357

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Chemical Soil Stabilization And Soil Aggregate Stabilization

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Contents

Stabilization of Soil with 4-Tert-Butylpyrocatechol

JOHN B. HEMWALL, Dow Chemical Company
DONALD T. DAVIDSON, Iowa State University, and
HENRY H. SCOTT, Dow Chemical Company

This investigation concerns the use of 4-tert-butylpyrocatechol (TBC) as a trace soil additive to mitigate the normally adverse effect of water on the soil, thus rendering the soil a more suitable engineering material. The TBC changes the surface character of the soil particles from a normally hydrophilic to a hydrophobic state.

The results reveal that although almost all soils can be rendered hydrophobic by the TBC, the silty-type soils are benefited most. These soils retain substantial unconfined compressive strengths, even when totally immersed in water and after several cycles of freezing and thawing.

The investigation reveals that the TBC gives best results when added to the soil with sufficient water to bring the soil up to optimum moisture content. The optimum curing conditions are shown to be those that result in drying of the soil after compaction. Thus, curing the treated compacted soil at a relative humidity of 50 percent is very effective.

The results presented are based entirely on laboratory studies. However, the properties imparted to the soil and the results obtained indicate that many soils are potentially suitable engineering materials when treated with TBC.

• THE USE OF CHEMICALS to modify or improve certain physical properties of soils has become a matter of considerable interest to engineers in the last 25 years. Attention has centered around improving the load-bearing capacity or strength of soils, especially under conditions of high moisture. The treatment of soils with chemicals, or chemical soil stabilization, is primarily intended to make soil a suitable alternative to crushed rock or aggregate in road bases and subbases.

Recently Sherwood (1) classified chemical soil stabilizers into the three categories of bonding agents, waterproofing agents, and combination bonding and waterproofing agents. Bonding agents stabilize soil by creating physical and/or chemical bonds between the soil particles which enhance the strength of the soil under both dry and wet conditions. Waterproofing agents do not affect dry strength significantly but allow the dry strength of the soil to be retained in the presence of water by reducing water adsorption.

Davidson and co-workers (2, 3, 4, 5) have studied soils waterproofed with a variety of chemicals. Many of these soils when waterproofed retained sufficient strength in the presence of water to be potentially suitable for base or subbase purposes. Some of the chemicals waterproofed soil more effectively than others. All of the chemicals, however, behaved more or less similarly in soil.

The purpose of this paper is to report results obtained with a new and chemically unique soil waterproofing agent, 4-tert-butylpyrocatechol, hereinafter referred to as TBC. The optimum conditions for the use of this chemical, the soil types in which it is most effective, and the optimum rate of use are discussed.

MATERIALS AND METHODS

Soils

The soils used in this investigation are given in Table 1 together with many of their

TABLE 1

PHYSICAL AND CHEMICAL PROPERTIES OF SOILS USED

Ref No	Mechanical Analysis[a]			pH[b]	$CaCO_3$[c] (%)	Organic Matter[d]	Cat Ex Cap [e](meq/g)	Plastic Index[f]	Liquid Limit[f] (%)	Opt Moist [g] (%)
	Clay	Silt	Sand							
A-1	16	41	43	7 5	1 1	1 1	14 4	5 4	25 8	14-16
B-1	22	52	26	5 3	0	10 7	39 6	11 1	76 6	33-35
B-2	12	46	42	7 2	3 5	4 1	4 0	1 1	33 6	17-19
C-1	25	46	29	7 8	7 9	3 1	16 7	8 8	39 1	20-22
D-1	12	46	42	6 0	0	1 0	8.0	[illegible]	17 8	10-12
E-1	22	62	16	7 7	4 5	1 4	23 4	13 8	36 6	15-17
F-5	23	35	42	7 1	0	1 1	16 0	5 9	22 2	11-13
F-7	38	35	27	7 4	0	1 6	22 0	47 5	27 5	13-15
H-1	6	14	80	5 5	0	0	3 0	0	13 5	8-10
J-2	55	40	5	7 5	1 7	1 6	31 8	21 4	48 3	16-18
K-1	18	43	39	7 6	1 4	2 3	22 4	6 0	35 1	14-16
L-2	67	27	6	7 6	11 5	1 7	33 0	24 0	63 0	26-28
M-2	42	36	22	6 3	0	1 2	11 2	43 0	38 8	24-26
N-1	48	39	13	6 75	0	2 7	31 1	21 4	49 1	17-19
P-2	57	27	16	7 7	1 2	1 8	18 2	25 0	58 0	19-21
R-1	18[h]	80[h]	2[h]	8 4	9 9[i]	0 1[j]	14 7	7 2	34 2	17-19[k]
R-7	39[h]	61[h]	1[h]	5 6	1 5[i]	0 2[j]	23 5	32 1	52 1	19-21[k]

[a]Determined by hydrometer method
[b]Determined on a non-plastic slurry
[c]By carbonate determination
[d]Determined by wet ash method
[e]Ammonium acetate method
[f]ASTM methods D423-54T and D424-54T
[g]Moisture at maximum density
[h]ASTM method D-422-54T
[i]By versenate method for total calcium
[j]Potassium bichromate method
[k]ASTM method D698-58T

physical and chemical properties. All soils were air dried, pulverized, and screened through a 10-mesh sieve before use.

Immersed Unconfined Compressive Strength

The TBC was added to soil in all cases as an emulsion in the molding water. The amount of water used was sufficient to bring the soil up to optimum moisture for maximum density. The soil was then compacted, cured for 7 days, and immersed for 24 hr in distilled water before determining the unconfined compressive strength. Whenever a modification of this general procedure was used, it has been noted in the description of the specific experiment. Auxiliary data such as weights, moisture contents, etc., were determined by classical procedures.

Some of the experiments reported were conducted at the Iowa Engineering Experiment Station, Ames, whereas others were conducted in the laboratories of the Dow Chemical Company in Seal Beach, Calif. Although the general procedure previously described was used in both laboratories, some of the details differed. The California procedure involved soil plug specimens 3.0 cm in diameter by about 6 cm in height. These specimens were compacted via two-end loading in a hydraulic press at a pressure of 750 psi on the plug specimen. The specimens were stressed to failure on a motor-driven Soiltest Model U-160 unconfined compression test apparatus using a loading rate of 0.07 in. per min.

The Iowa procedure involved specimens 2.0 in. in diameter and about 2 in. in height. The specimens were compacted via a conventional drop-hammer apparatus (5) and were stressed to failure on an unconfined compression test apparatus using a loading rate of 0.10 in. per min. The descriptions of the specific experiments indicate whether the Iowa or California method was used.

Freeze-Thaw Test

Soil specimens prepared via the Iowa method were subjected to 10 cycles of freezing and thawing as described previously (6). The apparatus and procedure were essentially the same as the British Method, except that the freezing temperature was 20 ± 2 F and the temperature of the water inside the vacuum flask was controlled at 35 F. The plastic

TABLE 2

EFFECTS OF TBC ON OPTIMUM MOLDING MOISTURE CONTENTS FOR MAXIMUM DRY DENSITY AND MAXIMUM IMMERSED STRENGTH, AND ON VALUES OF MAXIMUM DRY DENSITY AND MAXIMUM IMMERSED STRENGTH

Soil	TBC (%)	Optimum Molding Moisture (%)		Max Dry Density (pcf)	Max Immersed Strength[a] (psi)
		Max Dry Density	Max Immersed Strength		
R-1	0	18 0	--	106 5	0
	0 042	18 7	19 3	107 4	115
	0 085	17 3	17 3	106 8	132
	0 212	18 4	17.9	107.1	155
	0.425	19 6	19 3	107 7	180
	0 850	17.7	17.2	108.5	180
R-7	0	18 9	--	108 9	0
	0 042	17.8	19.0	108 1	40
	0.085	18 4	18 8	108 5	52
	0 212	18.6	19 7	107 6	45
	0 425	18.1	18 8	107.9	50
	0 850	18 4	19.1	107 9	42

[a] Cured for 7 days at 60 percent RH before 24-hr immersion.

specimen containers used restricted the volume change of the specimens to the upward direction.

RESULTS

Four major factors relative to the performance of TBC as a soil waterproofing agent were investigated: (a) the optimum conditions for treatment with TBC, (b) the limitations of TBC with respect to soil type, (c) the necessary treatment rates, and (d) the stability of the treated soil to freezing and thawing.

Moisture content at compaction is one of the major factors influencing the strength of stabilized soil. Generally, the optimum moisture content for maximum strength is that resulting in the maximum compacted dry density. Two soils were treated with several rates of TBC and were compacted at several moisture contents in an effort to determine the optimum moisture for both maximum dry density and maximum immersed strength. The samples were compacted by the Iowa method, cured for 7 days at approximately 60 percent relative humidity, and immersed for 24 hr in water before determining the unconfined compressive strength. The dry density of the samples was determined immediately after compaction. The soils used together with a summary of the results are given in Table 2. In addition, the complete results for soil R-1 treated with 0.085 percent TBC are shown in Figure 1.

The results in Figure 1 and Table 2 reveal that the optimum molding moisture contents for both maximum dry density and maximum immersed strength are essentially the same. Furthermore, the optimum molding moisture content is essentially the same regardless of the presence or absence of TBC or the rate applied. Thus, the optimum molding moisture content for soils being treated with TBC is

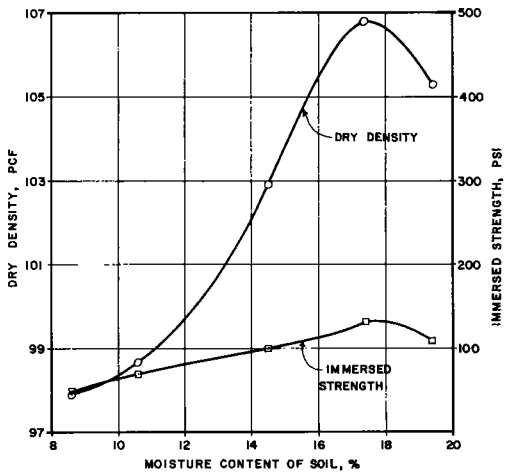

Figure 1. Dry density and immersed strength of soil R-1 treated with 0.085 percent TBC against moisture content of soil.

TABLE 3

INFLUENCE OF CURING METHOD ON IMMERSED STRENGTH OF SOIL R-1 TREATED WITH 0.425 PERCENT OF TBC

Method of Curing[a]	Immersed Strength (psi)
7 days at RH > 95%	54
14 days at RH > 95%	58
7 days at RH > 95% followed by 7 days at RH = 60%	178
2 days at RH = 49%	64
7 days at RH = 52%	218
14 days at RH = 56%	217

[a]RH = relative humidity.

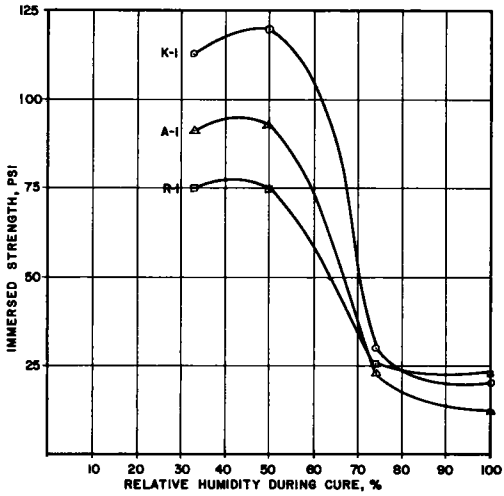

Figure 2. Immersed strength of soils A1, K1, and R1 treated with 0.1 percent TBC against relative humidity at which soils cured.

the optimum moisture content of the untreated soil for maximum dry density.

Maximum dry density for soil R-1 increased slightly with increasing TBC content. However, with soil R-7 the reverse was true. Because density can be controlled only to about ± 3 pcf in field operations, the changes noted are not considered especially significant.

Another factor influencing the strength attained by a chemically stabilized soil is the cure conditions. In a preliminary experiment several samples of soil R-1 treated with 0.425 percent of TBC were compacted at optimum moisture by the Iowa method, cured under the variety of conditions given in Table 3, and immersed for 24 hr in water before determining unconfined compressive strength.

The data in Table 3 indicate that curing at low relative humidity results in superior immersed strengths compared to curing at high relative humidity. At low relative humidity a cure period greater than 2 days and not more than 7 days is needed to maximize the strength. The data also show that cure at high humidity is somewhat inferior, although acceptable as long as it is followed by several days at a lower relative humidity. The untreated soil under all conditions of cure had zero immersed strength. Hence, the TBC-stabilized soil is superior regardless of cure conditions.

The reason that cure under drying conditions is superior to cure under humid conditions is undoubtedly because the drier a soil is before immersion, the drier it will be kept by the waterproofing agent during immersion. Also, the drier the soil after immersion, the more strength it will have. Furthermore, the maximum strength under any relative humidity condition will be attained at such time as the soil has dried to an equilibrium value.

The previous experiments have indicated that curing under drying conditions results in the treated soil attaining the greatest strength. The next experiment was designed to determine the optimum relative humidity conditions. Samples of three soils were treated with 0.1 percent TBC and compacted at optimum moisture by the California method. Samples of each soil were then cured at four different relative humidities for 7 days, immersed in water for 24 hr, and finally tested for unconfined compressive strength. The relative humidities used for curing are shown in Figure 2 together with the results.

It is quite clear that the optimum relative humidity is between 40 and 50 percent. Curing at relative humidities in excess of 70 percent results in strengths decidedly inferior to the strengths attained under optimum conditions. However, the untreated soils in all cases had an immersed strength of zero. Therefore, regardless of cure conditions, the treated soil is superior to the untreated.

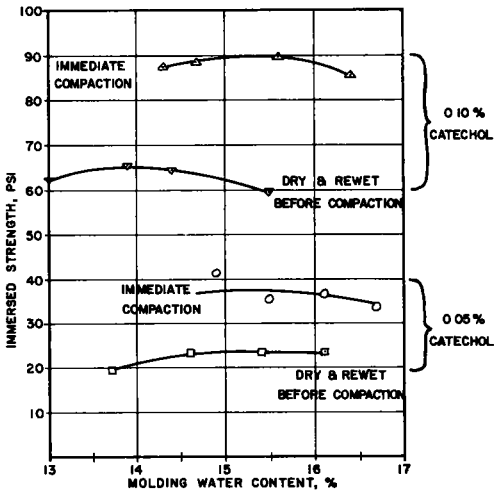

Figure 3. Immersed strength vs molding water content for soil K-1 treated with 0.05 and 0.10 percent TBC, both dried and rewet after treatment and before compaction, and compacted immediately after treatment.

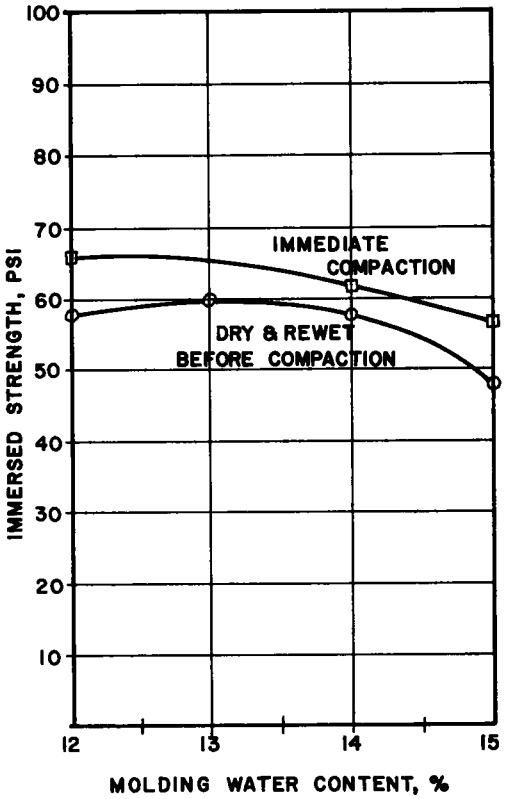

Figure 4. Immersed strength vs molding water content for soil R-1 treated with 0.07 percent TBC, both dried and rewet after treatment and before compaction, and compacted immediately after treatment.

Another factor that can influence the strength attained by a soil stabilized with TBC is the condition in which the soil is maintained subsequent to treatment and the time interval before drying of the compacted soil is initiated. Usually a soil will be compacted and drying will be initiated almost immediately after treatment, but situations in which this would not be so are possible.

An experiment was conducted to determine the effect of allowing the soil to dry before compaction. Samples of soil K-1 were treated with two rates of TBC and allowed to air dry for 72 hr prior to addition of molding water and to compaction by the California method. A second set of samples also were treated with two rates of TBC, but were compacted immediately after treatment. Several rates of molding water were used in both sets of samples and all samples were cured for one week at 50 percent RH prior to immersion and determination of strength. The levels of TBC used, the amounts of molding water, and the immersed strengths are shown in Figure 3. An analogous experiment was conducted using soil R-1 except that only one rate of TBC was used. The results of this experiment are shown in Figure 4.

Figures 3 and 4 reveal that the strength attained by the treated soil is reduced by allowing the soil to dry for 72 hr after treatment and before addition of molding water and compaction. An experiment was conducted to determine whether the drying or the time between treatment and final compaction and/or cure was responsible for this result. Soil K-1 was treated with 0.1 percent TBC at optimum moisture content and stored for 72 hr at a relative humidity of 100 percent. At various times during the 72-hr period, aliquots of the soil were taken, compacted by the California method, and returned to the 100 percent relative humidity condition for the remainder of the 72-hr period. After 72 hr, all the samples were cured for 7 days at a relative humidity of 50 percent. In addition, a sample of soil K-1 treated with TBC was compacted immediately after treatment and cured for 7 days at a relative humidity of 50 percent. All

TABLE 4

IMMERSED STRENGTHS OF SOIL K-1 TREATED WITH 0.10 PERCENT TBC[a]

Time at 100% RH (hr)		Immersed Strength[b] (psi)
Total	Before Compaction	
0	0	90
72	0	62
72	1	65
72	2	65
72	4	79
72	8	62
72	24	69
72	48	66
72	72	72

[a]As a result of compacting soil at various times during storage at 100 percent relative humidity as compared to compacting soil immediately after treatment and with no storage at 100 percent relative humidity.

[b]After 7 days cure at 50 percent relative humidity.

samples were then immersed for 24 hr before determining the unconfined compressive strength. The various times during the 72-hr period at which the samples were compacted are given in Table 4, together with the results.

Table 4 reveals that all samples which were kept for 72 hr at 100 percent relative humidity prior to cure at 50 percent relative humidity have essentially the same immersed strength regardless of when they were compacted. Further, all of these samples have lower strengths than the samples that were compacted immediately after treatment and immediately cured at 50 percent relative humidity. These results, together with the results shown in Figure 3, indicate that allowing 72 hr to elapse between treatment of the soil with TBC and commencement of drying of the compacted soil has an adverse effect on the strength obtained, regardless of whether the soil dries out before compaction and cure, and regardless of whether the soil is in a compacted or friable condition before cure.

The probable explanation for this is that TBC does not reach adsorption equilibrium with soil for a matter of 3 or 4 days, although within 1 hr most of it is adsorbed. Thus, when the soil is in a loose and friable condition for 3 days, whether it is drying or not, the TBC adsorbed onto the soil surfaces is homogeneously distributed. However, when drying takes place immediately after the soil has been compacted, water containing TBC flows through the capillary pores during the time that the soil specimen

TABLE 5

CLAY CONTENT, UNIMMERSED COMPACTED STRENGTH AT OPTIMUM MOISTURE FOR NATURAL SOIL, AND WATER CONTENT AFTER IMMERSION AS PERCENT OF PLASTIC LIMIT AND STRENGTH AFTER IMMERSION AT TREATMENT LEVELS OF TBC OF 0.10 AND 0.25 PERCENT FOR SEVERAL SOILS

Soil	Clay Content (%)	Untreated Compacted Strength at Optimum Moisture[a] (psi)	Water Content After Immersion of Treated Soil (% plastic limit)		Strength After Immersion of Treated Soil (psi)	
			0.25% TBC Treatment	0.10% TBC Treatment	0.25% TBC Treatment	0.10% TBC Treatment
H1	6	10	21	20	33	26
D1	12	27	28	28	88	87
B2	12	65	29	32	100	102
R1	16	50	31	33	83	82
A1	16	59	37	47	110	95
K1	18	97	43	48	157	139
B1	22	72	65	110	53	18
E1	22	143	82	94	57	46
C1	25	90	53	70	85	37
F7	38	118	53	61	65	42
M2	42	84	82	92	79	52
N1	48	112	91	116	31	5
J2	55	144	93	105	6	5
P2	57	149	>100	>100	0	0
L2	67	155	>100	>100	0	0

[a]All soils when immersed had zero strength.

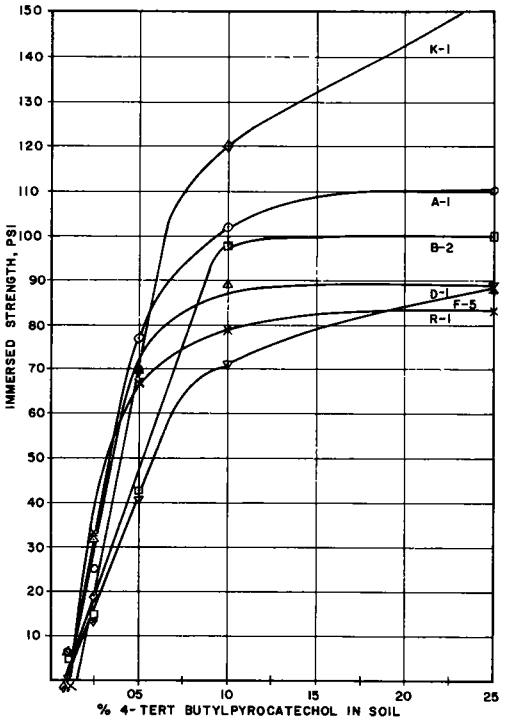

Figure 5. Immersed strength vs TBC content in six soils compacted by California method.

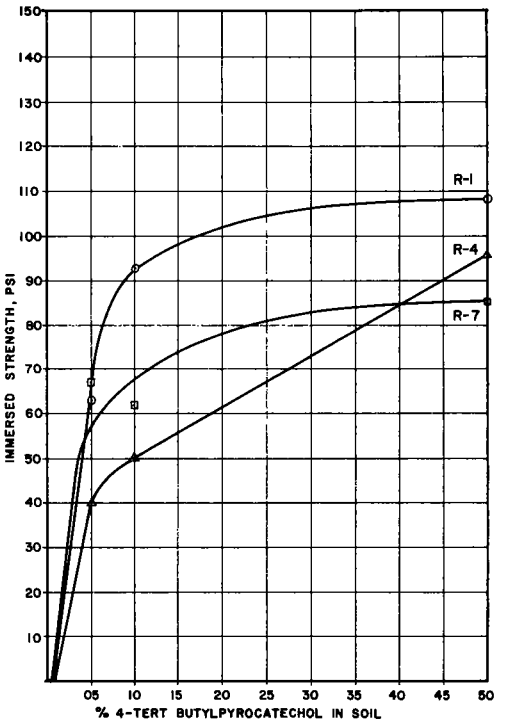

Figure 6. Immersed strength vs TBC content in three soils compacted by Iowa method.

is losing water via volatilization. This results in a somewhat disproportionate share of the TBC being adsorbed onto the soil surfaces adjacent to these capillary pores. Because it is these capillary pores that, when waterproofed, reduce the movement of water back into the soil, immediate compaction and drying results in a TBC distribution that is optimum for maximum exclusion of water, and hence, maximum immersed strength.

Soil Type Amenable to Treatment with TBC

The next series of experiments was devoted to determining what types of soils are most responsive to treatment with TBC. Several soils of widely varying properties were treated with two rates of TBC. They were compacted by the California method, cured for 7 days at 50 percent RH, and immersed for 24 hr before determining unconfined compressive strength and moisture content. In addition, the unimmersed compacted strengths of the untreated soils at optimum moisture were determined. The various soils used and the rates of TBC used are shown in Table 5, together with the results.

There are several soil factors that can influence the response of a soil to treatment with TBC. Clay content, however, appears to be the predominating factor. The results in Table 5, therefore, are listed in order of increasing clay content of the soils. They show that the unimmersed compacted strengths of the untreated soils at optimum moisture generally increase as the clay content increases. However, the data for the water contents of the treated soils expressed as a percent of the plastic limit reveal that as clay content increases, the effectiveness of TBC at keeping water out of the soil generally decreases. Consequently, the soils with the most natural strength are least amenable to having that strength protected from the deleterious effects of water. These two opposing effects of the clay content result in the net effect on the immersed strengths of the various soils given in Table 5. As clay content increases to about 20 percent, the

TABLE 6

LINEAR EXPANSION, MOISTURE CONTENT, AND UNCONFINED COMPRESSIVE STRENGTH OF SOIL R-1 TREATED WITH SEVERAL RATES OF TBC AFTER SUBJECTION TO 10 CYCLES OF FREEZING AND THAWING COMPARED TO SOIL NOT SUBJECTED TO FREEZING AND THAWING

Treatment Level (%)	Linear Expansion (%)		Moisture Content Increase (%)		Unconfined Compress Str. (psi)	
	Capillary Absorption	Freeze-Thaw	Capillary Absorption	Freeze-Thaw	Capillary Absorption	Freeze-Thaw
0.0	10.1	38 0	24.3	Mud	0	0
0.042	1.6	3.4	6.8	13.5	62	21
0 064	1.2	2.8	4.1	10 5	92	31
0.085	1.1	1.5	3 8	4.0	90	73

immersed strengths generally increase. As clay content increases beyond 20 percent, the immersed strengths generally decrease. Soils with clay contents in the range of 10 percent to 40 percent generally have reasonably high immersed strengths when treated with TBC. Thus, the soils that can be most effectively stabilized with TBC are limited to those with a moderate but not excessive percentage of clay.

Rates of TBC Necessary for Stabilization

The next series of experiments was designed to determine the rates of TBC necessary to obtain suitably stabilized soils. Several soils were treated with several rates of TBC, compacted by either the Iowa or California method, cured for 7 days at a 50 percent RH, and immersed for 24 hr before determining unconfined compressive strength. The soils, the rates of TBC, and the result obtained are shown in Figures 5 and 6.

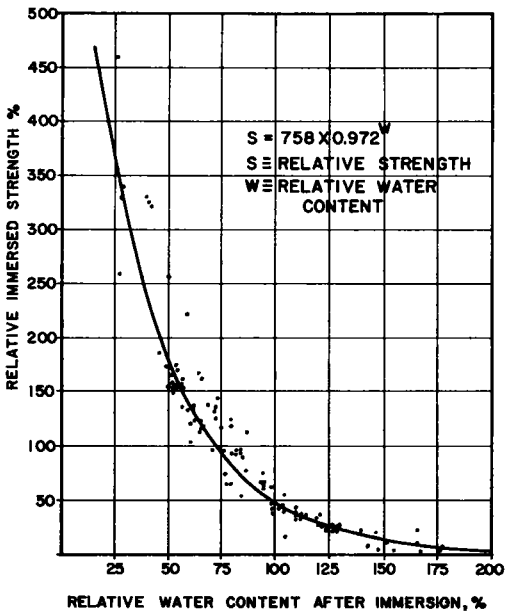

Figure 7. Plot of relative water content after immersion vs relative immersed strength for several soils treated with several rates of TBC under several varying treatment and cure conditions.

Figures 5 and 6 show that for most soils about 0.1 percent TBC is essentially optimum. Rates exceeding this do not generally result in appreciable additional immersed strength. Many soils even retain a substantial portion of the maximum immersed strength at the 0.05 percent treatment level. All soils, however, lose strength quite rapidly as the treatment level is reduced below 0.05 percent, although in some instances the strengths at these very low levels are quite substantial. Thus, treatment levels of 0.05 to 0.10 percent appear to be suitable for obtaining maximum or near maximum immersed strengths. Lower levels may be acceptable in some soils if the strength imparted to the soil meets engineering specifications. If the strength obtained at the 0.10 percent level does not meet specifications, it is doubtful that higher levels should be considered.

Effect of Freezing and Thawing on TBC Stabilized Soil

For a chemically stabilized soil to be of practical use it must be resistant to the deleterious effects of freezing and thawing. An experiment was conducted, therefore, to determine the effect of alternate freezing and thawing on TBC-stabilized soil.

Samples of soil R-1 were treated with several rates of TBC compacted by the Iowa method, and cured for 3 days at 100 percent RH and 4 days at 30 percent RH. One-half the samples from each treatment were then subjected to 10 cycles of freezing and thawing and the other half were placed on wet felt pads kept continually wet from a source of free water. The latter half of the samples were kept on the felt pads for a period of 10 days and allowed to adsorb an equilibrium amount of capillary water. All samples were then measured for linear expansion, moisture content increase from that at compaction, and unconfined compressive strength. The rates of TBC used in this experiment are given in Table 6 together with the results.

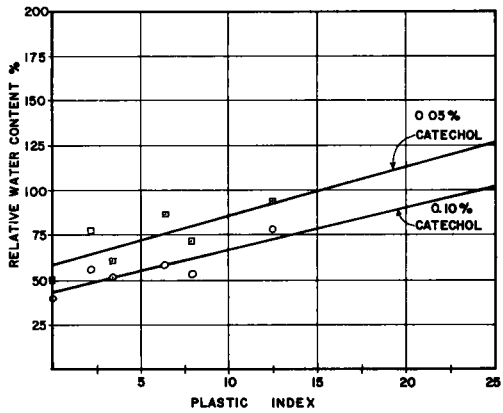

Figure 8. Relative water contents of several soils treated with 0.10 and 0.05 percent TBC vs plastic index of untreated soils.

It is evident that TBC greatly decreases the linear expansion and moisture absorption by the soil as a result of both freezing and thawing and capillary absorption. At the 0.042 and 0.064 percent rates of TBC there is some expansion and moisture absorption due to freezing and thawing over that resulting from simple capillary absorption. However, at the 0.085 percent level there is essentially no expansion or moisture absorption due to freeze-thaw over that caused by capillary absorption. The effect of the increased moisture content and consequent expansion caused by the freezing and thawing cycles on the unconfined compressive strength is shown in the last two columns of Table 6. The more the absorption of water due to freeze-thaw is decreased, the closer the strength of the soil subjected to freeze-thaw approaches that of the soil not subjected to freeze-thaw. Thus, at the 0.085 percent level of TBC the freeze-thaw cycles result in only a 19 percent loss in strength. Hence, the treated soil can be considered to be quite resistant to the deleterious effects of freeze-thaw.

Basic Function of TBC in Soil

The basic function of TBC in soil is strictly that of reducing the absorption of water by the soil. This becomes evident by plotting the relative immersed strength of any soil treated with any rate of TBC under any treatment conditions against the relative moisture content after immersion.

The relative moisture content can be expressed in percent as the ratio of the absolute water content after immersion to the optimum moisture content. The reason for using relative moisture content rather than absolute moisture content is that the relative moisture content is based on the free energy of the water in the soil. Soil physicists consider that the most valid statement of the moisture status of a soil is one based on the free energy, rather than the percentage of water in the soil. The use of the optimum moisture content as the base for the relative moisture content is somewhat arbitrary but nevertheless valid because all soils at optimum moisture contain water with approximately equal free energy.

The relative immersed strength can be expressed in percent as the ratio of the absolute immersed strength to the strength of the untreated compacted soil at optimum moisture. The reason for using the relative immersed strength is that this value removes the variation in natural strength between soils. The use of the strength of the untreated compacted soil at optimum moisture as a base is somewhat arbitrary. However, the natural strength of various soils should be reflected by their strength at any comparable moisture content on a free energy basis such as optimum moisture. Therefore, the strength of the untreated compacted soil at optimum moisture is a valid and convenient base on which to express relative immersed strength.

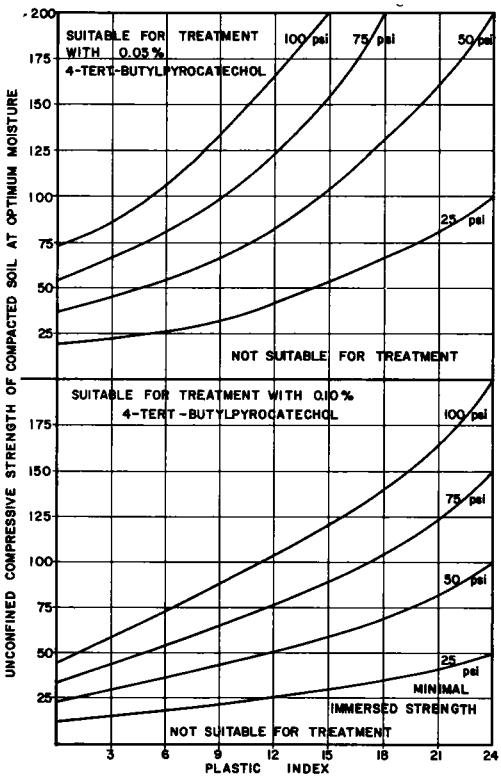

Figure 9. Nomograms of plastic index vs unconfined compressive strength at optimum moisture for untreated soil that will determine if soil will have above minimal immersed strength for treatment with 0.05 and 0.10 percent TBC.

Most of the data already presented were recalculated in terms of relative immersed strength. In addition, the corresponding relative moisture contents were calculated from unreported data. These results are plotted against each other in Figure 7, and the curve and its equation fitting these points is shown.

The points plotted in Figure 7 all fall quite close to the curve indicating the relatively exact nature of the relationship between relative immersed strength and relative water content after immersion. Thus it would seem that the relative strength is merely a function of the relative amount of water in the soil after immersion. Consequently, the relative strength imparted to a soil by TBC under any set of conditions and using any rate of TBC is simply a function of the ability of the TBC under the particular set of conditions and rate of use to keep water out.

REMARKS

The results presented thus far suggest several conclusions regarding the use of TBC as a soil waterproofing agent. As with all other chemical soil stabilizers, including cement, TBC will not do an acceptable job of stabilizing in all situations. The practicing engineer is, of course, interested in simple general criteria that will indicate the practicality of using the TBC in any given situation.

Such criteria can be presented, but will be contingent on certain assumptions regarding the ultimate design criteria for waterproofed soils in bases and subbases. Criteria for aggregate and cement or lime stabilized soils in bases and subbases have been established, but these criteria do not necessarily apply to waterproofed soil. Hopefully the criteria needed can be determined in the laboratory, but undoubtedly field experience with test roads also will be necessary.

Nevertheless, assuming that immersed unconfined compressive strength will be important in the ultimate design criteria for waterproofed soils, nomographs relating relatively simple soil properties can be prepared from which the suitability of any particular soil for treatment with TBC can be assessed. These nomographs can be prepared based on the results shown in Figure 7, which is a plot of relative immersed strength of soils vs relative water content after immersion.

First, it is necessary to determine if there is a relationship between the relative water content after immersion and any soil property. Figure 8 is a plot of relative water content of various soils treated with two levels of TBC vs the plastic indexes of the various soils. It is evident from Figure 8 that a reasonably good linear relationship exists. Thus, based on plastic index, it is possible to predict from Figure 8 the relative water content of any soil treated with either 0.05 percent or 0.10 percent TBC under optimum conditions. Then, referring to Figure 7, it is possible to determine the relative immersed strength of any soil based on the relative water content after immersion. Absolute immersed strength can then be determined on the basis of a knowledge of the maximum compacted strength of the particular soil. Thus, on the basis of maximum compacted

strength and plastic index for any untreated soil, the immersed unconfined compressive strength can be predicted for treatment levels of either 0.05 percent or 0.10 percent TBC.

In the absence of any accepted design criteria regarding the immersed unconfined compressive strength of a waterproofed soil, four strengths were arbitrarily chosen as minimal and the nomographs shown in Figure 9 were constructed from the data in Figures 7 and 8 by essentially reversing the previously described process for predicting immersed strength. It is apparent from Figure 9 that as the plastic index of soils increases, the compacted strength of the soils must increase in order to maintain any given constant immersed strength. The practical use of these nomograms, however, is that for any soil with a known plastic index and maximum compacted strength, it is possible to determine if the soil falls above or below the line delineating the minimum acceptable immersed strength. If the soil falls on or above the minimal line, it probably can be effectively stabilized with TBC. Conversely, if the soil falls below the minimal, it cannot be stabilized with TBC. As pointed out previously, the appropriate minimal line to use as yet has not been determined.

CONCLUSIONS

TBC is a potentially useful chemical for rendering some soils acceptable substitutes for aggregate in highway bases and subbases. The optimum method for treatment with TBC is to add it in sufficient molding water to bring the soil up to optimum moisture for maximum dry density. The soil should then be compacted immediately and allowed to cure for about a week, preferably at a relative humidity of 30 to 50 percent. These conditions are optimum although they are not necessary prerequisites for successful treatment.

Generally, the soils that are most responsive to stabilization with TBC are silty in nature and contain between 10 percent and 40 percent clay. The rate of TBC necessary to stabilize these soils is in the range of 0.05 percent to 0.10 percent.

Extensive results regarding freezing and thawing were not presented, but treatments with TBC in the indicated range appear to maintain a substantial portion of their strength through 10 cycles of freezing and thawing. Because the moisture content of the soil is maintained at a relatively low level by TBC, it is to be anticipated that soils that can be effectively treated will be resistant to freeze-thaw.

The function of TBC in soil is merely to keep water out. The material does not appear to bind soil particles together. However, by keeping water out it permits the soil, regardless of moisture conditions, to retain a substantial portion of its dry strength.

In the absence of accepted design criteria for waterproofed soils in bases and subbases, it is difficult to determine how broad a range of soils will be amenable to treatment with TBC. However, it is possible by assuming various minimal strength criteria to determine whether a soil is suitable on the basis of its plastic index and optimum compacted strength.

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Recent Investigations on Use of a Fatty Quaternary Ammonium Chloride as a Soil Stabilizing Agent

WAYNE A. DUNLAP, BOB M. GALLAWAY, EDWARD C. GRUBBS, and JOE E. HOUSE, respectively, Assistant Research Engineer, Research Engineer, and former Research Assistant, Texas Transportation Institute, College Station, Texas, and Manager, Market Development, Chemical Division, General Mills, Inc.

This paper reports the results of an investigation conducted to determine the soil-stabilizing potential of a fatty quaternary ammonium chloride. The study was divided into three general areas of interest. The initial portion of the paper deals with the laboratory investigation. Test procedures and results are given for the standard tests recommended by the Bureau of Public Roads for soil stabilization investigations, as well as procedures and results for non-standard tests used to determine the ability of the stabilizing agent to retard capillary absorption and frost heave. The results show that this material is an effective stabilizer for soils ranging from silts to highly plastic clays; however, it is most effective on materials containing moderate amounts of silt.

The second portion of the paper covers the actual field testing of the stabilizing agent. Two test sections totaling nearly 0.4 mi in length were constructed on an existing roadway. Results are presented for periodic strength and deflection measurements on one of the test sections. In addition, construction procedures for effectively applying this chemical are recommended.

Finally, based on the results of X-ray diffraction and other tests, a brief explanation of the possible stabilizing mechanism is presented.

• THE RAPIDLY-DWINDLING supply of suitable road-building aggregates in many areas of the United States has prompted numerous engineering investigations to determine means of improving or up-grading the more plentiful supply of poorer materials. Often, this improvement can be accomplished by the addition of chemicals to the soil. Engineering publications abound with results of research on the use of chemical soil stabilizers. From this multitude of research papers, it is easily observed that a practical, universal soil stabilizer has yet to be developed. Instead, certain chemicals are effective on one group of soils, but are ineffective on others. However, as opposed to the embryonic days of soil stabilization, the present-day engineer can usually select a material to suit his needs from the ever-increasing "library" of soil stabilizers. It is the purpose of this report to present the results of research on another chemical that should be added to the stabilizer "library."

The prime requirements for any stabilizer used in roadway construction are that it should retain or increase the shear resistance of the soil even in the presence of saturating amounts of water, and it should increase the volumetric stability of the soil. Chemical stabilizers may perform these functions by cementation and/or waterproofing action. The cementing-type stabilizers actually bond the soil grains together and improve the strength of the soil mass under certain conditions. Examples are portland cement and lime, although it is recognized that lime also stabilizes by virtue of cation exchange. Waterproofing stabilizers can render the soil grains hydrophobic thus improving their properties in the presence of water although they do not necessarily cement or bond the soil grains together.

The initial purpose of this investigation was determination of the stabilizing effect of a fatty quaternary ammonium chloride on several highly plastic clays located in the southwestern United States. These clays are well-known for their poor road-building characteristics, particularly in regard to volumetric changes. Later in the program, the scope was enlarged to include investigations on other soil types, potential of chemical to reduce frost heave, and construction of test roads to examine field application characteristics and techniques.

CHARACTERISTICS OF CHEMICAL STABILIZER

The chemical investigated in this research—known commercially as Aliquat H226 (trademark of General Mills, Inc.)—is a dihydrogenated tallow dimethyl ammonium chloride. Although Aliquat H226 is initially a wetting agent, once adsorbed on the surface of a material the surface becomes hydrophobic. For soil stabilization purposes it can be accurately defined as a waterproofing agent.

Aliquat H226 is normally supplied as 75 percent active material in isopropanol. In this state it can easily be dispersed in water heated to a temperature of 120 to 140 F. When dispersed, it forms a stable milke suspension of variable viscosity depending on the water content. Aliquat H226 is non-toxic and during approximately 3 years of laboratory and field research, no harmful effects were reported even in cases of direct contact with the skin.

PREVIOUS INVESTIGATIONS WITH LARGE ORGANIC CATIONS

In recent years, engineers and agronomists have reported numerous investigations concerning the adsorption of cationic organic complexes on clays. It has been shown that organic cations are at least partially exchanged for the inorganic cations and are subsequently held tightly to the clays. Clays saturated with organic cations exhibit hydrophobic or water-heating characteristics.

When introduced into clay-water suspensions, organic cations may be adsorbed between the clay layers or sheets, around the particle edges, or they may remain in suspension. Clays of the montmorillonite group, having the major portion of their exchange capacity between the layers or platelets, adsorb the majority of the organic cations on these positions with a resulting increase of the basal spacing. Jordan, investigating the effect of primary amines on montmorillonites, found that the basal spacing increased as the number of carbon atoms in the amine chain increased (1). Jordon's results indicate basal spacings of a maximum order of two molecular thicknesses of the cation. Other investigators have reported spacings equal to three molecular thicknesses of the adsorbed cation (2). Because large cations may occupy much of the surface area between the clay platelets, they leave little for the adsorption of water. Various investigations have shown that the water-adsorbing capacities of montmorillonite are reduced as the basal surfaces are coated with organic ions. As the size of the organic ion increases, the water-adsorbing capacity is generally reduced (3, 4, 5).

Little research has been performed on the amount of organic cations that might remain in suspension and affect the properties of the pore water. Brindley and Rustom have conducted studies on the adsorption of a non-ionic organic complex on montmorillonite in aqueous suspension (6). When the amount of organic material was less than 50 percent of the maximum organic adsorption, about 70 percent of the added organic material was adsorbed on the clay and 30 percent remained in suspension.

The influence of organic cations on clay minerals other than montmorillonite has not been investigated as thoroughly as organic-montmorillonite complexes. Experiments by Grim, Allaway, and Cuthbert have shown that water adsorption of kaolinite clays is reduced after treatment with organic ions (3). The decrease in water adsorption is relatively less for kaolinites than for montmorillonites.

Grossi and Woolsey studied the effect of a fatty quaternary ammonium salt on a Putnam silt loam (7). Small amounts of this organic chemical greatly increased the water-stability of the soil aggregates and aided in the aggregation of the soil. Treatment with this chemical also decreased the capillary moisture absorption and increased the immersed compressive strength. Although the properties of the soil were not given,

TABLE 1

SOIL PROPERTIES

Property	Sample[a] RF- 176-1	176-2	176-4	176-5	176-6	176-9	176-10	176-28	176-34	176-36	251-74	251-102	251-109
Liquid limit (%)	74.2	52.1	60.0	70.7	64.2	--	61.4	39.4	67.7	25.4	38.0	21.7	58.1
Plastic limit (%)	25 9	18.6	26.5	21.5	29.2	--	28.7	15.6	27.3	19.0	24.5	19.3	28.9
Plastic index (%)	48.3	33 5	33 5	49.2	35.0	--	32.7	23.8	40.4	6.4	13 5	2.4	29.2
Std AASHO compact test													
Max. dry density (pcf)	94.3	103.8	94.0	99.8	89.6	91.2	85.3	110.1	94.5	112.0	106.1	103.6	100.0
Opt. moist cont. (%)	24.9	18.2	24.3	22.1	27.9	23.6	29.4	15.9	24.0	14.8	18.8	18.0	22.4
Sand size (%)	10.0	8.0	1.0	0	--	--	12.0	15.0	--	--	6.0	8.0	--
Silt size (%)	33.0	44.0	42.0	69.0	--	--	48.0	60.0	--	--	72.0	81.0	--
Clay (%):													
Finer than 5μ	57.0	48.0	57.0	31.0	--	--	40.0	25.0	--	--	22.0	11.0	--
Finer than 2μ	53.0	28.0	44.0	27.0	--	--	28.0	22.0	--	--	16.0	9.0	--
Specific gravity	2.66	2.74	2.69	2.66	2.71	2.65	2.64	2.67	--	2.68	--	--	--
pH	4.6	7.96	7.74	7.18	6.03	7.70	7.96	5.54	--	5.2	--	--	--
Cation exch. cap. (meq/100 g)	32.5	28.0	32 3	28.6	--	--	51.7	11.4	--	--	--	--	31.4
Exchangeable cations (meq/100 g):													
Na	1.3	1.1	0.9	4.5	--	--	0.2	0.4	--	--	--	--	0.4
K	0.7	0.8	1.0	0.9	--	--	1.0	0.2	--	--	--	--	0.9
Ca	6.5	--[b]	--[b]	19.2[c]	--	--	--[b]	4.3	--	--	--	--	19.4
Mg	7 8	2.0	5.3	6.1	--	--	1.8	3.4	--	--	--	--	10.7

[a]Where value not given, test was not performed.
[b]Free $CaCO_3$ present, hence exchangeable calcium not determined.
[c]Free $CaCO_3$ possibly present.

the classification denotes only a small amount of clay fraction that was probably montmorillonite.

Davidson and Hoover experimented with a number of organic cationic chemicals as stabilizers for Iowa loess (8). In general, the organic cations increased the immersed compressive strength and decreased the water absorption and swelling of the loess. The only undesirable effect noted was a decrease in the air-day compressive strength of the soil. The clay fractions of the soils used in this study were predominately montmorillonite. The apparent success of Hoover and Davidson with one of the chemicals—Arquad 2HT (trademark of Armour & Co., Inc.)—on the loess soils, prompted the research described in this paper.

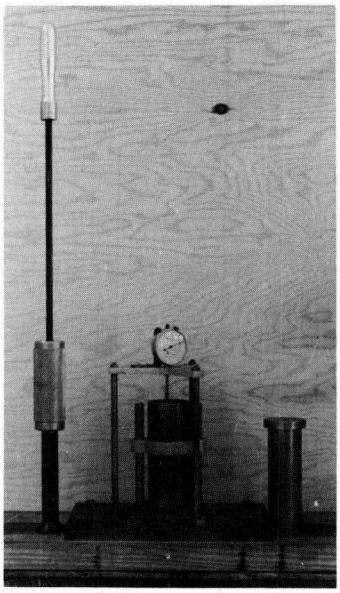


Figure 1. Vicksburg miniature compaction apparatus.

SOIL CHARACTERISTICS AND PREPARATION

In the initial phase of the research soils were selected on the basis of their reportedly poor engineering characteristics. Obtained primarily from Texas and Oklahoma, these soils are present in such large quantities that they must be utilized as highway building materials regardless of their poor properties. In later phases of the research, a few additional soils containing high percentages of silt were selected.

Properties of the soils used in this investigation are consolidated and given in Table 1. For ease of presentation, subsequent reference to soils is made by the laboratory identification number.

The soils were prepared for use by drying in a 140 F oven for a period of 12 to 24 hr depending on their original moisture content. The oven-dried materials were then ground in a muller until they passed a No. 18 sieve. All soils were fine grained and this type of grinding did not reduce the natural size of individual particles. After processing, the soils were stored in covered containers before use.

CHEMICAL PREPARATION

It was necessary to dilute the 75 percent active Aliquat H226 with water to reduce the viscosity so the solution could be easily mixed with the soils. The solutions were prepared in the following manner:

1. A weighed quantity (usually between 150 and 300 g) of 75 percent active Aliquat H226 was placed in a mixing bowl.
2. The amount of water required to produce the desired solution was heated to 140 to 150 F and poured into the mixing bowl.
3. The solution was vigorously stirred in a mechanical mixer until the odor of the solution indicated that most of the isopropanol had evaporated. The solution was then stored in a closed glass container.

A solution of 6.5 percent active Aliquat H226 by weight was found to be very convenient for the laboratory research.

LABORATORY TEST PROCEDURES

A variety of testing procedures have been used or recommended for the evaluation of soil-stabilizing agents. In an attempt to use an evaluation procedure that could be regarded as more or less standard, the research personnel turned to the Physical Tests

TABLE 2

MOISTURE CONTENT AND DENSITY RELATIONS FOR ALIQUAT H226-TREATED SOILS

Sample No.	Dry Soil Wt. (%)	Opt. Moist. Content (%)	Max. Dry Density (pcf)
176-1	0	24.9	94.3
	0.125	25.1	91.1
	0.250	26.5	91.0
	0.5	26.6	91.6
	1.0	23.3	92.6
	2.0	--	--
	3.0	--	--
176-2	0	18.2	103.8
	0.125	18.7	103.6
	0.250	21.0	101.0
	0.5	19.9	101.0
	1.0	20.3	101.0
	2.0	--	--
	3.0	--	--
176-4	0	24.3	94.0
	0.125	24.5	95.8
	0.250	24.5	94.9
	0.5	24.5	93.8
	1.0	24.5	93.7
	2.0	--	--
	3.0	--	--
176-5	0	22.1	99.8
	0.125	21.9	98.8
	0.250	22.7	98.5
	0.5	22.7	97.2
	1.0	22.5	97.7
	2.0	--	--
	3 0	--	--
176-6	0	27.9	89.6
	0.125	28.0	89.5
	0.250	27.3	90.4
	0.5	27.7	89.9
	1.0	28.4	88.8
	2.0	--	--
	3.0	29.0	85.8
176-9	0	23.6	91.2
	0.125	--	--
	0.250	--	--
	0.5	24.0	90.1
	1.0	25.5	90.8
	2.0	--	--
	3 0	--	--
176-10	0	29.4	85.3
	0.125	27.6	87.1
	0.250	28.2	86.8
	0.5	27.2	86.4
	1.0	27.8	85.4
	2.0	--	--
	3.0	--	--
176-28	0	15.9	110.1
	0.125	15.0	110.2
	0.250	16.6	107.4
	0.5	16.9	106.8
	1.0	16.4	105.4
	2.0	--	--
	3 0	--	--

Division, Bureau of Public Roads. For the most part the following procedures duplicate or closely parallel those procedures recommended by the Bureau of Public Roads for evaluation of chemical soil stabilizers.

Compaction Test

To determine the effect of Aliquat H226 on the compaction characteristics of soils, the standard AASHO compaction test (AASHO designation T99-57) was performed on soils in both the chemically treated and untreated states. Chemical concentrations of 0.125, 0.25, 0.5, and 1.0 percent of the dry soil weight were generally used if sufficient soil were available. The soil, water, and/or chemical solution were thoroughly mixed in a Lancaster PC mixer and then compacted in a standard AASHO mold. A portion of the mixture was also compacted in the Vicksburg miniature compaction apparatus shown in Figure 1. In the Vicksburg compaction procedure, a quantity of soil sufficient to produce a specimen 4 in. high and 2 in. in diameter was compacted in the mold in 4 equal layers. The number of blows per layer was selected to obtain a compaction curve similar to the standard AASHO curve. During the mixing and compaction process, it was observed that the addition of Aliquat H226 made the soils friable and easier to mix. It was also necessary to use a fresh specimen for each point on the compaction curve because soils treated with Aliquat H226 were effectively waterproofed to the extent that they did not readily accept additional water especially if some evaporation of the initial moisture had occurred.

Results of the standard AASHO compaction tests, Table 2, show that Aliquat H226 produces only minor changes in the maximum dry unit weights and optimum moisture contents. The general trend is a very slight reduction in unit weights with a slight but erratic variation in optimum moisture contents. An exception is RF 176-28 in which kaolinite is the predominant clay mineral. With high percentages of Aliquat H226 this soil became extremely spongy resulting in a fairly significant decrease in dry unit weight.

The Bureau of Public Roads has no criteria concerning the effect of stabilizers on the compaction characteristics of soils. However, with the possible exception of cementing-type stabilizers, significant

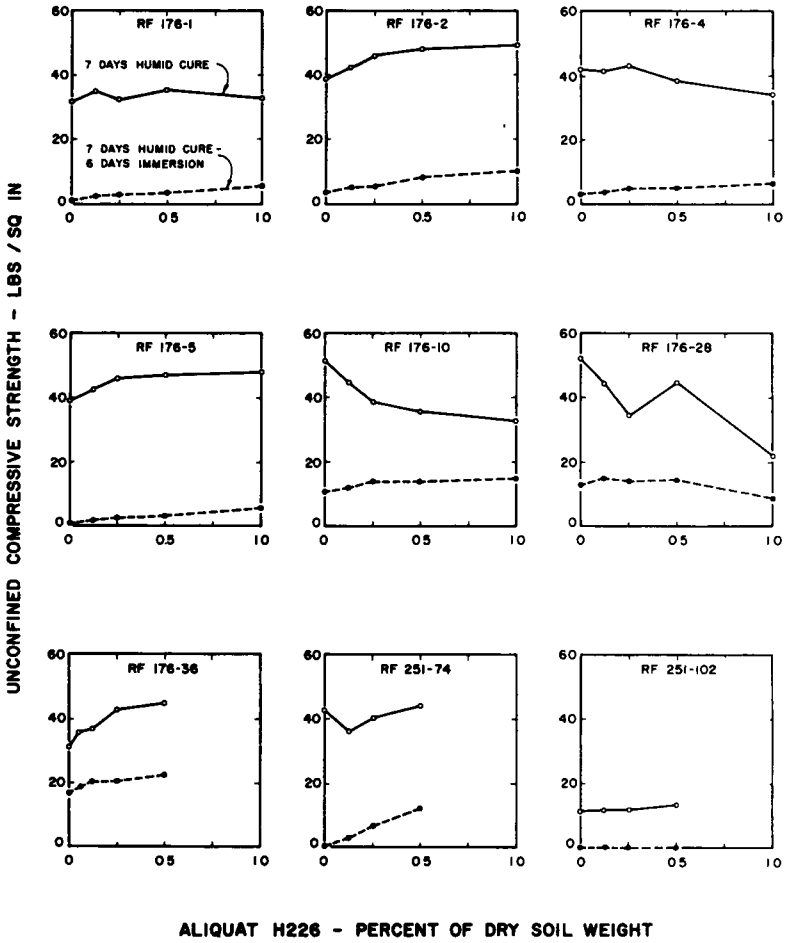


Figure 2. Unconfined compressive strength for 7 days of humid curing and 6 days of immersion.

decreases in the compacted unit weights are generally undesirable due to resulting decreases in soil strength.

Unconfined Compressive Strength

Specimens for the unconfined compression test were molded in the Vicksburg miniature compaction apparatus according to Bureau of Public Roads recommendations. Sufficient soil to make from 7 to 9 specimens was mixed with the amount of water or chemical solution needed to obtain the previously determined optimum moisture content. Specimens were then compacted to obtain the standard AASHO maximum unit weight with an allowable deviation of 0.25 pcf.

After being compacted and weighed, the specimens were placed in a humid room at 95 ± 5 percent relative humidity and 75 ± 5 F for 7 days. At the end of the 7-day humid-curing period each set of specimens was divided into two groups. One group was tested immediately to determine the unconfined compressive strength and the remaining group of specimens was completely immersed in water for 6 days and then tested. Compression tests were performed on a universal testing machine operated at a deformation rate of 0.05 in. per min. Results of these strength tests are shown in Figure 2.

The Bureau of Public Roads recommended that stabilized specimens meet the following specifications:

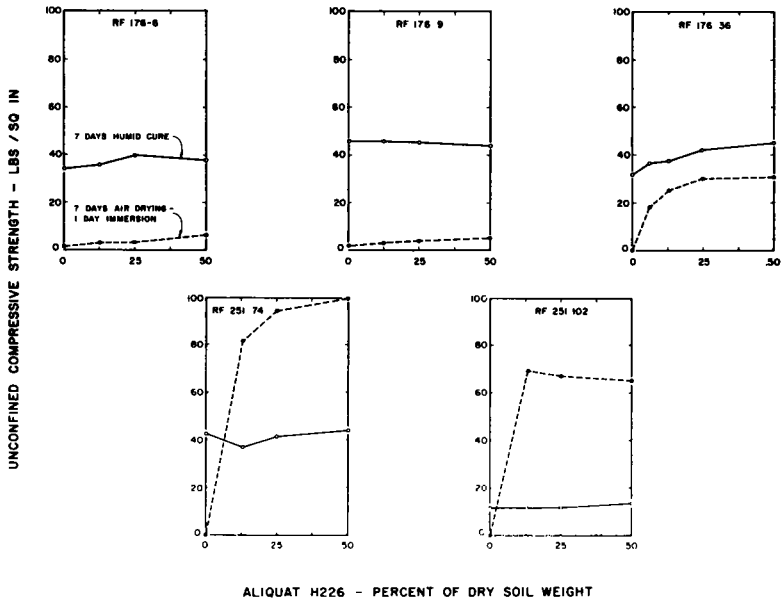


Figure 3. Unconfined compressive strength for 7 days of air drying and 1 day at immersion.

1. Attaining of a minimum compressive strength of at least 100 psi after 7 days of humid curing.
2. Loss of not more than 90 percent of their 7-day humid-cured strength after 6 days of immersion.

As seen from Figure 2, the effect of Aliquat H226 treatment on the 7-day humid cured specimens was variable; an increase in the chemical caused a strength increase in some of the soils and a decrease in others. RF 251-102 was virtually unaffected. In instances where the strength patterns appeared erratic, check tests disclosed no appreciable changes from the original data. Aliquat H226 treatment did increase the immersed strengths of 7 of the 9 soils reported. An increase in the chemical content generally increased the immersed soil strengths, but when used in economic quantities Aliquat H226 did not satisfy strength requirements of the Bureau of Public Roads.

With respect to other stabilization studies, the strengths shown in Figure 2 appeared comparatively low, and consideration was given to the curing conditions which were being used for this research. It is apparent that the condition of 7 days of humid curing before immersion is beneficial to cementing stabilizers such as lime and cement but this is not necessarily true for waterproofers. In their research on stabilization with large organic cations, Davidson and Hoover (8) obtained significant strength increases using a curing procedure of 7 days of air drying at approximately 35 percent relative humidity followed by 24 hr of immersion before the specimens were tested in compression.

The favorable results achieved by Davidson and Hoover prompted additional strength studies on 5 soils using the previously mentioned curing procedure. Results are shown in Figure 3. After the 7-day air drying period, untreated soils usually disintegrated or spalled badly within a few minutes after immersion. For the two clay soils RF 176-6 and RF 176-9, this curing procedure was less beneficial than 7 days of humid curing. The silty soils, RF 176-36, RF 251-74, and RF 251-102, were more favorably affected. RF 251-74 attained a strength of 100 psi with 0.5 percent Aliquat H226 and only slightly less strength with 0.25 percent. Figure 4 shows specimens of this soil after 24 hr of immersion.

Based on compressive strength alone, it appears that the original objective of stabilizing highly plastic clay soils with Aliquat H226 was only moderately successful.

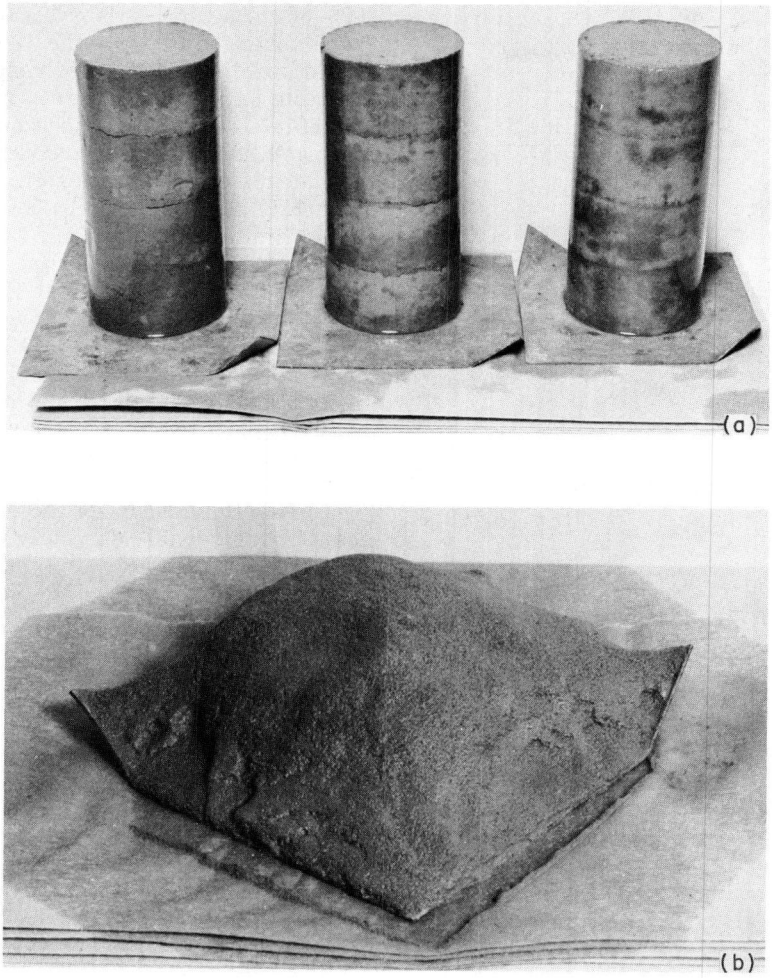


Figure 4. (a) Stabilized specimens of RF 251-74 after 7 days of air drying and 1 day at immersion. Aliquat H226 contents, left to right 0.125, 0.25, and 0.5 percent. (b) Untreated specimen after immersion.

On the other hand, the less plastic silty soils can be very effectively stabilized provided proper curing conditions are achieved. Preliminary studies have indicated that the optimum amount of drying varies with the properties of the soil.

Atterberg Limits

The effect of Aliquat H226 on the plasticity characteristics of the soils was evaluated with the Atterberg limits. Although the Bureau of Public Roads does not stipulate numerical criteria for this test, it is generally recognized that a decrease in the plasticity index is desirable.

Using the procedure recommended by the Bureau of Public Roads, samples were prepared for the tests by mixing approximately 600 g of soil with the required amount of Aliquat H226 solution and/or water needed to obtain the optimum moisture content determined from the compaction tests. After thorough mixing, each sample was divided into two equal parts and placed in separate air-tight jars. One portion was humid cured in the jar for 24 hr and the remaining portion cured for 8 days. At the end of their respective curing periods, the samples were removed from the jars, placed in open containers to air dry, and then crushed with a mortar and pestle until they passed a No. 40 sieve. The tests were then performed according to AASHO designation T89-60 and

TABLE 3

ATTERBERG LIMITS OF ALIQUAT H226-TREATED SOILS

Sample No.	Dry Soil Wt. (%)	Liquid Limit	Plastic Limit	Plasticity Index
176-1	0	74.2	25.9	48.3
	0.125	65.5	26.0	39.5
	0.250	62 0	30.0	32.0
	0.5	58.8	30.3	28.5
	1.0	55 0	31.0	24.0
176-2	0	52.1	18.6	33.5
	0.125	53.5	18.7	34.8
	0.250	49 5	19.5	30.0
	0.5	47.8	20.4	27.4
	1 0	44 2	20 4	23.8
176-4	0	60.0	26 5	33.5
	0.125	62.3	28.8	35.5
	0.250	59.0	27.4	31.6
	0.5	55.8	27.4	28.4
	1.0	50.3	30.9	19.4
176-5	0	70.7	21.5	49.2
	0 125	70.0	22.0	48.0
	0 250	68.4	23.2	45.2
	0.5	61.8	25.3	36.5
	1.0	57.0	28.4	28.6
176-10	0	61 4	28 7	32.7
	0 125	60.4	29.1	31.3
	0.250	60.0	31.0	29.0
	0.5	57.4	32.9	24.5
	1.0	54 0	36.1	17.9
176-28	0	39.4	15.6	23.8
	0.125	36.0	16.5	19.5
	0.250	35.5	19.0	16.5
	0.5	35.2	20.6	14.6
	1.0	33 8	20.5	13.3

T90-56. Because no significant differences were observed between the 1-day and 8-day cured specimens, the test results, given in Table 3, include only the values for 1-day humid curing. These results show that Aliquat H226 effectively reduced the plasticity indexes of all soils tested. This was generally accomplished by lowering the liquid limits and increasing the plastic limits. The most significant reductions occurred in those soils with the highest plasticity indexes. In many cases, small amounts of Aliquat H226 (0.125 to 0.25 percent) produced the greatest changes and additional quantities had slight effect. In general, it appears that Aliquat H226 in relatively small quantities will significantly reduce soil plasticity.

During the conduct of the Atterberg limit tests, considerable difficulty was experienced in mixing water into the Aliquat H226-treated specimens after the air-drying period. The water usually "balled up" on the soil and often 30 to 45 min of vigorous mixing was required to obtain a homogeneous mixture. This is further evidence of the excellent waterproofing characteristics of Aliquat H226. However, the fact that water could be eventually mixed into the soil seems to indicate that sufficient manipulation in the presence of water can at least partially overcome the hydrophobic characteristics imparted by the chemical.

Capillary Absorption Tests

Because of its excellent waterproofing characteristics it was thought that Aliquat H226 might also be effective in reducing moisture absorbed by capillarity. To test this theory, a rather simple laboratory procedure was used. Compacted specimens were placed on porous stones and then positioned in water so the water level was approximately ¼ in. below the base of the specimens. The specimens were covered with small jars. to prevent moisture loss to the atmosphere and then weighed at predetermined time intervals to determine the amount of moisture absorbed.

The first soil tested in this manner was RF 251-109, a clay from O'Hare International Airport, Chacigo, Ill. Because it had already been determined that drying affected the strength characteristics of treated soils, the program also included tests to determine the effect of drying on capillary absorption. This was accomplished by molding several specimens each of untreated, 0.125, and 0.5 percent Aliquat H226-treated soils at their optimum moisture contents and standard AASHO unit weights. Some of the molds were allowed to air dry to approximately 2, 4, 8, and 16 percent less than the as-molded moisture content. The partially dried specimens were then wrapped in foil, waxed and allowed to stand for 7 days to permit moisture redistribution before being tested.

Figure 5 shows the untreated soils gained moisture rapidly after being subjected to capillarity. Both the amount of moisture and the rate of moisture absorption increased with an increase in the amount of drying back from the as-molded moisture content. On the other hand, treatment with Aliquat H226 significantly reduced the amount and the rate of capillary absorption. Moreover, both levels of Aliquat H226 treatment appeared to be equally effective. The results also show that drying is not required to

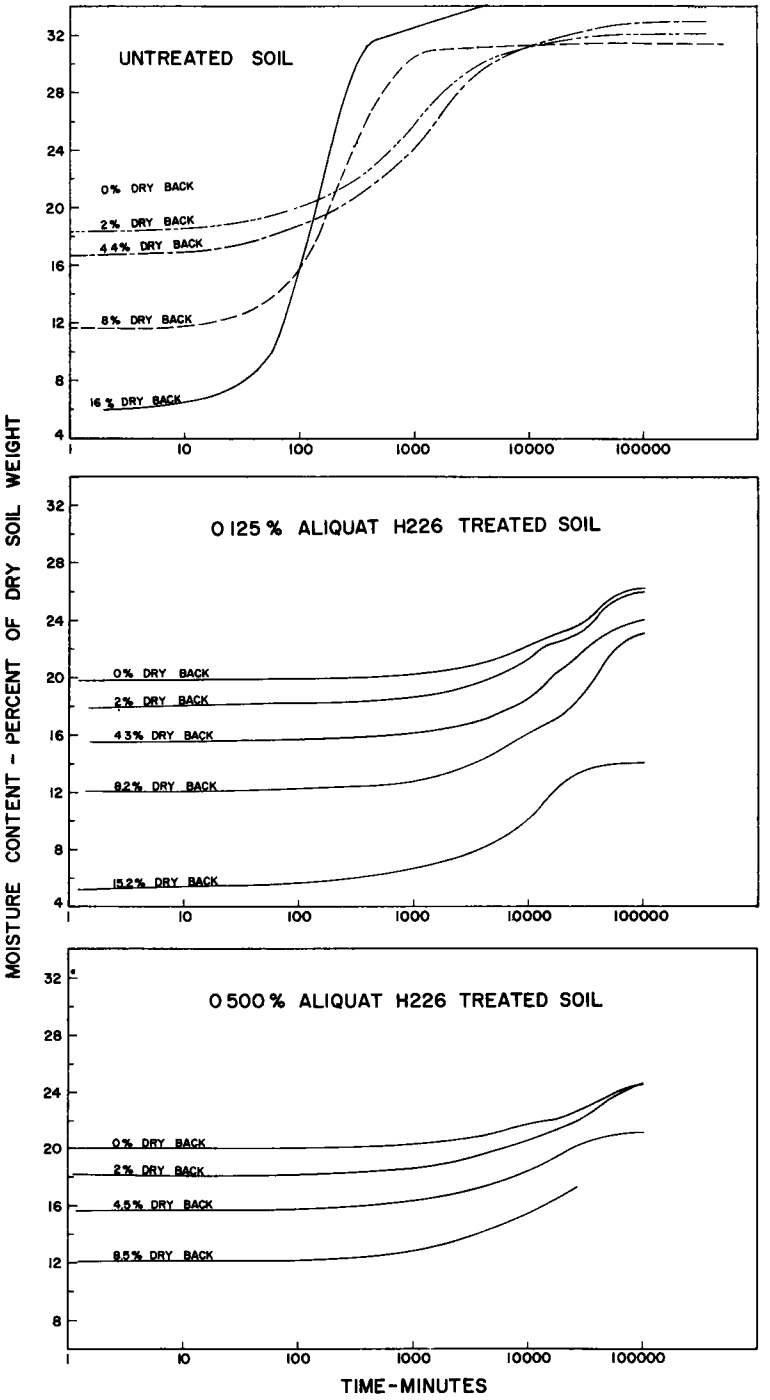


Figure 5. Results of capillary absorption tests on RF 251-109.

reduce capillary absorption of treated soils, but it does increase the effectiveness of the treatment significantly.

The effect of Aliquat H226 treatment was also evident in appearance and strength of the specimens as shown in Figure 6. With less than 1 day of capillarity the untreated

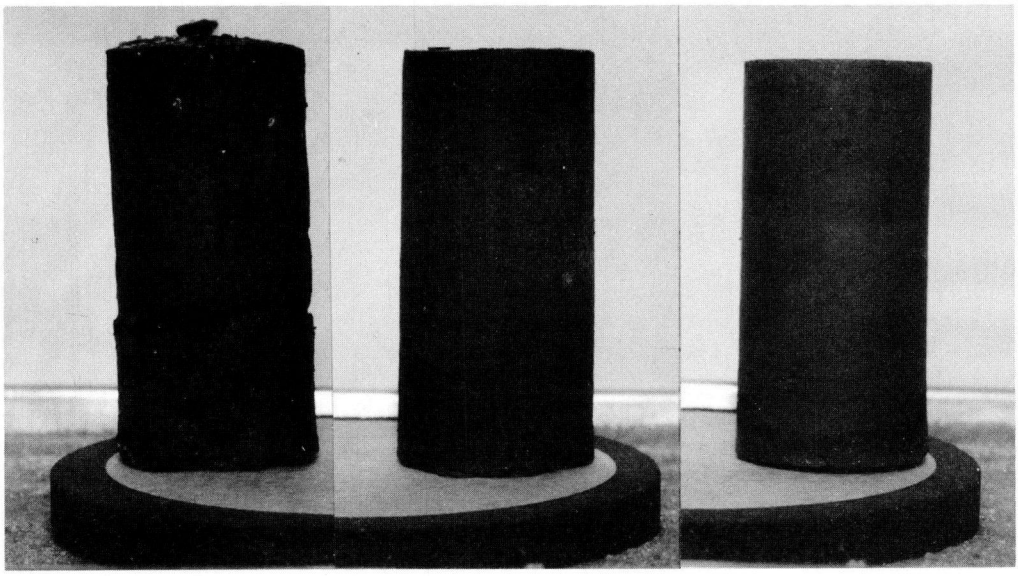


Figure 6. Capillary absorption specimens of RF 251-109. Treatment, left to right, untreated, 0.125 percent Aliquat H226, and 0.5 percent Aliquat H226.

soils became extremely fragile and required great care in handling. The treated soils exhibited high strength throughout the test period. Similar tests, not reported herein, were performed on several other soils confirming the observations made on the O'Hare Field clay. In all instances Aliquat H226 significantly reduced capillary moisture absorption.

As a means of comparing Aliquat H226 with other stabilizers, two soils (RF 176-5 and RF 176-34) were separately stabilized with 0.5 percent Aliquat H226 and with 4.0 percent lime and subjected to capillarity at their as-molded moisture contents. The results of these tests (Fig. 7) show Aliquat H226 to be superior to lime in reducing capillary absorption in these two soils.

COMBINATIONS OF ALIQUAT H226 AND OTHER STABILIZERS

From the test results shown in Figure 2, it was apparent that when used in economically competitive quantities on the highly plastic clays, Aliquat H226 could not satisfy the compressive strength requirements of the Bureau of Public Roads. The inherently low strength of the plastic soils necessitates the use of a cementing type of stabilizer rather than a waterproofer to attain the minimum strength requirement of 100 psi. However, it was felt that further addition of a waterproofer should retain the strength imparted by a cementing stabilizer during immersion, thereby resulting in stabilization superior to that obtained with either stabilizer separately. Accordingly, specimens were made using lime and portland cement in conjunction with Aliquat H226 and the effectiveness of these combinations was evaluated by unconfined compression tests. The results shown in Figure 8 for RF 176-5 are considered typical of tests conducted on several clay soils.

In general, it was found that small amounts of Aliquat H226 increased the strength of lime-stabilized specimens after 7 days of humid curing. When subjected to 6 days of immersion following the humid cure period, Aliquat H226 increased the strength of specimens containing small quantities of lime, but it was less effective at higher lime percentages. However, after the curing conditions of 7 days of air drying, lime-stabilized specimens collapsed when immersed unless they were treated with Aliquat H226.

Aliquat H226 had relatively minor effect on the strengths of cement-stabilized specimens except those cured with 7 days of air drying. At these conditions, it was again

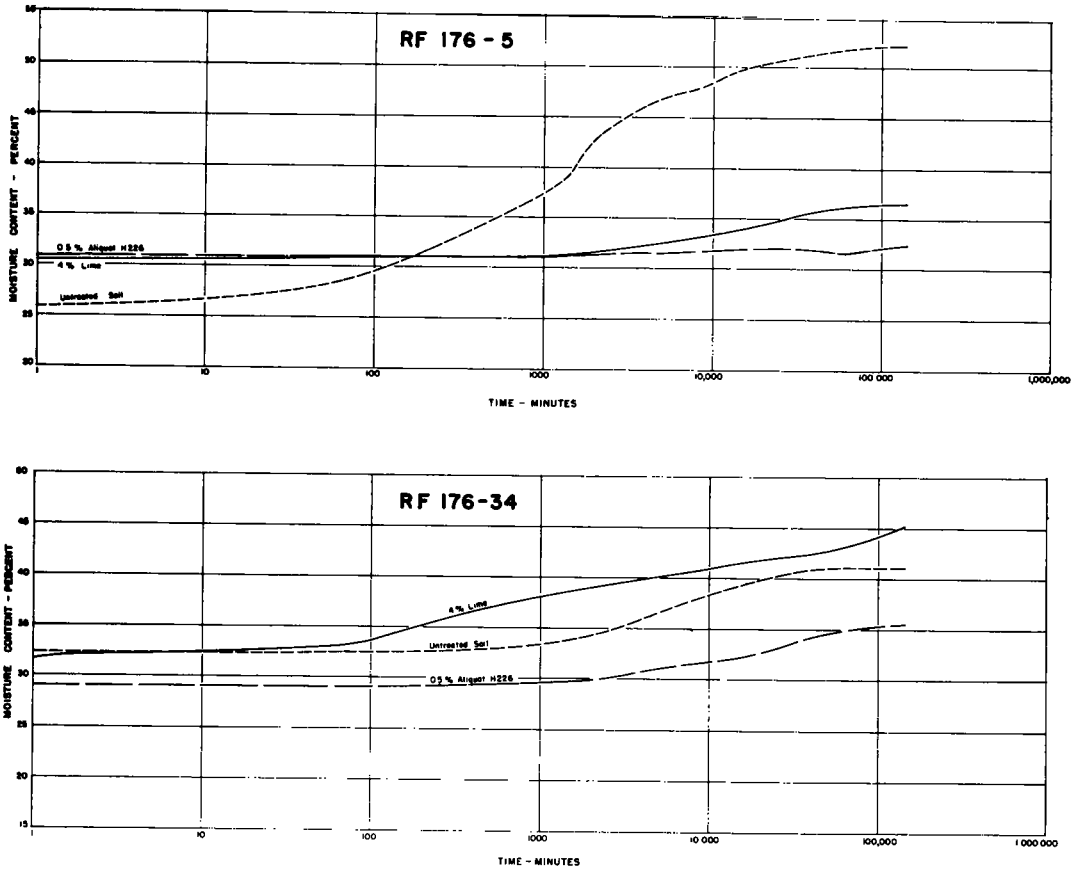


Figure 7. Results of capillary absorption tests on RF 176-5 and RF 176-34.

found that specimens which did not contain Aliquat H226 either disintegrated or had very low strengths when immersed.

FROST HEAVE STUDIES

Due to the ability of Aliquat H226 to reduce capillary absorption and render the soil grains hydrophobic, a brief study was conducted to determine its effectiveness as a frost heave inhibitor. Two soils, RF 251-74 and RF 251-102 were selected for the investigation. Soil properties are given in Table 1.

Apparatus

To perform tests under conditions resembling those encountered in the field, the freezing chest shown in Figure 9 was designed and constructed Several of the more important features of the freezing chest are considered pertinent. First, the chest was designed so that 20 Harvard miniature compaction molds could be tested during each freezing cycle. A plexiglas window was placed in the front for observations and measurements. The sides of the molds were insulated with styrofoam to insure freezing of the samples from the top and 4 of the 20 molds were equipped with thermocouples to record the downward progression of the freezing plane The rate at which the molds were frozen was thermostatically controlled. The freezing chest was placed in a large refrigerator to minimize the effect of fluctuations in laboratory temperature. To obtain a uniform temperature distribution throughout the chest, a small fan was installed to provide constant air circulation. Finally, a free water surface was made available to the bottom of the samples.

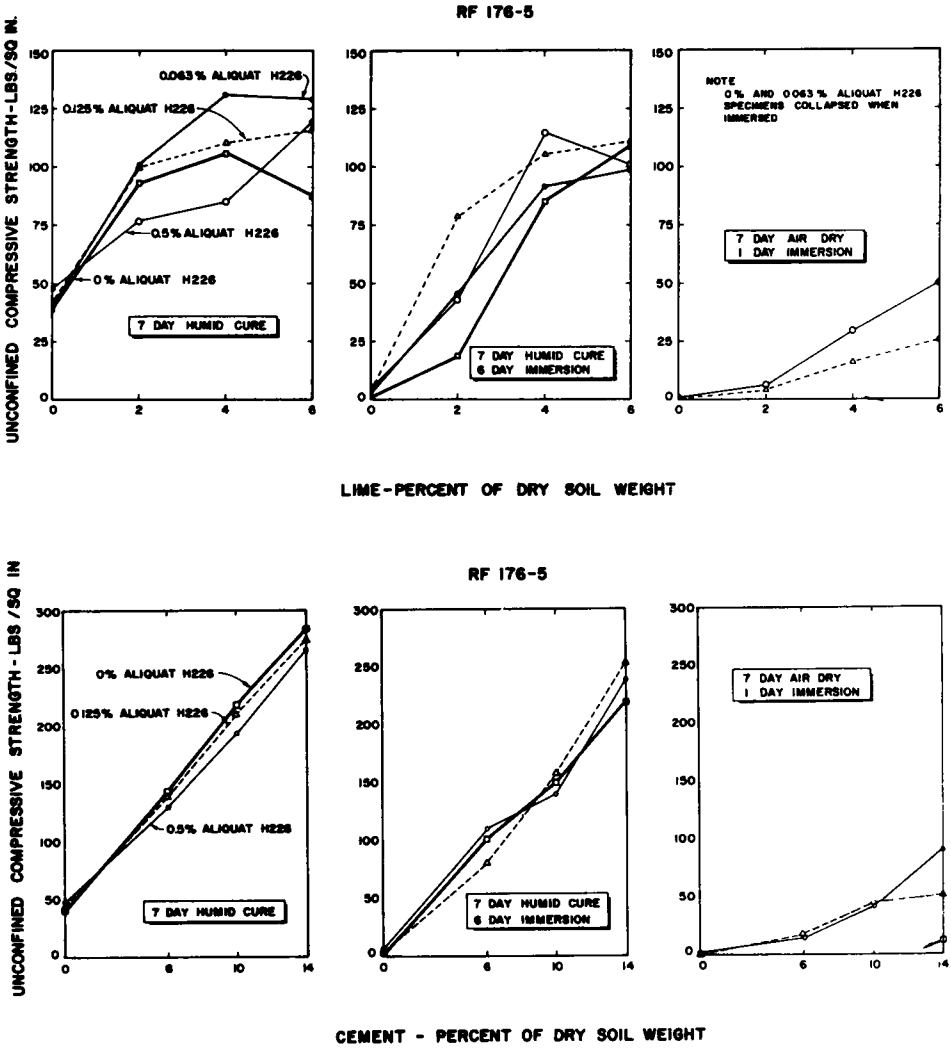


Figure 8. Unconfined compressive strength for RF176-5 using lime and portland cement in combination with Aliquat H226.

Laboratory Procedure

Test specimens were compacted in plexiglas molds using the Harvard miniature compaction apparatus and procedure. The specimens were generally broken down into four groups of five similarly stabilized specimens for each test cycle. One specimen in each group was instrumented with thermocouples (Fig. 10) and the heave of each treatment was taken as the average of the remaining four specimens. Each cycle contained an untreated group so that a comparison could always be obtained between the heave of the raw soil and treated specimens. Generally, the remaining three groups were stabilized with 0.125, 0.25, and 0.5 percent Aliquat H226.

Two different test methods were used. In the first method, specimens were placed on the base plate immediately after compaction and allowed to absorb water for 24 hr. The test was then carried out as described later. The second method allowed the specimens to dry back after compaction to approximately 50 percent of their as-molded moisture content, after which they were placed on the base plate to absorb water for 48 hr.

At the end of the soaking period, the heights of all specimens and the temperature gradient in the four instrumented specimens were recorded. The freezing chest was

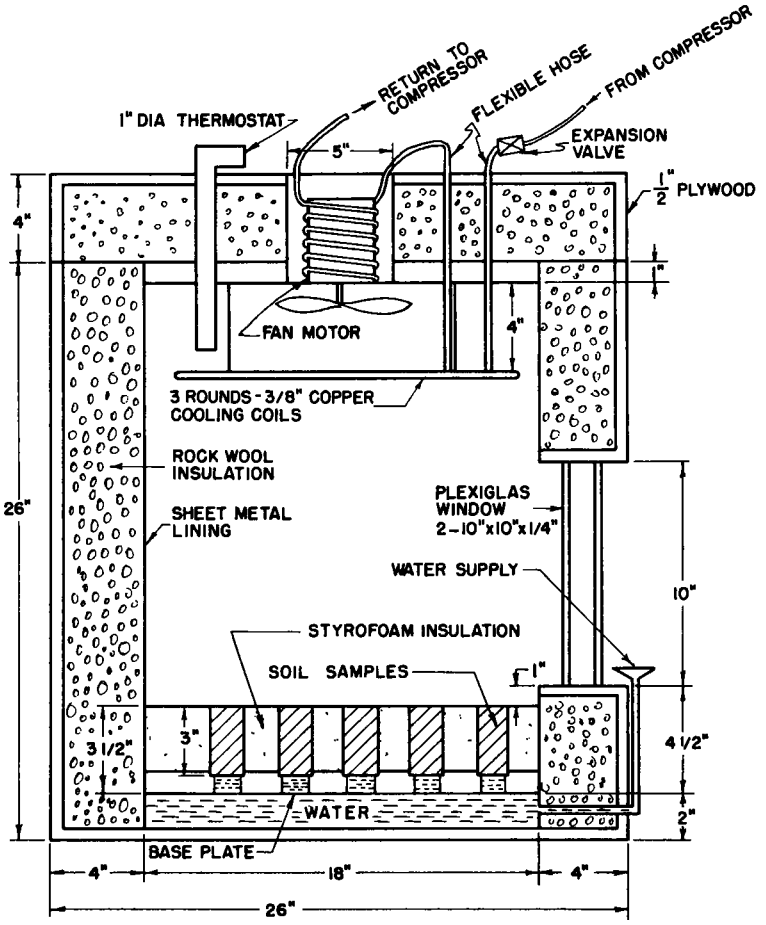


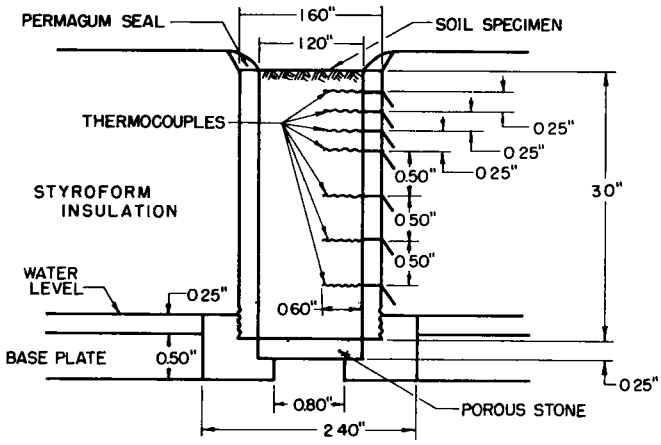
Figure 9. Cross-section of freezing chest.

sealed and the temperature inside the chest was gradually lowered so as to maintain a freezing gradient in the specimens of approximately ¼ in. per day. At predetermined time intervals, the specimen heights were measured and recorded. At the end of the freezing cycle the specimens were removed and allowed to thaw. Observations were made as to the conditions of the samples before and after thawing.

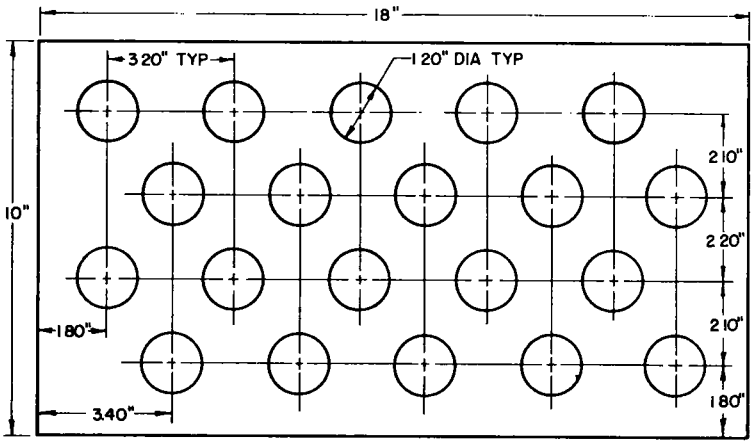
Results of Freeze Test

The effectiveness of Aliquat H226 as a stabilizer for frost susceptible soils was found to be markedly influenced by the curing method, as seen in Table 4.

Untreated specimens of RF 251-102 which were allowed to dry from the as-molded condition heaved 133 percent of their initial height. However, specimens tested simultaneously but treated with as little as ⅛ percent Aliquat H226 did not show any distress or change in height. The effectiveness of Aliquat H226 under these conditions can probably be attributed to the hydrophobic characteristics of the chemical. As the soil is dried back to a point where the capillary pores are no longer filled with continuous columns of water, the Aliquat H226 coats the sides of the pores. When the specimens are subsequently subjected to freezing, the forces normally pulling water into the soil; that is, capillary forces and suction forces due to freezing, are not great enough to overcome the repulsive force of the Aliquat H226 for water.



TYPICAL SECTION OF SAMPLE MOLDS SHOWING THERMOCOUPLES



PLAN OF FREEZING CHEST SHOWING SAMPLE SPACINGS

Figure 10.

The addition of Aliquat H226 did not improve the heave characteristics of soils tested at the as-molded moisture condition. The magnitude of the heave in the treated samples was, in fact, slightly greater than observed in the untreated soil. However, it was apparent that Aliquat H226 did influence the stability of the treated samples. This was evident several times when mechanical failures interrupted the freezing cycle and caused the ice on the cooling coils directly above the specimens to melt. The water dripping on the untreated specimens quickly disintegrated them but had no effect on the Aliquat H226-treated specimens. Also, when the tests were terminated and the specimens allowed to thaw after removal from the chest, the untreated specimens slumped whereas Aliquat H226-treated specimens retained their shape and strength. Apparently the migration of water within the specimens during freezing resulted in sufficient drying between the frost lenses to render the grains hydrophobic.

MINERALOGICAL STUDIES

Apparatus, Procedure, and Soils Studied

To investigate the action of Aliquat H226 on soils, a brief study was made of its influence on the basal spacings of certain clay minerals. This study was performed using

TABLE 4

RESULTS OF FROST HEAVE STUDIES

Soil	Average Compacted Density (pcf)	Treatment (% H 226)	Average Heave (cm)	Ratio of Heave[a]
RF 251-74	106 3	Untreated	0 81	--
	106 2	1/8	1 29	1.60
	106 2	1/4	1 33	1.64
	105 1	1/2	1 31	1 62
RF 251-74	105 6	Untreated	0 85	--
	104 9	1/8	1 52	1 79
	104.4	1/4	1 75	2 06
	102 4	1/2	1 20	1 41
RF 251-74	109 0	Untreated	1 31	--
	108 9	1/8	1 52	1.16
	109 3	1/4	1 40	1 07
	108 2	1/2	1 46	1 11
RF 251-102	103.2	Untreated	1 38	--
	102 2	1/8	1 64	1 19
	102.1	1/4	1 73	1 25
	102 0	1/2	1.80	1 30
RF 251-102[b]	102.9	Untreated	2 5	--
	102 8	1/8	0	0
	103.0	1/4	0	0
	102.1	1/2	0	0

[a]Untreated soil to treated soil

[b]Samples cured by drying before soaking and freezing.

a Phillips electronic X-ray diffraction unit, equipped with wide-range goniometer and copper target. Three clays were selected for X-ray study: Volclay (a sodium-saturated montmorillonite) and Panther Creek bentonite (a calcium-saturated montmorillonite), both obtained from the American Colloid Company, and a kaolinite, known as Hydrite 10 obtained from the Georgia Kaolin Company.

The specimens for the study were prepared by placing 1 g of air-dried clay in a 100-ml centrifuge tube. To this was added 25 ml of solution containing the desired treatment. The solution was shaken for 15 min in a reciprocating test tube shaker, allowed to stand for 48 hr and then shaken again for 15 min. A portion of the solution was placed on glass slides and allowed to dry to a constant moisture content at room temperature in order to obtain maximum particle orientation. X-ray patterns were then obtained.

Results and Evaluation

Because sodium montmorillonite is the most active of the clay minerals, first considerations were focused on the effect of Aliquat H226 on this mineral. As seen in Table 5, the pure clay mineral has an (001) spacing of 12.45 A. When 0.1 percent Aliquat H226 was added, no significant effect on the basal spacing was observed. However, 0.5 percent Aliquat H226 showed two equal intensity peaks at 13.19 A and 23.88 A. Because the peaks were of equal intensities, it would appear that approximately one-half of the clay had been expanded to 23.88 A, whereas the remainder was unaffected. X-rays for the 1 percent Aliquat H226 solution indicated that most of the montmorillonite had been expanded to 27.6 A, whereas the 14.25-A peak with the intensity of 4 was probably caused by some of the clay mineral remaining relatively unaffected by the treatment. Percentages of Aliquat H226 larger than one are believed to have completely blotted out all indications of the clay minerals. The 2.0, 4.0, and 6.5 percent Aliquat H226 solutions gave (001) spacings of 37.6, 39.3, and 39.3 A, respectively. These values check very closely with the 38.4-A spacing found for the Aliquat H226 (Table 6).

To determine if the expansion of the Na-montmorillonite was permanent, samples treated in the preceding manner with two percent Aliquat H226 were washed and new X-ray patterns were obtained. One sample washed four times with water showed an (001) spacing of 27.6 A which is identical to the high intensity peak obtained for the 1 percent treatment. A similar sample was washed four times in isopropanol, a known

TABLE 5

Na-MONTMORILLONITE X-RAY DATA

Treatment	Total Angle (deg 2θ)	Angle 2θ of Major Peaks	Basal Spacing (A)	Relative Intensity
1 g Na-montmorillonite in distilled water	40	7 1	12.45	10
1 g Na-montmorillonite + 25 ml 0 1% H226 sol	10	7 0	12.63	10
1 g Na-montmorillonite + 25 ml 0 5% H226 sol	10	6 65	13 19	10
		3 7	23 88	10
1 g Na-montmorillonite + 25 ml 1 0% H226 sol	10	3 2	27.6	10
		6 3	14 24	4
1 g Na-montmorillonite + 25 ml 2.0% H226 sol	10	2 35	37.6	10
		4 3	20.6	4
1 g Na-montmorillonite + 25 ml 4 0% H226 sol	20	2 25	39.3	10
		4 50	19 6	2
1 g Na-montmorillonite + 25 ml 6 5% H226 sol	20	2 25	39.3	10
		4 50	19 6	2
1 g Na-montmorillonite + 25 ml 2% H226 sol then washed 4 times with water	10	3 2	27.6	10
		6 65	13.2	2
1 g Na-montmorillonite + 25 ml 2% H226 sol then washed 4 times with isopropanol	10	3 4	26.0	10
		6 8	13 0	2
1 g Na-montmorillonite + 25 ml isopropanol	10	6 9	12.81	10

TABLE 6

ALIQUAT H226 X-RAY DATA

Treatment	Total Angle (deg 2θ)	Angle 2θ of Major Peaks	Basal Spacing (A)	Relative Intensity
Slide made of 6.5% solution of H226	40	2 3	38.41	10
		4.5	19.6	1
		6.75	13.1	2
		9.0	9 8	1

solvent of Aliquat H226, resulting in an (001) spacing of 26.0 A. Because a separate slide of the Na-montmorillonite, prepared using isopropanol without additional treatment, resulted in a spacing of 12.8 A, it is evident that the expansion was caused entirely by the Aliquat H226. The isopropanol washing was much more vigorous than any treatment normally applied to soils in the field; therefore, it can be concluded for all practical purposes that Aliquat H226 in sufficient quantity will expand the Na-montmorillonite, and that this expansion will be unaffected by leaching action of ground water.

The X-ray study performed on Ca-montmorillonite was similar to that cited above for the Na-saturated mineral. The results are given in Table 7. Once again, it can be seen that for percentages of Aliquat H226 greater than two the high intensity Aliquat H226 peak at 38.4 A completely obscured the clay mineral peak. However, when the samples were washed with water and isopropanol, the (001) peaks showed a spacing of 29.45 A and 26.0 A, respectively. Once again, a close similarity can be observed between the high intensity peak of 28.5 A for the 1.0 percent treatment and 29.45 A for the 2.0 percent sample after the latter had been washed four times in water. For both the Aliquat H226 treated Na- and Ca-montmorillonite samples, four washings with isopropanol resulted in a basal spacing of 26.0 A.

Table 8 gives the results of a limited X-ray study performed on kaolinite. With a 2.0 percent Aliquat H226 treatment, two equal intensity peaks were found at 7.196 A and 32.7 A. After four washings with water or with isopropanol, the larger peak was eliminated leaving only the peak characteristic of kaolinite. With this limited data, no

TABLE 7

Ca-MONTMORILLONITE X-RAY DATA

Treatment	Total Angle (deg 2)	Angle 2 of Major Peaks	Basal Spacing (A)	Relative Intensity
1 g Ca-montmorillonite in distilled water	40	6.25	14.26	10
1 g Ca-montmorillonite in 25 ml 0 1% H226 sol	10	5 95	14.85	10
1 g Ca-montmorillonite in 25 ml 0.5% H226 sol	10	5.90	14.98	10
1 g Ca-montmorillonite in 25 ml 1 0% H226 sol	10	3 1	28.50	10
		4 3	21.04	9
		5 8	15 24	8
1 g Ca-montmorillonite in 25 ml 2 0% H226 sol	10	2.3	38.41	10
		4 6	19 21	5
1 g Ca-montmorillonite in 25 ml 2% H226 sol then washed 4 times with isopropanol	10	3 5	26.0	10
1 g of Ca-montmorillonite in 25 ml 2% H226 sol then washed 4 times with water	10	3 0	29.45	10

TABLE 8

KAOLINITE X-RAY DATA

Treatment	Total Angle (deg 2θ)	Angle 2θ of Major Peaks	Basal Spacing (A)	Relative Intensity
1 g kaolinite in distilled water	40	12.4	7 138	10
		24.9	3 576	5
		37.7	2 386	1
1 g kaolinite in 25 ml 2% H226 sol	16	2 7	32 7	10
		8 0	11.05	1
		12.3	7 196	10
1 g kaolinite in 25 ml 2 0% H226 sol then washed 4 times in isopropanol	16	12 4	7 138	10
1 g kaolinite in 25 ml 2% H226 sol then washed 4 times in water	16	12 35	7.167	10

definite conclusion can be reached as to the significance of the 32.7-A peak, but it is apparent that Aliquat H226 is not irreversibly adsorbed or held by the kaolinite.

From the mineralogical studies, several additional observations can be made which would tend to explain the behavior of some soils after treatment with Aliquat H226. According to Grim (2), the charge deficiency of montmorillonite is about -0.66 per unit cell. Thus, on the average, in each 1.5 unit cell there is a deficiency of positive charges equal to unity. Assuming unit cell dimensions of $a_0 = 5.2$ and $b_0 = 9.4$ A, the total surface area per unit deficiency of charge would be approximately 75 sq A. The Aliquat H226 cation, with a positive one charge, however, has an area in plan which is almost three times as large. From this comparison of areas it can easily be seen that with an optimum arrangement of the Aliquat H226 ions, it would be necessary to have a minimum of three layers of Aliquat H226 in the interlayer spaces to satisfy the charge deficiency. Because this optimum arrangement will probably never be achieved, more than three layers will be required to satisfy the charge deficiency. Recalling the X-ray evidence cited earlier, both the Ca- and Na-montmorillonite remained at c-axis spacings of 26.0 A after being saturated and washed with isopropanol. This represents an increase in basal spacing of about 16 A above that of the theoretical mineral, or a total of 4 layers of Aliquat H226 in the interlayer spaces.

TABLE 9

ATTERBERG LIMITS OF CLAY MINERALS

Type of Mineral	Aliquat H226 (% dry soil wt)	Liquid Limit	Plastic Limit	Plasticity Index
Na-montmorillonite	Untreated	703	40	663
	0 25	664	43	621
	0 5	625	43	582
	1 0	547	49	498
	2 0	465	50	415
Ca-montmorillonite	Untreated	123 0	44 0	79 0
	0 25	113 8	47 8	66 0
	0 5	109 4	49 0	60 4
	1 0	100 2	54 3	45.9
	2.0	92 7	53 4	39 3
Kaolinite	Untreated	69 6	41 1	28 5
	0 125	82 9	44 6	38 3
	0 25	96 4	46 8	49 6
	0 5	105 4	50.6	54.8
	1 0	101 8	54 8	47.0

In addition to the previously mentioned relationships, a similar conclusion may be reached from a weight-charge relationship. Assuming that all of the Aliquat H226 would be effective, 0.5 g would have to be added to neutralize the charge deficiency of 1.0 g of clay. Once again, however, due to the extremely large size of the Aliquat H226 ion, it would appear to be practically impossible to locate all of the ions in the interlayer spaces. This is well illustrated by the X-ray patterns for both the Na- and Ca-montmorillonite samples. In Tables 5 and 7, 25 ml of 2.0 percent solution added to 1 g of clay would be approximately equal to the ratio stated. The (001) peak, however, corresponds very closely to that obtained for pure Aliquat H226, indicating that the amount of Aliquat H226 which has not been exchanged into the interlayer space is so great that it completely overshadows the expanded clay peak.

Table 9 gives the results of Atterberg limit tests performed on the three clay minerals. It was found that 2.0 percent Aliquat H226 (based on dry soil weight) reduced the liquid limit of Na-clay by 33 percent whereas the same amount reduced that of the Ca-clay by 24 percent. Thus, 2.0 percent Aliquat H226 sharply reduced the liquid limit even though this is only 0.04 of the amount required to completely satisfy the charge deficiency. The influence of Aliquat H226 on the Atterberg limits of kaolinite was not favorable. Addition of Aliquat H226 resulted in an increase in the liquid limit and the plasticity index.

The results of the X-ray analysis were not unexpected, and they confirmed the findings of others. It is apparent that up to 4 layers of Aliquat H226 molecules can be irreversibly adsorbed on the montmorillonite clays, but they are only loosely attached to the kaolinites. Even when used in quantities considerably less than required to satisfy the charge deficiency, Aliquat H226 significantly reduces the liquid limits and plasticity indexes of montmorillonites. This is probably due to the hydrophobic characteristics of the Aliquat H226 which lowers the adsorption of water in the interlayer spaces and to the flocculation resulting from reduction of the zeta potential. Lack of sufficient data precludes any definite statements concerning the effect of Aliquat H226 on kaolinite.

FIELD STUDIES

Although the laboratory investigation indicated that Aliquat H226 possessed some very desirable attributes as a soil stabilizer, it was apparent that the evaluation was only partially complete without field studies. Development of construction methods, correlation of field and laboratory tests, and determination of the efficiency and permanence of the treatment are items that can be determined only by construction of actual test sections. The two test sections reported were constructed to obtain this information.

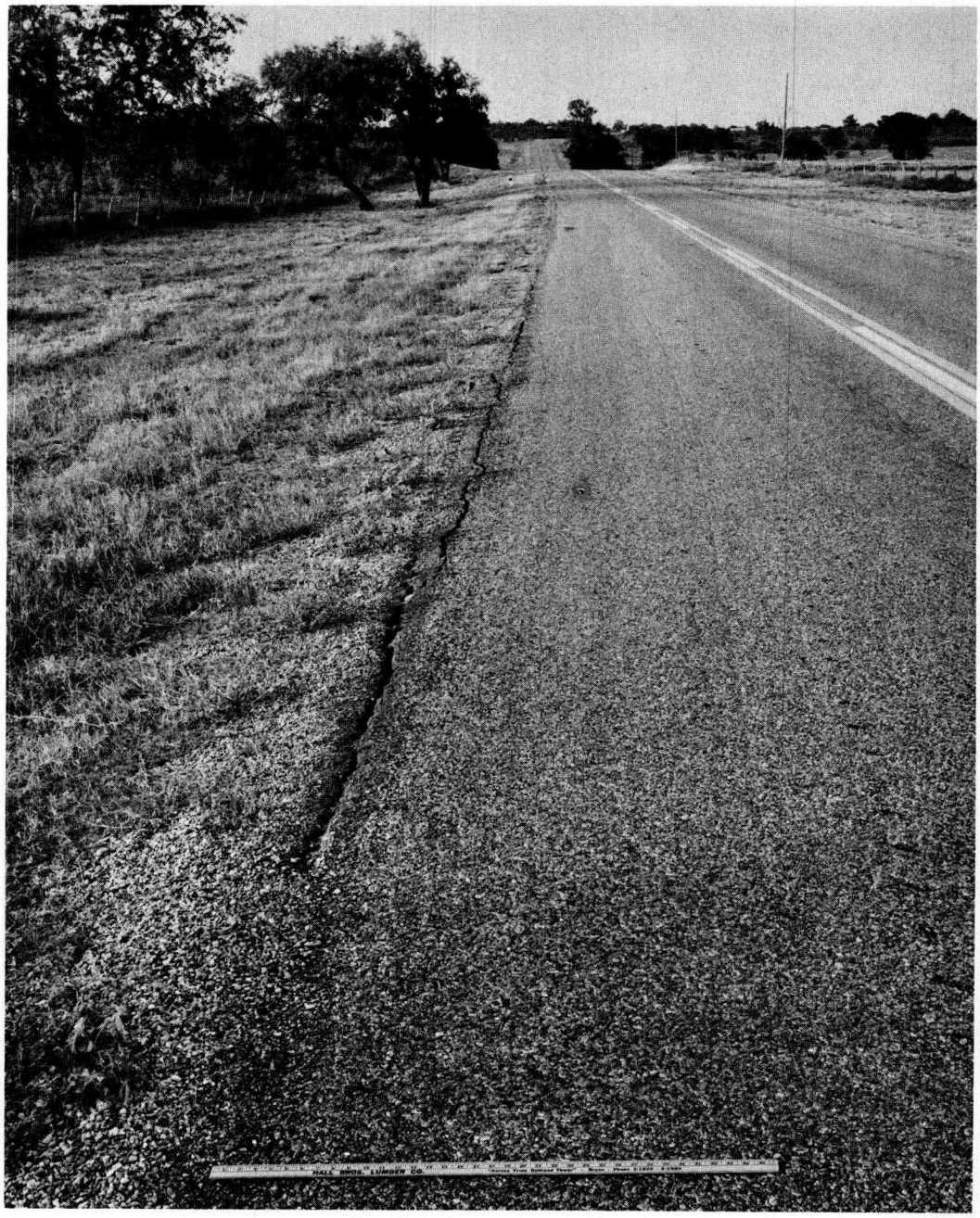


Figure 11. On this highly plastic clay, shrinkage cracks form at edge of pavement surface.

Test Section 1

The first test section was located on the Old San Antonio Road in Brazos Co., near Bryan, Texas. Classed as a State highway, this road receives an average daily traffic of 350 vehicles. Surface soils in the immediate area of the test section are characterized by extremely poor volumetric stability and low load-carrying capacity when saturated.

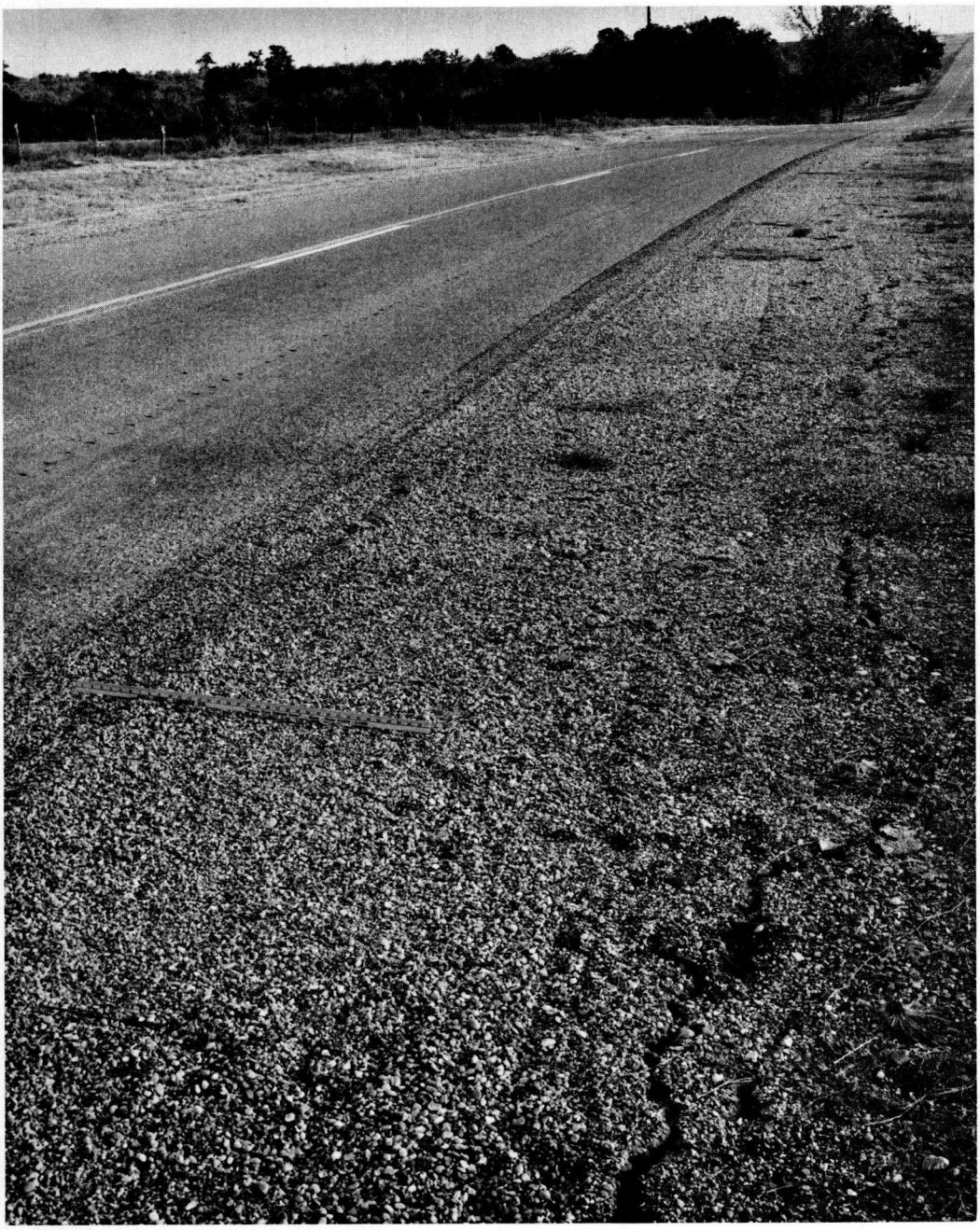


Figure 12. With addition of Aliquat H226 to subgrade soil, shrinkage cracks form at contact of stabilized and unstabilized areas.

In late October 1957, Texas Highway Department forces stabilized 700 ft of the subgrade soil. This period marked the rather drastic termination of a long drought in the area, and cool and rainy weather severely hampered construction operations. The major problem resulted from preparing a highly concentrated (approximately 17 percent) Aliquat H226 dispersion. When the isopropanol evaporated, the dispersion became extremely viscous in the unheated storage tank and required prolonged heating and agitation

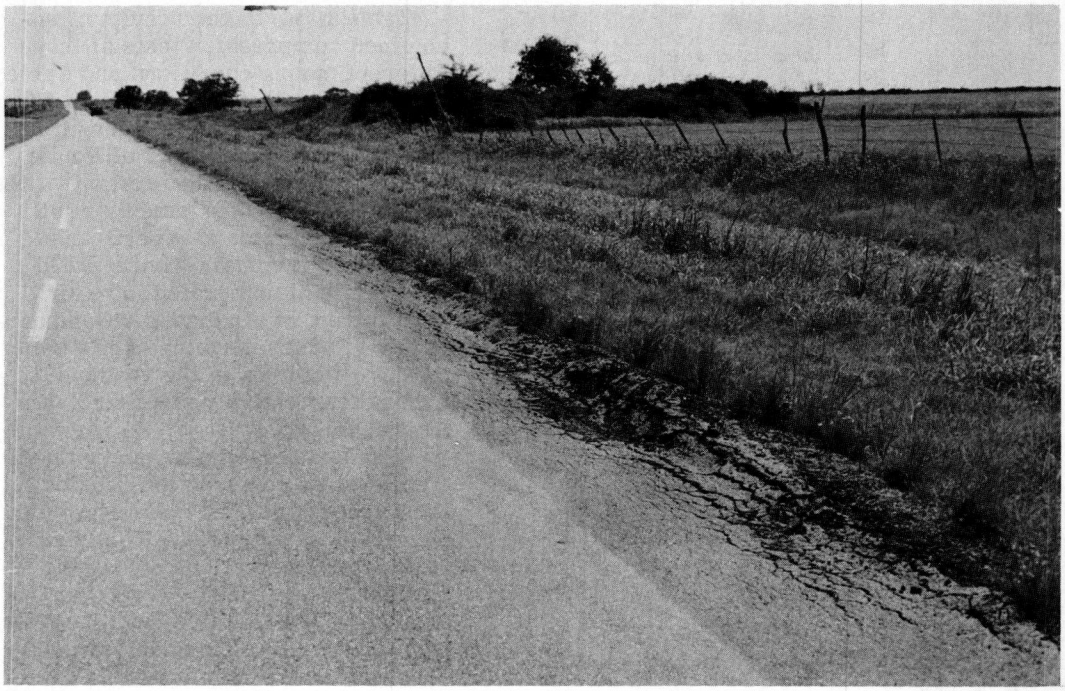


Figure 13. Typical edge failure on test section before reconstruction.

before sufficient water could be added. The saturated subgrade did not readily absorb the chemical dispersion and the rainy weather made it necessary to place and compact the subgrade soil well above the optimum moisture content.

The spotty and ineffective stabilization in some areas of the test section limited subsequent evaluation to little more than visual observations. However, the section has borne traffic for more than four years without significant distress. Perhaps the most striking evidence of the Aliquat H226 treatment is noted in the position of the shrinkage cracks which are prevalent in the area. During the dry season large shrinkage cracks, which provide easy passage of water to the base course and subgrade, form at the edge of the pavement on the untreated control section (Fig. 11). On the stabilized section (Fig. 12) the cracks form well beyond the pavement edge at the contact between the stabilized and unstabilized soil.

Test Section 2

Plans for a second test section were formulated during the summer of 1959. Texas Highway Department personnel suggested another area on the Old San Antonio Road approximately 10 mi from the first section. The roadway in this area was receiving a large volume of heavy truck traffic which had resulted in rapid and severe distress. The majority of the failures had originated as edge failures as shown in Figure 13 and progressed toward the center of the roadway. The test section was ultimately selected in this area due to the desirable high traffic volume and because a low wet area at one end appeared ideally suited for testing the waterproofing qualities of Aliquat H226.

Laboratory classification tests, performed on soils obtained from several auger borings in the test area, indicated that the subgrade could be divided into three general groups. Similar soils were combined and each composite group was tested to determine the stabilizing effect of Aliquat H226. Because the subgrade soils in the general area are often stabilized with lime, lime-Aliquat H226 combinations were also investigated. Evaluation tests were performed in accordance with the testing procedures previously described.

TABLE 10

RESULTS OF UNCONFINED COMPRESSION TESTS ON COMPOSITE SAMPLES[a], TEST SECTION 2

Composite Group No.	Lime (%)	Unconfined Compression Strength (psi)											
		7-Day Humid Cure				7-Day Humid Cure + 6-Day Immersion				7-Day Air Drying + 1-Day Immersion			
		0 % Aliquat	1/16 % Aliquat	1/4 % Aliquat	1/2 % Aliquat	0 % Aliquat	1/16 % Aliquat	1/4 % Aliquat	1/2 % Aliquat	0 % Aliquat	1/16 % Aliquat	1/4 % Aliquat	1/2 % Aliquat
1	0	31.5	32.5	32.5	31.0	8.0	9.0	10.0	9.5	0	15.5	40.5	68.0
	2	46.5	39.0	41.0	44.5	27.5	19.5	22.0	25.5	0	14.5	25.0	17.0
	4	62.5	61.0	62.0	53.5	51.5	47.5	47.0	52.5	4.5	17.8	21.5	19.0
2	0	28.0	26.5	24.0	25.3	7.0	7.5	9.5	9.5	0	7.5	19.5	31.0
	2	26.5	25.5	22.0	21.5	12.5	12.5	10.5	10.5	0	16.5	25.0	20.5
	4	31.5	37.5	37.0	33.0	21.0	23.5	25.5	27.5	0	17.0	20.5	21.5
3	0	32.0	32.5	28.5	31.0	3.0	4.0	9.0	8.5	0	2.0	9.5	12.5
	2	31.5	32.0	33.5	33.5	12.0	13.0	14.0	14.5	0	4.0	11.0	16.0
	4	62.5	63.0	62.5	67.5	21.5	19.5	21.0	23.0	0	2.0	4.5	4.5

[a] Percents of Aliquat H226 and lime are of dry soil weight.

Table 10 gives the results of unconfined compression tests at three different curing conditions and Table 11 gives the plasticity indexes. The unconfined compressive strength requirements of the Bureau of Public Roads were not attained for any of the treatments including lime by itself. With one exception all specimens which did not contain Aliquat H226 completely disintegrated in water after 7 days of air drying. Composite Group 1, which contained soils from the low, wet area of the roadway, was most effectively stabilized with Aliquat H226.

Based on these tests, and guided somewhat by construction considerations, the test section plan shown in Figure 14 was adopted. This plan called for five sections containing ½ percent Aliquat H226, ¼ percent Aliquat H226 and 2 percent lime, 1/16 percent Aliquat H226 and 4 percent lime, 4 percent lime, and one untreated control section. A stabilized subgrade depth of 6 in. extending over a roadway width of 32 ft was considered desirable.

Construction Operations

It was evident from the first test section that the viscosity of the Aliquat H226 dispersion was a factor or prime consideration. A concentrated dispersion would result in viscosity too high to flow through gravity spray-bars, whereas a very dilute dispersion would cause considerable water to be added to the soil while obtaining the desired percentage of chemical. Based on laboratory tests and observations, a dispersion of 5 percent active Aliquat H226 by weight appeared most desirable.

Dispersing the Aliquat H226 in water requires both heat and agitation. This was easily solved with equipment commonly used by all asphalt paving contractors— a portable storage tank and recirculating asphalt car heater as shown in Figure 15. The desired amount of water and Aliquat H226 was added to the 10,000-gal storage tank and mixed by the recirculating heater which also heated the liquid to 140 F. No hazards were encountered with this

procedure and the entire operation took less than 2 hr. The resulting mixture was homogeneous and free of foaming.

Actual construction of the test section began by stripping the surface and base course from the subgrade and stockpiling this material at the edge of the shoulder. Just before the stabilizers were scheduled to be added a 1-in. rainfall halted construction operations. Although most of the area drained rapidly, the stockpiled base course provided a barrier which caused ponding on part of the ½ percent Aliquat H226 section. This area became saturated and proved troublesome in subsequent operations.

TABLE 11

PLASTICITY INDEX OF COMPOSITE GROUP SAMPLES[a], TEST SECTION 2

Composite Group No	Lime (%)	Plasticity Index 0% Aliquat	1/16 % Aliquat	1/4 % Aliquat	1/2 % Aliquat
1	0	19.0	18 5	18.5	15 5
	2	14.0	14.0	13 7	13 7
	4	10 5	12 5	12 0	12 0
2	0	26.0	23.3	19.5	17.5
	2	16.0	18.0	14 5	12 0
	4	14 0	14 5	14.0	12 8
3	0	31.5	32.1	28.5	27.5
	2	16 0	19.0	21.5	24.0
	4	16.5	15.5	17.5	17.0

[a]Percent of Aliquat H226 and lime are of dry soil weight

Dry lime was added to the test section on the following day. About one-half of the required amount was added to the sections and lightly mixed into the soil, and the remaining lime was then added. Before spreading the lime (and the Aliquat H226), the soil was scarified to a depth of 4 in., rather than the full depth of 6 in. This procedure prevented the stabilizers from settling to the bottom of the layer where they could remain at a high concentration if subsequent mixing was not effective for the full 6-in. depth. After the lime was distributed, the soil was scarified to the full depth, thoroughly mixed with a Seaman pulvi-mixer, shaped, and lightly compacted to await the addition of Aliquat H226.

The Aliquat H226 sections were thoroughly pulverized to produce a loose porous layer that would readily absorb the dispersion. The desired amount of dispersion was added to the soil with a gravity spray-bar distributor as shown in Figure 16. Maintainers followed the distributor to surface-mix the chemical into the soil and the pulvimixer then mixed to the full stabilized depth of 6 in. Following the final mixing operation, the entire test section was compacted with a sheepsfoot roller and then surfaced-rolled with a light pneumatic compactor.

At no time during the construction operation did the addition of Aliquat H226 create any problems. In fact, it was observed that Aliquat H226 increased the workability of the plastic materials resulting in easier mixing and placement. The only construction problem that occurred during the entire operation could not be attributed to the Aliquat H226. The wet portion of the ½ percent Aliquat H226 section required scarifying and drying for the full depth of 6 in. before compaction could be completed. Although the stabilized layer was compacted within the allowable moisture content and density range, the underlying soil remained at a high moisture content.

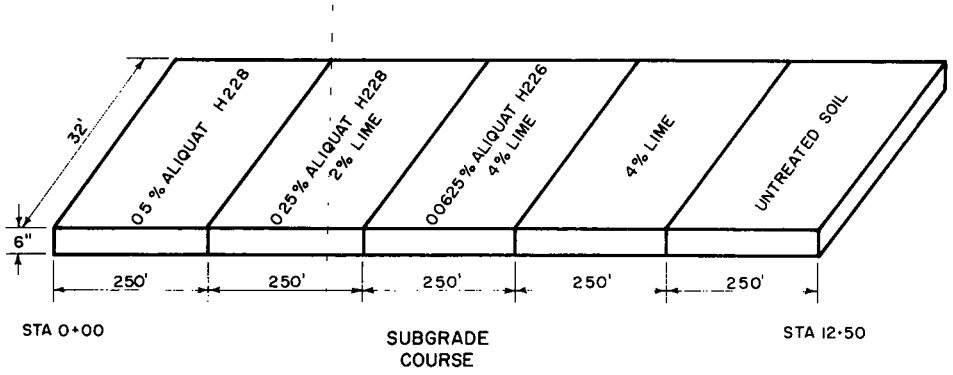


Figure 14. Plan at test section showing stabilizer contents and dimensions.

TABLE 12

RESULTS OF FIELD DENSITY TESTS, TEST SECTION 2

Section	Base Course			Subgrade		
	Unit Wt. (pcf)	Moist. Cont. (%)	Thickness (in.)	Unit Wt. (pcf)	Moist. Cont. (%)	% Std. Compact.
Untreated	134.2	1.7	7.25	119.0	13.4	--
	137.8	3.2	6.75	123.1	11.8	--
	130.1	6.7	7.25	126.9	9.9	--
Avg.	134.0	3.9	7.08	123.0	11.7	--
4% lime	123.3	6.6	6.75	111.3	17.2	98.2
	128.2	8.0	6.25	110.7	18.2	97.7
	130.1	7.5	6.25	109.1	17.1	96.2
Avg.	127.2	7.4	6.42	110.4	17.5	97.4
1/16% Aliquat H226, 4% lime	129.7	6.4	6.50	115.0	16.3	98.0
	130.2	6.7	7.25	112.9	16.6	96.2
	128.3	7.1	6.00	112.3	18.7	95.7
Avg.	129.4	6.7	6.58	113.4	17.2	96.6
1/4% Aliquat H226, 2% lime	129.1	5.6	8.00	119.0	15.5	99.1
	131.4	6.1	7.25	124.4	13.6	103.7
	132.2	6.9	6.25	117.4	16.5	97.8
Avg.	130.7	6.2	7.17	120.3	15.2	100.2
1/2% Aliquat H226	126.0	8.6	6.75	121.2	14.0	96.0
	130.4	5.0	7.00	119.9	10.8	94.9
	126.8	5.9	6.50	125.2	12.7	99.1
Avg.	127.7	6.5	6.75	122.1	12.5	96.7

On completion of the subgrade, the original base course was bladed back over the roadway and compacted. The road remained in this condition without further maintenance until a single asphalt surface treatment was applied approximately 6 weeks later. During this period the road received continued repetitions of heavily loaded vehicles without visible signs of distress.

Construction Tests

During construction, control tests were performed to insure that moisture content and density specifications were being met. After the base course was placed and compacted, several locations in each section were tested to determine the final moisture contents and unit weights of the subgrade and the base course, and the Atterberg limits of the subgrade.

Figure 15. Recirculating asphalt heater and 10,000-gal storage tank used to mix Aliquat H226 solution. Aliquat H226 shipped in barrels shown in foreground.

Results of the field density tests are given in Table 12. No control was exercised over the compaction or placement of the base course and the measured values were obtained only in case they were necessary to analyze future pavement behavior.

The relatively small range of Atterberg limit values (Table 13) indicates that the field mixing operation was very effective. With the exception of the ½ percent Aliquat H226 section, they also show the reduction of plasticity that was expected with stabilization. Plasticity indexes for the ½ percent Aliquat H226 section were higher than expected based on preliminary laboratory tests on composite group 1. It can only be assumed that sampling inconsistencies were responsible for this difference.

Figure 16. Aliquat H226 solution being applied to road from water distributor.

TABLE 13

RESULTS OF ATTERBERG LIMIT TESTS, TEST SECTION 2

Section	Liquid Limit	Plastic Limit	Plasticity Index	Shrinkage Limit
Untreated soil	63.6	24.8	38.8	9.5
	45.7	18.8	26.9	8.9
Avg.	54.7	21.9	32.8	9.2
4% lime	44.4	30.6	13.8	21.5
	50.1	34.8	15.3	23.1
	34.4	25.2	9.2	20.0
	42.4	32.2	10.2	20.8
	45.5	24.9	20.6	12.4
Avg.	43.4	29.6	13.8	19.6
1/16% Aliquat H226, 4% lime	44.8	29.9	14.9	18.8
	42.2	31.5	10.7	24.3
	44.8	32.4	12.4	23.2
	42.0	31.8	10.2	23.0
Avg.	43.5	31.4	12.1	22.3
1/4% Aliquat H226, 2% lime	40.7	23.9	16.8	17.2
	53.1	29.7	23.4	19.7
	45.3	27.7	15.6	18.4
	40.5	23.8	16.7	16.1
	40.1	23.4	16.7	15.8
Avg.	43.5	25.7	17.8	17.4
1/2% Aliquat H226	38.7	18.5	20.2	--
	37.2	15.7	21.5	11.9
	37.8	15.1	22.7	10.0
	39.7	17.4	22.3	11.8
	39.2	15.9	23.3	11.7
Avg.	38.5	16.5	22.0	11.4

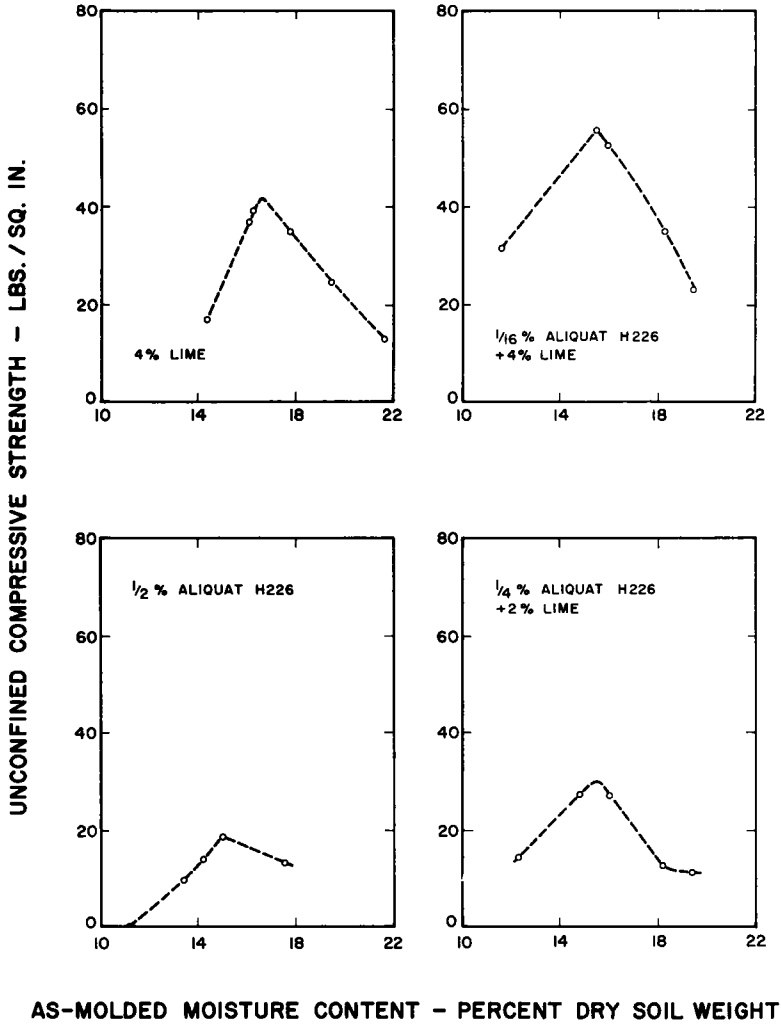


Figure 17. Unconfined compressive strength for test section specimens after 75 days of laboratory humid cure and 6 days of immersion.

Post-Construction Tests and Observations

Following subgrade completion, large composite samples from each treated section were taken to the laboratory and specimens were compacted in the Vicksburg miniature compactor at various moisture contents. These specimens were stored in a humid room for future testing. After 75 days of humid curing, selected specimens were removed from the humid room, immersed in water for 6 days, and tested to determine their unconfined compressive strengths. Results are shown in Figure 17. Similar tests, performed on specimens after nearly one year of humid curing, are shown in Figure 18.

At the laboratory curing conditions, the soil stabilized with 1/16 percent Aliquat H226-4 percent lime not only has the highest strength, but appears to be gaining strength with time. The 4 percent lime-stabilized soil suffered an unexpected decrease in strength with additional humid curing, whereas those soils stabilized with 1/4 percent Aliquat H226-2 percent lime and 1/2 percent Aliquat H226 essentially retained their original strengths.

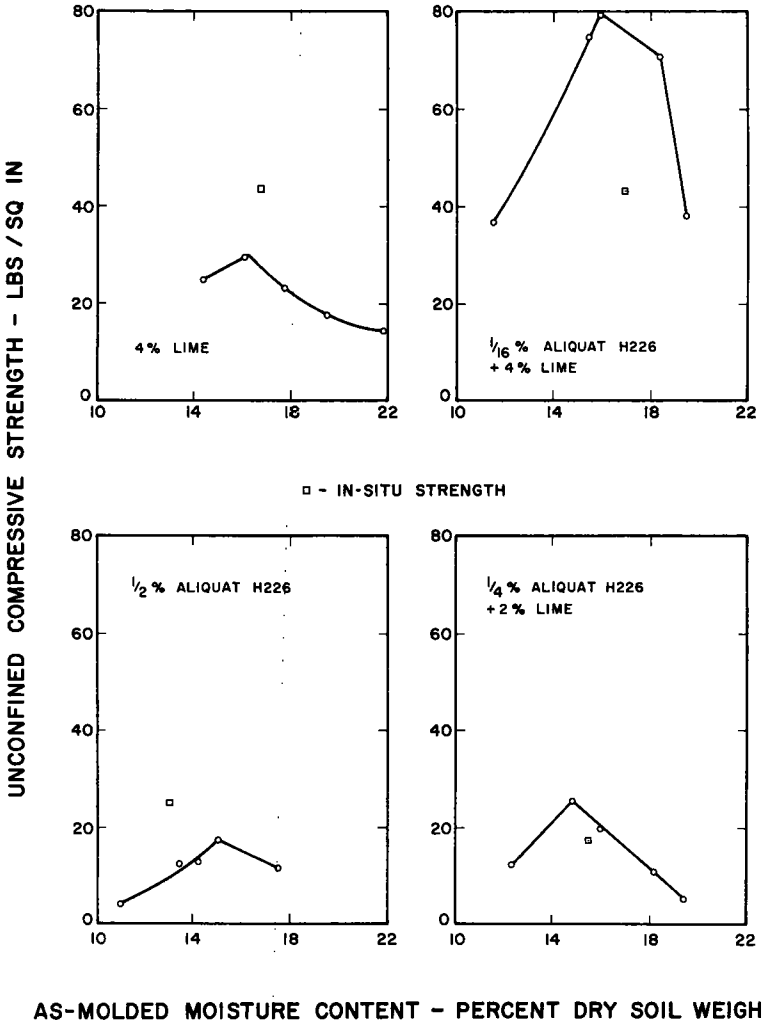


Figure 18. Unconfined compressive strength for test section specimens after 349 days of laboratory humid cure and 6 days of immersion.

Attempts to compare the laboratory strength results with field tests have been unsuccessful due to the difficulty experienced in obtaining undisturbed samples of the treated soils. Even with thin-walled samplers the recovery ratio is below 80 percent, indicating disturbed specimens. Figure 18 shows the average results of tests on field specimens obtained one year after construction, but little faith is placed in these values.

As an additional means of evaluating the performance of the roadway, deflection tests are being performed at approximately 6-mo intervals at 8 locations in each test section. Due to the relatively short life of the roadway, no definite conclusions can be reached from these measurements and they are not included herein.

As the roadway life increases, it will be possible to make more definite conclusions regarding the efficiency and permanence of Aliquat H226 stabilization. However, based on the available laboratory and field data, it appears that Aliquat H226 retains its effectiveness with time.

ACKNOWLEDGMENTS

The research described in this paper was sponsored by the Chemical Division of General Mills, Inc. Construction of both test sections was accomplished by maintenance forces of the Texas Highway Department, District 17. The authors are indeed indebted to the Texas Highway Department and especially to Robert E. Long, District 17 Laboratory Engineer, for the fine cooperation extended during the performance of this research. Appreciation is also expressed to G.W. Kunze for his advice in connection with the X-ray diffraction research reported herein. Photo credits are due Roland Chatham, Bryan, Texas.

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Soil Stabilization Field Trials, Primary Highway 117, Jasper County, Iowa

J.M. HOOVER, Assistant Professor of Civil Engineering, Iowa State University,
R.T. HUFFMAN, Captain, U.S. Army Corps of Engineers, and
D.T. DAVIDSON, Professor of Civil Engineering, Iowa State University

This paper presents the methods of construction, and the evaluation of three years of field and laboratory observations of 6,000 ft of stabilized soil base and subbase courses of primary highway 117, Jasper County, Iowa. The 6-in. subbase test sections were constructed by using the in-place subgrade loessial soil materials stabilized with lime, lime-fly ash, and commercial organic cationic chemical. The 7-in. base course test sections were constructed using a sand-loess soil mixture stabilized with lime-fly ash, lime-fly ash-accelerating agent, and type I portland cement. The fly ash was obtained from two sources. Sodium carbonate and sodium chloride were used as accelerating agents in two sections of lime-fly ash base course. The surface course was 3 in. of an asphaltic concrete mix.

The evaluation program of the test sections was divided into three phases: (a) laboratory analysis and development of project specifications before construction; (b) construction of base and subbase courses, accompanied by sampling and specimen molding of field mixed materials for laboratory testing; and (c) field and laboratory testing, and evaluation of the test sections under existing traffic and weather conditions covering the first three years of performance.

As far as possible, conventional construction practices were used: scarification, blading, spreading of stabilizing agent, single- and multi-pass mixing, sheepsfoot and rubber-tired compaction. Water for standard Proctor optimum moisture content was applied through the spray bar of the single-pass mixer. The chemical was applied in a water solution through the spray bar at a rate and water concentration necessary for desired optimum moisture content and chemical concentration in the soil.

Performance evaluation was accomplished through testing of laboratory specimens, core samples, Benkelman beam tests, crack studies, weather information, traffic volumes and road roughness measurements. Results indicate all sections of the test road have given three years of excellent service. The road has sustained severe freezing and moisture conditions and is in an equally excellent condition to the non-experimental sections of the road immediately adjacent which employ a 6-in. soil-aggregate subbase; 7-in. soil-cement base course, and a 3-in. asphaltic concrete surface course.

Appended to this paper are the results of soil-bacterial counts made on the chemical-treated subbase section. The results indicate that the presence of the chemical in the treated soil material has resulted in no net increase or decrease in the quantity of microorganisms present at the time of the study, approximately 3½ years after construction.

• DURING THE SPRING of 1957, the Iowa Highway Research Board, the Iowa State Highway Commission, and the Iowa Engineering Experiment Station Soil Research Laboratory, in joint effort, initiated a field trial of stabilized soil base and subbase courses at a test site on Primary Highway 117, Jasper County, Iowa (Fig. 1). The test program incorporated both generally accepted construction procedures, and laboratory-proven methods of soil stabilization.

The test site was a 6,000-ft portion of an otherwise 12-mi long soil-cement-base soil-aggregate subbase highway. The test sections were integrated into the regular construction, for a comparative analysis and examination between the experimental and the regularly designed sections.

During the life of the road, Iowa has had some of its coldest winters in over two decades. Adding to this severe condition is the fact that all occurring cracks have been left unsealed for the penetration of snow and rain. Since completion, traffic density has almost doubled. Increasingly heavy loads are being experienced, as trucking firms are using this route from US 30 to Interstate 35, which opened for traffic in early 1961.

The experimental program was set up in three phases:

1. A simplified laboratory analysis of the field materials, and a general development of project specifications.
2. Construction of the subbase and base course sections.
3. An analytical report covering the first three years' performance under existing traffic and weather conditions.

Since completion of construction in November 1957, the road has been under study by IEES Soil Research Laboratory personnel with assistance by the Iowa State Highway Commission in obtaining core samples, traffic data, and road roughness studies. Although some studies will necessarily continue for years to come, the preliminary studies for this test road were completed in 1961. The tests and observations to that time have indicated the positive effectiveness of the various stabilizing agents, their method of incorporation with the soil and the total design of the various experimental sections. The cost of materials used in the test sections, though not reported in this paper, were relatively low and reasonable.

The purpose of this paper is to present the background and results of the testing program, and the general evaluation of the experimental sections following three years of in-place service.

DESIGN

Timing on the experimental sections was such as to permit little opportunity for detailed laboratory studies and planning. Subsequently all mix design for the base and subbase sections were formulated on the experience with similar soils by personnel of the IEES Soil Research Laboratory. Soil stabilization research in the Laboratory indicated satisfactory stabilization with the mix design shown in Figure 2.

Preliminary Field and Laboratory Investigation

Preliminary investigation of the test sections was begun by using a soil survey made by the Iowa Highway Commission during 1955. This survey developed the AASHO classification of the subgrade soils and other information pertinent to design and construction of the subgrade (Table 1). Rather than locating the 6,000 ft of experimental sections by laboratory and statistical means, a visual inspection of the road was conducted jointly by Iowa State Highway Commission and IEES Soil Research Laboratory personnel to select a test area where terrain and subgrade soils would be relatively uniform. Some variations were unavoidable as the entire length of the road is in fairly rolling terrain involving a Tama-Muscatine soil series complex. The selected 6,000-ft test sector began at station 190 + 00, about 2 mi north of Colfax and ended at station 250 + 00 (Fig. 2). Subgrade soil variation had a pronounced effect on the subbase test sections, because the in-place subgrade soil was mixed with the various stabilizing agents, creating a subbase course. Nearby sand and loess "borrow" materials were decided on as a mixed base course soil.

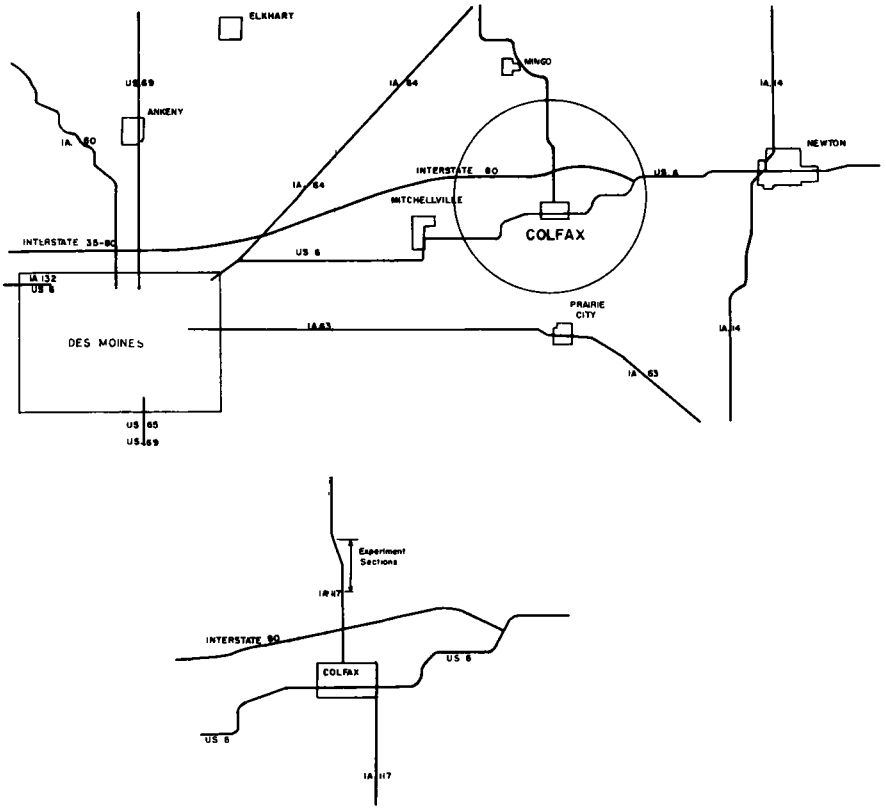


Figure 1. Soil stabilization field trials conducted at a site on Primary Highway 117 in Jasper Co., 2 mi north of Colfax, Iowa.

The in-place soil material used in the subbase test sections was predominantly a medium plastic loess: generally susceptible to capillary moisture from a fairly high water table, and by virtue of its location, subject to a major portion of the freeze-thaw cycles each winter. The in-place soil also included some gravel that had been used as a surfacing following completion of the subgrade in 1956.

The selection of the base course soil materials was primarily based on local availability and economy of utilization. Waste sand from a sand and gravel pit adjoining Highway 117 at the north edge of Colfax was the basic material finally decided on. To improve its workability for construction, and to create a near optimum sandy soil material for cement stabilization (for economical stabilization by portland cement as recommended by the Portland Cement Association, 17), the addition of a medium plastic loess was determined necessary. The sand-loess mixture adopted contained 18 percent loess by dry soil weight. This somewhat cohesive mixture did not have sufficient all-weather stability to serve as a base material without addition of some stabilizing agent.

Representative soil samples were removed from the in-place material in each subbase test section and from the base course borrow materials. Typical AASHO-ASTM analyses of the subbase soil and the base soil mixture are given in Tables 2 and 3, respectively.

Stabilizing Agents

Selection of the various stabilizing agents was based on previous research with soil materials similar to those in the experimental sections. The major consideration for

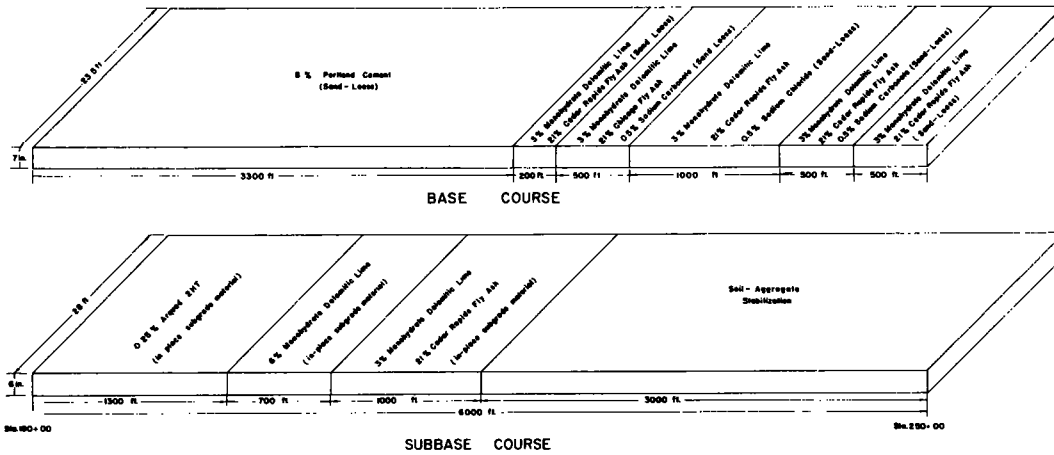


Figure 2. General layout and locations of soil stabilization sections constructed in field trials, Primary Highway 117, Jasper Co., Iowa.

TABLE 1

SOIL SURVEY OF SUBGRADE IN AND NEAR EXPERIMENTAL PORTIONS OF HIGHWAY 117[a]

Property	Station					
	192+00	196+00	221+00	180+00	183+00	184+00
Depth (ft)	0.0-2.4	2.4-12.0	5.6-10.0	7.2-9.0	0.0-3.0	2.6-16.0
Mechanical analysis (%)						
Gravel (over 2.0 mm)	0	0	0	5	0	0
Sand (2.0 - 0.074 mm)	1	0	1	36	1	0
Silt (0.074 - 0.005 mm)	68	70	81	25	62	71
Clay (0.005 - 0.0 mm)	31	30	18	34	37	29
Atterberg limits (%)						
Liquid limit	41	39	32	48	39	42
Plastic limit	25	20	21	26	25	21
Plastic index	16	19	11	22	14	21
Soil type	Silty clay	Silty clay loam	Silt	Clay	Silty clay	Silty clay loam
AASHO class.	A-7-6(11)	A-6(12)	A-6(8)	A-7-6(11)	A-6(10)	A-7-6(13)
Color	Br blk.	Mod yel br.	Lt. yel. br	Mod. br	Blk. br	Mod. yel. br

[a]From Iowa State Highway Commission Plan and Profile of Proposed Improvement of Primary Highway 117, Jasper County, Project F.772 (B) Date February and August, 1955.

stabilization of the in-place subbase material was the addition of agents known to arrest the shrinkage-swelling characteristics of the soil, and to waterproof it against moisture intrusion from either the surface or the subgrade.

Arquad 2HT.—An organic cationic chemical derived from meat packing wastes, Arquad 2HT has been found to waterproof and significantly reduce the shrinkage and swelling characteristics of many silty and clayey soils (4, 5, 6, 11, 14). The natural strength characteristics of a soil are little affected by the introduction of trace amounts of Arquad 2HT. However, silty or clayey soils treated with this chemical retain much of their load-bearing capacity under adverse moisture or freeze-thaw conditions.

TABLE 2

PROPERTIES OF UPPER 6 IN OF IN-PLACE SUBGRADE SOIL

Property	Soil
Geological description	Wisconsin-age loess, plastic, leached, thinly covered with surface dressing of gravelly sand
Location	Jasper Co , Iowa, 2 mi north of Colfax on Iowa 117
Soil series and horizon	Tama-Muscatine complex, silt loam, C-horizon
Textural composition (%)	
Gravel (>2.00 mm)	23.5
Sand (2 00-0 074 mm)	8.0
Silt (0.074-0.005 mm)	39.5
Clay (<0.005 mm)	29.0
Colloids (<0 001 mm)	24.0
Predominant clay mineral	Montmorillonite
Chemical properties	
Cat. exch. cap (meq/100 g)	21 9
Carbonates (%)	2.0
pH	7.5
Organic matter (%)	0.2
Atterberg limits (%)	
Liquid limit	40 5
Plastic limit	23.6
Plasticity index	16.9
Classification	
Textural[a]	Gravelly clay
Engineering (AASHO)	A-6(9)
Standard AASHO density (with 0.25% Arquad 2HT)	
Optimum moisture (%)	15.7
Maximum dry density (pcf)	114.0

[a]By triangular chart used by U S Bureau of Public Roads (19)

TABLE 3

PROPERTIES OF SAND-LOESS BASE COURSE SOIL MIXTURE

Property	Soil
Geological description	Loess—Wisconsin-age, plastic, leached. Sand—washed concentrate from Skunk River Valley
Location	Loess—borrow pit northeast edge of Colfax, Sand—Van Dusseldorp sand and gravel pit, 0 5 mi north of Colfax
Textural composition (%)	
Gravel (>2.00 mm)	0.5
Sand (2 00-0 074 mm)	57.5
Silt (0 074-0.005 mm)	30.0
Clay (<0.005 mm)	12.0
Colloids (<0 001 mm)	10 5
Predominant clay mineral	Montmorillonite
Chemical properties	
Cat. exch. cap. (meq/100 g)	11.0
Carbonates (%)	11.6
pH	8 3
Organic matter (%)	0.2
Atterberg limits (%)	
Liquid limit	18.9
Plastic limit	16.4
Plasticity index	4.0
Classification	
Textural[a]	Sandy loam
Engineering (AASHO)	A-4(1)
Standard AASHO density (with 3% lime, 21% fly ash)	
Optimum moisture (%)	18.0
Maximum dry density (pcf)	103.2

[a]By triangular chart used by U S. Bureau of Public Roads (19).

TABLE 4

ANALYSIS OF MONOHYDRATE DOLOMITIC LIME[a]

Property	Value
Physical (%):	
Loss on ignition	21.0
Fineness	92.1[b]
Chemical (%):	
CaO	45.36
MgO	36.29
SiO_2	0.40
R_2O_3	0.63

[a]Information furnished by manufacturer.
[b]Passing No. 200 sieve.

Lime.—In general, lime causes a reduction in plasticity, an increase in granulation, a reduction in swelling and shrinkage, and an increase in the strength characteristics of silty and clayey soils. Monohydrate dolomitic lime (Table 4) was selected for use in the lime-subbase section because previous investigations had shown it to be superior to other limes for soil-lime stabilization in Iowa, where the dominant soil clay mineral is montmorillonite (15, 16).

Fly Ash.—Fly ash, collected at power plants burning powdered coal, is predominantly composed of spherical particles of noncrystalline silica and alumina, and rounded particles of magnetic iron oxide, Fe_3O_4. Calcium oxide occurs alone or in combination with other ingredients of fly ash. Unburned porous bits of carbon act as a diluent in the pozzolanic reaction with lime; the content of carbon varying with the efficiency of fly ash production, but normally less than 10 percent in what is presently considered good quality ash. The pozzolanic activity of fly ash also appears to depend partly on fineness; normally 80 percent of a good-quality ash passes the No. 325 U.S. Standard sieve (13).

The fly ash used in the experimental sections was obtained from two producers; one located at Cedar Rapids, Iowa, and the second located at Chicago, Ill. (referred to in this report as Cedar Rapids fly ash and Chicago fly ash, respectively). Analyses of these fly ashes are given in Table 5.

Monohydrate Dolomitic Lime-Fly Ash.—The use of lime-fly ash as a stabilizing agent with clayey soils depends largely on the type and amount of clay present. The strength characteristics of montmorillonitic or kaolinitic clays may be improved to a higher degree with lime-fly ash than with lime only. The strengths of illitic-chloritic clays and non-cohesive sandy soils (generally too coarse to react with lime alone) stabilized with lime-fly ash are materially increased over those stabilized only with lime (16). Laboratory trials are necessary to determine the feasibility of stabilization of

TABLE 5

TYPICAL ANALYSES OF FLY ASH USED IN FIELD TRIALS

Property	Fly Ash	
	Chicago[a]	Cedar Rapids[b]
Chemical analysis (% by wt)		
SiO_2	49.8	43 72
Fe_2O_3	14.1	18 37
Al_2O_3	24.9	16.87
CaO	3.1	6 04
MgO	0 38	1.32
SO_3	1 2	1.09
S	---	---
Na_2O	---	---
$CaCO_3$	---	---
CO_2	---	---
SiC	---	---
C	0.36	8.75
Physical property		
Specific gravity	2 50	2.04
Specific surface (Blaine) (cm^2/g)	3,000	3,112
Passing No. 325 sieve (% by wt)	90.00	51.3

[a]Analysis furnished by Chicago Fly Ash Company, Chicago, Ill.
[b]Analysis furnished by Walter N. Handy Co., Springfield, Mo.

TABLE 6

ANALYSIS OF SODIUM CHLORIDE AND SODIUM CARBONATE[a]

Chemical Composition	Accelerator	
	Sodium Chloride	Sodium Carbonate
Compound (% by dry wt)		
$CaSO_4$	0.11	---
$CaCl_2$	0.02	---
NaCl	99.87	---
$(C_{18}H_{34}O_2)_2Ca$	0.022	---
$NaCO_3$	---	99.0
Na_2O	---	58.3
$NaHCO_3$	---	0.1
Na_2SO_4	---	0.02
Element (ppm)		
Fe	---	10
Al	---	5
Ca	---	130
Cu	---	1
Mg	---	30
Si	---	15

[a]Manufacturer's analyses.

silty soils with lime-fly ash rather than lime alone. Granular soils and crushed stone possess inherent mechanical stability, needing only slight improvement to meet requirements of a base course; however, the introduction of lime-fly ash is generally beneficial (7).

Fly ash is used primarily as a pozzolanic stabilization material with lime when the soil to be stabilized contains insufficient natural pozzolans. The benefits thus achieved are increased compressive strength, durability, and waterproofing improvements (7, 16).

Soil-lime and soil-lime fly ash base courses must be protected from traffic abrasion by a wearing surface, and for best results should be constructed and cured during the time of year when the air temperature averages 70 F or higher; the rate of hardening is a direct function of temperature. For proper moist curing as well as protection from carbonation by CO_2 in the air, lime or lime-fly ash-stabilized soil bases should be covered with a bituminous seal coat until such time as a permanent surfacing can be laid (7).

Accelerators.—Because the rate of pozzolanic hardening was expected to be slow due to the late season construction, sodium carbonate (covered by U.S. Patent No. 2,942,993) and sodium chloride were selected for evaluation as accelerators in three of the soil-lime-fly ash base course sections (8, 13). Analyses of the two accelerating agents used are given in Table 6.

Portland Cement.—Soil-cement is the hardened, freeze-thaw resistant material formed by curing a mechanically compacted uniform mixture of pulverized soil, portland cement, and water (9). A proven paving material, it was used in the experimental sections as a standard of comparison for other base course materials.

Type I, ordinary portland cement was used in both the experimental and non-experimental soil-cement portions of the road.

Final Design

Evaluation of all information obtained through field and laboratory investigations of the soils to be stabilized, provided a basis for the final design of the various test sections (Fig. 2). To compare the performance of the base and subbase test sections to the design employed by the Iowa State Highway Commission for the remainder of the 12-mi long road, all base test sections were underlain by the regularly designed soil-aggregate subbase, whereas the subbase test sections were overlain by the regularly designed soil-cement base.

TABLE 7

FINAL MIX DESIGN OF STABILIZING AGENT USED IN FIELD TRIALS

Course	Soil	Approx. Location by Stationing	Test Section Length (ft)	Stabilizing agent (% by dry wt.)[a]
Subbase	Using in-place soil material	190+00 to 203+00	1,300	0 25 (Arquad 2HT[b])
		203+00 to 210+00	700	6 (monohydrate dolomitic lime)
		210+00 to 220+00	1,000	3 (monohydrate dolomitic lime), 21 (Cedar Rapids fly ash)
Base	Sand-loess mixture	190+00 to 223+00	3,300	8 (Type I portland cement)
		223+00 to 225+00	200	3 (monohydrate dolomitic lime), 21 (Cedar Rapids fly ash)
		225+00 to 230+00	500	3 (monohydrate dolomitic lime), 21 (Chicago fly ash), 0.5 (sodium carbonate)
		230+00 to 240+00	1,000	3 (monohydrate dolomitic lime), 21 (Cedar Rapids fly ash), 0 5 (sodium chloride)
		240+00 to 245+00	500	3 (monohydrate dolomitic lime), 21 (Cedar Rapids fly ash), 0.5 (sodium carbonate)
		245+00 to 250+00	500	3 (monohydrate dolomitic lime), 21 (Cedar Rapids fly ash)

[a]Of soil-agent mixture.

[b]As supplied by manufacturer, Arquad 2HT is 75 percent active in isoproponal. Thus, 0.25 percent of 75 percent active Arquad 2HT would supply 0.1875 percent of pure Arquad 2HT by dry soil weight. Amount of Arquad used in field trials referred to herein as 0.25 percent by dry soil weight, as received in 75 percent active form from manufacturer

Thickness.—Design-compacted thicknesses of the subbase and base sections as selected by the Iowa State Highway Commission (largely on the basis of experience) were 6 and 7 in., respectively. The wearing surface was 3 in. of Type B asphaltic concrete constructed in accordance with Article 2304, Iowa State Highway Commission Specifications (12). The surface course was included as an integral part of the soil stabilization field trials only as the base and subbase courses might affect its performance.

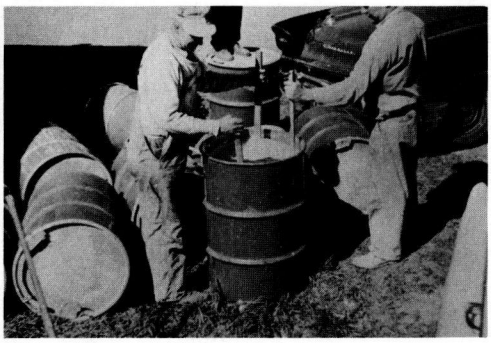
Mix.—The soil-cement base and the soil-aggregate subbase mixtures were designed by the Iowa State Highway Commission: the soil-cement by ASTM method D 560-57; the soil-aggregate in accordance with Art. 4118, Provision 41.06, Iowa State Highway Commission Specifications (12). All other mixes were designed by personnel of the Iowa Engineering Experiment Station, based on soil stabilization research using similar soils. Final mixes are given in Table 7 and Figure 2.

CONSTRUCTION

Subbase Course

The general construction and curing procedures used for the subbase sections were as follows:

1. Surface of subgrade was scarified to required depth, then bladed smooth.
2. The stabilizing agent was spread, with the exception of the Arquad 2HT.
3. AP & H single-pass stabilizer was used with the cutter depth established to produce enough processed material for a compacted depth of 6 in. Water was applied through the spray bar of the machine during mixing at a rate which produced standard Proctor optimum moisture content in the mixture. The Arquad 2HT was applied as a water dispersion through the spray bar, bringing the soil to the desired moisture content and chemical concentration.
4. Initial compaction was accomplished with a tamping foot roller.
5. After the tamping foot had walked out of the mixture, a blade grader was used to smooth the surface of the course, after which compaction was done with a Duopactor rubber-tired roller.



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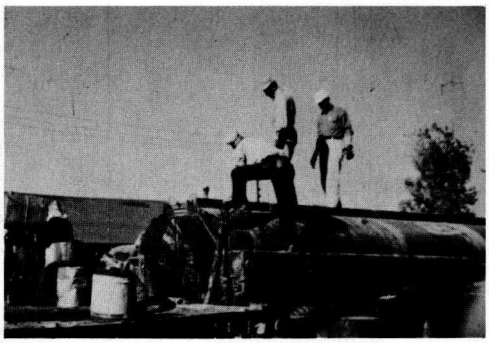

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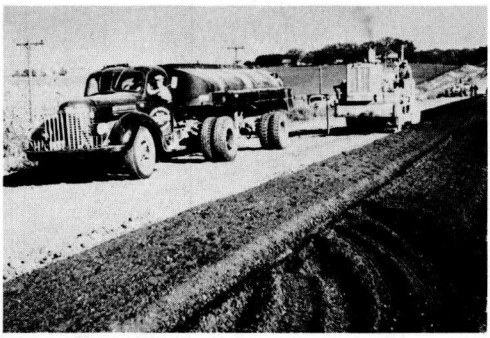

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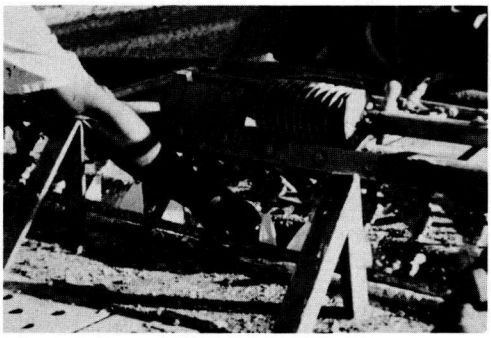

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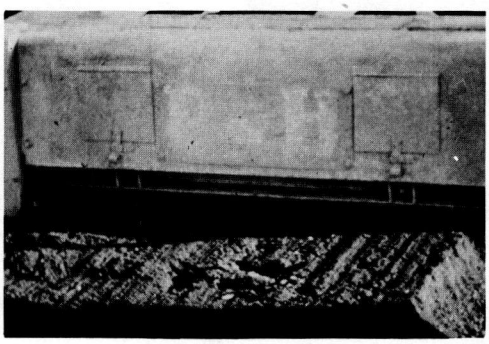

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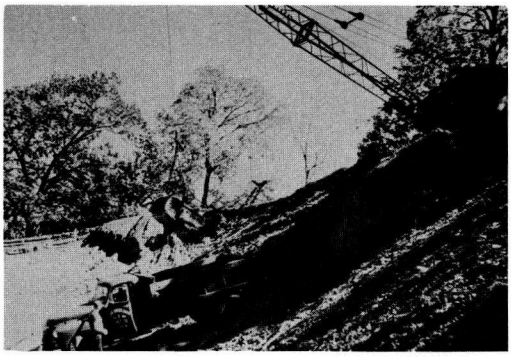
Figure 3. Mixing operations for Arquad 2HT subbase section. (a) Initial mixing of Arquad 2HT in 55-gal drums in which it was received, preparatory to pouring into tank truck. (b) Mixing Arquad 2HT with water in tank truck, using 3-in. centrifugal pump shown at left. (c) Tank truck and P & H single-pass stabilizer. (d) General procedure of mixing subbase materials with stabilizer. (e) Spray bar of stabilizer, showing Arquad 2HT-water dispersion being sprayed into subbase soil for mixing. (f) Tail strike-off of stabilizer, showing mixed material coming out of mixer box.



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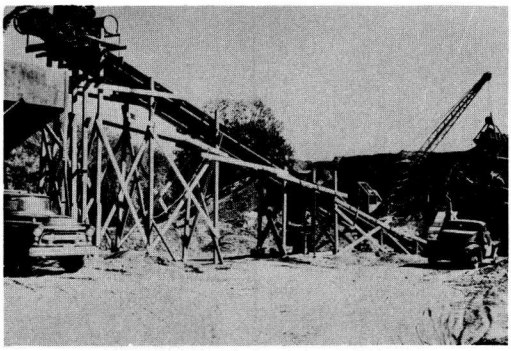

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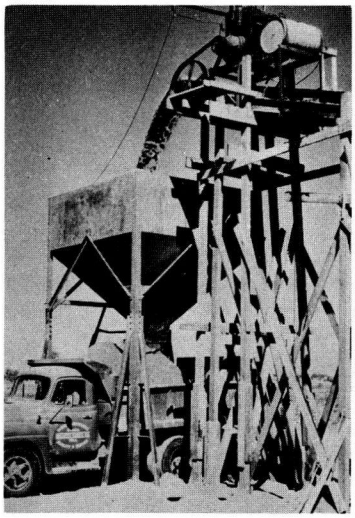

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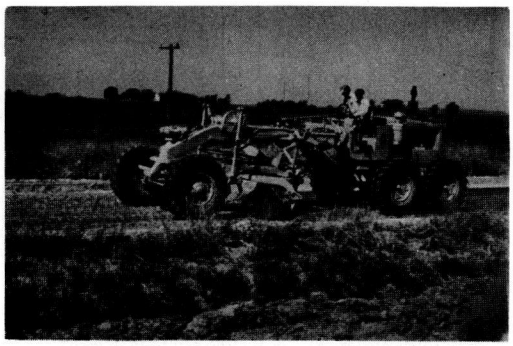

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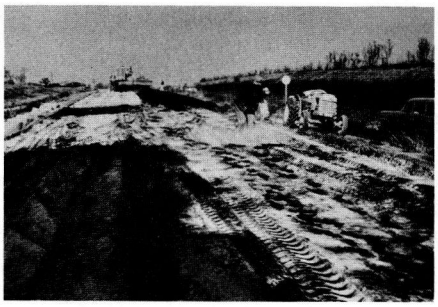
Figure 4. Preparation of sand-loess mixture for base course stabilization. (a) Loess in borrow pit at northeast edge of Colfax, Iowa, being loaded for hauling to mixing plant. (b) Feed hopper: Loess dropped into hopper, screened (to right) to 3/4-in. maximum lump size, and dropped into bottom of hopper containing a calibrated opening for discharge onto belt conveyer for proportioning of loess with sand. Conveyer deposited loess on belt carrying waste sand from pile in background. (c) General view of sand-loess mix plant, showing from right to left waste sand pile, loess hopper and screen, and belt conveyor to storage hopper. (d) Loading sand-loess mix for trucking to construction site. (e) After truck dumping, loess spread by blade grader to desired width and loose thickness before spreading of stabilizing agents.



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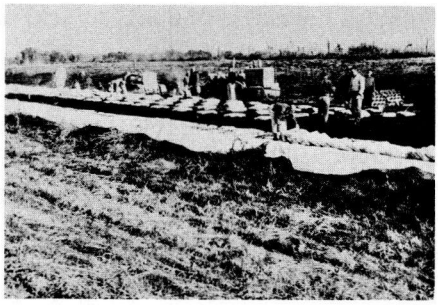

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Figure 5. Soil-lime-fly ash base course construction. (a) Fly ash was truck dumped, then spread by blade to desired width and loose thickness. In sections containing accelerator agents, sodium chloride and sodium carbonate spread on contrasting fly ash to aid uniform distribution. (b) Spotting and opening lime bags in the lime-fly ash section. Spike-tooth harrow drag used to spread transverse rows of lime. (c) Tractor-drawn Seaman Pulvi-mixer mixing 200-ft base section of soil-lime-Cedar Rapids fly ash. All other mixing accomplished with P & H single-pass mixer.

6. The blade grader shaped the course to the desired grade and crown.

7. A wire brush was used to roughen the surface slightly and a light self-propelled rubber-tired roller was used to seal the surface of the course.

8. Each test section was moist cured for 7 days, with the exception of the Arquad 2HT section which was allowed to air-dry for 7 days. Moist curing was accomplished by means of several surface applications of water daily.

One of the problems faced in the field trials was the preparation of the Arquad 2HT water dispersion. In the laboratory, this organic cationic chemical is normally mixed with water heated to approximately 140 F (60 C) to facilitate dispersion of the chemical. Not having a readily available supply of heated water in the field necessitated trying several methods of mixing. Because the quantity of Arquad 2HT used in the field trials was rather small, it was supplied in 55-gal drums. Each drum was given an initial mixing to insure uniformity throughout the depth of the drum. The required quantity of Arquad was then added to each tank truck to produce approximately a 4 percent Arquad concentration in water. This concentration was necessary to give the amount of Arquad desired (0.25 percent by dry soil weight) in the soil-Arquad mixture.

Several methods of mixing the chemical in unheated water in the tank truck were tried. An outboard motor was tried as a stirrer but produced excessive foaming. A compressed air line was next introduced into the tank but again excessive foaming resulted. A 3-in. circulating centrifugal pump was then tried, laying the intake line at the top of the tank and the exhaust at the bottom. This produced a satisfactory dispersion, though some fine particles of undispersed chemical remained, necessitating an occasional cleaning of the filter screen in the pump line of the P & H machine on the job site. Extremely fine air bubbles were also noted in the unheated dispersion which appeared to affect the pressure obtainable from the pump on the P & H machine, necessitating a slower rate of travel with the machine to insure that the proper amount of Arquad 2HT-water dispersion was added to the soil. The air bubbles broke immediately on contact with the soil and did not appear to affect the remainder of the construction procedure or the desired compacted density.

For one tank truck load of chemical dispersion 1,500 gal of water heated to approximately 50 C were obtained. By using a 2-in. pump, integrated to the lower rear of the truck, the dispersion was circulated continuously throughout the tank for a short period, producing excellent homogeneity. No undispersed chemical was noted and no air bubbles appeared. The construction effects noted at the P & H machine when this tank load was sprayed into the soil were extremely good.

The Arquad 2HT section of the subbase contained a rather plastic soil which was not pulverized with the P & H machine as effectively as was thought needed. Consequently, a tractor-drawn Seaman pulvi-mixer was used for further pulverization after the P & H machine had made its pass.

The lime used in the field trials was contained in 50-lb bags. The bags were spotted, opened, and dump-spread in rows transverse to the centerline of the roadway, adjacent rows being approximately 5 ft apart. A spike-tooth harrow was then used to drag-spread the lime uniformly over the surface of the subbase course. Mixing with the in-place soil was then accomplished with the P & H machine. Water added from a tank truck through the spray bars of the P & H machine brought the moisture content of the mix to near the standard Proctor optimum content.

Moistened fly ash was dumped from uncovered trucks, then spread with a blade grader to the required loose thickness and width. The lime was then spotted and opened as previously described. Premixing of the lime and fly ash was done with a spiked-tooth harrow, and final mixing and addition of water was accomplished with the P & H machine.

Base Course

The general construction and curing procedures used for the base course sections were the same as for the subbase, with the following exceptions:

1. The sand-loess mixture was hauled in and dumped according to the required spread for each stabilized section. Following blading to the full width of the base course, the various stabilizing agents were spread on top of it.
2. To provide moist curing, an RC-O cutback asphalt was applied at approximately 0.2 gal per sq yd immediately after final compaction.

The loess for the sand-loess mixture was hauled from the borrow pit to the sand and gravel plant where it was combined with the waste sand. A clam shell bucket loaded the loess into a hopper, from which it was screened and dropped onto a moving belt and delivered to a second belt coming from under the sand pile. The relative speeds of the two belts were adjusted to proportion the sand-loess mixture properly. The mix was then carried to a hopper from which it was dumped into trucks for transportation to the job site.

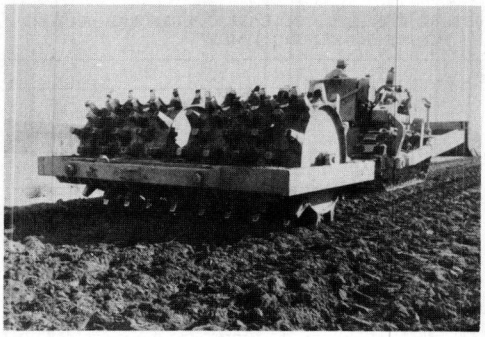
The lime-fly ash base course sections containing the two activating agents required an additional step from that previously described. After the fly ash had been spread, the sodium carbonate or the sodium chloride additive was spread uniformly over it; the former by a hand-seeding method, the latter by a whirlwind seeder. The contrast between the white color of the activators and the black fly ash facilitated uniform spreading.

The last 700 ft of fly ash construction, immediately north of the soil-cement section and including the 500 ft of Chicago fly ash (Fig. 2), was "dry" mixed with the tractor-drawn Seaman pulvi-mixer, after which the desired quantity of water was sprayed on the surface of the mixture from a tank truck equipped with pump and spray bar. Final mixing was done with the Seaman machine.

Mixing, compaction, and finishing procedures used for the 3,300 ft of soil-cement base were the same as in the lime and fly ash base course sections. However, the portland cement was spread with a conventional cement spreader.

General Remarks

Though it was originally thought that the experimental sections would be constructed in June or July, construction did not begin on the subbase sections until September 30, 1957. The final base course section was laid on November 1. In Iowa this is not a satisfactory time of year to obtain proper curing.



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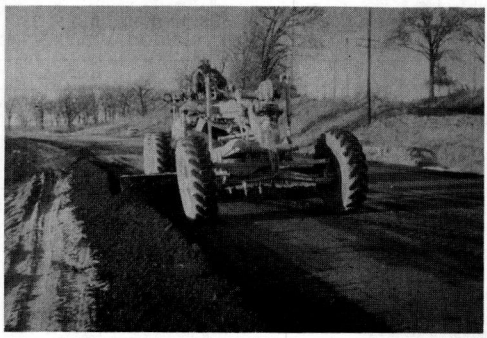

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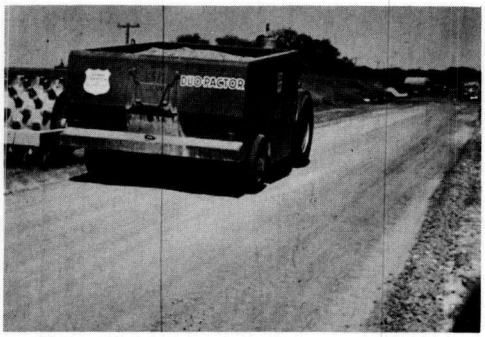

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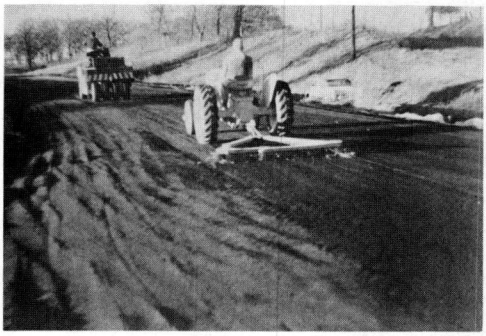

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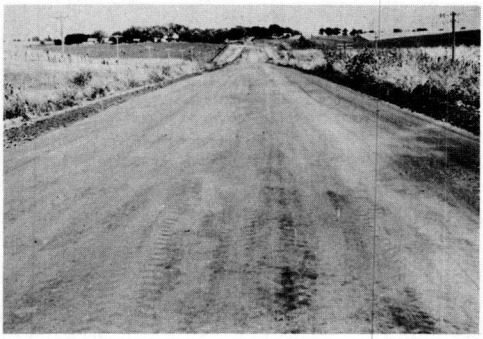


e

f

Figure 6. Compaction and finishing operations. (a) Tamping foot roller used for initial compaction. (b) Blade grader used to shape moderately cross-section before final compaction. (c) Final compaction accomplished by Duo-Packer rubber-tired roller. (d) After final blading to grade and cross-section, surface of course broom-dragged, then rolled by light rubber-tired roller. (e) Finished subbase course, looking north. In foreground, Arquad 2HT section, followed by lime section, and lime-fly ash section. (f) Finished lime-fly ash base course, looking south across soil-lime-Cedar Rapids fly ash-sodium carbonate base section.

TABLE 8

UNCONFINED COMPRESSIVE STRENGTHS, DENSITIES, AND MOISTURE CONTENTS OF BASE COURSE LABORATORY- AND FIELD-MIXED STABILIZED SPECIMENS AT DIFFERENT CURING TIMES[a]

Stabilizing Agent[b]	Curing Time (days)	Lab Mixed, Lab Molded		Field Mixed, Lab Molded		
		Avg. Unconf. Compr Str. (psi)	Dry Density[c] (pcf)	Avg. Unconf. Compr. Str. (psi)	Dry Density[c] (pcf)	Moist. Content[c] (% by dry wt.)
8% Type I portland cement	14	---	---	420	---	7.3
	26	---	---	550	---	6.2
	45	---	---	655	---	5.3
	183	---	---	834	---	4.0
	365	---	---	720	---	6.5
3% monohydrate dolomitic lime, 21% Cedar Rapids fly ash	7	115	102.2	---	---	---
	14	180	99.9	---	---	---
	17	---	---	195	123.2	13 8
	28	273	99 7	294	116.8	---
	45	---	---	545	109.4	18.5
	48	329	99 8	---	---	---
	60	231	---	---	---	---
	61	---	---	582	108.4	9.0
	183	166	---	605	---	6.2
	365	---	---	365	---	14.7
3% monohydrate dolomitic lime, 21% Chicago fly ash, 0.5% sodium carbonate	14	351	115 9	---	---	---
	17	---	---	609	117 9	8.5
	28	383	115.5	878	117 0	8 5
	45	488	117.7	1,320	116.7	6 3
	61	---	---	1,480	---	---
	183	---	---	1,710	---	3.8
	365	---	---	1,240	---	11 3
3% monohydrate dolomitic lime, 21% Cedar Rapids fly ash, 0 5% sodium chloride	7	77	109.3	163	112.8	15 6
	14	150	---	---	---	---
	27	---	109.0	354	112.8	14 8
	28	275	---	---	---	---
	45	274	110 0	420	113.8	10.2
	60	319	110.5	480	113.0	11 8
	183	---	---	512	---	5.0
	365	---	---	519	---	15.5
3% monohydrate dolomitic lime, 21% Cedar Rapids fly ash, 0.5% sodium carbonate	7	---	---	170	---	13 5
	28	---	---	235	---	12 8
	45	---	---	315	---	10 3
	60	---	---	360	---	6.2
	183	---	---	497	---	3 5
	365	---	---	220	---	21 2
3% monohydrate dolomitic lime, 21% Cedar Rapids fly ash	7	---	---	125	---	14 5
	28	---	---	238	---	12 8
	45	---	---	370	---	10.3
	61	---	---	440	---	7 0
	183	---	---	491	---	4 0
	365	---	---	409	---	16 5

[a]Laboratory specimens mixed and molded using representative sand-loess mix soil material and stabilizing agents Field specimens were molded from representative samples obtained immediately following construction mixing of sand-loess mix soil material and stabilizing agents. In each case 2-in. diameter by 2-in. high specimens were molded to near standard Proctor density, wrapped and sealed in waxed paper, and cured in a humidity cabinet for indicated periods at 70 F and near 100 percent relative humidity, molding and curing conditions approximately the same for lab- and field-mixed specimens.
[b]See Figure 2 for layout of experimental base course sections.
[c]At time of test.

The construction methods adopted for the test sections were the most effective means for the short sections involved and the consequent small quantities of materials used. Soil stabilization projects of larger scope might necessitate employing methods of construction other than those used on this project. For example, lime could be spread by a conventional cement or bulk spreader rather than from 50-lb bags. Figures 3, 4, 5, and 6 show the various phases of construction and the equipment used.

EVALUATION OF PERFORMANCE

Preliminary studies of the performance of the test sections cover the period from time of construction through January 1961, slightly over three years.

TABLE 9

UNCONFINED COMPRESSIVE STRENGTHS, DENSITIES, AND MOISTURE CONTENTS OF SUBBASE COURSE LABORATORY- AND FIELD-MIXED STABILIZED SPECIMENS AT DIFFERENT CURING TIMES[a]

Stabilizing Agent[b]	Curing Time (days)	Lab Mixed, Lab Molded			Field Mixed, Lab Molded		
		Avg. Unconf. Compr. Str. (psi)	Dry Density[c] (pcf)	Moist. Cont.[c] (% dry wt.)	Avg. Unconf. Compr. Str. (psi)	Dry Density[c] (pcf)	Moist. Cont.[c] (% dry wt.)
0.25% Arquad 2HT	7	220	109.3	4.3	---	---	---
	9	---	---	---	309	109.8	6.4
	14	300	---	3.6	---	---	---
	28	307	108.6	3.2	---	---	---
	30	---	---	---	435	110.2	3.5
	60	---	---	---	520	109.2	2.2
	63	216	108.0	2.9	---	---	---
	183	211	---	---	412	---	2.4
	365	---	---	---	372	---	3.4
6% monohydrate dolomitic lime	7	86	123 2	---	---	---	---
	10	---	---	---	69	121.6	---
	14	141	122.4	---	---	---	---
	28	188	---	---	---	---	---
	31	---	---	---	140	122.0	---
	60	228	121.7	---	---	---	---
	61	---	---	---	125	119.5	---
	183	141	---	---	261	---	---
	365	---	---	---	45	---	---
3% monohydrate dolomitic lime, 21% Cedar Rapids fly ash	7	52	104 9	21.0	---	---	---
	11	---	---	---	109	105.4	23.1
	14	61	107.2	20.5	---	---	---
	28	112	105.7	20 4	---	---	---
	32	---	---	---	116	117.5	23.0
	60	137	---	19.5	---	---	---
	62	---	---	---	137	111.4	22.8
	183	169	---	---	149	---	10.3
	365	---	---	---	100	---	20.5

[a]Laboratory specimens mixed and molded using representative in-place subbase materials and stabilizing agents Field specimens were molded from representative samples obtained immediately following construction mixing of in-place soil material and stabilizing agent. In each case, 4.0-in. diameter by 4.6-in high specimens were molded to standard Proctor density, wrapped and sealed in waxed paper, and cured in a humidity cabinet (Arquad 2HT specimens were not sealed and were air dried) for the indicated periods at 70 F and near 100 percent relative humidity, molding and curing conditions were approximately the same for lab- and field-mixed specimens

[b]See Figure 2 for layout of experimental subbase course sections.

[c]At time of test.

Laboratory Studies

Before construction, representative soil samples were removed from the base and subbase materials for laboratory mixing and molding of specimens containing the various stabilizing agents. During construction, immediately after mixing had been completed, samples of the soil-stabilizing agent mixture were taken from each major test section at the same locations represented by the laboratory-mixed samples, and were field-laboratory molded for subsequent testing. Compactive energy of molding was at standard Proctor density for all specimens. All base course materials were molded in 2-in. diameter by 2-in. high cylinders (18), whereas all subbase materials were molded in 4-in. diameter by 4.6-in. high cylinders (standard Proctor size). All specimens given moist curing were wrapped in waxed paper, sealed, and cured in a humid room at near 100 percent relative humidity and 70 F. The Arquad 2HT treated specimens were air cured.

Following curing, all molded specimens were tested to failure in unconfined compression (18). Average strengths of six or more specimens of mixtures from each stabilized section were determined for each curing period up to one year. The moisture content of each tested specimen was determined for comparison with the strength value obtained.

After three years of service, two 4-in. diameter core specimens were removed from the same locations in the base and subbase sections represented by the molded specimens. Each core was wrapped and sealed in waxed paper immediately after removal from the pavement and was stored in a humid room at near 100 percent relative humidity and 70 F until cut with a diamond saw to 4.6-in. height (Proctor size).

TABLE 10

AVERAGE UNCONFINED COMPRESSIVE STRENGTH AND MOISTURE CONTENT OF TWO OR MORE CORE SPECIMENS REMOVED FROM BASE AND SUBBASE COURSES, NOVEMBER 1960 (AGE OF TEST SECTION 3 YEARS)[a]

Course	Specimen	Station	Compr. Str.	Avg. Moist. Cont.[b] (% dry wt. of mix)
Base	Sand-loess mix, plus:			
	8% Type I portland cement	193+50	1,185	8.7
		198+50	1,670	8.5
		206+50	1,715	9.3
		212+50	2,263	7.7
		217+50	2,070	7.4
	3% monohydrate dolomitic lime, 21% Chicago fly ash, 0.5% sodium carbonate	227+50	1,840	9.4
	3% monohydrate dolomitic lime, 21% Cedar Rapids fly ash, 0.5% sodium chloride	232+50	690	29.8
	3% monohydrate dolomitic lime, 21% Cedar Rapids fly ash, 0.5% sodium carbonate	242+50	650	14.1
	3% monohydrate dolomitic lime, 21% Cedar Rapids fly ash	247+50	1,056	0.2
Subbase	In-place soil[c], plus 3% monohydrate dolomitic lime, 21% Cedar Rapids fly ash	212+50	668	25.4

[a]Core drilling accomplished by 4.0-in. inside diameter diamond bit core drill. One core removed from each traffic lane at locations indicated. Each specimen cut by diamond saw to 4.6-in. height and tested for unconfined compressive strength. Following testing, moisture content determined on duplicate samples from each specimen.

[b]At time of testing.

[c]Core specimens unattainable from lime and Arquad 2HT subbase sections though hard, lime subbase would not resist torsional effects of core bit, due to type of stabilizing agent, no cores anticipated from Arquad 2HT subbase section.

Unconfined compressive strengths and moisture contents of the core specimens were determined in the same manner as for the laboratory or field mixed, molded specimens.

Mixing.—The efficiency of field proportioning and mixing in base and subbase sections can be qualitatively evaluated by comparing the data on unconfined compressive strengths of field mixed, laboratory-molded specimens, and laboratory-mixed, laboratory-molded specimens given in Tables 8 and 9. Tables 8 and 9 also include some data on the densities and moisture contents of specimens at the times of testing.

Although the data are incomplete, a noticeable observance is that the strengths of field-mixed specimens exceeded the strengths of the comparable laboratory-mixed specimens with the exception of the specimens from the lime subbase. Because lab mixing is normally more efficient than field mixing, this relationship may be due to the higher density of the field mixed specimens. The field mixed specimens of the soil-lime subbase section generally had lower strength and density than the lab-mixed specimens.

The average 365-day cured strength of several of the mixes given in Tables 8 and 9 indicates a decrease from that at 183 days. This is believed to be due to the increase in moisture shown. During the last 60 to 90 days of curing, the waxed paper wrapping on specimens began to deteriorate allowing the specimens to absorb moisture from the humid air before this condition was discovered and the specimens were rewrapped.

In general, field proportioning and mixing appears to have been comparable to or even better than that done in the laboratory.

Carbon Content.—The superior strength values of the soil-lime-fly ash-activator specimens obtained with the low carbon content Chicago fly ash, as compared to strengths obtained with the high carbon Cedar Rapids fly ash, demonstrate the deleterious effect of high carbon content (Tables 8 and 10).

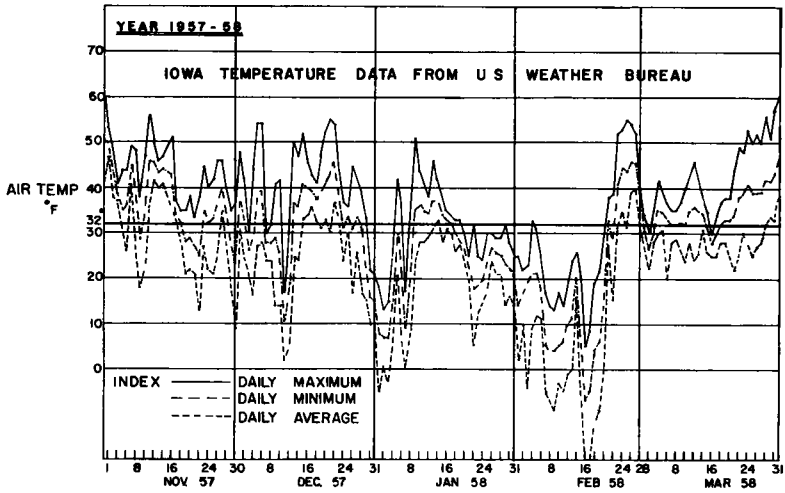


Figure 7. Winter 1957-58 temperature data compiled from U. S. Weather Bureau information.

Core Specimens.—Table 10 gives the unconfined compressive strength and moisture content data for the core specimens removed from experimental sections approximately 3 years after construction. (Removal of core specimens was attempted in April 1958, approximately 6 months following construction of the sections. With the exception of the soil-cement section, the remainder of the soil-stabilizing agent mixtures though hard and dense, would not withstand the torsional effect of the diamond core bit due to incomplete curing.) In comparing like sections, varying strength values seem to correspond directly with the moisture content of the cores. The lime-Cedar Rapids fly ash sections appeared the most adversely affected by the presence of high moisture. The increase in moisture above that at the time of construction was probably due to capillary moisture absorption from a high water table and/or infiltration of surface water.

The average moisture content in the soil-lime-fly ash sections was considerably higher than in the soil-cement base section, perhaps illustrating the relative permeability to moisture, or water-holding capacity, of these materials. However, the strengths of even the high moisture content specimens are indicative of high-bearing capacity-stabilized soil materials.

Visual inspection of core holes and specimens revealed close to design thicknesses and excellent contact between all courses of the pavement structure.

Inspection of cores taken to study surface cracks in the soil-cement and soil-lime-fly ash base and subbase sections, revealed that in some cases the cracks extended through the pavement structure to the subgrade. In two instances, the cores had broken normal to the vertical crack during drilling, revealing imprints of tamping foot roller teeth. The indentations had apparently provided planes of slight weakness which were not removed during construction by scratching the base with a spike-tooth harrow after each pass of the roller. Other cored cracks revealed from ½- to about 1½-in. thicknesses of soil material which had not been mixed with the stabilizing agent to the full, loose depth during construction. Although these conditions existed to a minor degree, and in general mixing was found to be efficient as well as effective, the importance of proper construction techniques cannot be overemphasized.

Curing

Time is required for lime and fly ash to react pozzolanically. The rate of reaction and hardening also depends on temperature, though the exact relationship has not been determined. The reaction most likely stops with temperatures at or below freezing (7).

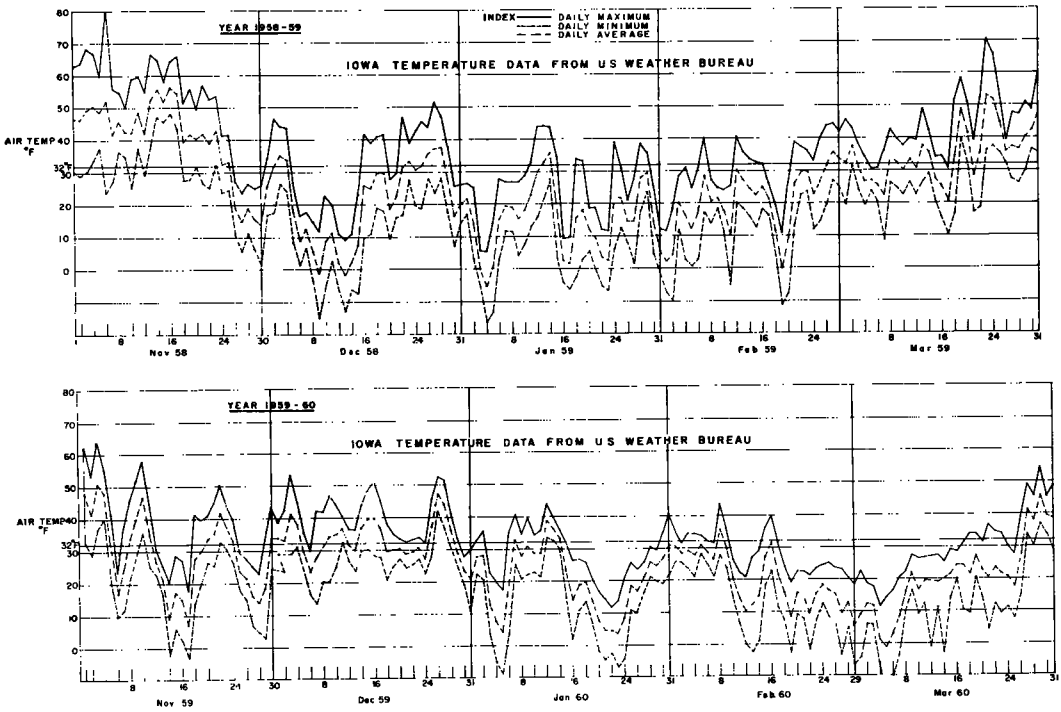


Figure 8. Winters 1958-59 and 1959-60 temperature data compiled from U. S. Weather Bureau information.

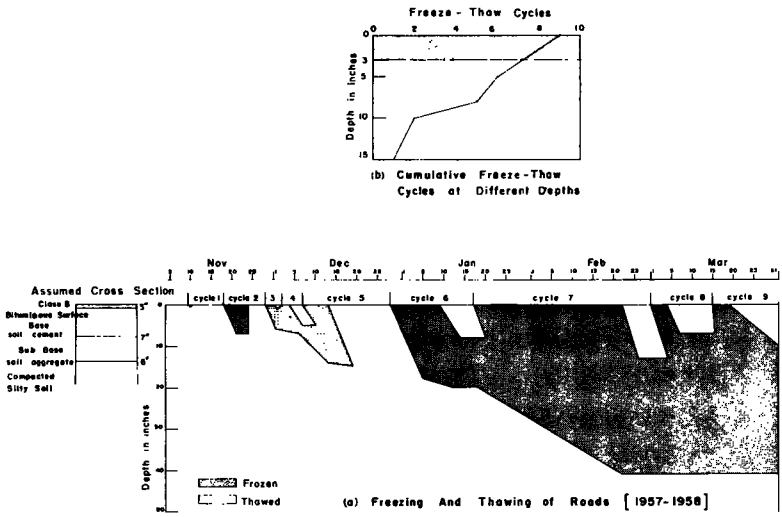


Figure 9. Winter 1957-58 computed freeze-thaw cycles.

At Colfax, the time from completion of the base course to the first freeze was about 6 days (Fig. 7). Except for several very brief intermittent periods, the average air temperature did not rise above freezing until mid-February 1958. Figure 8 shows the air temperatures to which the road was subjected during the winters of 1958-59 and 1959-60. Figures 9, 10, and 11 show the number of freeze-thaw cycles and depth of

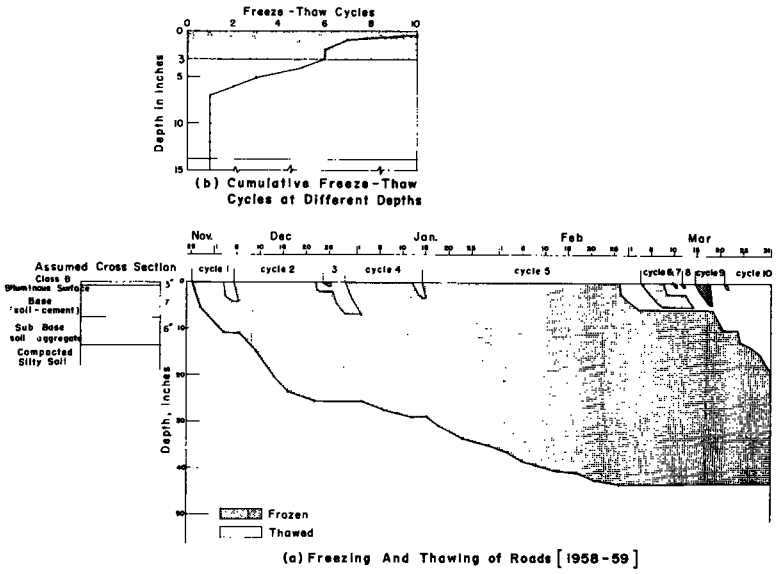


Figure 10. Winter 1958-59 computed freeze-thaw cycles.

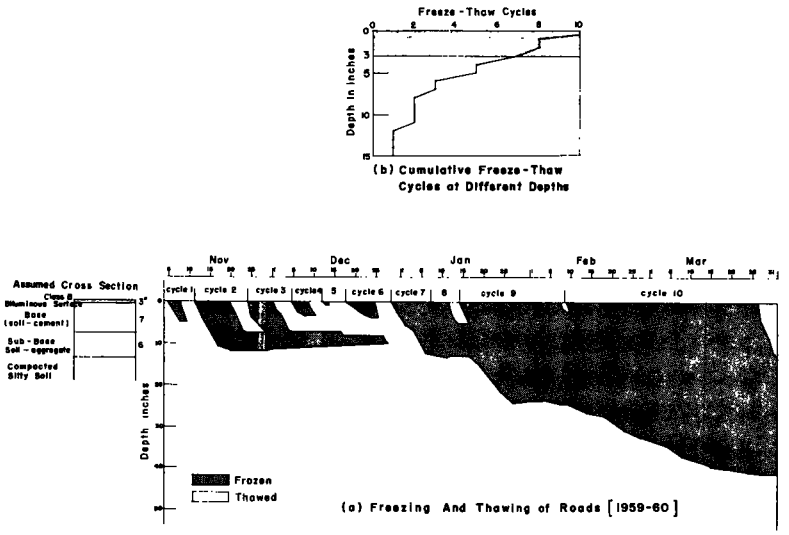


Figure 11. Winter 1959-60 computed freeze-thaw cycles.

freezing calculated for the assumed pavement using the temperature data shown in Figures 7 and 8, and the modified Berggren formula (1); these computations were considered sufficiently accurate for a qualitative analysis of the number of freeze-thaw cycles to which the test sections were subjected.

The qualitative effect of freezing on field curing can be estimated by a comparison of the unconfined compressive strengths of 6-mo and 1-yr laboratory-cured specimens molded from field mixes, to the strengths of the 3-yr field-cured core specimens from each test section (Tables 8, 9, and 10). In each case, core strength greatly exceeded laboratory-cured specimen strength, indicating that field curing of the pavement mixes continued over the 3-yr period even though interrupted by winter freezing.

TABLE 11

PAVEMENT[a] ROUGHNESS MEASUREMENTS[b]

Item	Section No.	Length (mi)	Run 1 Roughness (in./mi)	Run 2 Roughness (in./mi)
Terminus:				
Colfax RR tracks	1	1.00	71	72
	2	1.00	67	68
Approx. south end of test portion of hwy	3	1.00	58	72
Approx. north end of test portion of hwy	4	1.00	59	71
	5	1.00	55	60
	6	1.00	63	65
	7	1.00	61	70
	8	1.00	64	66
	9	1.00	66	67
	10	1.00	65	64
Jct. Iowa 64	11	0.98	63	68
Miles measured, total		10.98		
Average roughness				
Run			63	68
Test portion of pavement			65.00	
Regular pavement			65.26	

[a]May 9, 1958, from Colfax, Iowa, north to junction of Iowa 64 and Primary Highway 117.
[b]Information furnished by Iowa State Highway Commission.

During the winter of 1957-58 only a leveling course (1½ in. thick) of asphaltic concrete protected the experimental sections from traffic abrasion. This deficiency, coupled with the lack of cured strength in all the soil-lime-fly ash sections, was the cause of some isolated surface "alligator" cracking during the spring thaw of 1958. The most severe cracking occurred in the east lane of the soil-lime-Cedar Rapids fly ash-sodium carbonate base, approximately between stations 240+00 and 241+05. Before application of the additional 1½ in. of asphaltic concrete, the damaged areas of asphaltic concrete, were removed. Inspection revealed the base, in all cases to be free of cracks and in excellent condition, though in need of additional curing. The areas were resurfaced without further base treatment and no further difficulties have been observed.

TABLE 12

ROUGHNESS CRITERIA[a]

Roughness Index (in./mi.)	Riding Quality
Old pavements:	
Below 60	Excellent
60-74	Good
75-90	Fair
91-150	Poor (possible resurfacing)
Above 150	Very poor (resurfacing required)
New pavements:	
Below 75	Good (acceptable)
75-90	Fair (acceptable)
Above 90	Poor (not acceptable)

[a]From Holloway (10).

Pavement Roughness

The Iowa State Highway Commission measured pavement roughness with a roughometer during May 1958 (Table 11). The roughometer is a device which, when towed over a paved surface, is assumed to stay in a relatively fixed plane due to its own inertia. Changes in elevation are measured by means of a floating wheel which follows the paved surface and deviates from the machine proper (19). Values of pavement roughness obtained are relative, therefore, it is necessary to correlate them with known surface behavior. Table 12 gives roughness criteria suggested by Holloway (10).

A comparison of data in Tables 11 and 12 indicates that the riding quality of all sections of the road would be classified as "good." The 65.00 mean inches of measured roughness per mile within the test

TABLE 13

RESULTS OF SURFACE CRACK STUDY OF VARIOUS EXPERIMENTAL SUBBASE AND BASE COURSE SECTIONS, AND 2,300 FT OF REGULARLY DESIGNED NON-EXPERIMENTAL 7-IN. SOIL-CEMENT BASE OVER 6-IN. SOIL-AGGREGATE SUBBASE

Surface Crack		Surface Cracking per 100 Lineal Feet of Pavement (ft)																	
		Subbase Sections						Base Sections											
Location	Class[a]	0.25% Arquad 2HT		6% Lime[b]		3% Lime[b] 21% Fly Ash[c]		3% Lime[b] 21% Fly Ash[c]		3% Lime[b] 21% Fly Ash[d] 0.5% Na_2CO_3		3% Lime[b] 21% Fly Ash[c] 0.5% NaCl		3% Lime[b] 21% Fly Ash[c] 0.5% Na_2CO_3		3% Lime[b] 21% Fly Ash[c]		Non-Experimental Section[e]	
		1959	1960	1959	1960	1959	1960	1959	1960	1959	1960	1959	1960	1959	1960	1959	1960	1959	1960
Longitudinal																			
Outside edge[f]	1	37.3	74.5	42.8	50.0	13.0	34.0	10.0	0.0	8.0	12.0	6.0	3.6	0.0	10.0	0.0	6.0	33.6	22.7
	2	1.2	40.0	15.7	50.0	0.0	9.0	35.0	95.0	0.0	40.0	2.8	10.8	2.0	14.0	0.0	0.0	3.6	1.0
	3	0.0	0.0	0.0	10.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.2	0.0	0.0	0.0	0.0	0.0	0.0
Center of lane[g]	1	0.0	10.8	4.3	0.0	5.0	2.0	0.0	0.0	4.0	0.0	1.4	2.6	0.0	5.0	0.0	0.0	21.8	9.1
	2	0.0	1.9	0.0	5.7	4.0	23.0	0.0	0.0	0.0	18.0	0.0	1.4	0.0	0.0	0.0	0.0	36.3	35.4
	3	0.0	0.0	0.0	0.0	0.0	3.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	8.2
Centerline[h]	1	4.6	29.6	37.2	11.4	11.0	13.0	0.0	0.0	0.0	0.0	2.2	5.0	0.0	3.0	0.0	0.0	37.3	42.7
	2	0.0	11.2	0.0	5.6	0.0	11.0	0.0	15.0	0.0	0.0	0.0	4.2	0.0	0.0	0.0	0.0	14.6	23.6
	3	0.0	2.3	0.0	0.0	0.0	4.0	0.0	0 0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	27.3
Transverse	1	7.2	0.6	15.7	0.3	16.8	0.0	29.0	0.0	51.6	0.8	48.8	0.0	21.2	0.0	18.4	1.6	24.0	65.3
	2	59.5	69.1	54.8	10.6	44.4	1.4	67.0	11.0	30.0	36.8	21.6	49.0	39.6	18.4	51.6	20.4	54.3	24.0
	3	0.0	6.0	2.3	62.8	0.0	60.2	0.0	87.0	0.0	44.8	0.0	26.5	0.0	44.8	9.6	79.6	4.4	52.1

[a]All pavement surface cracks plotted on a scaled sketch of sections noted. Class indicates width of crack 1 = 0-1/8 in., 2 = 0-1/4 (with some raveling, 3 = 0-1/2 (with definite raveling).

[b]Monohydrate dolomitic type.

[c]Cedar Rapids fly ash.

[d]Chicago fly ash.

[e]Regularly designed 7-in. soil-cement base over 6-in. soil-aggregate subbase, 2,300 ft long, immediately adjacent to north end of experimental sections and including both cut and fill subgrade conditions.

[f]Outside edge indicates 0 to 3 ft from edge of each lane.

[g]Center of lane indicates 3 to 10 ft from pavement edge.

[h]Centerline indicates a strip 1 ft on each side of centerline.

TABLE 14

SUMMARY OF CRACK STUDY

Agent	Section (ft)	Total Length of Cracking (ft/100 lin ft of pavement)					
		Longitudinal			Transverse		
		April 1959	Dec. 1960	Increase	April 1959	Dec. 1960	Increase
Subbase-stabilizing							
0.25% Arquad 2HT	1,300	43.1	170.2	127.1	66.7	75.7	9.0
6% lime	700	100.0	132.7	32.7	72.8	73.6	0.8
3% lime, 21% Cedar Rapids fly ash	1,000	33.0	99.0	66.0	61.2	61.6	0.4
Base-stabilizing							
3% lime, 21% Cedar Rapids fly ash	200	45.0	110.0	65.0	96.0	98.0	2.0
3% lime, 21% Chicago fly ash, 0 5% Na_2CO_3	500	12.0	70.0	58.0	81.6	82.4	0.2
3% lime, 21% Cedar Rapids fly ash, 0 5% NaCl	1,000	12.4	29.8	17.4	70.4	75.5	5.1
3% lime, 21% Cedar Rapids fly ash, 0 5% Na_2CO_3	500	2.0	32.0	30.0	60 8	63.2	2.4
3% lime, 21% Cedar Rapids fly ash	500	0.0	6.0	6.0	79.6	101.6	22.0
Typical non-experimental section[a]	2,300	147.2	176.0	28.8	82.7	141.4	58.7

[a]Includes cut and fill

sections compares favorably to the 65.26 mean inches of measured roughness for the portions of the road outside the test area.

Surface Crack Studies

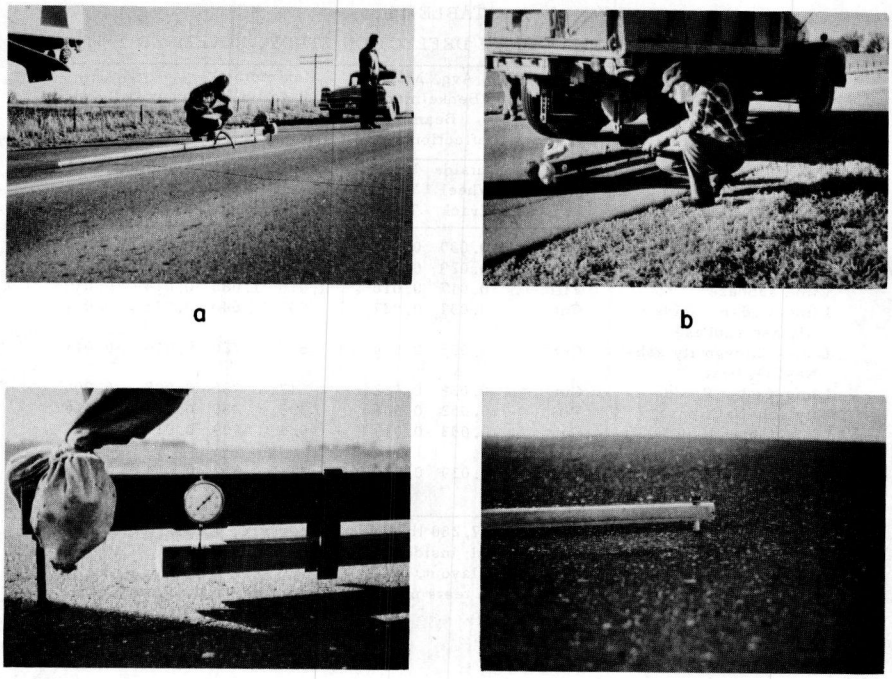
Two studies of pavement surface cracking were made on the experimental portions of the highway and on 2,300 ft of regularly designed pavement immediately north of the experimental sections. These studies were conducted during April 1959 and December 1960 as a means of measuring the comparative performance of the experimental pavement with that of the regularly designed pavement, and also as a means of evaluating possible pavement structural failure.

From visual examination and measurement the class and length of surface cracking in each test section was determined, differentiating between transverse and longitudinal cracks. Three designations were employed to classify the cracks by width: Class 1, 0 to ⅛ in.; Class 2, 0 to ¼ in. (with some raveling); and Class 3, 0 to ½ in. (with definite or pronounced raveling).

Table 13 gives a breakdown of the surface cracking according to the three classes, for both studies, and Table 14 gives a summary of the total cracking per each experimental section, and the increase in development of cracks over the 1½ years between studies.

Comparison of the two studies, indicates increased longitudinal and transverse cracking in all surface areas of the experimental portion of the highway since the first study in April 1959. Comparison of the subbase test sections (overlain by the regularly designed soil-cement base) indicates surface cracking over the lime-fly ash portion was less than the surface cracking over either the Arquad 2HT or the lime-stabilized portions. The largest increase in total longitudinal cracking occurred in the surface over the Arquad 2HT-stabilized section, whereas the largest increase in total transverse cracking occurred in the surface of the regularly designed pavement. The greatest total surface cracking (that is, both longitudinal and transverse) as of December 1960 was in the regularly designed portion of the pavement north of the experimental sections.

Comparison of the soil-lime-fly ash-stabilized base sections indicates the least total



a b c d

Figure 12. Benkelman beam deflection test. (a) Side view of beam. Deflection reading being taken with left rear dual of test truck at point 5 ft in front of probe of beam. (b) Zero reading being taken for outside wheel track. Beam placed between dual wheels of test truck having a rear axle load of 17,280 lb. (c) Rear view of beam showing extensiometer for determining deflections, and bag of lead weights for stabilizing beam carriage. (d) Probe of beam placed in direct contact with surface after surface is brushed free of loose material.

surface cracking in the sections where the Cedar Rapids, high carbon content, fly ash was used.

Some transverse and longitudinal cracking is to be expected in soil-cement, and to a more limited extent, in soil-lime fly ash if enough hardness is attained. The reasons are not yet completely known. Thermal expansion and contraction affect the width of cracks (widest in winter) whereas some cracking can be created by uneven settlement and/or shrinkage and swelling of subgrade materials. Soil-cement cracks are not regarded as failures unless structural evidence of failure can be attributed to the cracks (13). The principle objection to cracks is their reflection through a bituminous surface.

Some longitudinal cracking near the outside edges of the pavement was caused by heavy construction equipment, particularly scoops, which rode the pavement edges during shoulder construction in the spring of 1958. The overloading of the outside edges apparently caused shear and/or flexural failures, particularly along the slower curing portions of the experimental sections.

Lack of surface crack maintenance has undoubtedly permitted surface moisture to penetrate into and weaken the total pavement structure. However, an over-all observation of the test areas and the adjoining regularly designed roadway is that cracking is generally less in the test sections.

Flexural Strength Analysis

To analyze the flexural strength of the experimental sections, a Benkelman beam (2) was used to measure deflections induced by a 17,280-lb single rear-axle load of an Iowa

TABLE 15

RESULTS OF BENKLEMAN BEAM DEFLECTION STUDY, NOVEMBER 1960[a]

General Test Section[b]	General Subgrade Condition	Avg. Max. Benkelman Beam Deflection[c] (in.)		Maximum Relative Stiffness[d] (kips/in.)		Avg. Benkelman Beam Deflection 5 Ft Ahead of Probe (in.)	
		Outside Wheel Track	Inside Wheel Track	Outside Wheel Track	Inside Wheel Track	Outside Wheel Track	Inside Wheel Track
Arquad 2HT subbase	Cut	0.039	0.026	443	665	0 022	0.019
	Fill	0.029	0.022	596	785	0 016	0.012
Lime subbase	Fill	0.017	0.016	1,016	1,080	0 014	0.011
Lime-Cedar Rapids fly ash subbase	Cut	0.031	0.027	557	640	0.022	0.017
Lime-Chicago fly ash-Na_2CO_3 base	Cut	0.028	0.024	617	720	0.010	0.011
Lime-Cedar Rapids fly ash-NaCl base	Cut	0.028	0.022	617	785	0.008	0 006
	Fill	0.052	0.036	332	480	0.022	0.016
Lime-Cedar Rapids fly ash-Na_2CO_3 base	Cut	0 033	0.019	524	909	0.012	0.007
Lime-Cedar Rapids fly ash base	Cut	0.039	0.028	443	617	0.014	0 010

[a]Rear single-axle load of test truck, 17,280 lb, tire air pressure, 75 psi, tire contact pressure outside wheel track 79.4 psi, inside wheel track, 77.6 psi.
[b]See Figure 2 for detailed test section layout.
[c]Average of minimum of two deflection tests per wheel track, per traffic lane, per subgrade condition in each test section.
[d]Load divided by average deflection.

State Highway Commission test truck (Fig. 12). Because the maximum allowable single-axle in Iowa is 18,000 lb, deflections thus found were near maximum values.

With the rear duals of the test truck placed so that loading in each traffic lane occurred at points 2 and 9 ft from the pavement edge, the beam was placed between each rear dual wheel, in turn. Three readings were observed and recorded for the inside and outside wheel tracks at each of the following locations: at the point of placement (zero reading) at maximum deflection as the truck moved slowly ahead past the probe and at the point when the rear axle of the truck reached 5 ft in front of the beam probe. The deflection for each location was taken as twice the mean of the maximum reading minus the zero reading of each wheel track, because the deflection shown by the extensiometer is one-half the actual deflection. Air and pavement surface temperatures were recorded for each station; however, due to an overcast sky, temperature variance during the entire test operation was negligible, showing an average surface temperature of 60 F.

Table 15 gives the results of the Benkelman beam deflection studies as conducted in early November 1960. As expected, the outside edges showed the greatest deflection; that of the soil-lime-Cedar Rapids fly ash-NaCl section over fill showed an average maximum deflection of 0.052 in. A limiting deflection of 0.2 in. has been selected by the Navy Department for use in the design of flexible pavements (19). As a qualitative measure of the flexibility of an experimental section, a relative stiffness factor was computed by dividing the axle load by the maximum deflection; the more flexible the material, the lower the relative stiffness factor. Using the axle load of the test truck (17,280 lb), the maximum relative stiffness for the Navy Department design criteria (19) would be

TABLE 16

AVERAGE ATTERBERG LIMITS[a] OF SUBGRADE AND ARQUAD 2HT-TREATED SUBBASE SOILS SAMPLED OVER 3 YEARS FOLLOWING CONSTRUCTION

Arquad 2HT Additive (%)	Atterberg Limit (%)		
	Liquid Limit	Plastic Limit	Plasticity Index
0.00 (subgrade)	45.6	21.1	24.5
0.25 (subbase)	38.9	22.5	16.4

[a]Performed on minus No. 40 sieve fraction in accordance with ASTM designations D423-59T and D424-59.

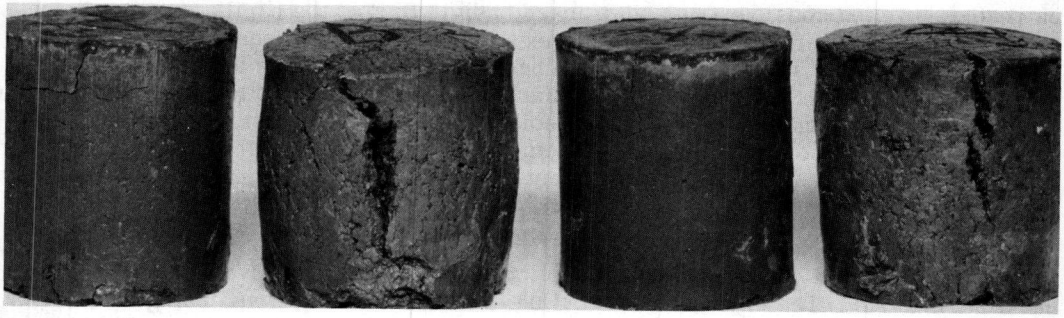


Figure 13. Alternately, 3-yr-old Arquad 2HT-treated subbase soil and untreated subgrade soil molded into 2-in. diameter by 2-in. high specimens. Specimens molded to standard Proctor density, air cured for 7 days, and subjected to capillary moisture absorption in open air for 7 days.

86.4 kips per in.; the lowest relative stiffness factor of the experimental sections was 332 kips per in. Thus the experimental sections appear to have adequate flexural strength for performance under 18,000-lb single-axle loads.

Traffic

Iowa State Highway Commission traffic information for the portion of Iowa 117 which includes the experimental sections is as follows:

From 1956 to 1959 the average daily traffic increased from 425 to 836 vehicles per day. Of the latter total, 556 were local passenger cars, 35 were foreign passenger cars, 42 were pickups and panels (subtotal, 633); 14 were 4-tire and 114 were 6-tire 2-axle single-unit trucks, 10 were 3-axle single-unit trucks; 19 were 3-axle truck tractor semi-trailer combinations, 36 were 2-axle TT, 2-axle ST truck tractor semi-trailer combinations, and 7 were 5-axle combinations, 3 were busses (subtotal 203). Total section length for 1959 was 3.43 mi, truck and bus vehicle-miles were 696, and total vehicle-miles were 2,867. In late 1960, Interstate 80 was opened between Des Moines and Newton with interchange facilities at Iowa 117. As of mid-1961 the average daily traffic over the experimental sections had increased to 874 vehicles per day.

Arquad 2HT-Treated Subbase

In May 1961 over three years after construction, the Arquad 2HT subbase section was given some additional testing and evaluation due to its more highly experimental nature.

Several 4-in. diameter core holes were opened in the soil-cement base overlying the Arquad 2HT subbase. Readings of a pocket penetrometer (Model CL-700, Soiltest, Inc., Chicago, Ill.) were used to qualitatively measure the general strength characteristics of the exposed subbase in each hole. The average unconfined compressive strengths thus determined were over 3.5 tons per sq ft.

Disturbed samples of the full depth of the subbase were removed by auger from each hole, sealed in containers, and returned to the laboratory for determination of Atterberg limits and capillary moisture absorption observations of remolded 2-in. diameter by 2-in. high cylinders compacted to near standard density. From each of the same holes the upper 6 in. of subgrade was also removed by auger, sealed in containers, and returned to the laboratory for tests duplicating those noted. Duplicate 5- to 10-g samples of these materials were removed directly from the auger for bacteriological studies reported in the Appendix. Table 16 presents the results of the Atterberg limits tests on the disturbed samples.

All remolded cylindrical specimens were air cured for 7 days and then placed on water soaked felt pads, subjecting them to capillary absorption in open air for 7 days. Figure 13 shows the effect of capillary moisture on four of the specimens. The remolded Arquad 2HT-treated soil specimens showed only slight swelling; the same soil,

untreated, swelled considerably and cracked badly. During the capillary test, the untreated specimens became fully wetted in less than 30 min, whereas the 3-yr-old Arquad-treated soil, required 4 to 5 hr to become fully wetted due to its retained hydrophobic properties. During the performance of the plastic limit test, it was observed that the rate of drying of the Arquad 2HT soil was considerably faster than that of the untreated subgrade soil. At the time of removal of samples from the subbase and subgrade the average in-place moisture contents were 21.7 and 24.1 percent, respectively.

CONCLUSION

The experimental base and subbase sections of Iowa 117 have given over 3 years of excellent service. The experimental portion has sustained severe freezing and moisture conditions and has shown a general performance at least equal to that of the regularly designed pavement structure employing a 7-in. soil-cement base and a 6-in. soil-aggregate subbase.

ACKNOWLEDGMENTS

The subject matter of this report was obtained as part of the research being done by the Iowa Engineering Experiment Station, Soil Research Laboratory, Iowa State University, under sponsorship of the Iowa Highway Research Board, Iowa State Highway Commission.

Special appreciation is extended to the following people and organizations: the Iowa Highway Research Board and the Iowa State Highway Commission for the opportunity to field test these soil stabilizing agents; Hallett Construction Company for the cooperation and interest shown during the construction phase of the field trials; those members of the Iowa Engineering Experiment Station staff who assisted in the field laboratory at time of construction and those who assisted in gathering field performance information; and the following companies who generously donated their respective products for use in these field trials: Armour Industrial Chemical Company, Chicago (Arquad 2HT); Walter N. Handy Company, Springfield, Mo. (fly ash); Morton Salt Company, Chicago (sodium chloride); and Dow Chemical Company, Midland, Mich. (sodium carbonate).

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Appendix

BACTERIAL COUNTS OF ARQUAD 2HT-TREATED SUBBASE

P.A. Hartman, Associate Professor of Bacteriology
Iowa State University

Materials and Methods

In May 1961, four core holes were opened in the soil-cement base exposing the Arquad 2HT subbase of Iowa 117. Two core holes were also opened exposing the soil-aggregate subbase immediately south of the experimental sections. From each of the six holes four 5- to 10-g samples were collected into sterile screw-cap test tubes for bacteriological analysis; one sample each, from near the top and bottom of the Arquad 2HT subbase, near the top of the subgrade and about 6 in. into the subgrade. The samples were kept cool until returned to the laboratory for storage at 4 C before plating about 6 to 11 hr later.

One gram of each sample was shaken 100 times with 99 ml of 0.1 percent peptone buffer (1). Subsequent dilutions were made in peptone buffer, after which appropriate dilutions were plated in duplicate into Trypticase soy agar (Baltimore Biological Laboratory, Inc., Baltimore, Md.) and inoculated in triplicate into tubes of steamed and cooled Thioglycollate medium (without indicator) (Difco Laboratories, Inc., Detroit, Mich.). In like manner, two media containing 0.25 percent added Arquad 2HT were inoculated, the pH of the Arquad media being adjusted so that it was 7.2 following sterilization of the media.

Trypticase soy agar plates were incubated for 4 days at 30 C. Tubes of Thioglycollate medium were incubated in candle jars for 7 days at 30 C. Total aerobic and anaerobic counts were calculated using 3-tube most probable numbers tables (2, p. 152).

Results

Data obtained from the tests are given in Table 17. Bacterial counts on the medium containing 0.25 percent added Arquad 2HT were negligible and thus were omitted from Table 18. Differences in moisture content of the various roadway samples would not have affected bacterial counts sufficiently to alter any conclusion made in this report.

It would appear that the vast majority of the microorganisms encountered in this study were susceptible to the presence of Arquad 2HT in the medium. In addition,

TABLE 17

BACTERIAL COUNTS ON SAMPLES OF ARQUAD 2HT-TREATED SUBBASE SOIL-AGGREGATE SUBBASE, AND UNDERLYING SUBGRADE, IOWA 117

Type of Bacteria	Sample Location		Type of Subbase[a]	Logarithm of Bacterial Count per Gram Wet Weight of Soil			
				Subbase		Subgrade	
	Station	Lane		Upper	Lower	Upper	Lower
Aerobes	193+50	East	A	4.54	4.78	5.48	4.28
	198+50	East	A	4.59	4.57	4.83	3.65
	200+50	West	A	4 60	4.66	4.78	4.66
	196+00	West	A	4.79	4.71	4.80	4.38
	187+00	West	B	4 68	4.86	4.40	4.49
	184+00	East	B	4.77	5.00	5.08	5.29
Anaerobes	193+50	East	A	3.40	5.04	4.40	3.65
	198+50	East	A	5.04	5.04	4.65	3.40
	200+50	West	A	4.65	4.65	4 40	4.65
	196+00	West	A	3.65	3.65	3 40	3.65
	187+00	West	B	4.65	>5.40	3.40	4.40
	184+00	East	B	4 40	4.65	>5.40	>5.40

[a]A = Arquad 2HT-treated subbase, B = soil-aggregate-stabilized subbase.

bacterial counts of all samples were rather uniform, indicating that the presence of Arquad in the subbase resulted in no long-term net increase or decrease in the quantity of microorganisms present.

The results, however, do not preclude the possibility that some inhibition or enrichment of the microbial population might have occurred early in the history of the road's construction and utilization. This possibility is suggested by the fact that slightly higher counts of aerobes were obtained in the upper portion of the subgrade, just under the Arquad 2HT-treated subbase, than in the other areas of sampling. Furthermore, substantial increases in bacterial counts were noted in Arquad-treated, but not in untreated, soil samples held for some time in the laboratory. Bacteria that were resistant to the Arquad could be isolated with ease from both types of samples using enrichment procedures where Arquad 2HT served as the sole carbon source.

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A Method for In-Place Mix Control in Reconstruction of Soil-Aggregate Roads

J. B. SHEELER, Associate Professor of Civil Engineering, Iowa State University

The design method in this paper is specific for the case in which an old aggregate surface is to be scarified and incorporated in the new surface and the binder material is taken from the roadbed. The quantities of existing material are calculated from trench sample data. Equations are presented from which quantities of additional material are calculated and spotted. Quantitative control is accomplished by requiring a percentage of material finer than some specific size in the final mixture. The quantities also depend on the dimensions desired.

• SEVERAL METHODS of blending natural soil materials are used in the construction of the wearing surface for secondary roads. In the most common methods an aggregate and an imported clayey soil are mixed by some means on a newly constructed grade or the materials are plant-mixed before placement. The methods for proportioning materials are well established for soil aggregate mixtures, and the stationing of materials for the preceding construction procedures is easily calculated. An excellent method of blending natural earth deposits is given by Ritter and Shaffer (5). The method requires an electronic computer and uses linear programming for determining materials and sources for least cost. Inasmuch as most county engineers do not have ready access to an electronic computer and are not trained in linear programming, this method is somewhat limited in use for secondary road construction.

In another method of secondary road construction a worn soil-aggregate surface is salvaged; the old surface is scarified, new aggregate is added to the old material, and the deficiency of fine binder material in the resulting mixture is corrected by incorporating some soil from the subgrade. Usually the soil from the subgrade is loosened during the scarification process and is bladed into a windrow with the salvaged aggregate before the new material is added. This paper describes a rational method of determining the material placement needed to obtain a reasonably uniform thickness and gradation throughout a roadway to be constructed by the procedure just described. The method may also be used in chemical stabilization of soil-aggregate materials.

DEVELOPMENT OF EQUATIONS

In the utilization of granular surface material in the construction of new surface courses the engineer has problems of quality and quantity control. Because the amount of granular material varies from place to place, the cross-sectional area of the granular material varies throughout the length of a road. Therefore, if the surface is scarified to a constant depth over the length of the road, the amount of soil scarified from the subgrade will vary from place to place, and the amount of fines in the mixture will vary considerably. This variation can be greatly reduced by creating a constant cross-sectional area of granular material over the length of the road. Scarification to a constant depth will then produce a constant cross-sectional area of loosened subgrade soil, and the amount of fines will be relatively constant over the entire road.

The amount of granular material located at all points on a road must be known in order to have control over the quantities of material involved at any point in question. The amounts of granular materials may be determined by digging trenches normal to

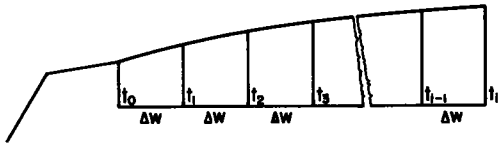

Figure 1. Cross-section of sampling trench showing meaning of symbols and subscripts used in calculating average thickness of granular material.

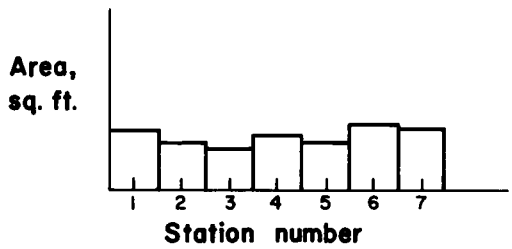

Figure 2. Histogram of trench sample data Trenches are located at mid-point of each section and identified by station number

the centerline of the road. A determination of the exact amount of material is out of the question because of the great number of trench samples required. Therefore, some accuracy must be sacrificed in the interests of economy, trench samples must be taken at intervals and assumed to be representative of their respective intervals.

The cross-sectional area of granular material at each trench sample location may be determined by dividing the length of the trench into i subintervals Δw long, and measuring the thickness t at each of these subintervals as shown in Figure 1. The total cross-sectional area is then the summation of the small areas. However, it is more convenient and useful to measure the full width of the granular material and to determine an average thickness as follows:

The area of increment No. 1 is $(\frac{t_0+t_1}{2})\Delta w$, if the increment is considered to be trapezoidal, that of increment No. 2 is $(\frac{t_1+t_2}{2})\Delta w$, and so on. The total area is then

$$\text{Area} = \frac{t_0+t_1}{2}\Delta w + \frac{t_1+t_2}{2}\Delta w + \ldots\ldots\ldots\ldots + \frac{t_{i-1}+t_i}{2}\Delta w$$

$$= \Delta w\left(\frac{t_0}{2} + \frac{t_1}{2} + \frac{t_1}{2} + \frac{t_2}{2} + \frac{t_2}{2} + \ldots\ldots + \frac{t_{i-1}}{2} + \frac{t_{i-1}}{2} + \frac{t_i}{2}\right)$$

$$= \Delta w\left[\frac{t_0+t_1}{2} + \sum_{i=1}^{i-1} t_i\right] \qquad (1)$$

The area can also be expressed in terms of an average value of thickness, t_{avg}:

$$\text{Area} = (i)\,(\Delta w)\,t_{avg} \qquad (2)$$

Equating Eqs. 1 and 2 gives

$$t_{avg} = \frac{\frac{t_0+t_i}{2} + \sum_{i=1}^{i-1} t_i}{(i)} \qquad (3)$$

Since (i) (Δw) is equal to the width w, the cross-sectional area becomes equal to wt_{avg}.

Henceforth, the average thickness is designated by t, and subscripts on t and w refer to the stations. The amounts of granular material on a road may be represented by a histogram in which the cross-sectional area is plotted against distance or station number (Fig. 2).

The addition of fresh granular material to a road increases the cross-sectional area as shown in Figure 3. By carefully controlling the quantities of material added to each section, the cross-sectional area of the total granular material can be made constant throughout the length of the road. The amounts of granular material can now be represented by the plot of cross-sectional area vs distance shown in Figure 4. Scarification to a constant depth (d) by using the fresh surface as a reference plane (see Fig. 3) then

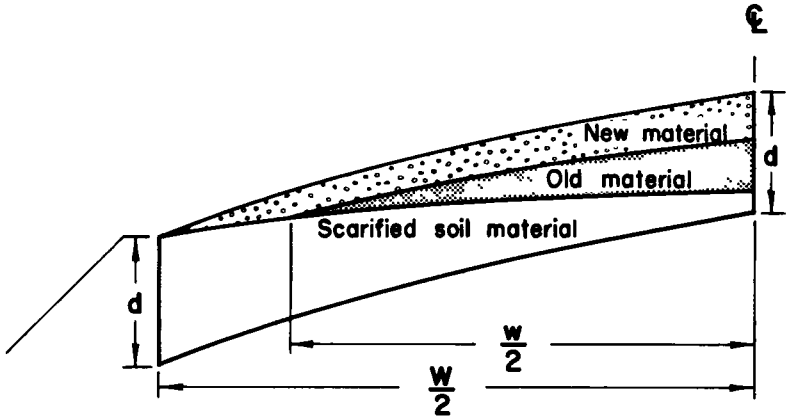

Figure 3. Cross-section of a roadway surface.

insures a constant cross-sectional area of loosened soil to be mixed with the overlying old and new granular materials.

Because the amount of any material in a section is the unit weight times the volume, the total amount of existing granular material G can be calculated by summing the amounts in each section.

$$G = \gamma_G \Delta L w_1 t_1 + \gamma_G \Delta L w_2 t_2 + \ldots\ldots + \gamma_G \Delta L w_n t_n + \gamma_G (L - n\Delta L) w_n t_n \tag{4}$$

in which
L = total length in feet;
ΔL = length of a section in feet;
n = number of sections; and
γ_G = average unit weight of the in-place granular material in pounds per cubic foot.

The final term represents a remainder that must be included, because the length of a section is seldom an exact divisor of the length of the road. The values of the last trench sample w_n and t_n are assumed to also represent the final segment. Because γ_G and ΔL are constant, they may be factored from the equation so that

$$G = \gamma_G \Delta L \sum_{n=1}^{n} w_n t_n + \gamma_G (L - n\Delta L) w_n t_n \tag{5}$$

The quantities of materials necessary for a desired final mixture must be proportioned by some means. Davidson and Sheeler (2, 3) indicate that the plasticity index of a given type of soil is linearly dependent on the amount of clay present in the soil. The relationship is also dependent on the type of clay mineral. If a given soil changes in gradation from place to place but the type of minerals remains constant, the linearity of the plasticity index to clay relationship is preserved. The amount of clay contained by the materials then appears to be a useful control factor. The percent clay to be expected in a mixture of old granular material, new granular material, and scarified soil is given by

$$p = \frac{\frac{p_G}{100} G + \frac{p_A}{100} A + \frac{p_S}{100} S}{G + A + S} \times 100 \tag{6}$$

in which
G = quantity of in-place granular material in pounds;
A = quantity of added granular material in pounds,

S = quantity of scarified soil in pounds;
p = desired percent clay in the final mixture;
p_G = clay content of G, percent;
p_A = clay content of A, percent; and
p_S = clay content of S, percent

Revision of Eq. 6 gives

$$S = \left(\frac{p - p_G}{p_S - p}\right) G + \left(\frac{p - p_A}{p_S - p}\right) A \tag{7}$$

or

$$S = K_G G + K_A A \tag{8}$$

in which

$$K_G = \frac{p - p_G}{p_S - p} \tag{9a}$$

and

$$K_A = \frac{p - p_A}{p_S - p} \tag{9b}$$

If the final dimensions (in feet) of the road are to be T = thickness, W = width, and L = length, and the material is to be compacted to a unit weight γ (in pounds per cubic foot) the total weight of the road must be γ LWT or the sum of G, S and A from which

$$\gamma\ LWT = G + S + A \tag{10a}$$

Substitution of Eq. 7 gives

$$\gamma\ LWT = G + A + (K_G G + K_A A) = (1 + K_G)\ G + (1 + K_A)\ A \tag{10b}$$

Rearranging Eq. 10b shows the necessary addition to be

$$A = \frac{\gamma\ LWT - (1 + K_G)\ G}{1 + K_A} \tag{11}$$

To control the cross-sectional area throughout the length of a road so that the cross-sectional area of the scarified soil will also be constant at all points, several more equations must be developed. This may be done by using a synthetic average unit weight γ_{AG} of the new granular material plus the in-place granular material. The total weight of these materials is given by $\gamma_{AG} L\ (wt)_{avg}$ where wt_{avg} represents the average cross-sectional area produced if the two materials were combined and spread evenly over the length L and compacted to γ_{AG}. The total weight is also given by G + A. The average unit weight is the total weight divided by the total volume so that

$$\gamma_{AG} = \frac{G + A}{\frac{G}{\gamma_G} + \frac{A}{\gamma_A}} = \frac{\gamma_A \gamma_G (G + A)}{\gamma_A G + \gamma_G A} \tag{12}$$

Also,

$$\gamma_{AG} L (wt)_{avg} = G + A \tag{13}$$

Substitution of Eq. 12 in Eq. 13 gives

$$(wt)_{avg} = \frac{\gamma_A G + \gamma_G A}{\gamma_A \gamma_G L} \tag{14}$$

Eq. 11 gives the amount of fresh granular material required to add to the existing granular material and the scarified soil to make a road with a unit weight γ, a thickness T, a width W, and a length L. The depth of scarification required to give the weight of soil S necessary to produce a roadway surface of these dimensions can be found as follows:

Wd = total area scarified, square feet;
$(wt)_{avg}$ = area of granular material, square feet; and
$Wd - (wt)_{avg}$ = area of scarified soil, square feet

The weight of the scarified soil is then

$$S = \gamma_S L \left[Wd - (wt)_{avg} \right] \tag{15}$$

Substitution of Eq. 8 in Eq. 15 gives

$$d = \frac{K_G G + K_A A}{\gamma_S LW} + \frac{(wt)_{avg}}{W} \tag{16}$$

All equations necessary for the calculation and distribution of all quantities of materials have now been derived.

SAMPLE PROBLEM

The proper use of the equations is illustrated by working through a sample problem. First, the trench sample data (widths and thicknesses) must be obtained as well as mechanical analyses and unit weights of the soil and aggregates. The trench sample data are best obtained by digging short trenches on opposite sides of the road to locate the edges of the granular surface. Then, assuming that the cross-section is symmetrical, the trench is extended from one side of the center of the road, and the necessary measurements for computing the average thickness are taken and the width measured. Errors introduced by the symmetry assumption can be minimized by trenching on alternate sides. Baylard (1) used a similar system in a performance study of calcium chloride-treated roads.

The mechanical analyses and unit weights of the subgrade must be determined as well as the same values for the granular material to be added. The unit weight of the additional granular material γ_A must be determined in a noncompacted state, because this is the way in which the material will be used before mixing.

The trench sample data used to illustrate the application of the equations are given in column 2 of the schedule of materials in Table 1. The values of w and t have been determined and appear as w times t in column 2. The trench samples are 500 ft apart, the proposed road will be 3.5 in. thick, compacted to 130 pcf over a width of 24 ft and a length of 1 mi. The unit weights of materials are as follows: the in-place granular material is 130 pcf, the additional granular material is 100 pcf (noncompacted), and the in-place soil is 98 pcf. The proportioning of materials will be on the basis of a 9 percent 5-μ clay content in the final mixture. The mechanical analyses and plasticity indexes of the three materials are given in Table 2 which shows the 5-μ clay contents of the materials as 6 percent p_G, 3 percent p_A, and 30 percent p_S. A specified minimum addition rate of new material is 500 tons per mile or 47.35 tons per 500-ft section which amounts to a cross-sectional area of 1.89 sq ft for a 500-ft section (calculated with γ_A = 100 pcf).

The control coefficients are calculated from Eqs. 8 and 9b as follows:

$$K_G = \frac{p - p_G}{p_S - p} = \frac{9 - 6}{30 - 9} = \frac{3}{21} = \frac{1}{7};$$

$$K_A = \frac{p - p_A}{p_S - p} = \frac{9 - 3}{30 - 9} = \frac{6}{21} = \frac{2}{7};$$

$$1 + K_G = 8/7; \text{ and}$$

$$1 + K_A = 9/7.$$

The total weight of the in-place granular material is found from Eq. 5:

TABLE 1

SCHEDULE OF MATERIALS

Station	$w_n t_n$ (sq ft)	Remove (sq ft)	Remove at 130 Pcf (tons)	Possible to Add Old Material: Sq Ft	Possible to Add Old Material: At 100 Pcf (tons)	Add Old Material at 100 Pcf (tons)	Source of Material in Col 7 (station)	Old Material Added (sq ft)	Total Old Material (sq ft)	Add New Material: Sq Ft	Add New Material: Tons	Corrected Addition (tons)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
2 + 50	2 50								2 50	4.07	101 8	102 6
7 + 50	2.12								2 12	4 45	111 2	112 0
12 + 50	2.59								2 59	3 98	99 5	100 3
17 + 50	2 37								2 37	4 20	105 0	105 8
22 + 50	4.12								4 12	2 45	61 3	62 1
27 + 50	3.81			0 87	21 8	21 8	32 + 50	0 87	4 68	1 89	47 3	48 1
32 + 50	5.97	1 29	41 9						4 68	1 89	47 3	48 1
37 + 50	4 24			0 44	11 0	11 0	32 + 50	0 44	4 68	1 89	47 3	48 1
42 + 50	2 71			1 97	49 3	9 1	32 + 50	0 36	3 07	3 50	87 5	88 3
47 + 50	2 79								2 79	3 78	94 5	95 3
	33 22											
52 + 50	2 51								2 51	4 06	56 8	57 3
			41 9			41 9					859 5	868 0

$$G = (130)\,(500)\,(33.22) + 130\left[5{,}280 - (10)\,(500)\right](2.51)$$
$$= 2{,}159{,}300 + 91{,}400 = 2{,}250{,}700 \text{ lb.}$$

The amount of additional aggregate material needed is found from Eq. 11:

$$A = \frac{(130)\,(5{,}280)\,(24)\,(3\ 5/12) - (8/7)\,(2{,}250{,}700)}{9/7} = 1{,}736{,}400 \text{ lb} = 868.2 \text{ tons.}$$

The average cross-sectional area after the addition of all material is found from Eq. 14:

$$wt_{avg} = \frac{(100)\,(2{,}250{,}700) + (130)\,(1{,}734{,}400)}{(100)\,(130)\,(5{,}280)} = 6.57 \text{ sq ft.}$$

Because the specified minimum addition rate is 500 tons per mile, which makes a cross-section of 1.89 sq ft, the maximum cross-sectional area to which 1.89 sq ft can be added to give the average cross-section is 4.68 sq ft (6.57 - 1.89). This value is then compared with those in column 2 of Table 1, and the value at station 32 + 50 is found to exceed 4.68 by 1.29 sq ft which is entered in column 3 and converted to tons in column 4 by

$$\frac{1.29 \text{ sq ft} \times 500 \text{ ft} \times 130 \text{ pcf}}{2{,}000 \text{ lb per ton}} = 41.9 \text{ tons.}$$

Column 5 is found by subtracting the values in column 2 from 4.68 sq ft. Only the sections immediately adjacent to the station having a surplus of material need be considered. The values in column 5 are then converted to tons in column 6 and an allocation of this material is made in column 7. The sum of the values in column 7 equals 41.9 tons in this case. If there were several surplus stations indicated in column 4, the sum of the tons of redistributed material indicated in column 7 should equal the sum of the surplus in column 4. Column 8 indicates the source of the material in column 7. The values shown in column 7 are converted back to square feet and entered in column 9. The values in column 9 are added to those in column 2 to give the total cross-sectional area of old material in each section after redistribution.

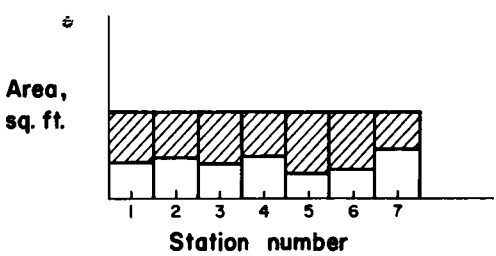

Figure 4. Histogram representing constant cross-sectional area of granular material after new granular material has been added.

Column 11 is found by subtracting the values in column 10 from 6.57 sq ft, the final cross-sectional area of the aggregate only. The values of column 11 converted to tons are entered in column 12.

The sum of column 12 will be slightly less than the calculated amount of additional material. The difference is mainly due to

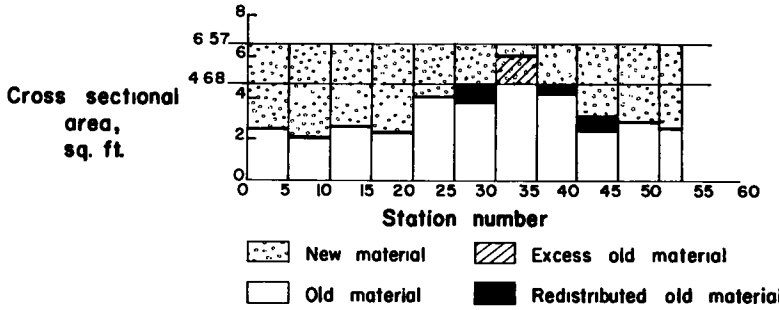

Figure 5. Sample problem cross-sectional areas of granular material after redistributing old material and adding new material.

the change in volume caused by redistributing the in-place granular material. The change in volume involved in this example is an increase of 0.3 cu ft per cubic foot of in-place material moved. This means then that for every cubic foot of in-place material moved the amount of fresh material must be reduced by 0.3 cu ft to stay within the computed constant cross-sectional area. The discrepancy also partially results because the number of significant figures does not permit a completely accurate material accounting.

The calculated additional material was 868.2 tons, which is 8.7 more than the sum of column 12. A correction is made by distributing the difference evenly to each of the sections to give column 13. This amounts to about 0.8 ton per 500-ft section and 0.5 ton for the final 280 ft. The sum of column 13 is then 868.0 tons, which agrees more closely with the calculated addition.

The final disposition of all granular material is shown in Figure 5. The example problem shows only one station with an excess of in-place granular material in order to simplify the calculations. Figure 5 is a graphical representation of Table 1 and is included to show more clearly the means of obtaining a constant cross-sectional area over the length of a road before scarification. A constant amount of soil is then assured.

After scarification, the soil and aggregate should be mixed. Any chemical stabilizer can then be added and mixed with the soil materials before final spreading and compaction.

The depth of scarification is calculated by Eq. 16:

$$d = \frac{(1/7)\,(2{,}250{,}700) + (2/7)\,(1{,}736{,}400)}{(98)\,(5{,}280)\,(24)} + \frac{6.57}{24}$$

$$= 0.0658 + 0.2738 = 0.3396 \text{ ft} = 4.075 \text{ in.}$$

Roads with a stabilized surface course must be constructed according to some set of specifications which usually include gradation limits and plasticity index limits. The gradation of the mixtures resulting from the calculations are compared with the gradation specifications of the Iowa State Highway Commission (4) in Figure 6. Because the amount of scarified soil is constant in all sections, the sections in which there was a minimum and a maximum amount of in-place granular material will represent the extremes of gradation in the completed road.

The outer smooth curves of Figure 6 represent the gradation limits specified by the "Standard Specifications" of Iowa (4) for stabilized surface courses. The two inner curves represent the gradations of the extremes previously mentioned. The two extremes fall well within the necessary limits. The dust ratios of the two mixtures are 0.57 and 0.55 (minimum and maximum in-place material, respectively) which are also well below the specified (4) maximum value of two-thirds.

The equation used to calculate the plasticity index of the two extreme gradations does not predict the exact value of the plasticity index but gives an approximation (6). The PI calculated for the section with a minimum in-place material is 9.5, and that for the maximum in-place material is 9.3. Both of these values lie well within the specified (4) range for a plasticity index of not less than 5 or more than 12. A quick

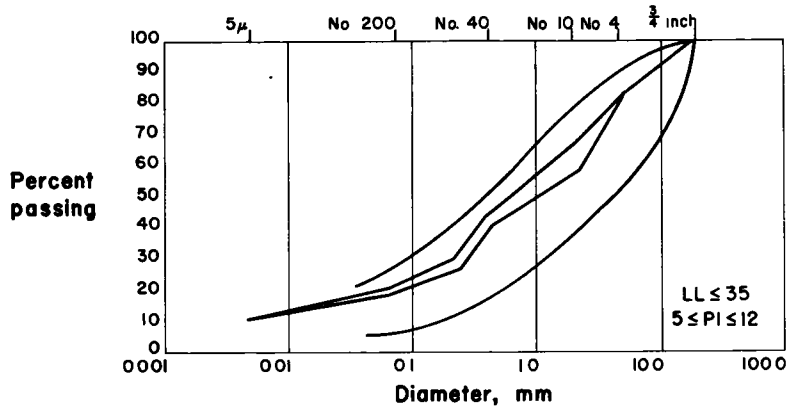

Figure 6. Comparison of gradations of final mixtures with limits specified. Smooth curves represent limiting values.

check on the calculated values can be made with Eq. 10 by

$$T = \frac{(8/7)\ (2,250,700) + (9/7)\ (1,736,400)}{(130)\ (5,280)\ (24)}$$

$$= 0.2917 \text{ ft} = 3.4999 \text{ in.}$$

which corresponds to the design thickness.

The thickness of the various sections will differ because the material is proportioned by cross-sectional area rather than by weight. After redistribution and addition of fresh material, there are essentially four different materials which are defined by cross-sectional area and density. These materials are loose (redistributed) old granular material at γ_{GR} pcf, compacted old granular material at γ_G pcf, loose new granular material at γ_A pcf, and compacted soil at γ_S pcf.

If these materials are combined and compacted to a unit weight γ, the total material in any section is $(\Delta LWR\gamma)/12$, where R is the resulting thickness in inches. The total material is also the sum of all the materials just described. The two can be equated as follows:

$$(\Delta LWR\gamma)/12 = \Delta L(wt)_S\gamma_S + \Delta L(wt)_A\gamma_A + \Delta L(wt)_G\gamma_G + \Delta L(wt)_{GR}\gamma_{GR} \qquad (17)$$

in which the subscripts on the cross-sectional areas (wt) correspond to those on the unit weights γ.

If $\gamma_{GR} = \gamma_A$

and $(wt)_A + (wt)_{GR} = (wt)_{avg} - (wt)_G$,

then substituting in Eq. 17 gives

$$(\Delta LWR\gamma)/12 = \Delta L\left[(wt)_S\gamma_S + (wt)_{avg}\ \gamma_A + (wt)_G\ (\gamma_G - \gamma_A)\right]$$

Substitution of the values from the example gives

$$R = 1/2\left[1.19 + 6.57\ (100/130) + (wt)_G\ (130 - 100/130)\right]$$

$$= 3.12 + 0.115\ (wt)_G.$$

The cross-sectional area $(wt)_G$ represents only the undisturbed granular material, after redistribution in any one section. The final compacted thickness at each station is given in Table 3.

Comparison of the thickness of the various sections shows them to be very near the design value of 3.5 in. in all cases. The greatest deviations are 0.14 in. below and

TABLE 2

MECHANICAL ANALYSES AND PI

Analysis	Diameter (mm)	In-Place Material (G) (%)	Soil (S) (%)	Additional Material (A) (%)
Sieve·				
3/4-in.	19.05	100	100	100
No. 4	4.76	79	91	78
No. 10	2.00	66	88	40
No. 40	0.42	37	72	24
No. 60	0 25	28	66	14
No 200	0 074	18	55	11
5μ	0.005	6	30	3
PI		5	21	2

0.16 in. above the design value, giving a maximum over-all difference of 0.30 in. between the thinnest and the thickest sections in the road.

REMARKS

The amount of 5-μ clay was used as the design criterion because it frequently correlates well with the plasticity index of a material. Nine percent clay was chosen, because research data indicate this to be the average amount of clay used in secondary roads in Iowa. However, the method of design described is not necessarily restricted to the use of the amount of clay as the design criterion but does depend on some size fraction. The size fraction used will depend on experience with local materials or perhaps on a conveniently determined size fraction such as the amount of material passing the No. 40 sieve or the No. 200 sieve. Known correlations of plasticity index with the size fractions may possibly influence the choice. If there are no correlations, the best size fraction for control can be determined by making trial mixes and correlating the amount of the various size fractions with the plasticity index. The equations still hold regardless of the size fraction used for control; however, the values of the constants, K_G and K_A, change accordingly, which in turn change the relative proportions of materials.

Several assumptions are made that are not strictly true and therefore introduce errors. However, if the assumptions are understood, the errors can be kept within the bounds permitted by specifications. The main variation occurs in gradation and hence in plasticity index, but because considerable leeway is usually allowed in these items, the quantities of materials can be controlled so that the final variations lie within the allowable range.

Some error is introduced by assuming that the histogram developed from trench sample data truly represents each section. The error thus introduced is not serious provided the difference between ordinates of adjacent sections is not too large. Large differences indicate that the trench samples are too far apart, and the distance between sites should be reduced. Occasionally a sample site will fall at an intersection and will show an excessive cross-sectional area. Samples should not be taken at or too near an intersection for this reason but should be taken on either side to describe adequately the quantities of material.

Another source of error in the histogram is in the fact that there is a tendency for the granular material to be bladed off the top of hills. The histogram will give a false representation of the amount of material on such a crest if the trench sample is taken elsewhere. If this discrepancy is not corrected, the final mixture will contain too much fine material and is apt to be too soft for good performance.

Other sources of error are due to the use of average values of density and gradation. Density values generally do not vary greatly from place to place for the in-place granular material, the in-place soil, or fresh bulky granular material. The use of an average value of density to describe the in-place material plus the added material introduces some error, but this is not too serious as indicated by the final thicknesses previously calculated.

TABLE 3

FINAL COMPACTED THICKNESS AT EACH STATION

Station	$(wt)_G$ (sq ft)	R (in.)
2 + 50	2.50	3.41
7 + 50	2.12	3.36
12 + 50	2.59	3.42
17 + 50	2.37	3.39
22 + 50	4.12	3.59
27 + 50	3.81	3.56
32 + 50	4.68	3.66
37 + 50	4.24	3.61
42 + 50	2.71	3.43
47 + 50	2.79	3 44
52 + 50	2.51	3.41

Differences in gradation from place to place cause the most serious errors and variability in the final mixture. The new material is usually rather uniform as it comes from a gravel pit or stone quarry, thus gradation errors introduced from this source are small. The main gradation differences result from the in-place granular material and the underlying soil material at the various stations. Of these two, the differences in gradation found in the soil are the most serious, because the soil furnishes most of the fine material and therefore influences the plasticity index of the final mixture more than any other ingredient. The amount of fine material in the mixture can be kept within the allowable limits by adjusting the control percentage used as a criterion for proportioning materials. If a road passes through several soil types having wide gradation differences each area must be treated as a separate problem.

The success of this method of design depends on the success of the method of construction as well as on the use of the equations. The new material must be accurately spotted and spread to form a reference surface from which to scarify to a calculated depth. The depth of scarification must be constant over the entire length and width to insure a reasonably uniform final gradation and thickness of stabilized material.

Present methods of secondary road construction, in which old road surfaces are salvaged, rely chiefly on the judgment of the engineer. Arbitrary rates of addition of granular material are used, and the depth of scarification is an estimation. Such practice sometimes leads to poor quality roads, although after long experience some engineers achieve considerable success with these methods. The scheme described in this paper gives the engineer a means of control over the amounts of materials and permits a more effective usage of old road materials. The quality of the resulting surface course is more controlled, and a good deal of guesswork is removed from the design and construction procedures. However, the method must be tempered with common sense and the engineer must be alert to recognize any deviations from the predicted results and should make field corrections accordingly.

ACKNOWLEDGMENT

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Effectiveness of Certain Derivatives of Furfural as Admixtures in Bituminous Soil Stabilization

HANS F. WINTERKORN, Princeton University, and
THEODORE REICH, Soil Stabilization Development Corp.

The effectiveness of furfural derivatives, especially those obtained by reaction with plain and substituted aniline and other aromatic and aliphatic amines, as soil stabilizers by themselves and as additives in bituminous stabilization, has been known since the early 1930's. To obtain maximum benefit from such use, it is necessary with some soil materials to employ catalysts and reaction modifiers that are compatible or even synactive with the furfural derivative in proportions that are fitted to the particular soil-stabilizer combination. This may require more experience and judgment on the part of the user than he is able to muster or afford. It is desirable, therefore, to develop combinations that are effective with all soils likely to be encountered within a certain region, especially with those soils in it that would require special attention if the most economical combination (that is, aniline-furfural, were used). Following this reasoning, laboratory experiments were made in which either or both special catalytic and synactive substances were added to the aniline-furfural combinations or chlorinated anilines were employed in the mixtures. The results of these experiments are discussed.

•THE EFFECTIVENESS of furfural derivatives, especially those obtained by reaction with aromatic amines, as cementing and waterproofing agents for soils has been known since the early 1930's (1, 2). Their stabilization potential for both cohesive and granular soils, by themselves and also as additives in bituminous stabilization, was thoroughly investigated during the 1940's, and the general principles underlying their effectiveness were established during that time (3, 4, 5, 6). Within recent years, some of the knowledge of the 1940's has been rediscovered, indicating that a good deal of this knowledge has been forgotten or had never reached those for whom the publications had been intended. It is the purpose of the present paper to add to the available information on this subject and to clarify certain extant questions.

The basic problem in the stabilization of cohesive soils is to maintain part of the water-induced cohesion, but to control excessive water intake (7). Systems of this kind possess sufficient cohesion for all practical purposes and they also have the ability to breathe; that is, to prevent water accumulation, which is likely to occur beneath water-impervious layers. The water affinity of a soil is a function of the extent and character of the internal surface and of the type and number of the exchangeable ions. Treatment with organic amines aims to control the water affinity of soils by replacing some of the water-attractive ions by the amines that possess the amino group as cationically active head and a water-repellent body represented by the remainder of the organic molecule. If such cation exchange takes place on the basal planes of the clay minerals, then the lumpy organic molecule will interfere greatly with the parallel force fields on such planes and will thus reduce the amount of osmotic swelling possible in such soils. The relationship between electric structure on the basal plains and osmotic swelling has been discussed elsewhere (8).

TABLE 1

SIZE COMPOSITION OF N.J. HAGERSTOWN SOIL

Separate	Size (mm)	Bouyoucos Method (%)	ASTM Method (%)
Sand	1 - 0.05	10 5	12.2
Silt	0.05 - 0.005	51.5	74.3
Clay	<0.005	38.0	13.6
Colloids	<0.002	20.0	6.7

To effect a replacement of the natural cations present in the soil for cations derived from the organic amines, enough moisture must be present to induce these cations to get out of their places in the mineral surfaces and into the water phase. In addition, it is necessary that the system be acid either throughout or locally, by having close combinations or associations of the amines with inorganic or organic acids or acid-salt complexes. It is also advantageous if the released exchange ions form insoluble salts with the respective acids. In the case of calcium carbonate particles (such as coral beach sands) coatings of calcium salts, with either inorganic or organic anions, possessing very low solubility must be produced in the stabilizing process.

Osmotic swelling increases with decreasing charge of inorganic exchange ions. Also, in the monoionic soils, except for the hydrogen exchange ions, the pH increases with decreasing charge of the exchange ions. Hydrogen soils tend to become iron and aluminum soils when stored long because of the interaction of the hydrogen with the skeleton of the clay mineral. With respect to mechanical resistance of soils, it is, of course, well known that such resistance increases with increasing densification of the soil. However, if increased dry density is purchased at the price of too low a moisture content (that is, one that is insufficient to bring about the desired exchange of the natural ions on the soil for the amine ions), then compaction to higher densities may actually result in lesser stabilization effectiveness, especially with respect to weathering resistance.

The following presentation is divided into three parts:

1. The preparation of the monoionic modifications of N.J. Hagerstown soil and their response to stabilizing treatment with two percentages of the two aniline -1 furfural reaction product, on specimens prepared at two different densities and different moisture contents.
2. The effect of aniline-furfural admixtures to bitumen (MC-2) on the stabilization of natural N.J. Hagerstown soil in comparison with that of other admixtures and stabilizing agents.
3. The effect of A-F admixtures on bituminous stabilization of local Princeton red clay and is also concerned with questions of general importance, such as overcompaction, water requirements, and physical significance of testing procedures.

Characterization of Soil Materials Employed

The soil employed was a Hagerstown clay (B-horizon of the profile) obtained in the vicinity of Peapack, N.J., on a location designated as typical by the Soils Department of Rutgers University, New Brunswick, N.J. The latter institution has been using material from the same location for its own investigations. This allowed a certain mutual checking of the experimentally determined soil characteristics.

Hydrometer analyses were made according to the Bouyoucos and the ASTM methods. The results are given in Table 1. As observed on other occasions, the ASTM method often fails to disperse and register the finer soil fractions properly. The natural soil sample had the following chemical characteristics; for a cation exchange capacity of 9.6 meq per 100 g of soil, the exchangeable cations per 100 g of soil are H, 0.0 meq; Ca, 5.83 meq; Mg, 4.27 meq; and K, 0.007 meq.

The value for the cation exchange capacity given is that obtained by electrometric titration; this value is identical with the one obtained by Rutgers College in their investigations; a slightly smaller value was obtained using the ammonium acetate method. The value of 9.6 was employed in the making of the homoionic soil modifications.

PREPARATION OF HOMOIONIC MODIFICATIONS

The H-, Na-, K-, Mg-, Ca-, Al-, and Fe- soils were prepared and employed in this investigation. They were made in batches of 60 lb of soil (obtained by quartering of the total soil sample) dispersed in about 50 l of distilled water. As reaction vessels, paraffined galvanized iron cans were employed, the paraffine preventing reaction of the soil and of the chemicals with the metal.

The H- soil was prepared by adding hydrochloric acid in sufficient amount to reduce the pH of the soil suspension to about 4, and washing off the chlorides formed in the process.

The Na-, K-, Mg-, and Ca-soils were made by adding the calculated amounts of hydroxides to the previously prepared H-system. It is of interest that approximately 7,000 gal of distilled water were required to wash the chlorine ions out of the H- soil batches.

The Fe- and Al- soils were prepared from the natural soil by adding an excess of the respective chlorides, and by washing the ionic reaction products and the excess Fe- and Al- salts by means of distilled water until only a faint trace of chlorine ion was found in the soil-water system.

The seven homoionic modifications and the natural soil were dried at a temperature of about 150 F, pulverized to pass a No. 10 sieve, and stored in galvanized cans.

The physical and chemical properties of the different soil modifications are given in Table 2. Though the data speak for themselves, attention is called to the following points:

1. The liquid limits and consequently the plasticity indexes of the Na- and K-modifications appear to be rather low. This may possibly be due to a sampling error. In the preparation of the K- and Na- modifications, the clay material separated from the remainder of the soil and formed very hard crusts in drying. Although this material was ground with the remainder of the soil and well mixed with the latter, it is possible that, in getting the -40 sieve fraction for the consistency tests, a disproportionate amount of these hard aggregations remained on the sieve.

TABLE 2

PHYSICAL AND CHEMICAL PROPERTIES OF THE NEW JERSEY HAGERSTOWN HOMOIONIC SOILS

Soil	pH	Liquid Limit (%)	Plastic Limit (%)	Plastic Index	Specific Gravity (H_2O)	Specific Gravity (xylene)	Homoionic Ion Saturation[a] (meq/100 g)	Shrinkage Limit (%)	Moisture-Density Relationship: Proctor H_2O (%)	Proctor Max Dry Dens. (pcf)	Mod. AASHO H_2O (%)	Mod. AASHO Max Dry Dens. (pcf)
Nat	7.72	29.5	21.8	7.7	2.68	2.71	7.1	20.5	18.8	103.0	16.5	113.0
Na	9.00	22.5	18.7	3.8	2.68	2.71	5.5	22.5	17.7	105.6	16.4	110.4
K	8.62	24.5	21.3	3.2	2.68	2.70	6.0	23.3	20.7	102.0	17.8	110.2
H	5.62	32.2	28.6	3.6	2.68	2.72	0.5	26.2	19.5	102.3	17.8	108.9
Al	4.92	30.3	25.3	5.0	2.68	2.71	0.5	27.9	20.2	103.2	18.3	108.2
Fe	5.80	33.3	26.0	7.3	2.68	2.71	0.0	28.4	20.8	100.3	19.4	108.9
Mg	7.01	29.3	20.0	9.3	2.68	2.71	7.6	22.8	17.7	104.9	16.0	112.3
Ca	7.70	29.0	21.0	8.0	2.67	2.70	9.5	24.2	18.3	105.0	16.7	112.8

[a] NH_4Ac extractable ions.

TABLE 3

BATCH COMPOSITION OF SPECIMENS MADE AT PROCTOR DENSITY

Property	Soil							
	Nat	Na	K	Ca	Mg	H	Fe	Al
Hygroscopic moisture (%)	3.4	2.9	7.3	4.4	2.0	7.8	6.9	4.2
Specimen weight (g)	209.0	211.5	210.0	212.0	210.5	209.0	207.0	212.0
Air-dry soil (g)	1,945	1,975	2,000	1,995	1,950	2,015	1,965	1,965
H_2O added (cc)	289	284	249	266	300	219	255	302
1 percent[a] (cc):								
Aniline	12.4	12.5	12.1	12.5	12.5	12.1	12.0	12.4
Furfural	5.4	5.5	5.4	5.5	5.5	5.4	5.3	5.4
2 percent[a] (cc):								
Aniline	24.8	25.0	24.2	25.0	25.0	24.2	24.0	24.8
Furfural	10.8	11.0	10.7	11.0	11.0	10.7	10.3	10.8

[a]Amount of water employed in these specimens was reduced equivalent to volume of liquid stabilizers added.

2. The pH of the H- soil changed after the wet preparation from about 4 to 5.6, the soil having been dried and redispersed. The Hagerstown soil is a reddish soil and as such contains surface-adsorbed iron and aluminum hydroxides. Because these hydroxides were not entirely removed, they had a chance to react with the exchange positions, probably resulting in at least partial Fe- and Al- modifications.

3. The specific gravities in xylene were somewhat larger than those in water. This also seems to be related to the coating of the soil particles with iron and aluminum hydroxides.

4. The low values for ionic saturation of the H-, Al-, and Fe- soils are due to the fact that the trivalent Fe- and Al- ions cannot be replaced by ammonium; therefore, these values do not indicate actual ionic saturation.

Preparation and Testing of Specimens

The stabilizers employed were 1 and 2 percent based on the dry weight of the soil of a combination of 2 parts of aniline and 1 part of furfural in liquid form. To replace every exchangeable ion in the soil with the cation produced by the reaction of 2 mols of aniline and 1 mol of furfural would have required 2.68 percent of the 2A-1F combination. Cylindrical test specimens (2 in. by 2 in.) were prepared using the floating ring compaction method. The dry compacted densities corresponded to those previously determined for the different ionic modifications by both the Proctor and the modified AASHO methods, as did the quantities of moisture employed. Ten specimens were made in each batch and cured for 7 days without loss of moisture in a moist room (90 to 100 percent relative humidity). Duplicate specimens were submitted to the following exposures before being tested for compressive strength:

1. One day of capillary absorption followed by 7 days water immersion.
2. One cycle of wetting and drying.
3. Four cycles of wetting and drying.
4. One cycle of freezing and thawing.
5. Four cycles of freezing and thawing.

One cycle of wetting and drying consisted of 24 hr of complete immersion in water followed by 24 hr of drying in an oven at 60 C (140 F). All specimens undergoing the wetting and drying test were immersed in water for 24 hr after the last drying cycle before they were tested for compressive strength.

One cycle of freezing and thawing consisted of 24 hr of freezing at -10 F followed by 24 hr of immersion in water. All specimens undergoing freezing and thawing cycles were immersed in water 24 hr before the first freezing cycle.

TABLE 4

BATCH COMPOSITION OF SPECIMENS MADE AT MODIFIED AASHO DENSITY

Property	Soil							
	Nat	Na	K	Ca	Mg	H	Fe	Al
Hygroscopic moisture (%)	3.4	3.5	7.3	4.4	2.0	7.8	6.9	4.2
Specimen weight (g)	225.0	220.0	221.0	224.5	222.5	219.0	222.5	219.0
Air-dry soil (g)	2,121	2,065	2,150	2,106	2,080	2,135	2,120	2,060
H_2O added (cc)	269	260	210	252	286	218	233	261
1 percent (cc):								
Aniline	13.3	13.1	13.1	13.3	13.3	12.9	12.9	12.9
Furfural	5.9	5.8	5.8	5.9	5.7	5.7	5.7	5.7
2 percent (cc):								
Aniline	26.6	26.2	26.2	26.6	26.6	25.8	25.8	25.8
Furfural	11.8	11.6	11.6	11.8	11.8	11.4	11.4	11.4

Tables 3 and 4 give the batch compositions for the specimens and densities employed. Table 5 gives the compressive strength of the corresponding, compacted unstabilized monoionic and natural soils after curing for 7 days without loss in moisture content.

Test Results

The test results are given in Table 6 and Figure 1. The discussion of the test data is based mainly on the compressive strength values, because of the close relationship between moisture content and compressive strength of the specimens, and the obviously primary importance of the latter for engineering purposes.

It is of interest to present first a few general relationships on the following items—(a) the beneficial or deteriorating effect of the different weathering exposures, (b) the effect of density on the initial compressive strength and on the resistance of the specimens to deterioration, and (c) the average compressive strength of the untreated and unweathered specimens as compared with that of the treated but weathered specimens:

1. The average compressive strength was 50 and 128 psi, respectively, for untreated, unweathered specimens compacted at the Proctor and modified Proctor densities. The modified Proctor specimens were therefore two and one-half times as strong as the standard Proctor specimens.

2. The average compressive strength over all ionic soil modifications, all stabilizing treatments, and all weathering exposures was 64 psi for the standard Proctor and 136 psi for the modified Proctor density. The treated modified Proctor specimens were, therefore, more than twice as good as the treated standard Proctor specimens.

TABLE 5

COMPRESSIVE STRENGTH OF UNSTABILIZED HOMOIONIC N. J. HAGERSTOWN SOIL

Soil Modification	Compressive Strength (psi)	
	Proctor Density	Modified Proctor Density
Nat	50	140
H	75	175
Na	50	140
K	50	145
Mg	50	110
Ca	50	125
Al	50	95
Fe	25	95
Avg.	50	128

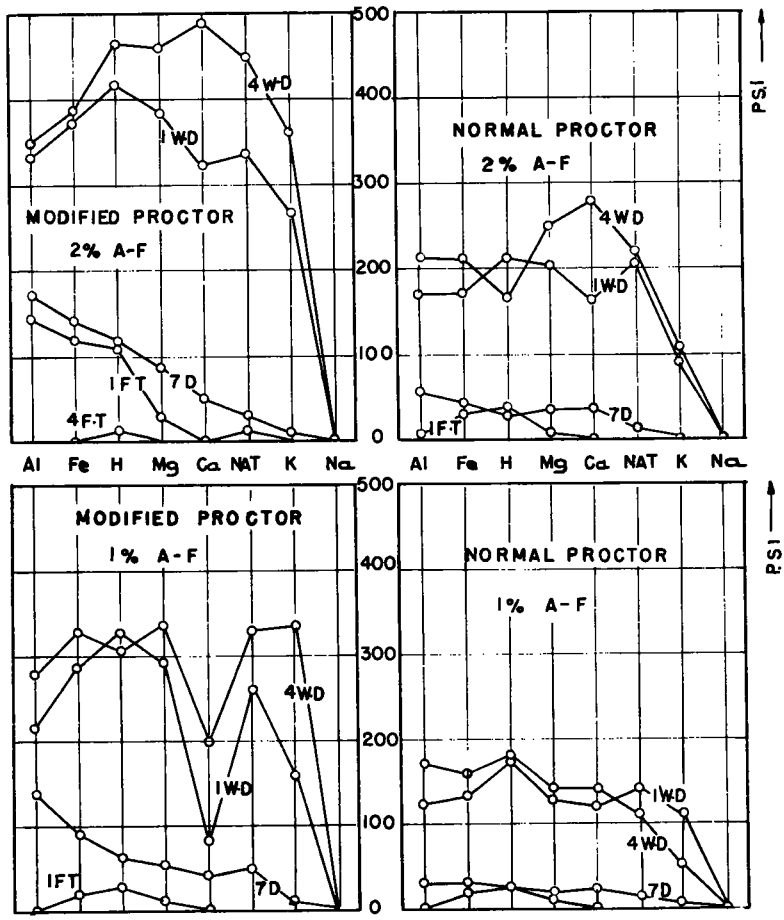

Figure 1. Compressive strength of treated soil specimens.

3. The average compressive strength over all ionic modifications, all stabilizing treatments, and all weathering exposures was 128 and 106 percent, respectively, of that of the average value of the non-treated and non-weathered specimens at the Proctor and modified Proctor densities.

4. The beneficial or detrimental effect of the different weathering exposures is shown by the average compressive strength values given in Table 7 for the Proctor and modified Proctor densities.

The relative susceptibility to aniline-furfural stabilization of the different ionic soil modifications is indicated by the strength values averaged for each ion over the two percentages of admixture, the two densities, and all pretest treatments (Table 8).

INFLUENCE OF RESIN ADMIXTURE ON STABILIZING EFFECTIVENESS OF MC-2 CUTBACK

In the investigation presented in the preceding part, the specimens were permitted to cure for 7 days without loss of water before being submitted to the different exposures. In previous tests, partial drying was allowed during the curing period. It is still an open question which curing procedure is preferable; that is, which corresponds most closely to field conditions. As a contribution to the eventual settlement of this question, and also for the purpose of a direct comparison between resinous and bituminous soil stabilization, a second set of experiments was run, employing the natural Hagerstown

TABLE 6

COMPRESSIVE STRENGTH OF TREATED SOIL SPECIMENS AFTER DIFFERENT WEATHERING EXPOSURES

A-F	Compacting Density	Treatment	Exchange Ion							
			Al	Fe	H	Mg	Ca	Nat	K	Na
A_o 1%	Mod Proctor	7-day immers.	138	96	63	54	42	50	21	0
		1 W-D	217	284	326	292	80	268	163	0
		4 W-D	276	326	305	334	200	330	234	0
		1 F-T	84	63	67	21	0	15	0	0
		4 F-T	0	0	8	0	0	0	0	0
	Normal Proctor	7-day immers.	29	29	25	21	21	13	6	0
		1 W-D	122	134	173	130	121	142	117	0
		4 W-D	171	166	180	142	142	117	52	0
		1 F-T	0	21	27	13	0	0	0	0
		4 F-T	0	0	4	0	0	0	0	0
B_o 2%	Mod. Proctor	7-day immers.	173	142	129	89	49	31	11	0
		1 W-D	333	372	422	382	324	333	266	0
		4 W-D	350	386	466	460	488	475	364	0
		1 F-T	146	120	111	31	0	13	0	0
		4 F-T	0	0	13	0	0	0	0	0
	Normal Proctor	7-day immers.	58	44	32	36	35	13	0	0
		1 W-D	173	173	213	204	164	208	93	0
		4 W-D	213	213	168	248	275	222	107	0
		1 F-T	7	31	36	9	0	0	0	0
		4 F-T	0	0	7	0	0	0	0	0

TABLE 7

AVERAGE COMPRESSIVE STRENGTH FOR PROCTOR AND MODIFIED PROCTOR DENSITIES

Type of Weathering	Average Compressive Strength (psi)		
	Proctor	Modified Proctor	Avg.
(1) 7 days immersion	22.5	65	44
(2) 1 wetting-drying	135.5	254	195
(3) 4 W-D	151	313	232
(4) 1 F-T	8	42	25
(5) 4 F-T	4	0.7	2.3
(0) none	50	128	89

TABLE 8

Property	Ionic Modification							
	H	Fe	Al	Mg	Nat	Ca	K	Na
Avg. compres. str. (psi)	139	130	125	123	112	97	72	0
pH of soil mod.	5.6	5.8	4.9	7.0	7.7	7.7	8.6	9.0

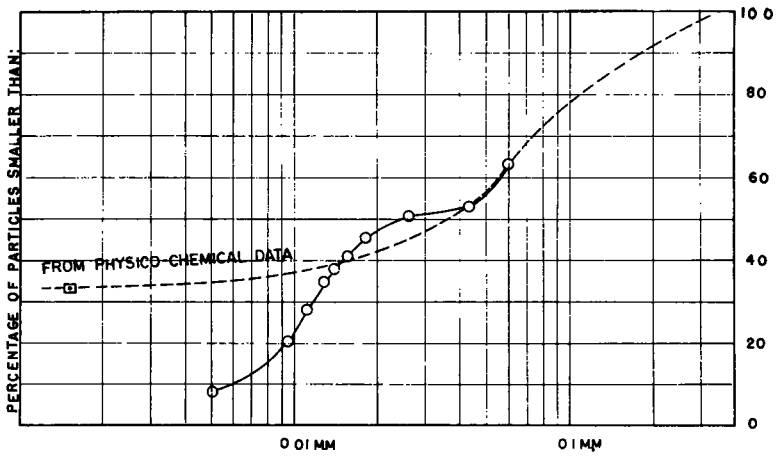

Figure 2. Grain-size cumulative curve.

TABLE 9

Identification	Stabilizing Admixture[a]
I	10% MC-2
II	10% MC-2 + 2.5% $Ca(OH)_2$
III	10% MC-2 + 10% $Ca(OH)_2$
IV	8% MC-2 + 0.06% aniline-furfural
V	8% MC-2 + 0.12% aniline-furfural
VI	8% MC-2 + 0.24% aniline-furfural
VII	1% Tall oil (25% neutralized with NaOH)
VIII	10% MC-2 + 0.24% aniline-furfural
IX	10% MC-2 + 0.24% high resin tall oil 1/4 neutralized
X	10% MC-2 + 0.24% normal tall oil 1/4 neutralized

[a]Percentages based on dry soil weight.

soil, moist and dry curing, modified Proctor compaction, and stabilizing admixtures as given in Table 9.

Tests Performed and Results

Two curing methods—wet and dry curing for a period of 7 days—were employed. Thereupon, duplicate specimens for each type of curing were submitted to the same exposure conditions as described in the preceding part and tested for compressive strength. The test data are given in Table 10. The over-all average performance of the stabilized specimens as a percentage of the average compressive strength of the non-treated, non-weathered compacted specimens was 134 percent for the dry cure and 118 percent for the moist cure.

This difference was due mainly to the poor performance of the moist-cured specimens in the freeze-thaw cycles. If the freeze-thaw data are omitted in the averaging, then one obtains 187 percent for the dry cure average and 174 percent for the moist cure average.

Because the freeze-thaw test may be too severe for many actual construction conditions, the effectiveness of the different stabilizers has been evaluated in two ways: (a) employing all test data including those obtained in the freeze-thaw tests; and (b) employing all but the freeze-thaw data. This evaluation is given in Table 11. The respective

TABLE 10

AVERAGE COMPRESSIVE STRENGTH AFTER DIFFERENT EXPOSURES

Cure	Stabilizer	Average Compressive Strength (psi)						
		7-Day Immersion	1 Wet-Dry	4 Wet-Dry	1 Freeze-Thaw	4 Freeze-Thaw	Avg.	Perf.[a]
Dry	I	34	182	394	28	19	131	93
	II	204	358	439	250	161	282	201
	III	350	546	483	353	283	403	287
	IV	32	191	358	24	14	124	89
	V	50	293	454	32	18	169	121
	VI	91	296	461	54	21	185	132
	VII	38	42	41	27	0	39	27
	VIII	127	342	532	115	25	228	163
	IX	49	205	430	32	19	147	105
	X	83	218	431	48	21	160	114
	Avg.	106	277	402	96	58	188	134
	Perf.	76	197	287	68	41	134	
Wet	I	32	135	322	24	0	103	74
	II	251	318	455	204	94	264	188
	III	309	474	600	325	188	379	270
	IV	35	169	402	17	0	125	89
	V	54	185	400	10	0	130	93
	VI	72	233	496	8	0	162	116
	VII	73	43	35	26	0	35	25
	VIII	107	264	546	29	0	189	135
	IX	90	126	361	16	0	119	85
	X	104	178	425	19	0	145	103
	Avg.	113	213	404	68	28	165	118
	Perf.	80	152	289	48	20	118	

[a]Average compressive strength as a percentage of compressive strength of untreated and unweathered compacted specimens, which was 140 psi.

data speak for themselves. Of course, their engineering evaluation must take into account such factors as materials costs, effect of admixtures on the ease and hence economics of mixing and compaction, cost of transportation, etc.

STABILIZATION STUDIES ON LOCAL RED CLAY SOIL FROM PRINCETON, N.J.

The local red clay soil from Princeton, N.J., possesses certain features that are of significance in bituminous soil stabilization, but which are likely to be overlooked by the average practitioner. The soil is derived from a triassic red shale which itself was a well-leached clay soil previous to geologic consolidation. The grain-size composition obtained by the ASTM method and the moisture density relationships under compaction by the 2,000-psi static load method were determined by Tschebotarioff whose plots of these properties are shown in Figures 2 and 3 (9).

The ASTM grain-size analysis method gave less than 10 percent -2-μ material and there was a definite break in the cumulative size curve at 0.03 mm. On request by Tschebotarioff, a physico-chemical study of this soil had been undertaken by the senior author (10). This study included the following:

1. Separation of the -2-μ fraction from the whole soil and determination of its mineralogical character by means of the DTA method.

2. Determination of the base exchange capacity of the -2-μ fraction and of the whole soil by potentiometric titration.

3. Determination of water adsorption isotherms for the whole soil and the separated -2-μ fraction, and calculation of the base exchange capacity and clay content of the whole soil from known relationships between these properties and water vapor absorption under specified conditions.

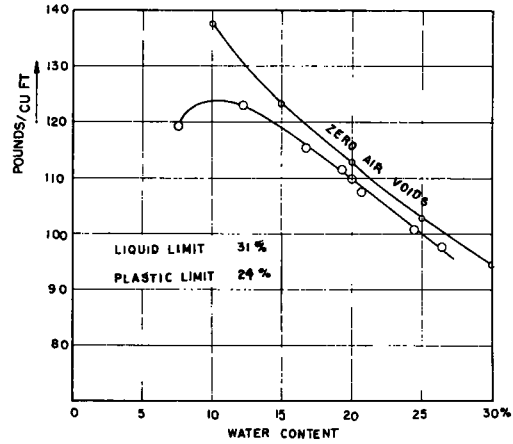

Figure 3.

The extracted clay fraction consisted almost entirely of kaolinite with an effective particle size smaller than 0.1-μ. Because of this small particle size, the experimentally determined base exchange capacity of the -2-μ fraction had the relatively high value of 30.4 meq per 100 g of clay. The base exchange capacity of the whole soil was about 9.6 meq per 100 g of soil. This gives a calculated clay content for the whole soil of about 32 percent, whereas clay contents calculated from the isothermal absorption data ranged from 28.4 to 32 percent.

According to the physico-chemical evidence, the red soil contained about 32 percent of -0.1-μ clay. In its geological history, this clay had been aggregated with sufficient force that even after weathering to the present soil, this clay content did not become apparent in the ASTM analysis nor in the moisture-density data obtained by compaction tests. On the other hand, intensive mechanical working of the soil changed its plastic and liquid limits in a similar manner as known and reported for certain lateritic soils (11) showing that the clay could be activated by mechanical working in the presence of water and probably also in a new chemical environment; for instance, such as created by chemically active soil stabilization agents.

Using the 2,000-psi static compression method, Tschebotarioff had obtained a maximum density of about 123 pcf at an optimum moisture content of about 12 percent.

TABLE 11

RATING OF TREATMENTS[a]

Treatment	Rating	
	Incl. F-T Data	Excl. F-T Data
III 10% MC-2 + 10% $Ca(OH)_2$	270	330
II 10% MC-2 + 2.5% $Ca(OH)_2$	188	244
VIII 10% MC-2 + 0.24% A-F	135	218
VI 8% MC-2 + 0.24% A-F	116	191
X 10% MC-2 + 0.24% Tall oil, normal	103	174
V 8% MC-2 + 0.12% A-F	93	152
IV 8% MC-2 + 0.06% A-F	89	144
IX 10% MC-2 + 0.24% Tall oil, high rosin	85	137
I 10% MC-2	74	116
VII 1% Tall oil, 25% neutralized with NaOH	25	36

[a]Moist curing, modified Proctor compaction. Percentages of all admixtures based on dry weight of soil. For complete base exchange with A-F combination, 2.68 percent of latter would have been required. Hence, maximum A-F admixture of 0.24 percent represents 9 percent of base exchange capacity of soil involved.

Approximately the same values were obtained in the laboratories of the Soil Stabilization Development Corporation with the Dictert compactor (20 blows), whereas the standard Proctor hammer method gave about 105 pcf at an optimum moisture content of 20 percent. The liquid limits and plastic limits, found by Tschebotarioff to be 31 and 25 percent, respectively, were duplicated by the SSDC in normal procedure, but both values became smaller with increased working of the moist soil. The same happened in the case of the shrinkage limit which was found to range between a high of 25 percent and a low of 17 percent.

TABLE 12

COMPOSITION OF ADDITIVES

Designation	Component	Weight Proportion
A	3, 4 bichloraniline	3
	furfural	1
B-1	Aniline	2
	Furfural	1
	Het acid	4 17
B-2	Aniline	2
	Furfural	1
	Het acid	2 1
C-1	Aniline	2
	Furfural	1
	Isophthalic acid	1 78
C-2	Aniline	2
	Furfural	1
	Isophthalic acid	0 89

Stabilization Experiments

Despite the warning signs that could be read in the results of the physico-chemical and consistency tests, the SSDC desired to utilize the advantage that seemed to lie in the great density obtained by both the static and the Dictert compaction method. Accordingly, soil-bitumen-water mixtures were designed to give dry soil densities of 2-in. cylindrical test specimens by the use of the Dictert method in the order of 120 pcf. Various contents of MC-2 cutbacks were used in the mixtures, whereby the lubricating effectiveness of the volume of cutback employed was considered as one-half that of the

TABLE 13

COMPRESSIVE STRENGTH OF SPECIMENS MADE WITH 8 PERCENT MC-2 CUTBACK ASPHALT CONTAINING 0.5 TO 2 PERCENT ADDITIVES[a]

Additive		Compressive Strength (psi)					Water Intake (% of air dry wt.)			
No.	%	0 Days	1 Day	7 Days	14 Days	28 Days	1 Day	7 Days	14 Days	28 Days
A	0.5	403	243	103	50	38	4.5	8.5	13.7	18.3
	1.0	390	253	108	65	28	2.3	7.4	12.3	17.2
	1.5	350	238	123	65	48	2.6	6.6	11.8	16.1
	2.0	365	222	157	60	53	3.0	7.0	13.3	15.8
B-1	0.5	333	212	57	32	30	3.3	11.0	18.0	20.3
	1.0	325	225	77	40	43	3.1	9.6	15.2	17.8
	1.5	370	233	87	50	50	2.8	9.3	14.8	18.4
	2.0	375	220	67	45	35	3.5	10.7	15.5	18.6
B-2	0.5	318	213	72	35	30	3.4	11.3	16.3	19.2
	1.0	350	233	90	50	37	2.5	9.0	15.5	18.0
	1.5	325	225	80	47	43	2.8	8.7	15.0	18.3
	2.0	343	260	95	55	50	3.0	9.2	15.2	17.8
C-1	0.5	280	180	37	25	22	3.0	17.9	18.3	20.3
	1.0	293	211	76	43	42	2.7	10.1	15.9	16.5
	1.5	318	206	65	36	45	2.2	10.3	15.3	16.0
	2.0	330	243	110	65	67	1.8	8.3	17.0	14.8
C-2	0.5	325	187	41	25	18	2.9	14.5	12.8	17.3
	1.0	317	209	75	53	42	2.0	9.6	15.1	16.0
	1.5	305	213	78	30	44	2.3	10.0	14.8	16.5
	2.0	308	236	93	50	53	1.8	8.6	13.8	15.7

[a]Based on weight of cutback in air-dry condition and after 1, 7, 14, and 28 days of immersion.

TABLE 14

COMPRESSIVE STRENGTH OF SPECIMENS AS FUNCTION OF ADDITIVE COMPOSITION[a]

Additive	Compressive Strength (psi)					Avg. Water Intake (% of air-dry wt.)			
	0 Days	1 Day	7 Days	14 Days	28 Days	1 Day	7 Days	14 Days	28 Days
A	377	241	123	60	42	3.1	7.4	12.8	16.8
B-1	351	222	72	42	40	3.2	10.2	15.9	18.8
B-2	334	233	84	47	40	2.9	9.5	15.5	18.3
C-1	305	210	72	42	44	2.4	11.6	16.6	16.9
C-2	314	201	72	40	40	2.3	10.7	14.1	16.4

[a]Averaged over four different percentages of additive employed.

same volume of water. This means that the total compaction water requirements were reduced by one-half the volume of the cutback employed. The amount of MC-2 employed ranged from 6 to 10 percent of the weight of the dry soil. The cutback was used by itself as well as with various additives. The bitumen-treated specimens were cured in the laboratory air to constant weight and then submerged in water at room temperature.

All these high-density specimens, irrespective of bitumen and additive percentage and composition, cracked sooner or later after immersion in water, showing expansion failure typical for overcompacted specimens. It was then decided to design test specimens at the Proctor dry density of 105 pcf and an optimum liquid content of 20 percent by weight of the dry soil. The latter was to be supplied by 12 percent of water, which corresponded to the lubricating needs for the maximum density in the 2,000-psi static and in the Dictert compaction, and 8 percent of the MC-2 cutback. With this mix design and speciman density, stabilization of the red clay soil could be achieved with the help of certain additives to the bitumen.

Effectiveness of Certain Furfural-Derived Additives

In the course of stabilization studies with this soil, it was decided to check the possibility of obtaining better results than using the plain aniline-furfural compound by employing dichlor aniline instead of the aniline or by using the chlorinated Het acid instead of isophthalic acid which in previous work had proven itself, similarly to pentachlorophenol, as a most effective catalytic and synactive agent of the 2A-F additive (12). Accordingly, specimens were fabricated with the previously stated mix design and density, but in which the MC-2 cutback contained 0.5, 1, 1.5, and 2 percent, respectively, of the additives designated as A, B-1, B-2, C-1, and C-2 whose compositions are given in Table 12.

The weight of the compacted 2- by 2-in. cylindrical specimens averaged 208 g as fabricated. They were permitted to dry in air to constant weight which was in the vicinity of 187 g. Ten specimens were made of each composition, two of which were weighed and tested for compressive strength after air curing, and two after air curing plus 1, 7, 14, and 28 days, respectively, of immersion in water. The results of these tests are given in Table 13. Table 14 gives the compressive strength and water intake values as a function of the additive composition, obtained by averaging over all four different percentages of the additive used, whereas Table 15 gives compressive strength and water intake data of the most effective compositions as judged by the test results after 28 days of immersion.

In accordance with the test results, the additive compositions can be rated in the following order of decreasing effectiveness: C - 1 > A = C - 2 > B - 1 = B - 2. If the ionically effective mass is calculated per unit weight of additive,

$$\begin{array}{ccccccccc} A & > & C-2 & > & C-1 & > & B-2 & > & B-1 \\ 0.78 & & 0.77 & & 0.63 & & 0.59 & & 0.42 \end{array}$$

TABLE 15

COMPOSITIONS GIVING BEST DATA AFTER 28 DAYS OF IMMERSION FOR EACH TYPE OF ADDITIVE

Additive		Compressive Strength (psi)					Water Intake (% of air-dry wt.)			
No.	%	0 Days	1 Day	7 Days	14 Days	28 Days	1 Day	7 Days	14 Days	28 Days
A		365	222	157	60	53	3.0	7.0	13.3	15.8
B-1	1.5	370	233	87	50	50	2.8	9.3	14.8	18.4
B-2	2.0	343	260	95	55	50	3.0	9.2	15.2	17.8
C-1	2.0	330	243	110	65	67	1.8	8.3	--	14.8
C-2	2.0	308	236	93	50	53	1.8	8.6	13.8	15.7

If the additive compositions are ranked in accordance with the relative proportions of cation equivalents per unit mass of admixture,

$$\begin{array}{ccccccccc} C-2 & > & C-1 & > & A & > & B-2 & > & B-1 \\ 1.00 & & 0.81 & & 0.78 & & 0.76 & & 0.55 \end{array}$$

The C-2 composition contains about 5.5 mmole of amine per gram of additive; that is, about 0.9 mmole per 100 g of soil if the additive is 2 percent of the weight of the MC-2 employed. Because the soil had a base exchange capacity of 9.6 meq per 100 g, the maximum amount of additive employed represented less than one-tenth of the amine requirements for complete base exchange. The highest ranking in stabilizing effectiveness was achieved by the C-1 composition which had third rank in effective ionic mass and second ranking in cationic equivalents. This indicates a special synactive benefit produced by the isophthalic acid component.

Duration of Water Exposure

In analogy with the short-cut testing on soil-cement specimens, it has become customary in many places to judge the quality of stabilized soil systems by compressive strength tests after 7 days of water immersion of 2-in. diameter cylindrical test specimens. Such a procedure is justified only after previous establishment of the actual relationship between the compressive strength after 7 days of immersion and that obtained after immersion of a duration, following which no further decrease in strength is obtained. The latter cannot be predicted a priori, but is a function of such factors as water affinity of the specimen, initial and final permeability and others. Thus, in the reported test series, judgment from the compressive strength obtained after 7 days of immersion would have given first rank to the A-composition which is much more expensive and considerably less effective than the C-1 composition.

The confusion that exists in many places concerning the proper time of water immersion for specimens of a certain size, water affinity and permeability is paralleled by one concerning the change in exposure time required by change in size of test specimens. Although it is always best to determine such time requirements by direct test, the problem is illustrated by the following calculations made for cylindrical test specimens under the assumption that all water movement is normal to the cylindrical peripheral surface and that the hydrostatic head is negligible as compared with the physico-chemical water suction forces. These calculations have been made by J. Riera, undergraduate research assistant at the Soil Physics Laboratory of Princeton University.

Referring to the cross-sectional cylindrical slice of unit thickness shown in Figure 4, the driving force is $D = f(R - x)\, 2\pi.1$ where f is a constant that depends on the nature of both the liquid and the specimen. Consequently, the gradient

$$i = \frac{2\pi f(R - x)}{x}$$

and

$$\frac{dx}{dt} = k\frac{2\pi f(R - x)}{x}$$

If

$$2\pi fk = A, \text{ then}$$

$$\frac{dx}{dt} = A\frac{R - x}{x}$$

$$\frac{dxx}{R - x} = A\ dt$$

and

$$A\int_0^T dt = \int_0^W \frac{x}{R - x}\ dx$$

Performing this integration yields

$$T = \frac{1}{A}\left[R \ln \frac{R}{R - u} - u\right]$$

in which u is the distance from the specimen perimeter to the saturation line; that is, $u = x$.

Theoretically, the saturation time is infinite. Hence, u-values must be chosen that are physically significant but fall short of reaching R.

1. Assuming a u-value equal to 0.95 R,

$$T_S = \frac{R}{A}\left[\ln \frac{1}{1 - 0.95} - 0.95\right]$$

$$= \frac{R}{A}\left[3.00 - 0.95\right] = \frac{2.05}{A}R$$

Accordingly, the time necessary for saturation of the specimen would be proportional to its radius.

2. It is assumed that physically significant saturation is reached when a core of predetermined radius r is not yet saturated. The size of r should be a function of material and pore characteristics of the specimen. In this case,

$$T_S = \frac{1}{A}\left[R \ln \frac{R}{r} - (R - r)\right]$$

r, of course, is assumed to be constant for all specimens of a certain soil composition even if the specimens vary in size. Considering two specimens, one of radius R, the other of radius 2R, their respective saturation times will be

$$T_{S1} = \frac{1}{A}\left[R \ln \frac{R}{r} - (R - r)\right]$$

and

$$T_{S2} = \frac{1}{A}\left[2R \ln \frac{2R}{r} - (2R - r)\right]$$

Therefore,

$$\frac{T_{S1}}{T_{S2}} = \frac{\ln \frac{2R}{r} - \left(1 - \frac{r}{2R}\right)}{\ln \frac{R}{r} - \left(1 - \frac{r}{R}\right)}$$

$$= 2 \frac{\ln \frac{R}{r} + \ln 2 - 1 + \frac{r}{R} - \frac{r}{2R}}{\ln \frac{R}{r} - 1 + \frac{r}{R}}$$

$$= 2 \left(\frac{\ln \frac{R}{r} - 1 + \frac{r}{R}}{\ln \frac{R}{r} - 1 + \frac{r}{R}} + \frac{\ln 2 - \frac{r}{R}}{\ln \frac{R}{r} - 1 + \frac{r}{R}} \right)$$

$$= 2 + 2 \frac{\ln 2 - \frac{r}{R}}{\ln \frac{R}{r} - 1 + \frac{r}{R}}$$

If $r = 0.1R$, then $\frac{T_{S2}}{T_{S1}} = 2 + 2 \frac{\ln 2 - 0.05}{\ln 10 - 1 + 0.1} = 2 + 2 \left(\frac{0.69 - 0.05}{2.30 - 0.9}\right) = 2 + 2 \left(\frac{0.64}{1.4}\right)$ or $T_{S2} \cong 3\, T_{S1}$. For $r \to 0$, $T_{S2} = T_{S1}$.

These calculations show that times of immersion can not be chosen arbitrarily for specimens of different sizes, if they are to yield significant and comparable information.

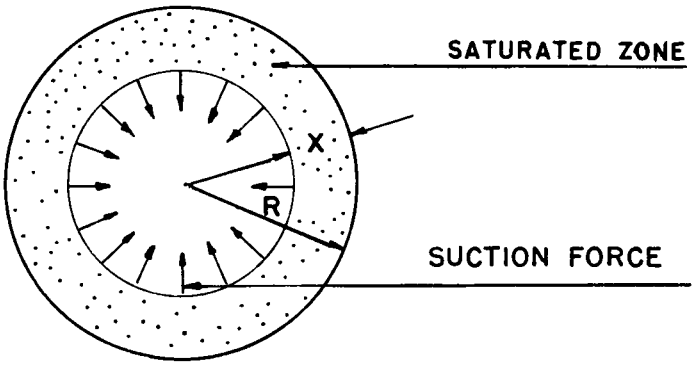

Figure 4.

SUMMARY AND CONCLUSIONS

Tests have been described and results presented on the use of cationically active condensation products of furfural with aniline and dichlor aniline with and without other admixtures by themselves and as additives to asphaltic bitumen in the waterproofing stabilization of clay soils.

1. One of these clay soils was used in the natural condition as well as in several monoionic modifications. The influence of the exchange ions on the stabilization susceptibility of the soil went parallel with that on the pH value. This is logical because the amines require association with H^+ ions to become cationically effective.

2. Increase in density of test specimens resulted in an increase in strength and weathering resistance as long as such density increase was not purchased at the price of using an amount of water insufficient to satisfy the affinity requirements of the soil, or in the case of soils that are susceptible to overcompaction because of strong aggregation of their clay mineral components in the natural state.

3. In the case of cationically active organic admixtures, it is important that (a) they contain as many potential cation equivalents per unit mass as possible and (b) that there

be enough H- ions in the system to change them into the ionically active ammonium ion derivative. If the natural soil is lacking in H- ions (that is, if its exchange complexes are largely saturated by other cations), H- ions must be introduced into the system by means of acid admixtures, preferably of types that their anions form insoluble salts with the exchange cations present in the natural soil.

4. Test methods employed for the evaluation of the effectiveness of stabilizing materials and procedures must be carefully analyzed with respect to the pertinent physical and physico-chemical phenomena involved if they are to give significant information.

By proper consideration of the known facts pertinent to bituminous soil stabilization and to the action of chemical additives, there is no greater uncertainty in the design of stabilized soil mixtures than there is in the design of hydraulic mortars and concretes in which air-entraining and dispersion agents are employed.

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