 would be important to the exchange complex. The changes with time for the soil with higher NaOH content are irregular.

PLASTIC LIMIT (PL)

The additions of $CaCl_2$ or NaCl cause no significant change in the plastic limit. Those small changes that do occur are not conducive to analysis because of the nature of the test itself. The soil did not become noticeably less sticky or tough as a result of the addition of the chlorides. As opposed to this, the PL was considerably affected by all but the smallest additions of lime. The soil feels and behaves as a silty soil. For the highest addition of lime, the PL tends to rise with time, but for the additions of 2 to 7 percent there occurs (after the initial increase) a drop that tends to decrease after 6 months. The initial increase in PL occurs with the 2 or 7 percent addition of lime (expressed on the basis of CaO) and further increases are of no further significance, indicating perhaps a flocculation. However, the tests after a longer period show the positive effect of amount and an aggregation is indicated.

The smaller additions of NaOH cause a small decrease in the PL such as might be expected to be brought about by medium entering the exchange complex. This apparently requires some time since the initial effect is in the opposite direction. Larger additions of NaOH cause the PL to rise rapidly. The PL does not change significantly after 4 months. The effect of NaOH is consistent with an aggregation that is however smaller than in the case of lime additions.

PLASTIC INDEX (PI)

The influence of the additives on PI is presented in Figures 1 and 2. It is not thought justified to enter into a detailed discussion of the changes that occur, since the salt concentration at LL and PL are quite different and thus changes that are influenced by electrolyte concentration do not occur at the same salt content expressed in terms of dry soil. The PI is not in itself a specific independent property, but it is interesting to note that for all additions of NaCl or $CaCl_2$ the soil remains (on the plasticity chart used in the Casagrande soil classification) a CH soil. However, the larger additions of NaOH and lime cause the soil to cross the A-line on the plasticity chart and would, on its basis for classification, be termed a MH or OH (Fig. 3).

Terzaghi (3) ascribed the properties of a halloysite clay that falls in the same region below the A-line to the "simple assumption that most of the clay fraction of the soil occurs in the form of clusters or hard porous grains with rough surfaces, each of which consists of a number of firmly interconnected clay mineral particles." This might very well be applicable in the case of some of the soil-lime mixtures (which obviously hardened) and in the case of some of the soil-NaOH mixtures.

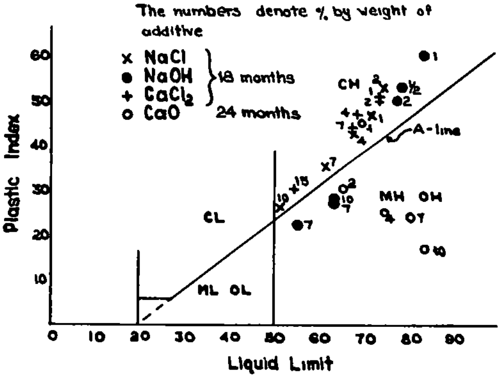

Figure 3. Plot of soil-chlorides and alkalies mixtures on plasticity chart.

SHRINKAGE LIMIT (SL)

In general the addition of chlorides tends to produce an increase in the SL, which is effected mainly by the smallest addition, further additions being of little influence. With additions of both NaCl and $CaCl_2$, the SL rises initially and then tends to drop. The large variations that occur during aging of the mixtures are of considerable interest and it might be said quite unanticipated. It is not very likely that the large amounts of chlorides were rendered insoluble or absorbed. The appearance of a green slime in the jars of some soil-$CaCl_2$ and NaCl mixtures leads to the thought that the addition of the salts, and particularly of the calcium salt, promoted the activity of soil microbes that effected changes in the organic matter. Thus the soil reverted to a more dispersed state.

Winterkorn (4) has pointed out, in relation to cohesive soil stabilization and the effect of soil microbes, that "in fact, with time some of the stabilized soils become less stable than the untreated natural soils." There exists the interesting possibility of promoting and utilizing this phenomenon where an increase in soil dispersion is required, such as in impermeable earth blankets for water ponds.

There is a general tendency for the addition of the hydroxides to cause an increase in the SL within a certain period of time. After that time a drop occurs relatively more significant for the smaller additions of the hydroxides. These results are consistent with a change in aggregation or texture, and changed interface properties are probably also of importance. A countereffect or partial neutralization is indicated by the tendency of the SL to drop after a period of time.

The effect of additions of NaCl and $CaCl_2$ on the pH are very similar except that immediately after addition, the smallest quantities of NaCl cause a slight rise that does not occur in the case of $CaCl_2$. Otherwise, additions of these salts generally cause a progressive decrease in pH. The additions of both lime and NaOH cause an increase of pH to that of the pH of a saturated aqueous solution. The pH subsequently drops, more so for the mixtures with smaller additions of hydroxides. The higher pH readings for NaOH should be treated with caution due to the difficulty of true pH determination of solution with such high concentration of Na^+ at high pH.

The large amounts of lime required to bring the pH of the soil slurry to 12.6 is remarkable. The concentration of lime is about 8 gm per 100 gm water whereas the solubility in water is less than 0.1 gm per 100 gm water. This rapid "absorption" of the lime might very well not be necessarily dependent on prior solution. It is interesting to note that the findings of Clare and Cruchley (5) with regard to changes in pH with time (based, of course, on the initial pH determinations) can be related to the amount of lime absorbed. If this be really so, it would appear that more lime is "absorbed" with time for the richer lime-soil mixtures.

CRITIQUE

It is felt, in retrospect that a program of the nature described is of a limited value that by itself does not give as conclusive an answer as might be desired. While it is true that the Atterberg Limits and the corresponding Plasticity Chart are of considerable local value for purposes of comparison and classification, it is felt that an excessive reliance has been placed on them in spheres in which they are less applicable. The over-faithful use of the PI concept has tended to obscure the need for a more precise classification.

The PI does not necessarily reflect the mechanical properties of the soil. A stiff, non-slaking clay or shale may be described as CH and a lime-stabilized clay, or for that matter any cemented or aggregated clay, might be described as OH or MH, etc. The classification based on the plasticity properties of the ground soil is of very doubtful value. It is no reflection of compressibility or swelling and certainly does not reveal the high water stability of the clay nor its strength. The texture is to an important extent a function of the degree of pulverization, and so accordingly are the consistency limits. It is therefore felt that to anticipate the engineering behavior of soil materials on the basis of their consistency properties alone might very well lead to the neglect of otherwise suitable materials.

CONCLUSIONS

1. There is no essential difference between the influence of NaCl and $CaCl_2$ in the range of quantities added to the consistency properties of the soil. Neither of these salts causes any significant alteration that remains for the duration of the period of testing.
2. The hydroxides have a similar influence on the soil except for a difference in effect on LL. This is (at higher concentration of NaOH) probably a reflection of the weaker cementation that occurs at this high moisture content (and low density). This is reflected in the smaller changes effected by NaOH in PL and SL.
3. The essential difference between the influence of the hydroxides and the chlorides is in the relatively high increases in SL and PL that the former cause. These increase with higher hydroxide content and accordingly with increase in pH.

4. In all probability, the degree of "stabilization" of a soil is most likely to be reflected by changes (increases) in the PL (and to a lesser degree in SL). (It has been previously reported (1) that additions of the chlorides to this soil did not impart further strength nor resistance to slaking, whereas the addition of equivalent amounts of the hydroxides did.)

5. Large variations with time occur and reactions should be checked over an extended period of time.

6. In order to minimize the influence of changes in the soil due only to rewetting, it is felt that soils should be wetted well in advance of the commencement of research programs.

ACKNOWLEDGMENT

The author's warm thanks are due to J.G. Zeitlen, Professor of Soil Engineering and Dean of the Faculty of Civil Engineering, for his interest in the work and for making it possible, as well as to Dr. J. Rubin, Head of the Soils Department, Rehovot Agricultural Experiment Station, for his valuable advice.

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A Problem in Highway Slope Stability

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This paper describes the application of soils engineering to the correction of an unstable highway slope.

During construction of a highway cut through clays, silts, and sands of glacial outwash origin, the cut slope became very unstable. Extensive sloughing and erosion, with some slope movement, developed as excavation of the cut proceeded. A heavy flow of ground-water seepage developed in the cut slope. Drainage methods used to control this seepage are described.

After excavation of the cut area was complete, further slow movement of the slope face was observed. Successive cross-sections showing rate and nature of movement are presented. Additional field explorations employing continuous sampling techniques were made to determine cause of movement. Methods of sampling and testing are described. Investigation revealed slickensides, resulting in low-strength zones, in a soil mass that had general high-shear strength. Also, a zone of soft low-strength elastic clay was disclosed at a critical location below subgrade.

Observations indicated movement on a failure plane approaching the shape of a circular arc. Analyses by the Swedish circular arc method and the critical height of slope method are discussed. Results of further study to determine the slope section required to attain stability are presented. Corrective treatment used is described, and principles and techniques used in this solution that are considered to have general application to similar problems are summarized.

Pictures illustrate conditions from the beginning of construction to final completion of corrective work in September 1960.

● THE PROBLEM described in this paper occurred during the construction of a 21-mi section of Interstate 70 on new location in central Ohio, between Columbus and Zanesville, at a point immediately east of the structure carrying Ohio 668 over Interstate 70. The general location with respect to the Ohio highway system is shown by Figure 1.

GENERAL ROADWAY DESIGN FEATURES

In general, the roadway section is comprised of two 24-ft pavements separated by a depressed 50-ft median. The pavement consists of 9 in. of reinforced portland cement concrete on 6 in. of granular subbase. The median is graded to an 8:1 slope downward to a center ditch.

Paved shoulders are provided adjacent to both edges of each concrete pavement, with the outside shoulders 10 ft wide and the median shoulders 5 ft wide. Composition of the paved shoulders consists of 6 in. of granular subbase, 6 in. of crushed aggregate base, 3 in. of bituminous macadam base, and a double-seal surface treatment.

Subsurface drainage is provided by four lines of longitudinal pipe underdrains, with a line located under each shoulder 5 ft outside each pavement edge. The underdrains consist of 6 in. extra-strength drain tile, with porous backfill extending up through the subbase to contact with the bottom of the stabilized crushed aggregate base in the shoulder.

Standard plan slopes employed are 2:1 (horizontal: vertical) for fills over 10 ft high, 4:1 for fills 10 ft or less in height, 2:1 for cuts in excess of 5 ft deep, and 3:1 for cuts of 5 ft or less in depth.

GEOLOGY, TOPOGRAPHY, AND GENERAL SOIL CONDITIONS

Geologically, the problem area lies at the southeastern boundary of glacial advances, which, in past geologic history, covered approximately two-thirds of the State. The slope instability developed at a location within the limits of the Illinoian ice advance and beyond the limits of the Wisconsin ice advance. The location in relation to glaciation is shown in Figure 2. Soil deposits involved in the cut slope are of marginal glacial outwash origin of considerable depth. Topographically, the terrain is hilly primarily as a result of stream abrasion. The general surface relief is shown by Figure 3. Maximum relief of the area in the general vicinity of the problem is approximately 150 ft.

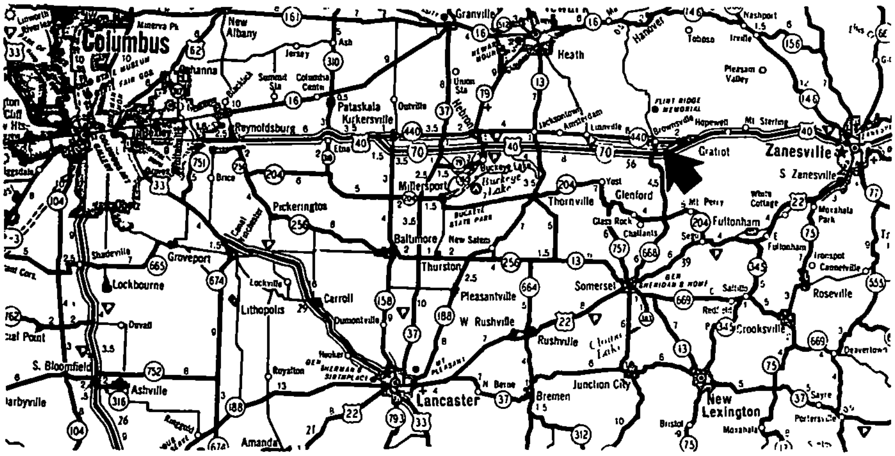


Figure 1. Partial Ohio Highway Map, showing location of slope instability with respect to highway system between Columbus and Zanesville.

In this area, the alignment traverses a gentle upland slope of only about 6 deg. At the location of instability, the alignment traverses a gentle topographic depression that tends to concentrate surface water runoff from higher terrain to the left of the alignment. The terrain before construction is shown by Figure 4, taken on centerline viewing forward and by Figure 5, taken at about 125 ft left of the centerline and viewing forward. Both views show the gentle topographic depression.

A cross-section of the proposed cut is shown by Figure 6. This figure also shows logs of test borings of the roadway subsurface investigation drilled prior to detailed design and construction plan preparation. These borings disclosed soil deposits of much greater depth than anticipated at this location. They further disclosed the deposits to be of marginal glacial origin and to consist of silts and fine sands underlain by clays directly overlying bedrock. Bedrock is composed of firm fine-grained sandstone and argillaceous shale with thin sandstone interbeds.

The subsurface investigational practice for this project entailed test borings for the general roadway to a depth of 8 to 10 ft below proposed grade at intervals along centerline averaging approximately 300 to 400 ft. Test borings of anticipated depth of 20 ft or less were drilled by faster truck-mounted bit-on-kelly-type augers and were 9-in. diameter holes. These borings were advanced in increments of from 0.5 to 1.0 ft, the bit being retracted after each increment of advance and the material inspected. Representative samples of each layer penetrated were taken and submitted for laboratory classification tests. Borings in excess of 20 ft in depth were drilled by slower split-spoon sampling techniques, with samples taken at about 5-ft increments of depth and submitted for laboratory classification tests.

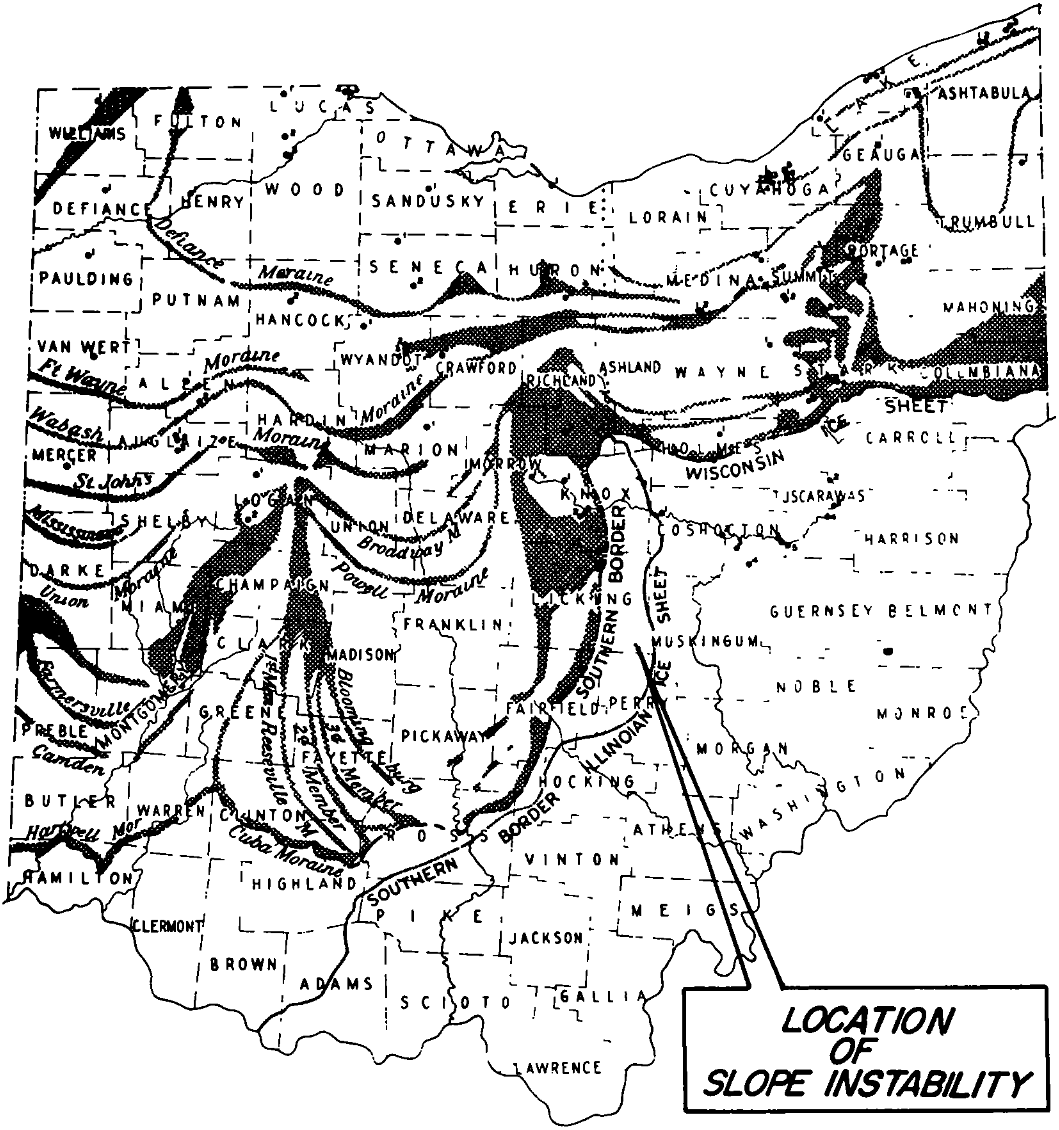


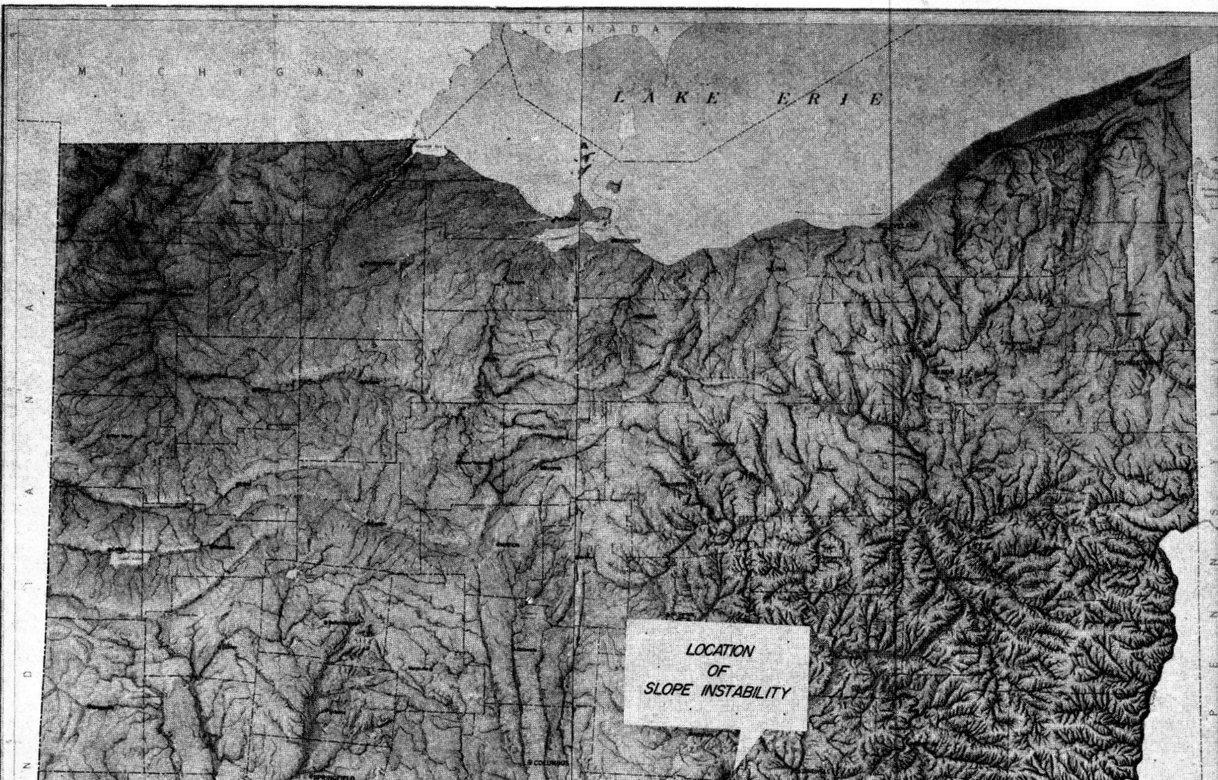
Figure 2. Map of Ohio, showing boundaries of glacier advances, and location of slope instability.

PROGRESS OF CUT EXCAVATION

Excavation of the roadway cut, about 45 ft deep, between Stations 1509 and 1516 was begun in May 1958 and completed to rough grade in October of the same year. High moisture content of the soil and strong flow of ground water from the north slope of the excavation at numerous locations made working conditions difficult and rate of progress slow.

The excessively high moisture content of the soil resulted in adequate bearing for efficient operation of heavy earth-moving equipment. A drag line, working on mats, was used for much of the excavation as shown by Figure 7.

Much of the soil from the excavation was wasted at the option of the contractor. Ohio specifications provide that soil with moisture content in excess of optimum plus 8 percent may be wasted, even though suitable when drier.



LOCATION OF SLOPE INSTABILITY

Figure 3. Map of Ohio, showing surface relief of State and location of slope instability.

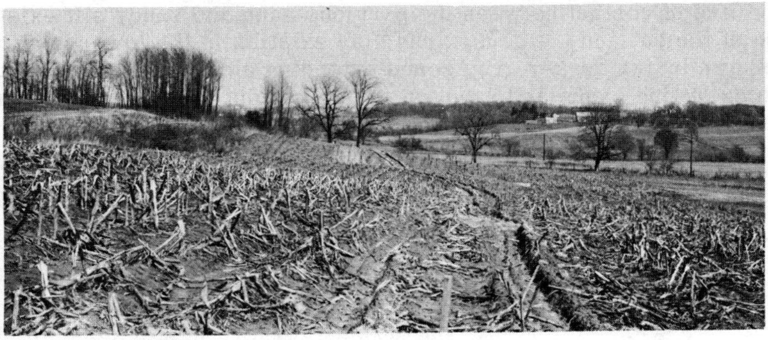


Figure 4. Foreground shows topography prior to construction along proposed centerline in area where cut is to be made.

Figure 5. This farm drive, prior to construction, is located at about the top of the proposed cut slope. Note sag in topography, and direction of surface drainage along gentle slope from left to right.

During excavation, longitudinal side ditches 10 to 15 ft deep were maintained between the bottom of slope and the material being excavated. The purpose of these ditches was to collect and divert seepage water from the slopes and to drain water from the saturated material being excavated, thereby improving conditions for operating earth-moving equipment. A view of one of these ditches (Fig. 8) shows that seepage from the side slopes caused erosion and localized sloughing and required frequent restoration of the ditch flow line by repeated excavation. The generally soft condition of the material during progress of excavation is shown by Figure 9, taken at the time the excavation was approaching rough grade.

Construction was suspended for the winter, with the cut completed to rough subgrade elevation.

COMPLETION OF CONSTRUCTION

When work resumed in this area in the early spring of 1959, the first operation was to excavate from the left ditch material that had accumulated there during the winter. This ditch-cleaning operation was repeated several times during April and May.

Ground water seepage occurred at a number of locations along the slope. This seepage caused localized sloughing of the slope, severe erosion of the slope face, obstruction of the roadway ditch, and ponding of water in the ditch and shoulder area. This became a major obstacle to completion of installation of roadway subgrade drainage and construction of the paved shoulder through this area. Seepage was particularly pronounced along the line of contact between the pervious sand and sandy silt existing in the upper portion of the cut and the dense gray clay existing in the lower portion of the slope. This is shown by the darker gray zones extending along the slope in Figure 10. Localized sloughing of fine sands and sandy silts at the point of seepage breakout and erosion channels on the face of the slope are shown in detail by Figure 11. To control this seepage and associated slope damage, a system of subsurface drains was installed in the slope.

The pavement and other roadway items of this contract were completed and with this, a 21-mi continuous section of Interstate 70 was opened to traffic.

DEVELOPMENT OF MAJOR SLOPE INSTABILITY

Between May 1959 and January 1960, concurrently with completion of this construction contract, a number of observations were made disclosing very slow but progressive movement of the slope, indicative of major instability.

The first significant observation was that of a tension crack opening in the service road at the top of slope. The crack appeared approximately parallel to and 190 ft left of centerline, between Stations 1512+50 and 1516+00. The occurrence of the crack was also accompanied by some vertical displacement at the top of slope. This is shown by Figure 12.

Tension cracks at the toe of slope, as a result of bulging of the lower portion of the slope, are shown by Figure 13. Such tension cracks resulting from bulging may easily be misinterpreted as erosion channels, particularly in the early stage of development. Close inspection of such cracks will disclose that they cross high points of the slope surface and therefore are not characteristic of surface runoff channels.

In the foreground of Figure 14, diagonal tension cracks are shown in the mid-portion and upper portion of the slope. Such cracks also might be misinterpreted initially as

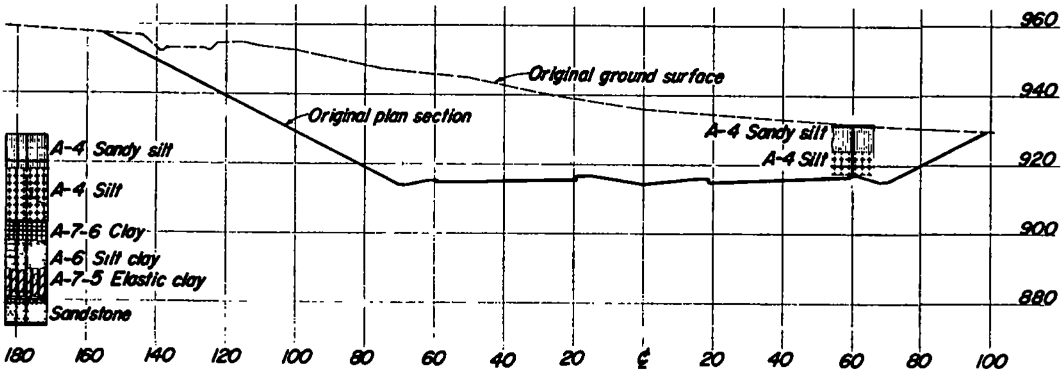


Figure 6. Cross-section of proposed cut, showing original ground surface, original plan section, and logs of borings for roadway subsurface investigation.

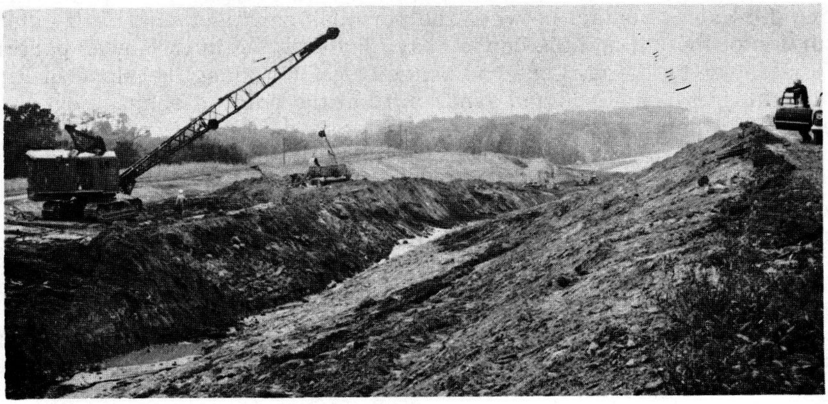


Figure 7. Excavating soft wet soil by using drag line working on mats.

Figure 8. Deep side ditch maintained through cut during excavation, showing erosion.

Figure 9. View showing soft wet condition of soil during progress of excavation.

early development of surface erosion courses. Close inspection discloses characteristics similar to the bulging cracks mentioned. Diagonal cracks normally occur in the area of the upper lateral limits of the sliding mass. Also in the upper right portion of the view of Figure 14, a sharp break and vertical downward displacement of the slope is seen.

Figure 10. View showing seepage in left cut slope along line of contact between sand and sandy silt in upper portion of cut and clay in lower portion of cut.

In Figure 15, at the left and at the right of the view, two cracks are shown running generally along the face of the slope, with the intermediate soil mass displaced downward. This denotes general outward movement having occurred in the lower portion of the slope, accompanied by block failure in the upper portion of the slope.

Figure 16 shows the material forming the lower portion of the slope. This material consists of layers of clay size particles separated by thin layers of silt size particles. Planes of slippage were observed in this material. Sag at the top of the slope after a 6-month period of movement is emphasized by the row of guard rail posts shown in Figure 17. Heaving of the ditch, shoulder, and outer pavement lane is shown by Figure 18.

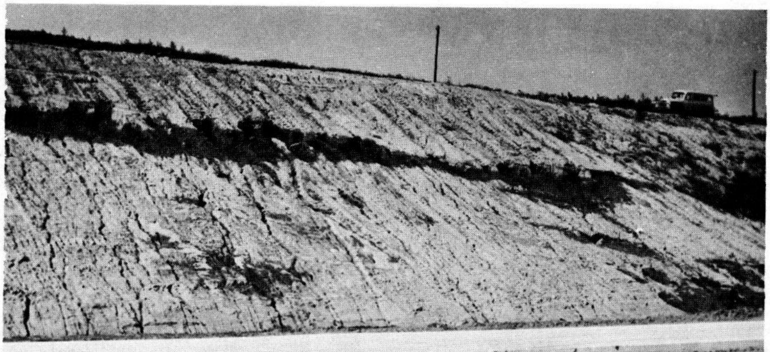


Figure 11. Sloughing of fine sands and sandy silts at point of seepage breakout, and erosion channels on face of slope.

Figure 12. Crack at top of slope, indicating slope failure.

DETAILED SUBSURFACE INVESTIGATION

Very slow movement of the slope continued and resulted in heaving of the shoulder and pavement, creating a hazard to traffic. The heaved pavement lane was closed to traffic and major corrective action was initiated.

In the development of corrective treatment, the initial step was the performance of a detailed subsurface investigation of slope conditions. For the purpose of procuring detailed information on the structural characteristics of the soil mass, continuous sampling procedures were employed from the ground surface to 10 ft below the bedrock surface. In soil, thin-wall samplers were pressed continuously, except in hard or dense zones where split-tube drive samplers were used. Upon encounter with bedrock, continuous diamond coring was employed. Two such borings were drilled, one at the top of slope and one at the toe of slope. Truck-mounted rotary drilling equipment of the type shown in Figure 19 was employed.

For more complete coverage of the condition, a number of faster and less costly borings were drilled along the toe of slope for procurement of disturbed samples to develop information at additional locations. These borings were drilled by means of truck-mounted bit-on-kelly auger of the type shown in Figure 20. Additional borings on the slope were drilled by hand auger.

All samples were subjected to examination, moisture content determination, and classification tests. Upon ejection from the tubes, undisturbed samples were closely examined and specimens were selected for laboratory determination of strength characteristics.

Figure 13. Tension cracks at toe of slope, resulting from bulging.

Figure 14. Diagonal tension cracks in the mid-portion and upper portion of the slope.

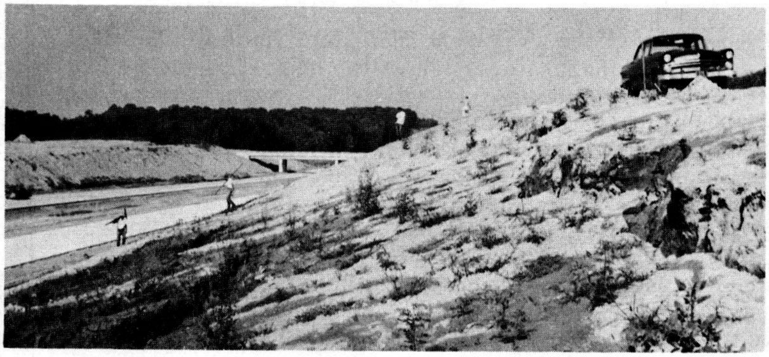


Figure 15. Cracks and downward displacement of intermediate soil mass.

Figure 16. View of lower portion of slope.

Results of the detailed investigation disclosed the soils for the most part to be erratic in deposition. The materials comprising the higher portion of the slope consisted of fine to coarse sands, silts, sandy silts, and clayey silts. Silty clays and clays were found to comprise the lower portion of the slope and the foundation. Findings of test borings are presented by the cross-section shown in Figure 21.

Figure 17. Sag at top of slope, shown by drop of guard rail posts.

In general, appearance of the samples and strength determinations indicated strengths of considerable magnitude. In the silts, sandy silts, and sand, values for angle of internal friction ranged from 25 to 50 deg, and values for apparent cohesion ranged from 0.02 to 0.3 tons per sq ft. In the silt clays and clays, values for angle of internal friction ranged from 15 to 35 deg and values for cohesion ranged from 0.10 to 1.20 tons per sq ft.

Test borings disclosed a persistent zone of elastic clay occurring in the slope foundation. In this zone, samples disclosed the natural moisture content to be consistently above the plastic limit of the material. Strength tests of specimens resulted in apparent values for angle of internal friction of only 3 to 4 deg and values for cohesion

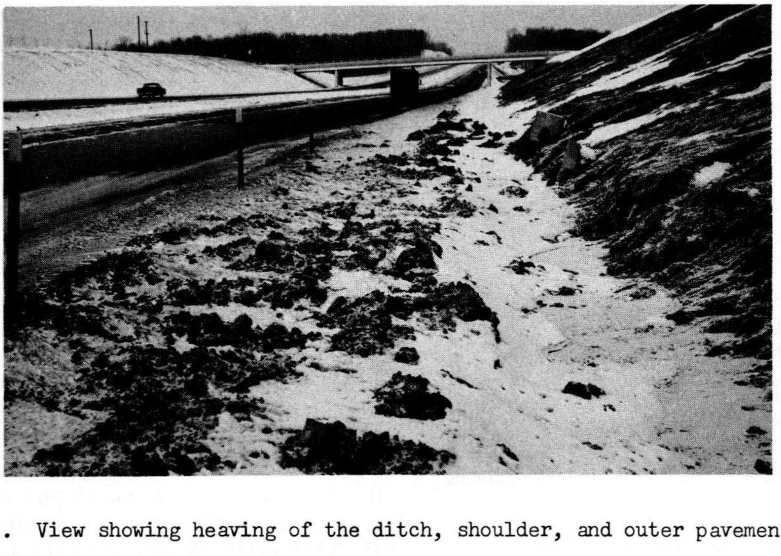


Figure 18. View showing heaving of the ditch, shoulder, and outer pavement lane.

of 0.28 to 0.65 tons per sq ft. Slippage planes were also observed in undisturbed samples from this zone.

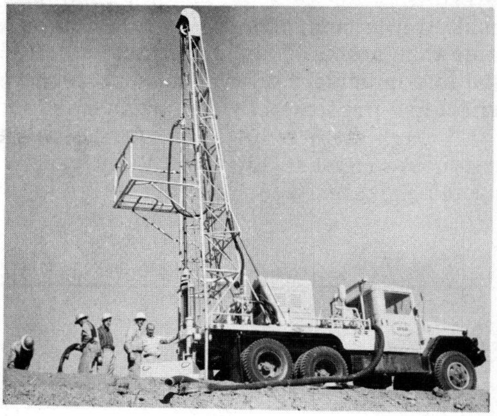


Figure 19. Truck-mounted rotary equipment used for continuous press sampling and coring.

ANALYSIS OF FINDINGS AND OBSERVATIONS FOR CORRECTIVE TREATMENT

Environmental conditions adjacent to the right-of-way were conducive to correction by flattening of the slope. Procurement of additional right-of-way for this purpose involved only tillable land of average quality. This method of treatment involved only construction of the least costly type. Of further importance, this method of treatment did not involve further aggravation of slope instability during construction operations.

From the aforementioned field observations and subsurface investigation, a number of factors relative to the nature of the slope failure were approximately defined, thereby simplifying analysis. The approximate limit of the failure at the top of slope was defined by the tension crack and associated subsidence of the mass. The approximate limit of failure at the base of the slope was defined by the limit of heaving of the paved shoulder and pavement lane. The maximum depth to which movement could occur was defined by the test borings, sampling, and laboratory tests. They disclosed a softer plastic clay zone underlain by hard clay immediately over bedrock. Successive cross-section observations, showing the progress of slope movement, were interpreted to define the approximate location of a vertical plane containing an axis about which rotational movement appeared to be occurring. For purposes of analysis, these factors were assumed to define a circular arc approximately encompassing the failure mass. This is shown by Figure 22.

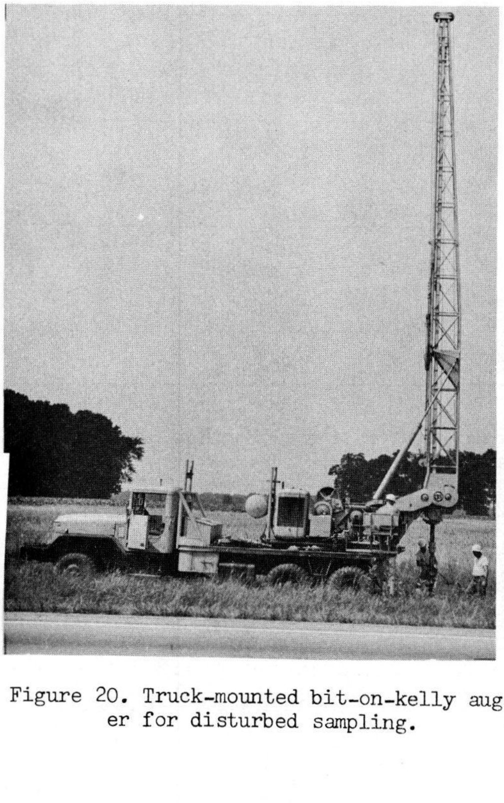


Figure 20. Truck-mounted bit-on-kelly auger for disturbed sampling.

As previously mentioned, strength values as disclosed by laboratory tests varied over a wide range. The effective strength of the mass, therefore, was not defined as readily as the geometric limits of the failure mass. Computations employing the higher shear strength values (15 to 50 deg) resulted in a critical height of slope greater than the height of the failed slope. However, computations employing the lower strength values (3 to 4 deg) indicated that these more nearly coincided with the effective strength of the soil mass.

For purposes of stability analysis, the entire mass was assumed to have an effective cohesion of 0.2 ton per sq ft and angle of internal friction of 5 deg. Critical height of slope computations indicated these values to be somewhat lower than the effective strength in the slope. Also, employing these values in the Swedish circular arc method of slices in the analysis of the failure arc, as approximated from the observations and subsurface investigation, resulted in a computed factor of safety of 0.85, with this considered to be slightly lower than actual. The observed extremely slow movement of the slope was interpreted to be indicative of a factor of safety with respect to sliding more nearly approaching 1.0.

Although slightly low, the aforementioned assumed strength values were considered reasonably representative of effective strengths developed in the mass and were used in subsequent analyses for corrective treatment. The analyses pursued the course of

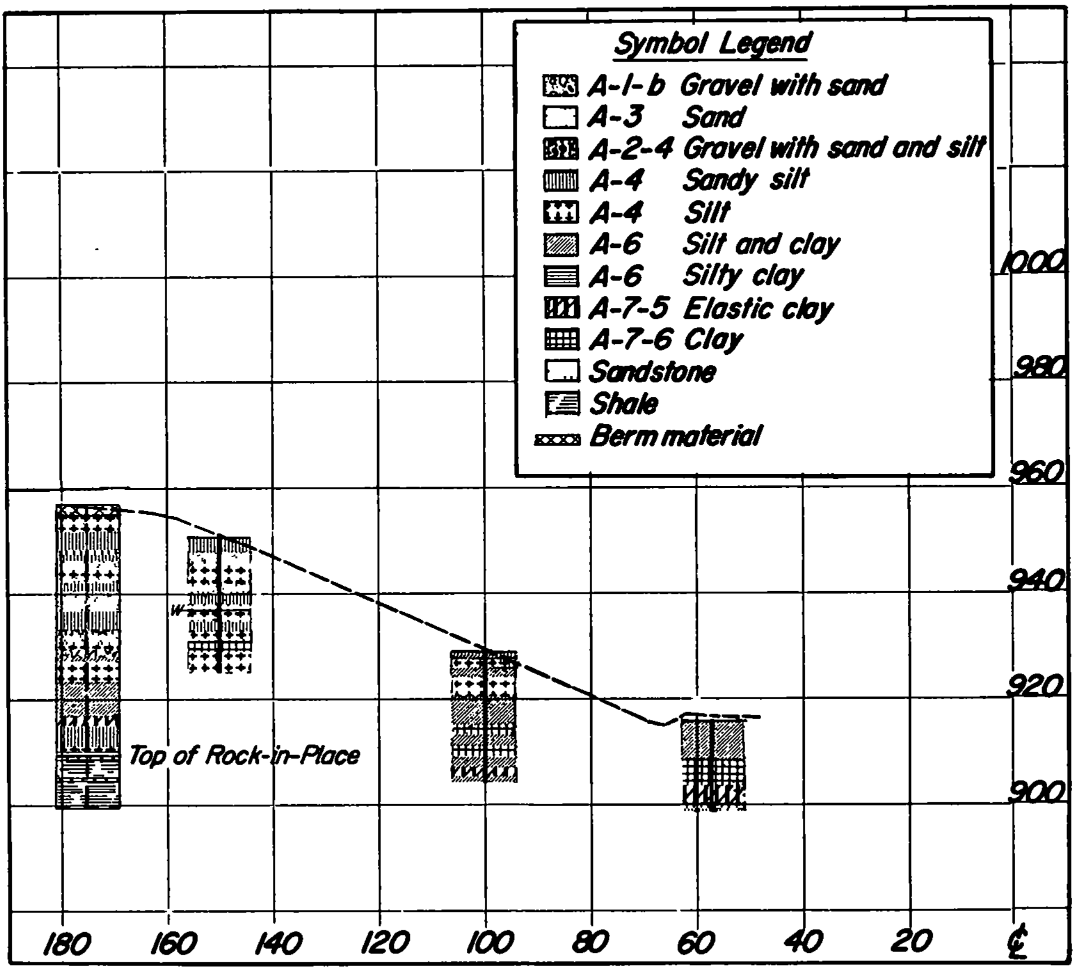


Figure 21. Cross-section of original cut, showing logs of several borings made for the detailed slope investigation.

investigating various trial slope sections by the Swedish circular arc method of slices. The driving forces, tending to actuate slope movement, were reduced through employing a combination of benches and reduced degree of slope.

The slope section selected for final corrective treatment of the problem was computed to have a factor of safety with respect to sliding of approximately 1.2. It is considered that a factor of safety slightly in excess of 1.2 most probably will be realized,

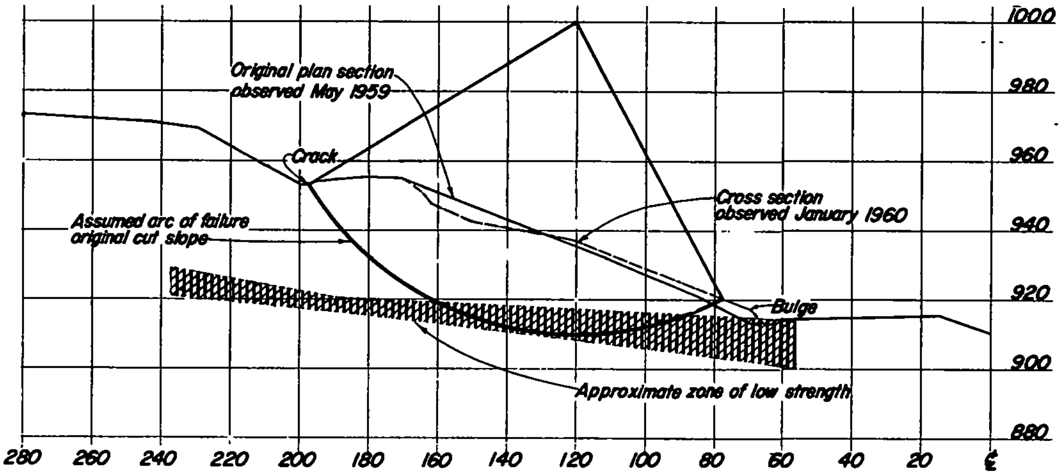


Figure 22. Cross-section of original cut, showing original plan section, failure section (including crack and bulge), zone of low strength, and approximated arc of failure.

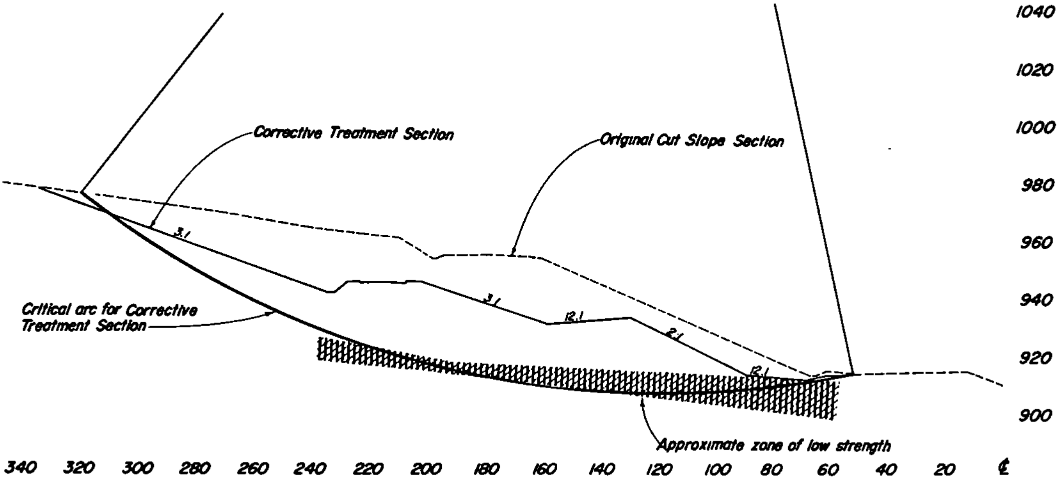


Figure 23. Cross-section of corrective treatment, showing zone of low strength and critical arc for the section.

for reasons mentioned in the foregoing paragraph. Design details of the slope section for corrective treatment are shown by Figure 23.

CORRECTIVE TREATMENT

Construction by change order was started in July and completed in October 1960. The correction involved a quantity of 65,000 cu yd of excavation at a cost of $52,000. Total cost of correction was $74,000. In addition to the earthwork, this total includes cost for removal and replacement of one pavement lane and paved shoulder, restoration of subgrade drainage, installation of subsurface drainage on the two upper benches, restoration of the service road, and other miscellaneous items.

Figure 24 is a view taken after completion of corrective work, showing the benches and slopes.

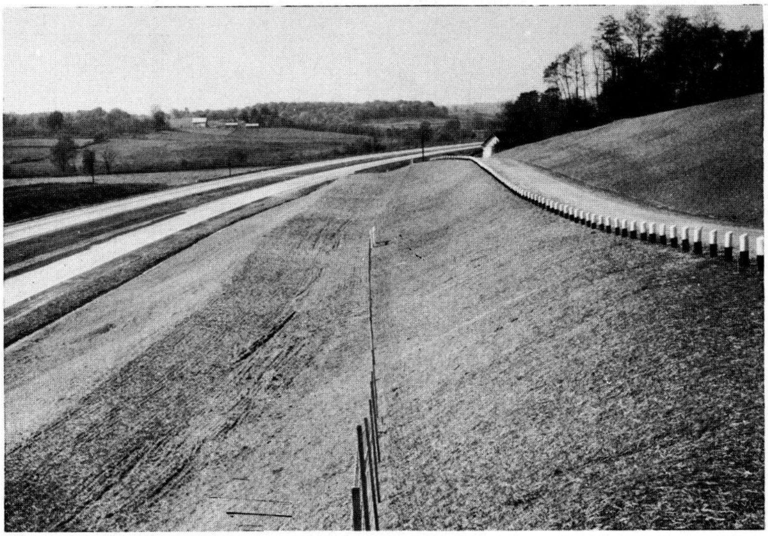


Figure 24. Slope after construction to correct slide.

CONCLUSIONS

This problem in general illustrates the effectiveness of applied soil mechanics in the investigation, analysis, and treatment of problems of slope stability. A number of considerations are emphasized:

1. Detailed subsurface information for reliable analysis of critical soil conditions is of utmost importance. Extreme care in sampling and a thorough record of observations during drilling is of basic importance. Information of critical importance can be missed when employing intermittent sampling procedures or procedures that totally disrupt soil structure characteristics in procurement of samples. Continuous sampling techniques, preferably pressed undisturbed samples, offer greatest assurance in developing reliable information.
2. Laboratory test results to be employed in the analysis of problems must be reviewed with extreme care for use in analysis. Close inspection of samples during each step in testing, from opening the samples through completion of tests, is required, with detailed observations recorded for correct evaluation of test data.
3. Field observations are of utmost importance in the definition of environmental factors and boundary conditions directly related to critical soil conditions and analyses of problems associated with highway design and construction. A thorough photographic record of such observations is invaluable.
4. Most effective solution of problems associated with critical soil conditions justifies the attention of specialists with technical training and experience in both theoretical and applied soil mechanics.

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