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Bulletin 304

Lime Stabilization: Properties, Mix Design, Construction Practices and Performance

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Contents

Accelerated Curing for Lime-Stabilized Soils

M. C. ANDAY, Highway Research Engineer, Virginia Council of Highway Investigation and Research, Charlottesville

This paper is concerned with the preliminary steps of a general study the ultimate goal of which is to develop specifications for the required unconfined compressive strength of lime-stabilized base and subbase materials. Specifically it deals with the possibility of establishing a correlation between the strengths of field-cured specimens and those of laboratory-cured specimens that might be used to predict the former from the latter.

Two soils were chosen for study. Harvard mold-size specimens were tested for unconfined compressive strength after being subjected to field curing and to laboratory curings of 120 and 140 F. Evaluation of the data consists of determining the time of accelerated curing required to achieve 45-day field strengths.

The limited data indicate that a strength prediction based on laboratory curing can be made with reasonable accuracy.

●IT OFTEN has been found difficult to obtain suitable base or subbase materials within reasonable hauling distances of construction projects. If the addition of a small percentage of lime can condition the in-place subgrade or bring a nearby borrow material within the limits of specifications, economy will result. In several instances in Virginia, lime stabilization could have been used to bring about such economy. In these instances, although a blanket material was available near the job, no material meeting the specifications could be found. The soils laboratory of the Virginia Council of Highway Investigation and Research has been asked on several occasions to investigate the possibility of stabilizing, with lime, some of the materials found in place or near certain projects.

On the basis of laboratory investigations several experimental projects were constructed but because no method of test or design had been correlated with field performance it was considered necessary that a method of determining the rate of strength gain of materials stabilized in the field be developed. Because no single soil property such as bearing value or strength can be considered as the most essential in highway performance, a practical laboratory test, which can be easily replicated, such as the unconfined compressive strength test, was favored for this study. It was believed that a specification requiring a given unconfined compressive strength of soil-lime mixtures and based on an accelerated curing of the test specimens could be developed (1).

This paper covers the preliminary data obtained during the summer of 1960 in the first step of the general study, the investigation of the possibility of predicting field strengths of lime-stabilized soil specimens on the basis of accelerated laboratory curing. In this paper, only the data obtained on the two soils tested is presented and discussed; no generalizations are made. It is hoped the findings can be generalized in the future, as more types of soil of different origins are studied.

The soils used in the study were taken from experimental lime-stabilization projects previously installed in Virginia (2). This was done in an effort to specify an unconfined compressive strength based on the accelerated laboratory curings and the performance of these soils under traffic.

LIMITATIONS OF THE STUDY

The measurement of the compressive strength of soils under pavements by coring involves such a number of variables that strength determinations obtained in this way are often not comparable to laboratory values. Therefore, no coring will be attempted. The specified value of the unconfined compressive strength would be based on observations of the performance of the previously installed projects and the results of tests on soil samples obtained from these projects.

This study did not consider such variables as movement of moisture or repetition and magnitude of loads but was concerned with the effect of maturity (temperature x time) on gain of strength. Only one percentage of lime, 5 percent by weight of the soil-lime mixture, was selected for the study.

TABLE 1

PHYSICAL PROPERTIES OF SOILS TESTED

Property	Soil A	Soil B
Percent passing No. 4	100	100
Percent passing No. 10	86	99
Percent passing No. 20	68	97
Percent passing No. 40	45	89
Percent passing No. 60	31	76
Percent passing No. 80	28	63
Percent passing No. 100	26	55
Percent passing No. 200	25	39
Percent silt (0.05 - 0.005 mm)	3	29
Percent clay (x 0.005 mm)	21	4
LL	35	40
PI	11	N.P.
Sp Gr	2.70	2.75
Max den. plain soil, pcf (AASHO-T99)	118.3	96.0
Max den. with 5% lime, pcf (AASHO-T99)	117.0	94.8
Opt moist cont plain soil, %	13.0	23.2
Opt moist cont with 5% lime, %	13.4	24.4
HRB classification	A-2-6(0)	A-4(1)

The precision of the equipment used, such as the oven and the scales, is of the same order as that of an average laboratory. The temperatures maintained during laboratory curings were constant within ± 3 F.

PROPERTIES OF THE SOILS TESTED

Two soils were chosen for study. The physical properties of these soils are given in Table 1.

Soil A is a clay gravel, common in the northeastern part of the state. The particular sample tested was taken from a borrow pit in Fairfax County. Previous tests on soil from this pit showed that it reacts very well with lime. When stabilized with lime, it produces fairly high unconfined compressive strengths and a very high CBR value, and resists the freeze-thaw and wet-dry tests satisfactorily (3). Due to the size of the unconfined compression test specimens only the portion passing the No. 4 screen was sampled. However, the pit contains from 30 to 60 percent material retained on the No. 4 screen. It is believed that when stabilized with lime this material could be used as an alternate to the available local crushed aggregate.

Soil B is a micaceous silty soil of fairly common occurrence in Virginia and is usually regarded as being a "troublesome soil". A similar soil was stabilized with various percentages of lime, cement, and lime-fly ash and used as a subbase on an experimental section of project 1770-03, US 58, Patrick County. The project was built in 1956 and since then valuable data have been obtained on the performance of this soil (2).

About 600 lb of each soil were air dried, sieved through the No. 4 screen, and stored in closed containers. The moisture content of the soil in the containers was determined prior to each molding or testing.

TEST PROCEDURE

In this study the unconfined compressive strengths of field-cured specimens were compared to those of specimens cured at 140 and 120 F in the laboratory when both were stabilized with 5 percent lime. However, because the Virginia Department of Highways uses the CBR method of design for flexible pavements, it was thought appropriate to include some CBR specimens in the field curing. The values obtained from the field-cured CBR's were compared to the values obtained from specimens cured in the moist room.

Size and Number of Specimens

Statistical calculations made in pilot studies indicated that for the unconfined compression test at least 8 specimens for each period of oven curing and at least 11 specimens for each period of field curing had to be used to obtain satisfactory precision at the 95 percent confidence level. For this reason the Harvard miniature mold size (1.34-in. diameter and 2.8-in. height) was chosen as the size of the unconfined compression test specimens to keep the amount of soil needed to a minimum and to obtain a homogeneous mixture. However, due to the large size of the CBR specimens, only two were used for each period of field and moist-room curings.

Molding

The maximum density and the optimum moisture content of each soil, with and without lime, were determined in accordance with AASHO T99-57 Method A.

The Harvard miniature mold was used for the compression specimens; however, the spring hammer furnished by the manufacturer was replaced with a homemade drop hammer. The drop hammer was constructed to have a 1-lb weight falling 10 in. The diameter of the hammer head was one-half that of the mold. Using the Harvard miniature mold and this hammer, it was observed that at optimum moisture content five layers and 15 blows per layer would produce a density very close to the maximum density obtained by AASHO T99-57 Method A. The Virginia standard procedure was used to mold the CBR specimens (4).

All specimens were molded close to their optimum moisture contents, wrapped with aluminum foil, and coated with a special wax to preserve the molding moisture content during the curing period. The CBR specimens were wrapped in their molds.

Field Curing

Forty-five days of field curing was assumed to be a reasonable period, because it was believed that this would be the maximum time allowed for lime to react before the roads were opened to traffic. However, to obtain a better estimate of what happens when a soil is cured under simulated field conditions, both the CBR and the unconfined compression test specimens were cured for 30, 45 and 60 days.

Field Curing Schedule.—The study was started in May 1960. At that time 33 unconfined compression test specimens and 6 CBR specimens were molded and subjected to field curing. At 30, 45 and 60 days, respectively, 11 of the compression specimens and two of the CBR specimens were tested. These 39 specimens are referred to in this paper as Series I.

In July 1960 another set of 33 unconfined compression test specimens and 6 CBR specimens were molded, subjected to field curing, and then tested. These specimens are referred to as Series II.

Specimens for Series III were molded and subjected to field curing in September

1960. However, because tests on these specimens were not completed at the time of this writing, this paper is concerned with only Series I and Series II. The outline of the field-curing schedule and the accumulated maturities at each date of testing are shown in the Appendix (Table 3).

Figure 1 shows the depths at which the test specimens were located in the simulated road. These dimensions are typical flexible pavement design coverages for bases and subbases used in Virginia. For this reason in some instances reference will be made to soils A and B as base and subbase soils, respectively.

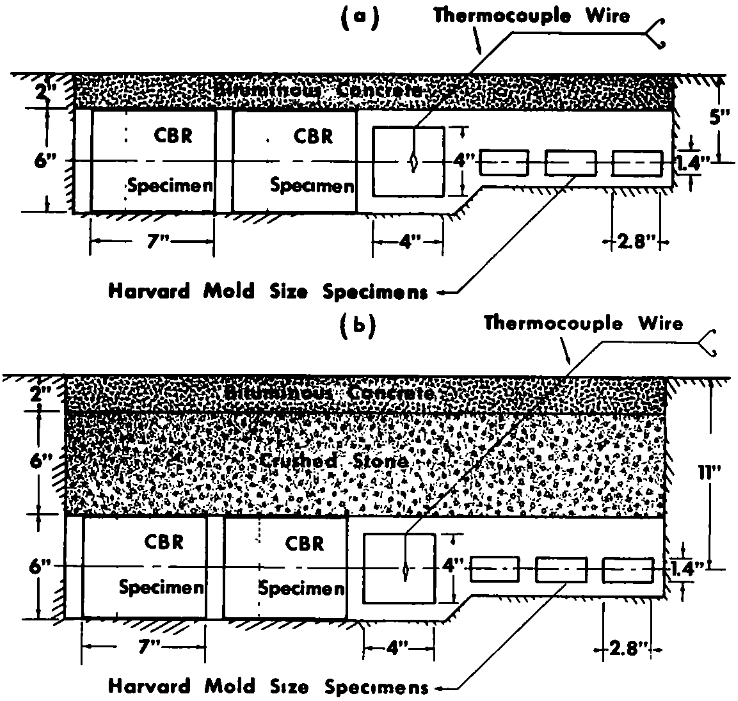

Figure 1. Sketch illustrating the depth of field-cured specimens: (a) base soil specimens, and (b) subbase soil specimens.

Laboratory Curing

On the basis of previous experience with accelerated curing, 0.5, 1, 3 and 5 days of oven curing were employed, both at 120 and 140 F for the unconfined compressive strength correlation. CBR specimens were cured for 3, 7, 14 and 21 days in the moist room at 70 F.

Testing

Unconfined compression test specimens were tested at a rate of strain of about 0.12 percent per second. The Virginia standard procedure was used for testing the CBR specimens. Moisture contents of the test specimens were determined after testing to indicate the loss of moisture during each curing period.

Maturity

It is known that time and temperature have important effects on the reaction and hardening of cemented mixtures. Experiments with concrete indicate that, other variables being constant, the strength of concrete is a function of its maturity (reckoned in temperature-time) at any curing temperature (5, 6).

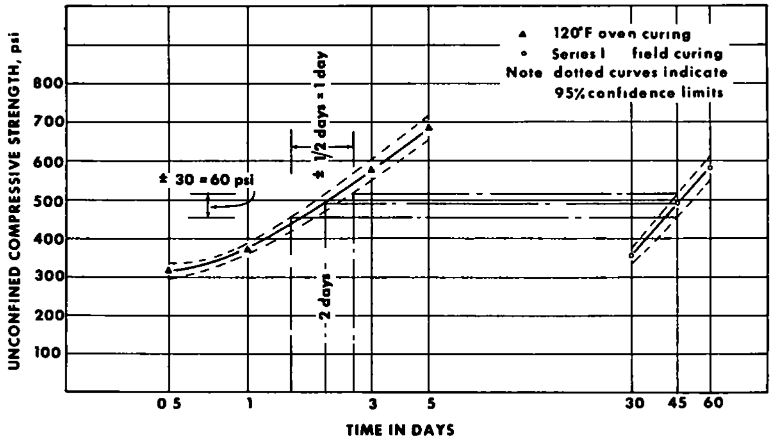

Figure 2. Comparison of Series I field curing and 120 F oven curing for Soil A.

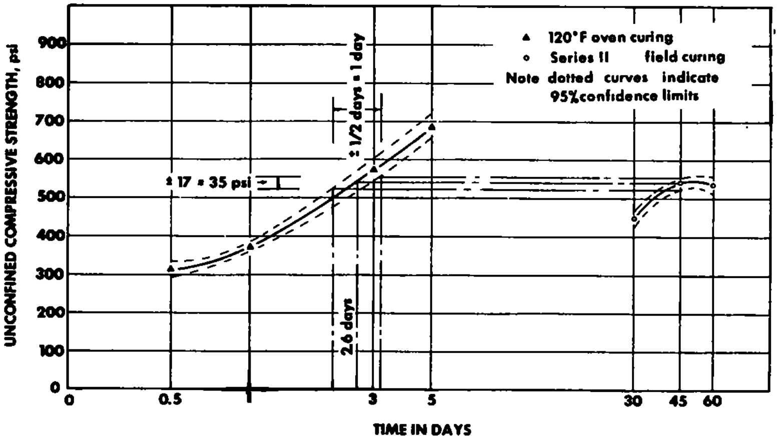

Figure 3. Comparison of Series II field curing and 120 F oven curing for Soil A.

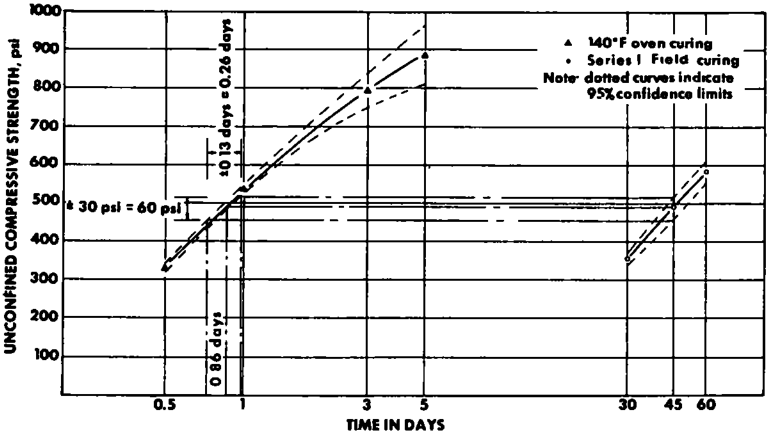

Figure 4. Comparison of Series I field curing and 140 F oven curing for Soil A.

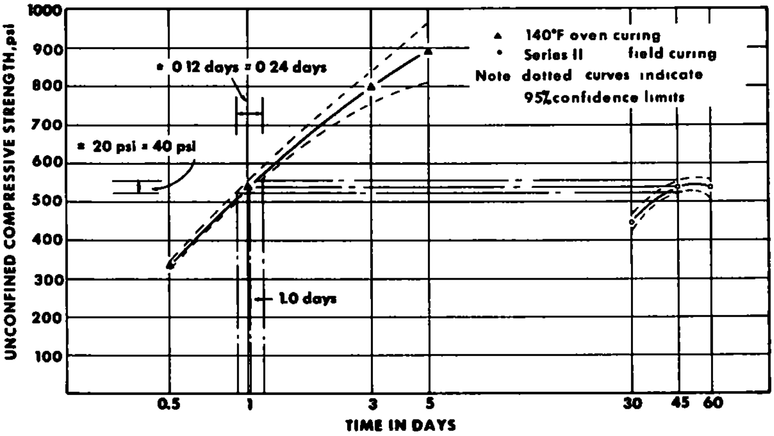

Figure 5. Comparison of Series II field curing and 140 F oven curing for Soil A.

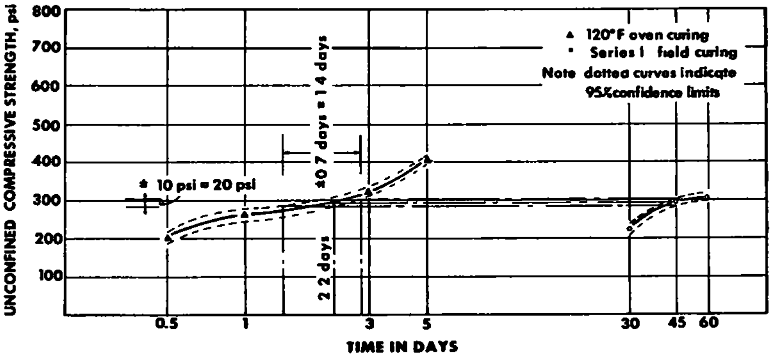

Figure 6. Comparison of Series I field curing and 120 F oven curing for Soil B.

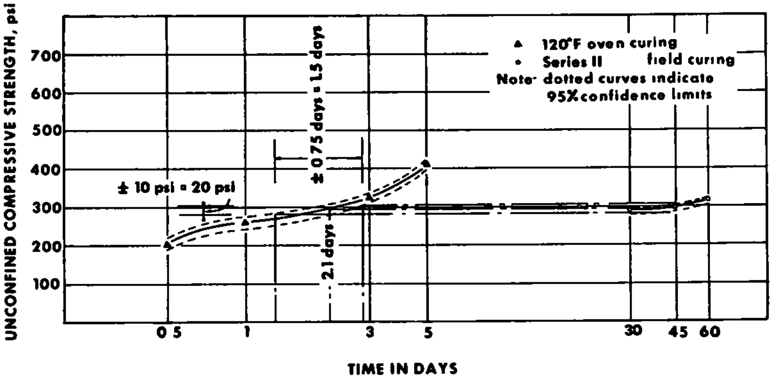

Figure 7. Comparison of Series II field curing and 120 F oven curing for Soil B.

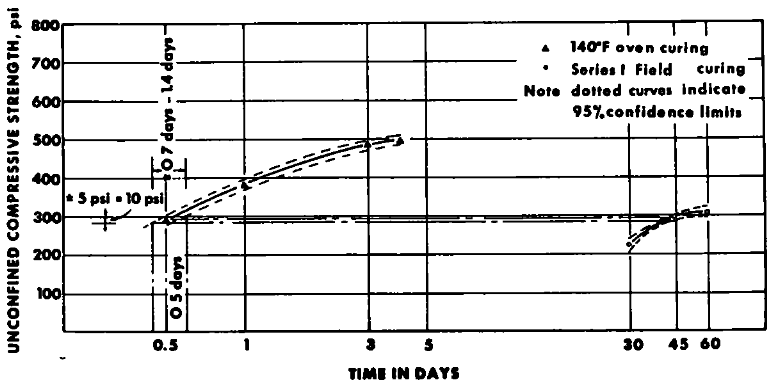

Figure 8. Comparison of Series I field curing and 140 F oven curing for Soil B.

To determine the feasibility of this approach for soil-lime mixtures it was decided to investigate the maturity of field-cured specimens. Because no temperature recorder was available at the time, a thermocouple was installed for each soil at its average depth (Fig. 1). It was hoped that the temperatures of the subbase and base specimens could be predicted by conducting 36-hr soil temperature surveys and correlating these surveys to the air temperature data available at the Council.

TEMPERATURE STUDIES

Three temperature correlation studies were conducted, two in the middle of the summer and one late in the summer. During these studies the air, base, and subbase temperatures were recorded hourly for about 30-36 hr by the use of the thermocouple. The results of these studies are shown in the Appendix (Fig. 12 and Table 4). The purpose of these studies was to investigate the possibility of using the available data on air temperatures, with a correction factor, to predict the temperatures prevailing at the base and subbase levels.

RESULTS

The unconfined compressive strengths of the field-cured and laboratory-cured specimens vs their ages at the time of test are shown in Figures 2 through 9.

In these figures the average unconfined compressive strength and the 95 percent confidence limits are plotted against curing period. (For actual values see the Appendix (Tables 5 and 6)).

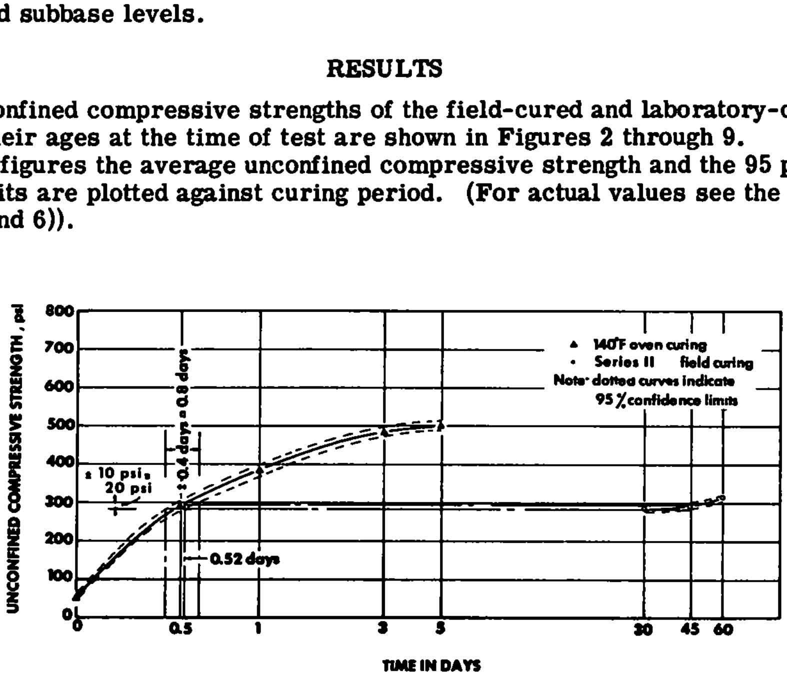

Figure 9. Comparison of Series II field curing and 140 F oven curing for Soil B.

TABLE 2

TIME OF ACCELERATED CURING REQUIRED TO ACHIEVE 45-DAY FIELD STRENGTH

		Accelerated Curing Days				Strength, psi			
Soil	Oven Curing Temp and Series	Min	Max	Avg	Range	Min	Max	Avg	Range
A	120-I	1.5	2.5	2.0	1.0	455	515	490	60
	120-II	2.1	3.2	2.6	1.1	520	555	540	35
	140-I	0.72	0.98	0.86	0.26	455	515	490	60
	140-II	0.92	1.15	1.00	0.24	520	555	540	35
B	120-I	1.4	2.8	2.2	1.4	290	305	295	15
	120-II	1.3	2.8	2.1	1.5	280	300	290	20
	140-I	0.44	0.60	0.50	0.16	290	305	295	15
	140-II	0.44	0.59	0.52	0.15	280	300	290	20

From these curves the time required for laboratory-cured specimens to reach a strength equivalent to that of a field-cured specimen cured 45 days can be determined. These determinations are summarized in Table 2.

From Table 2 it is interesting to note that the average laboratory-curing periods, corresponding to 45 days of field curing, are 2.0 to 2.6 days at 120 F and 0.5 to 1.0 days at 140 F (Col. 5), and that the range of strength values does not exceed 60 psi (Col. 10). Because the temperatures and their durations, namely the maturities of Series I and Series II, were different but close to each other (Appendix, Table 4), this amount of variation in values was not unexpected.

Figure 10 shows the effect of maturity on the unconfined compressive strength of both soils. In this figure the strength values obtained from Series I and II are plotted against the maturity of the specimens at the time of test. It should be pointed out that the maturities plotted are those of the air and not of the subbase or base. However, as can be seen from the Appendix, the average values of air, base and subbase temperatures obtained during the first two temperature correlation studies in the middle of the summer are very close.

Figure 10 indicates that although the strength values of the two soils did not follow the same pattern as would be the case with concrete, each gained strength in relation to its maturity.

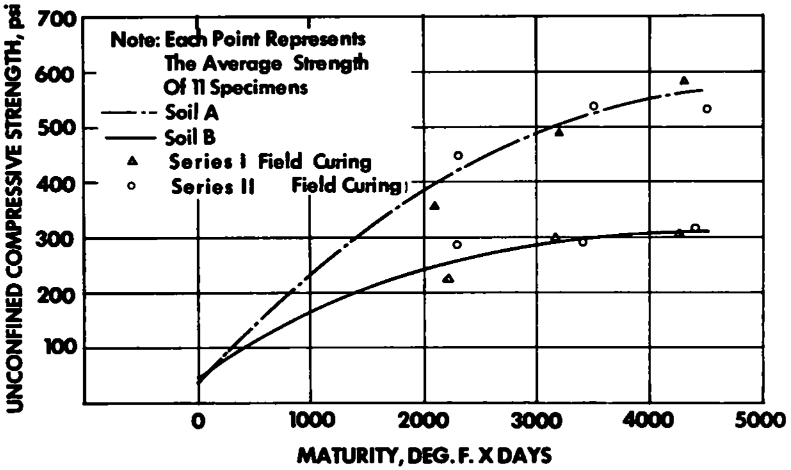

Figure 10. Effect of maturity on unconfined compressive strength.

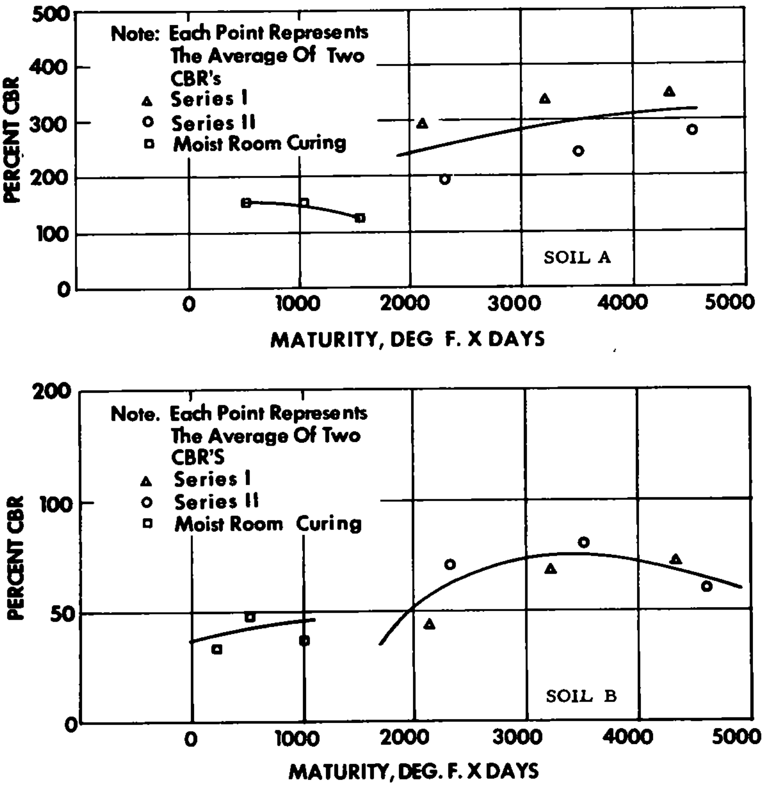

Figure 11. Effect of maturity on the CBR value.

The attempt to establish a field-moist-room curing correlation for the CBR specimens proved to be an unrealistic approach. However to arrive at a significant conclusion testing was continued through Series II. Figure 11 summarizes the results of tests on both soils. The actual strength values of the CBR specimens are given in the Appendix (Table 7).

CONCLUSIONS

The following conclusions derived from this research are restricted to the two soils studied and the use of 5 percent lime as the stabilizing additive.

1. The unconfined compressive strength of specimens field cured for approximately 45 days at summer temperatures could be predicted by an accelerated laboratory curing of either 18 hr at 140 F or two days at 120 F. However, 120 F curing is preferred for the following reasons: (a) less condensation between the specimen and the protective coating during curing; (b) a lower, therefore, a more realistic temperature; (c) convenience of curing time; and (d) increased accuracy obtained with small slopes of the strength-time curves.
2. The strengths of these stabilized soils will be a function of their maturities, when subjected to field curing.
3. The soils' CBR values will increase manyfold. However, these values are sometimes so high as to be unrealistic. Also, due to the size of the specimens and the amount of soil involved, not enough CBR test specimens can be made for statistical evaluations. Therefore the attempted field-moist-room curing correlation for the CBR specimens proved unsuccessful.

FURTHER WORK PLANNED

It is planned to continue the study during the summer of 1961 to generalize the findings reported here. More soils from lime stabilization projects previously installed in Virginia will be used.

Once an accelerated curing period is established, specifications for testing lime-stabilized soils based on strength data obtained from the soils studied will be devised.

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Special thanks go to Mr. T.E. Shelburne, Director of Research, for his guidance and supervision.

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Appendix

TABLE 3

FIELD CURING SCHEDULE

		Series I		Series II	
Soil	Age of Field Curing	Date 1960	Maturity[a] (deg Fx days)	Date 1960	Maturity[a] (deg Fx days)
A	Molding	May 17	0	July 18	0
	30 days	June 16	2,103	Aug 17	2,298
	45 days	July 1	3,194	Sept 2	3,510
	60 days	July 16	4,301	Sept 16	4,502
B	Molding	May 25	0	July 25	0
	30 days	June 25	2,192	Aug 24	2,278
	45 days	July 8	3,142	Sept 8	3,412
	60 days	July 23	4,267	Sept 23	4,430

[a] Maturity = Air temp (in deg F) x days.

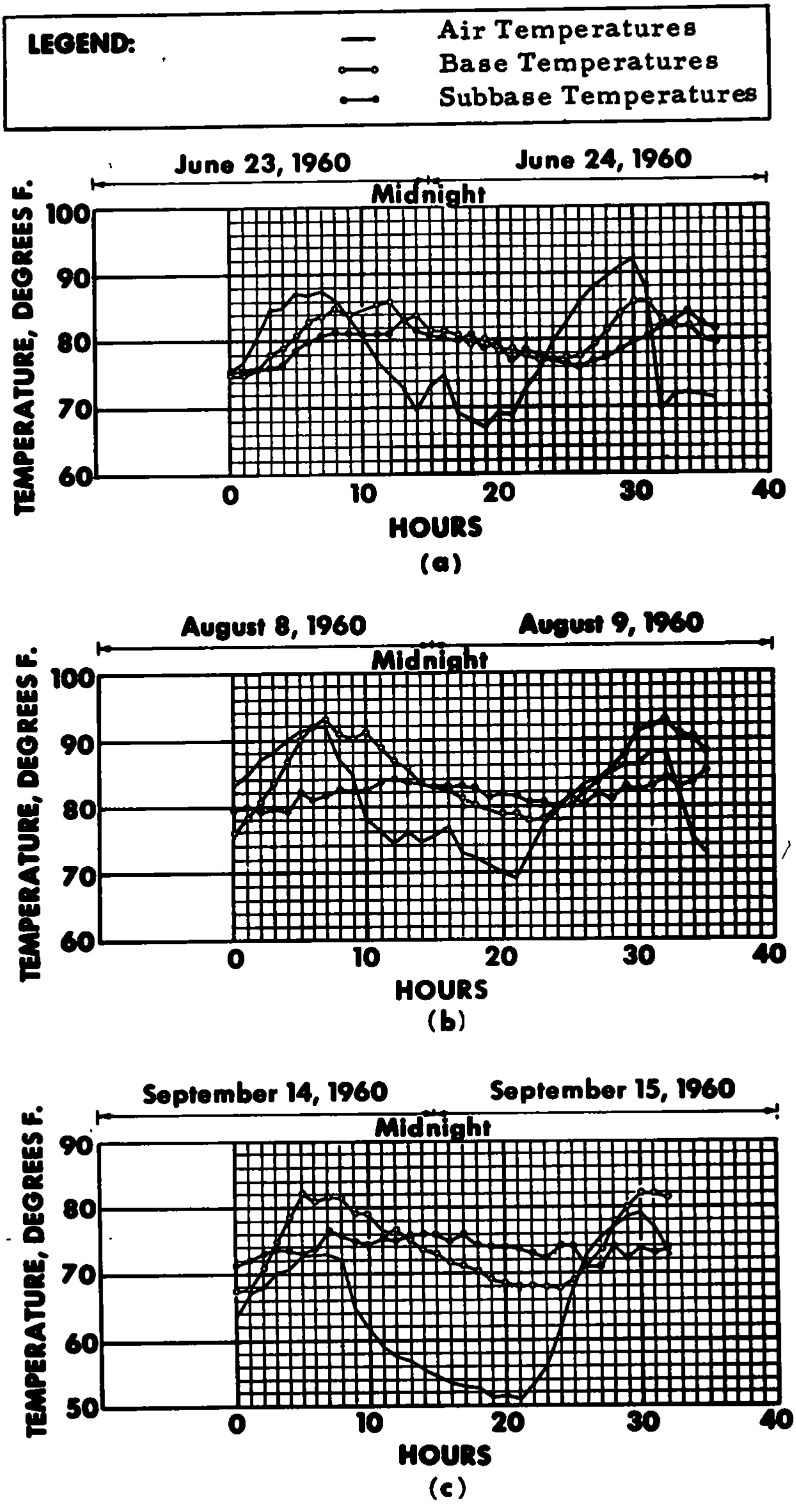

Figure 12. Charts from temperature correlation.

TABLE 4

SUMMARY OF SUBBASE, BASE AND AIR-TEMPERATURE CORRELATIONS

Trial No.	1			2			3		
Date	June 23, 1960			August 8, 1960			Sept 14, 1960		
Depth of Thermocouple, in.	11	5	Air	11	5	Air	11	5	Air
Min daily temp, deg F	76.0	77.0	67.5	80.0	78.0	69.3	71.0	67.8	51
Max daily temp, deg F	83.0	86.0	87.5	84.0	92.8	92.6	76.5	82.0	73
Mean temp, deg F[a]	79.5	81.5	77.5	82.0	85.4	81.0	73.8	74.9	62
Dev from mean air temp, deg F	+2.0	+4.0	-	+1.0	+4.4	-	+11.8	+12.9	-
Avg temp, deg F[b]	79.4	80.9	78.4	82.0	85.1	80.9	73.9	74.4	64.3
Dev from avg air temp, deg F	+1.0	+2.5	-	+1.1	+4.3	-	+9.6	10.1	-

[a] Mean temperature = average of maximum and minimum temperatures.
[b] Average temperature = average of hourly temperatures.

TABLE 5

EFFECT OF AGE AND TYPE OF CURING ON STRENGTH OF SOIL A

Age in Days	Type of Curing	Moisture Cont at Test, %	Unconfined Compressive Strength, psi		
			Mean	95% Confidence Limits, ±	V[a]
0	Field, Ser I	12.6	39	-	-
	Field, Ser II	12.6	39	-	-
30	Field, Ser I	12.9	354	16	6.1
	Field, Ser II	12.5	445	22	7.3
45	Field, Ser I	12.3	487	31	9.3
	Field, Ser II	12.9	538	15	4.2
60	Field, Ser I	12.1	580	27	6.9
	Field, Ser II	12.7	532	23	6.5
0.5	-	12.1	309	10	4.0
1	120 F	12.2	371	8	2.5
3	Oven	12.3	574	27	5.6
5	-	12.3	681	27	4.7
0.5	-	12.7	331	8	3.0
1	140 F	12.5	535	14	3.1
3	Oven	12.9	794	45	6.8
5	-	12.8	887	80	8.6

[a] V = coefficient of variation.

TABLE 6

EFFECT OF AGE AND TYPE OF CURING ON STRENGTH OF SOIL B

			Unconfined Compressive Strength, psi		
Age in Days	Type of Curing	Moisture Cont at Test, %	Mean	95% Confidence Limits, ±	V[a]
0	Field, Ser I	23.9	48	3	-
	Field, Ser II	23.9	48	3	-
30	Field, Ser I	26.4	222	17	9.8
	Field, Ser II	23.7	286	7	3.6
45	Field, Ser I	23.2	295	6	3.0
	Field, Ser II	22.9	289	8	4.0
60	Field, Ser I	23.2	307	12	5.8
	Field, Ser II	23.4	314	7	3.1
0.5	-	23.5	201	14	7.6
1	120 F	23.4	262	16	8.1
3	Oven	23.4	321	10	3.8
5	-	23.3	407	11	3.2
0.5	-	23.1	291	12	4.8
1	140 F	23.1	383	15	5.1
3	Oven	23.2	483	12	3.0
5	-	23.2	498	12	3.0

[a] V = coefficient of variation.

TABLE 7

RESULTS OF CBR TESTS

	Soil A with 5% Lime					Soil B with 5% Lime				
		Density at	Moisture (%)				Density at	Moisture (%)		
Type of Curing	Days Curing	Molding (%)	At Molding	At Test	CBR (%)	Days Curing	Molding (%)	At Molding	At Test	CBR (%)
Moist room		-	-	-	-		98.7	24.3	27.0	34
	3	-	-	-	-	3	98.5	24.5	29.0	34
		101.5	13.1	13.5	152		98.5	22.8	25.1	51
	7	101.0	13.2	14.3	150	7	99.4	22.7	27.1	45
		102.8	13.0	13.4	148		98.1	24.9	26.1	36
	14	97.6	12.9	13.7	146	14	98.2	24.7	23.7	38
		100.8	13.3	14.0	111		-	-	-	-
	21	101.1	13.3	13.5	120		-	-	-	-
Series I	30	100.6	12.8	13.5	300	30	95.2	28.1	26.9	29
		102.1	12.6	12.4	287		98.6	22.9	26.2	56
		100.2	12.9	13.1	347		99.9	23.4	26.3	68
	45	100.2	13.0	13.4	320	45	99.2	22.6	27.0	68
		99.8	13.0	12.8	410		99.5	22.8	26.7	68
	60	101.3	13.3	12.1	292	60	99.1	23.4	25.9	75
Series II		100.9	13.2	13.0	203		98.6	23.1	25.9	72
	30	101.0	13.5	13.1	193	30	100.0	23.3	25.7	70
		100.0	13.6	14.3	220		-	-	27.6	76
	45	100.6	13.4	13.6	265	45	-	-	24.7	83
		100.9	13.2	12.9	328		-	-	22.7	61
	60	101.1	12.8	13.8	238	60	-	-	25.3	56
No curing Plain soil	0	102.0	12.3	12.9	28	0	100.0	23.0	26.6	8

Influence of Time Between Mixing and Compaction on Properties of a Lime-Stabilized Expansive Clay

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The effectiveness of 4 percent dolomitic hydrated lime as a stabilizer for an organic expansive clay soil has been investigated. This soil is typical of some of the "adobe" soils encountered in California. Samples of treated and untreated soil were compacted to 90 percent of maximum density for each mixture, as obtained by modified AASHO compactive effort, over a range of water contents. The effectiveness of stabilization was measured, in most cases, by triaxial compression tests at various ages after compaction for both soaked and unsoaked samples.

It was found that the time interval between mixing (of the soil, water and lime) and compaction could have a pronounced effect on the properties of the treated soil. For samples prepared using constant compactive effort, a delay of 24 hr between mixing and compaction led to as much as 8 pcf decrease in density and 30 percent decrease in as-cured strength from the values for samples compacted immediately after mixing. The expansion and soaked strength characteristics of treated samples were also adversely affected by a delay between mixing and compaction.

The principal factor responsible for this behavior was found to be the decrease in density (at constant compactive effort) accompanying delays between mixing and compaction. The unsoaked and soaked strength and the swell values for samples compacted immediately after mixing and 24 hr after mixing were essentially the same if the samples were prepared to the same density. Thus, in practice, the beneficial effects of a delay between mixing and compaction in terms of the "mellowing" action of the lime and the consequent improvement in mixing and handling properties may justify an increased compactive effort to maintain the desired density.

A 4 percent lime treatment was found to be an effective stabilizer of the organic expansive clay. Strength improvement in the as-cured samples was not great; however, the effectiveness of lime in reducing the swell of samples soaked under a surcharge pressure of 0.1 kg per sq cm was very marked and resulted in soaked strengths

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of treated samples up to seven times greater than for untreated samples. Peak compressive strength was developed at considerably lower strains for treated than for untreated samples.

A consideration of the data and the results of chemical analyses suggests that, for the soil studied, the beneficial effects of lime treatment are caused primarily by flocculation of the soil structure and a decrease in the water sensitivity of the clay minerals. Cementation effects are believed to be of minor importance in this soil.

● ORGANIC expansive clays, locally termed "adobe," are encountered over large areas of California. Their high water sensitivity leads to a high swell and loss of strength on wetting, and shrinkage and cracking on drying. Thus, in their natural state these soils are undesirable for use as highway subgrade, subbase or base course materials. Effective low-cost stabilization might be expected to have wide application. Lime treatment of such soils for use as secondary road base and subbase courses has been employed in some areas of the state. Although in general the results have been very satisfactory, little specific information concerning the over-all efficiency of lime treatment or the nature of the reactions in these soils is available. Consequently, a laboratory study was initiated for the purpose of better defining the effects of lime treatment. A soil typical of some of the adobes was used.

It was noted early in the investigation that, for a constant compactive effort, the sample densities were sensitive to the time interval between mixing (of the soil, lime and water) and compaction. It has long been believed that one of the practical advantages of lime stabilization is that the time interval between mixing and compaction is not critical, and that the mixed material may be allowed to stand for as long as two days and then be reworked prior to compaction without detrimental effects (1, 2). It therefore appeared desirable to initiate a more detailed study of the effect of time between mixing and compaction, with particular reference to ascertaining whether or not the decreases in densities were accompanied by decreases in strength. Such a study was carried out concurrently with the initially planned investigations by maintaining careful control of sample preparation times throughout the test program.

This paper reports the results of studies both on the effects on properties of the time interval between mixing and compaction and the efficiency of lime as a stabilizer for an organic expansive clay.

Two features of this investigation are considered of particular importance in evaluating stabilizer effectiveness. One of these was that a comparison between treated and untreated samples was made on the basis of identically processed treated and untreated samples, with the untreated samples subjected to the same curing and testing conditions as the treated samples. The use of such a control procedure has the obvious advantage of showing up any effects such as thixotropy in the untreated soil, which, if undetected, might tend to give a false picture of the stabilizer effectiveness. A second feature was to compare treated and untreated samples compacted to essentially the same relative compaction based on their own maximum density for a given compactive effort, rather than samples compacted to the same density. Inasmuch as the addition of lime reduces the maximum density and increases the optimum moisture content relative to the values for untreated soil, this meant comparing samples of treated soil with samples of untreated soil compacted to a higher dry density. Comparison of treated and untreated samples on this basis would appear to more closely duplicate probable field behavior than a comparison on the basis of the same density, because specifications would likely require compaction to a relative density based on the compaction curve for the particular mixture used.

MATERIALS

The clay soil used in the studies was a gray expansive material containing about 8 percent organic matter by weight. It was obtained from the area adjacent to the Soil Mechanics and Bituminous Materials Laboratory of the University of California in

Richmond, Calif. Pertinent characteristics of this soil are summarized in Table 1. It may be noted that on the basis of the plasticity characteristics, the soil would be classified as CL in the Unified Soil Classification System. Analysis of the exchangeable cation complex has shown that calcium and magnesium account for 20.4 milliequivalents per 100 gm of the total exchange capacity of 27.0 milliequivalents per 100 gm. X-ray analyses have indicated a mixture of montmorillonite and illite to be the dominant clay material. The soil possesses undesirable physical characteristics in that it swells and becomes sticky when wet and shrinks excessively with the formation of large shrinkage cracks during dry periods.

TABLE 1

SOIL PROPERTIES[a]

1. General description: Gray expansive clay containing about 8% organic matter; some small roots, sticky when wet and exhibiting high shrinkage when dried.
2. Grain size data:

Size, mm	% Finer (by wt)
0.074	77
0.005	43
0.001	26

3. Plasticity:

	Untreated Soil	Soil + 4% Lime (by wt) 24 Hr after Mixing
Liquid Limit	46%	53%
Plastic Limit	21%	35%
Plasticity Index	25%	18%

4. Dominant clay minerals: Illite and montmorillonite mixture.
5. Cation exchange and free cation data:

Cation exchange capacity - 27 milliequivalents/100 gm.
Specific ion concentrations: (all values in milliequivalents/100 gm).

Ion	Exchangeable	Water Soluble	Total
Sodium	0.72	0.80	1.52
Potassium	0.40	0.22	0.62
Calcium + Magnesium	20.4	2.2	22.8

[a] Data obtained by Goldberg (3), formerly of ITTE, Univ. of Calif., Berkeley.

The lime used in these studies was a dolomitic hydrated lime having the composition indicated in Table 2. It should be noted that this lime is a commercial agricultural hydrated lime.

EXPERIMENTAL PROCEDURES

The soil was air dried and passed through a No. 8 sieve prior to preparation of any samples. A lime treatment level of 4 percent by weight of oven dry soil was selected for all tests. The appropriate weights of air dry soil and lime were blended in a paddle-type mixer and then sufficient water was added from a spray bottle to give the desired water content. Mixing was continued for 3 min. The mixed samples were then placed in

TABLE 2

COMPOSITION OF DOLOMITIC HYDRATED LIME

Compound	Percentage (by wt)
Calcium hydroxide	37
Magnesium hydroxide	41
Calcium carbonate	22

plastic bags and stored in a humid room until needed for compaction. Compaction was carried out at specified time intervals after the end of the initial mixing period. The samples were hand mixed in the plastic bags immediately prior to compaction.

Large samples (4 in. by 4 in.) were prepared at the beginning of the investigation for determination of compaction curves using modified AASHO compactive effort. Compaction curves were determined for the untreated soil, soil plus 4 percent lime compacted immediately after mixing, and soil plus lime compacted 24 hr after mixing. The samples used to obtain these compaction curves were put aside in an unwrapped condition in shallow pans in a humid room. These samples were later used as a source of supplementary information on the effect of the time interval between mixing and compaction on the properties of the treated soil. At the time these samples were tested they had aged for 215 days, and free water had accumulated in the pans to a depth of slightly more than 1 in. The strength of these samples was evaluated by means of the unconfined compression test using a strain rate of 0.05 in. per minute.

All other samples were prepared by kneading compaction using the Harvard miniature compaction apparatus. Samples were compacted over a range of water contents to a relative density of 90 percent based on the modified AASHO compaction curves. The compactive efforts required to prepare samples to the desired densities were determined by trial for each water content investigated. The resulting samples, which were 1.4 in. in diameter and 3.5 in. long, were carefully weighed and measured and then placed between a lucite cap and base and wrapped with two thin rubber membranes with a layer of silicone grease between. The samples were allowed to cure under water for specified periods of time. At the end of the curing period the samples were removed from the storage tank, unwrapped, weighed, and measured to insure that they had maintained their initial composition during storage. The samples were then rewrapped with new membranes and either tested in triaxial compression under undrained conditions using rate of strain control (0.058 in. per min) and a confining pressure of 1 kg per sq cm, or soaked prior to testing. Specimens to be soaked were placed in triaxial cells with porous stones at each end of the sample. The samples were maintained under a confining pressure of 0.1 kg per sq cm and given access to free water through the porous stones. Soaking was continued for a period of 7 days, after which the samples were weighed, measured and tested in undrained triaxial compression in the same manner as the unsoaked specimens.

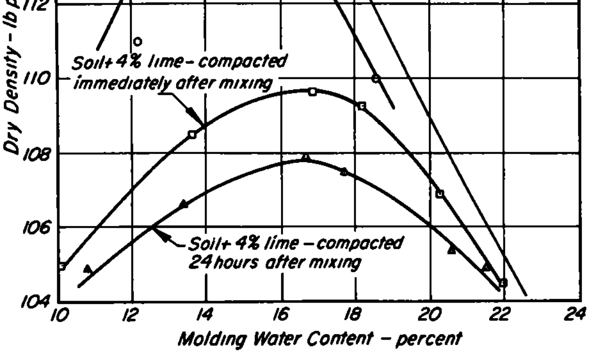


Figure 1. Modified AASHO compaction curves for treated and untreated soil.

RESULTS AND DISCUSSION

Density vs Water Content Relationships

Modified AASHO compaction curves for the untreated soil, soil + 4 percent lime compacted immediately after mixing, and soil + 4 percent lime compacted 24 hr after mixing are shown in Figure 1. Two effects may be noted from these curves. First, the effect of lime treatment is to increase the optimum moisture content and decrease the maximum density, a commonly observed consequence of lime addition. This may be attributed primarily to the flocculating effect of lime on the soil structure. The second obvious effect is the lower densities obtained by the modified AASHO effort for the treated samples allowed to age for 24 hr between mixing and compaction in comparison with the densities obtained for the treated samples compacted immediately after mixing.

Effect of Time Interval Between Mixing and Compaction on Density and Strength of Samples Prepared Using Constant Compactive Effort

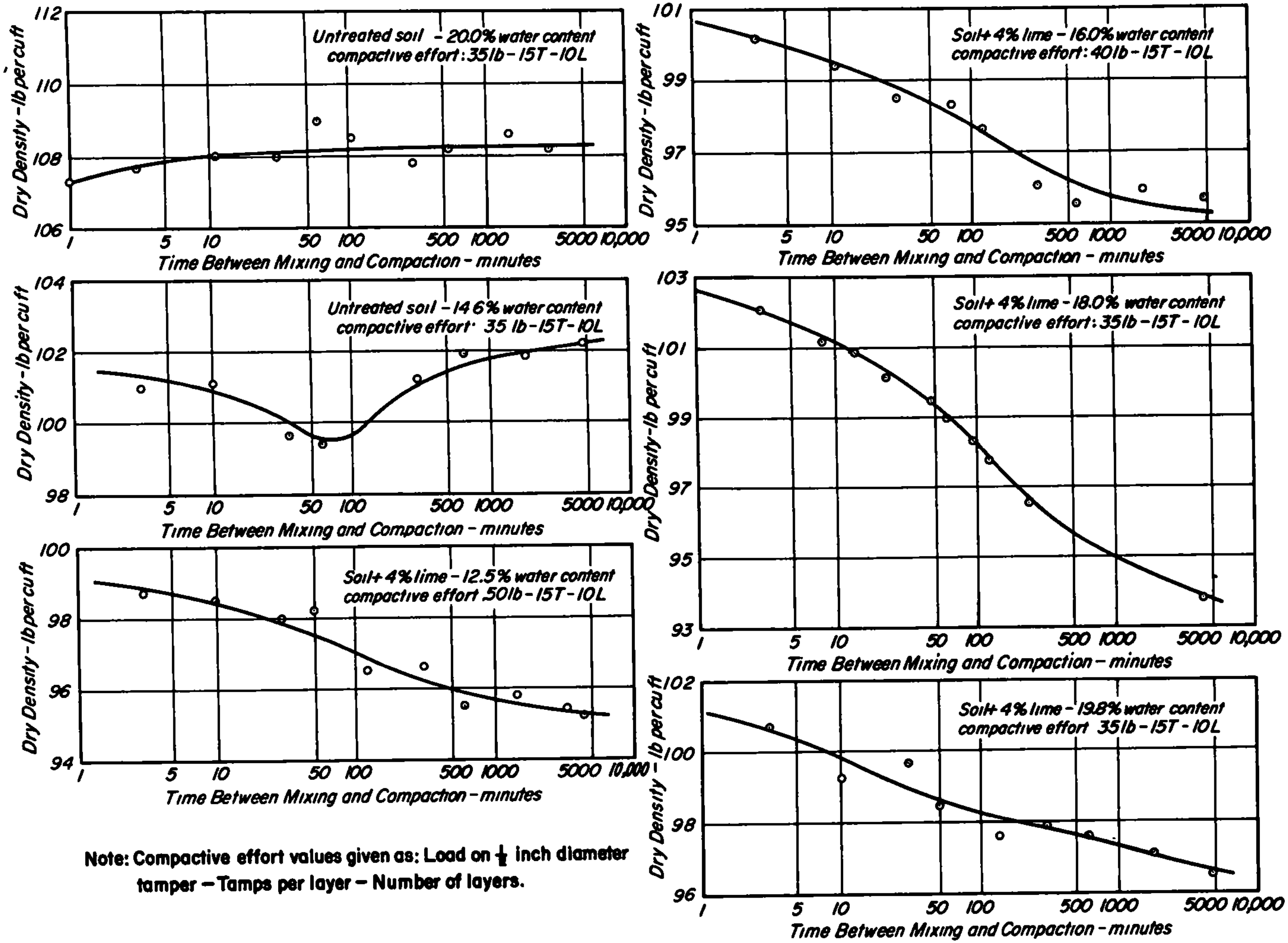
Series of samples of treated and untreated soil were prepared by kneading compaction at several water contents. For each series a compactive effort was selected such that a sample compacted immediately after mixing would have a relative density between 90 and 95 percent based on the modified AASHO compaction curves (Fig. 1). Samples within any given series were then prepared at different times after mixing, using the same, preselected compactive effort for each sample.

Figure 2 shows the effect of time interval between mixing and compaction on density, and Figure 3 shows the effect of time interval between mixing and compaction on compressive strength. The compressive strengths were determined after a 14-day curing period at constant water content. The same compactive effort was used for the preparation of all samples in any given series.

Figure 2 shows that the time interval between mixing and compaction has little effect on the densities obtained for the untreated control specimens at the two water contents investigated. No explanation can be offered for the slight dip in the density vs time interval curve for samples prepared at 14.6 percent molding water content. On the other hand, the data for the lime-treated specimens show a pronounced decrease in density with increased time intervals between mixing and compaction for all water contents investigated. It may be seen from Figure 3 that, although the compressive strengths of as-cured specimens of untreated soil are insensitive to variations in time between mixing and compaction, the lime-treated samples show a very significant decrease in compressive strength as this time interval increases. On a percentage basis, if compaction was delayed 24 hr the strength decrease was as much as 30 percent of the strength of samples prepared immediately after mixing. It should be kept in mind, however, that these strengths were determined for samples of progressively decreasing density.

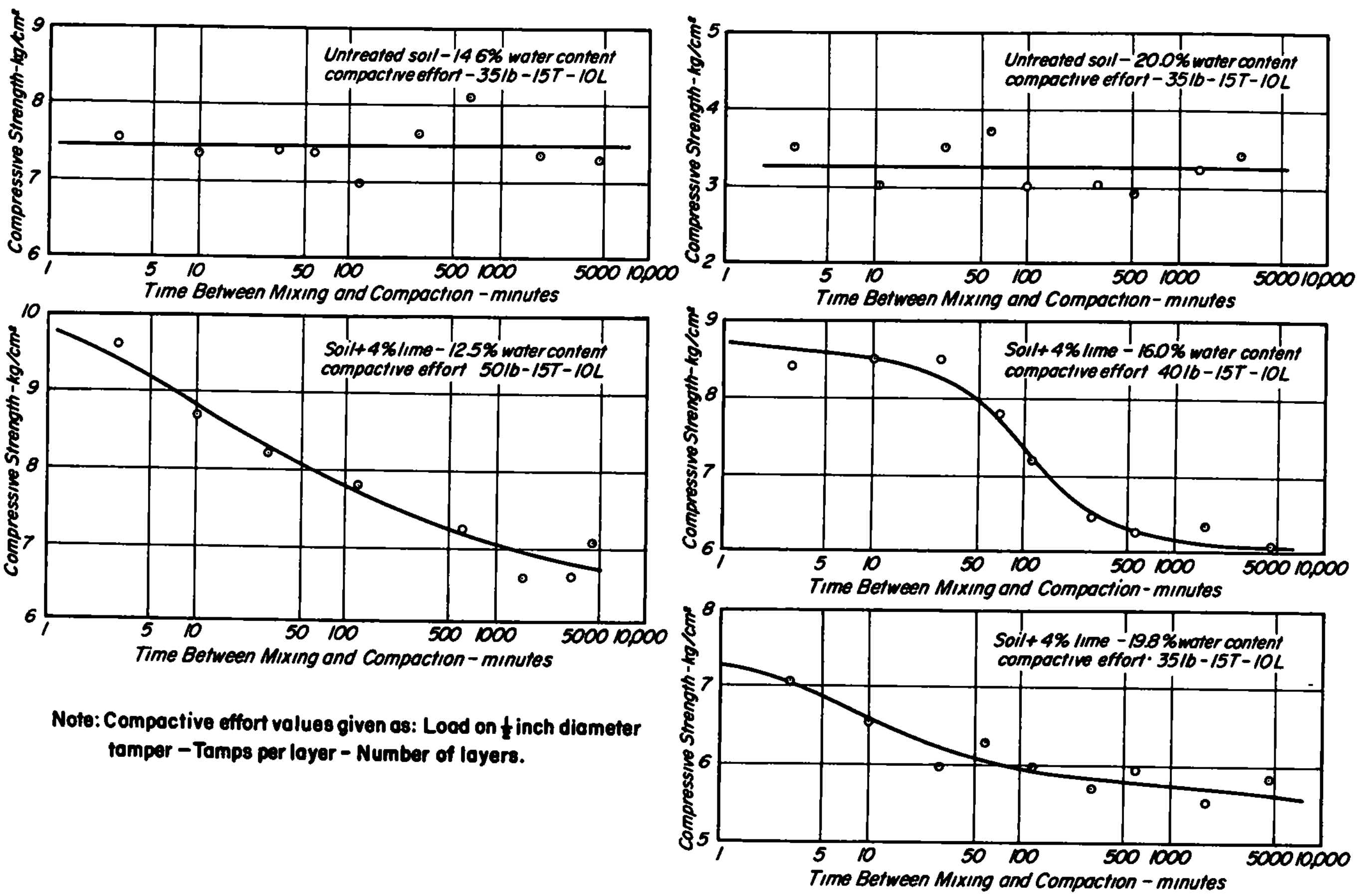
The data in Figures 2 and 3 suggest that the magnitude of the detrimental effects of long time periods between mixing and compaction varies with the molding water content. It may be noted, also, that for all treated samples investigated (prepared by kneading compaction) the density decrease resulting from a 24-hr time interval between mixing and compaction is greater than the 2 pcf density decrease observed for samples prepared using modified AASHO compactive effort and impact compaction (Fig. 1). It appears, therefore, that both density and method of compaction may influence the effect of variation in time interval between mixing and compaction on properties.

Additional data on the effect of the time interval between mixing and compaction on the properties of lime-treated samples prepared at constant compactive effort were obtained from a study of the 4- by 4-in. cylindrical samples used to establish the compaction curves shown in Figure 1. As previously noted, these samples had been exposed to a moist atmosphere for 215 days and were resting in about 1 in. of free water at the time they were removed for testing. Thus, the samples had been exposed to rather severe conditions. At the end of the 215 days the samples ranged from very poor to very good condition. The samples of untreated soil prepared at the lower



Note: Compactive effort values given as: Load on $\frac{1}{2}$ inch diameter tamper — Tamps per layer — Number of layers.

Figure 2. The effect of time between mixing and compaction on the dry density of lime-treated samples prepared using constant compactive effort.



Note: Compactive effort values given as: Load on $\frac{1}{8}$ inch diameter tamper – Tamps per layer – Number of layers.

Figure 3. The effect of time between mixing and compaction on the as-cured strength of lime-treated samples prepared using constant compactive effort.

moisture contents were generally soft and mushy. The lime-treated sampes were in better condition, with those that were compacted immediately after mixing appearing much better than those compacted 24 hr after mixing. The samples prepared at the lowest water contents were generally the poorest, with those compacted wet of optimum remaining firm and exhibiting less surface deterioration.

The effects of the storage period on the composition of the samples are shown in Figure 4. The open symbols represent the as-compacted densities and water contents, and the closed symbols represent the density and water content at the end of the aging period. In Figure 5 the volumetric swell and compressive strength are shown as functions of the molding water content. It is immediately evident from Figures 4 and 5(a) that lime treatment had a beneficial effect on reducing the swell and moisture content increase. Figure 5(b) illustrates the marked strength increase effected by lime treatment.

The effects of a 24-hr delay between mixing and compaction on the properties of the treated soil are clearly shown in Figure 5. The water resistance is considerably impaired by the delay, as reflected by the swell values. This higher swell is undoubtedly one of the principal factors responsible for the lower strengths of the samples compacted 24 hr after mixing as compared with the strengths of the samples compacted immediately after mixing.

Behavior of the type shown in Figures 3 to 5 may be examined in terms of soil-lime reactions and their effects. The more significant reactions would probably include:

1. Base exchange of calcium (and magnesium if dolomitic hydrated lime such as employed in this investigation is used) for the existing exchangeable cations in the soil (2, 4, 5).
2. A flocculation of the soil structure resulting from both base exchange and increase in free electrolyte content caused by the addition of lime (4, 5).
3. A reaction of calcium and magnesium with available colloidal silica and alumina forming slow-setting cementitious compounds (2, 4, 5).
4. Attack and breakdown of the clay

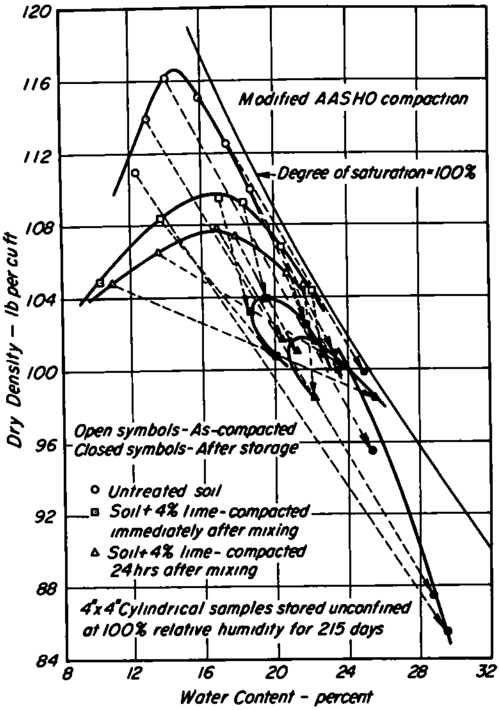


Figure 4. Effect of lime treatment on water content and density characteristics of expansive soil subjected to moist storage.

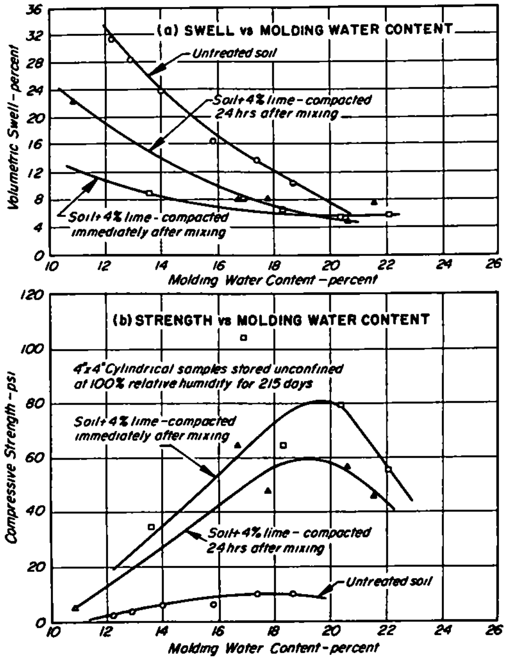


Figure 5. Effect of lime treatment on the swell and strength characteristics of expansive soil subjected to moist storage.

mineral structure with or without the formation of new crystalline compounds (5).

5. Absorption of CO_2 from the air which reacts with $Ca(OH)_2$ to form $CaCO_3$ which may act as a cementing material (2, 5).

McDowell (2) points out that for reactions 3 and 5 to be fully effective compaction should take place prior to the reaction. It would seem logical that the same should hold true for reaction 4. Because these reactions may be presumed to begin as soon as water comes into contact with the soil and lime, the sooner the mixture is compacted the better. These reactions are probably irreversible and each increment of the reaction uses up a part of the available lime and water. Reaction prior to compaction will result in cementation of particles in a loose structure. Subsequent compaction will break the bonds formed. At the end of compaction there will be less available lime and water to enter into further reactions. Thus, it would be expected that if a significant part of the available lime is consumed by reactions of this type during the early stages after mixing, then the strength of the treated soil will be adversely affected by a delay between mixing and compaction. It should also be noted that cementation into loose aggregates prior to compaction will cause the soil to offer a greater resistance to compaction. Thus, a given compactive effort would be expected to produce lower densities as the time available for these reactions increases.

Due to the relatively high calcium and magnesium content of the exchange complex in the untreated soil (Table 1), base exchange reactions might not play a significant part. Studies by Goldberg (3) show, however, that exchange of calcium and magnesium for sodium and potassium does occur in this soil, but the rate is slow. It should also be noted from the exchangeable cation data in Table 1 that some 5.5 me per 100 gm of exchangeable cations were unaccounted for by the analyses. Calcium exchange for these ions could be important.

Flocculation can have a pronounced effect on the physical properties of a soil. It is known that flocculating a given soil will lead to a higher optimum water content and lower dry density for a given compactive effort. In lime-stabilized soils the effects of increased flocculation might develop slowly due to the low water solubility of lime and other factors, such as the time required for calcium and magnesium penetration between the unit cells of expansive clay minerals. However, the increased flocculation might not have any appreciable effect on strength, because the loss in strength caused by the density decrease could be offset by the strength increase due to the greater degree of flocculation.

It is difficult to attribute the observed strength decreases to one or more specific reactions on the basis of the data thus far examined, because both density and time are variable factors. To evaluate the effectiveness of lime treatment on the expansive clay, however, samples of treated soil prepared 40 min and 24 hr after mixing were studied at essentially the same density. As previously noted, it was originally intended to compare samples at the same relative compaction. A value of 90 percent was chosen, which would mean a density of 98.7 pcf for treated samples compacted immediately and 97.2 pcf for treated samples compacted 24 hr after mixing, based on the curves for modified AASHO compactive effort. Because this difference in densities is not large for this method of compaction, it was decided that preparation of all treated samples to the same density might provide the most information. Consequently, all treated samples for these studies were prepared to an average density of between 97.5 and 98.0 pcf, thus permitting an analysis of

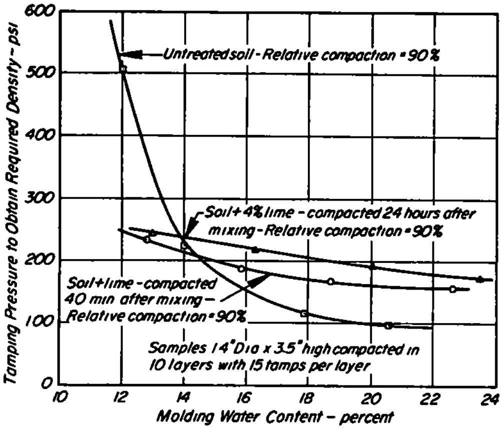


Figure 6. Compactive effort required to obtain 90 percent compaction for lime-treated and untreated expansive clay.

the effect of time between mixing and compaction on strength for samples compared at constant density. Further speculations as to the reactions causing the behavior shown in Figures 2-5 are deferred until after presentation of these additional data.

Effect of Lime Treatment on Strength and Volume Change Characteristics of Expansive Clay

Compactive Effort.—In Figure 6 are shown the kneading compactive efforts required to obtain the desired densities of 97.7 pcf for the treated soil and 105 pcf for the untreated soil. These densities correspond to relative compactions of 89.1 percent for the soil plus 4 percent lime compacted 40 min after mixing, 90.6 percent for the soil plus 4 percent lime compacted 24 hr after mixing, and 90 percent for the untreated soil. As would be anticipated from Figure 2, a greater effort is required to achieve the desired density for samples compacted 24 hr after mixing than for samples compacted soon after mixing. Figure 6 shows that the compacted effort required to achieve the desired density is much more sensitive to water content for the untreated soil than for the lime-treated soil. This might be anticipated from the shape of the compaction curves in Figure 1, which show steep branches and a sharp peak for the untreated soil and relatively broad, flat curves for the treated soil.

It is also important to note that inasmuch as compactive effort decreased with increasing water content, all samples were prepared dry of the optimum water content corresponding to the compactive effort used for their preparation as shown in Figure 7.

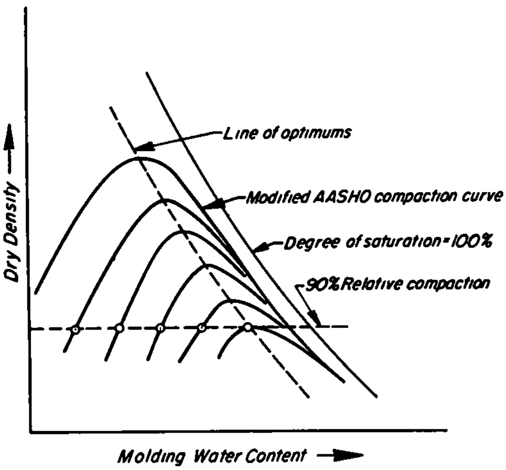


Figure 7. Schematic diagram illustrating that all water contents studied were dry of optimum.

Strength, Volume Change and Water Content Relationships.—The relationships between strength, volume change and water content for samples prepared to a constant density are shown in Figures 8, 9 and 10. In part (a) of each figure is shown the relationship between as-cured compressive strength and molding water content; part (b) indicates the volumetric swell of the soaked samples as a function of molding water content; part (c) presents compressive strength vs molding water content for samples soaked under a surcharge pressure of 0.1 kg per sq cm for seven days after curing; and part (d) shows soaked strength vs final water content. Values of the as-cured compressive strength are indicated for curing periods of 0, 7, 14, and 35 days. Properties of soaked samples are indicated for curing periods of 0, 7 and 28 days followed by a 7-day soaking period. Figure 8 refers to the results for control specimens of untreated soil compacted to 105 pcf; Figure 9 refers to samples containing 4 percent lime compacted 40 min after mixing to a dry density of 97.7 pcf; and Figure 10 refers to samples containing 4 percent lime compacted 24 hr after mixing to a dry density of 97.7 pcf. (Variations of ± 0.5 pcf were permitted.) Examination of these results show:

1. For both treated and untreated samples prepared at constant density the as-cured compressive strength decreases markedly with increasing molding-water content (Figs. 8, 9, 10(a)).
2. If increase in as-cured compressive strength with time is used as a measure, then thixotropic effects are not large in the untreated soil. Figure 8(a) shows a slight strength increase in the first 14 days after compaction, but essentially the same strength after 35 days as immediately after compaction.
3. The as-cured compressive strength of the lime-treated specimens compacted

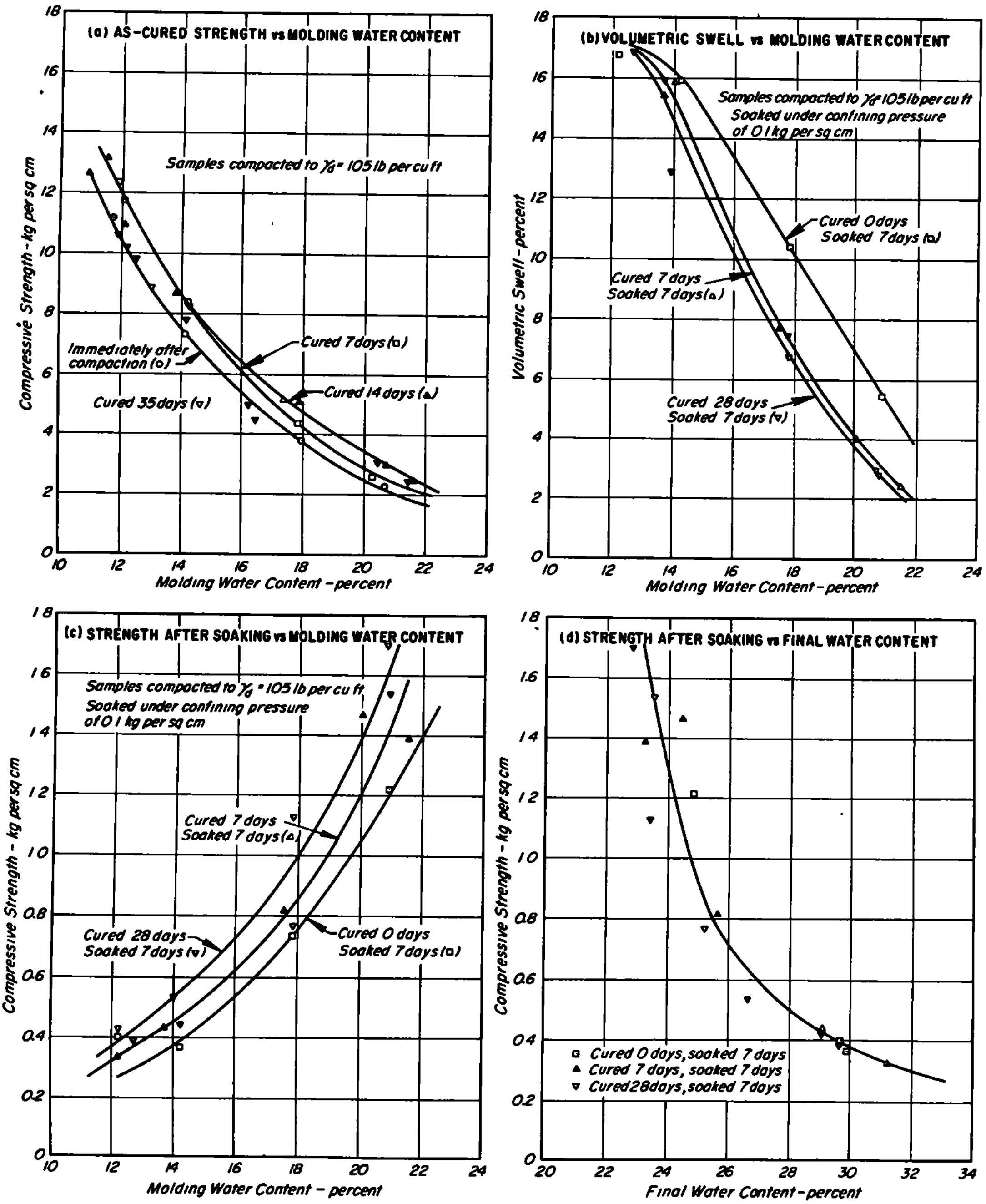


Figure 8. Strength, swell and water content relationships for samples of untreated expansive clay.

24 hr after mixing, Figure 10(a), increases significantly with curing time. Scatter in the results, Figure 9(a), does not lead to such a conclusion for samples compacted 40 min after mixing. Reasons for this large scatter are not known; the samples appeared satisfactory in all respects. Perhaps it reflects a non-uniformity in water and lime distribution in the soil as-mixed, which gradully dissipated in the case of samples tempered for 24 hr prior to compaction.

4. The swell under 0.1 kg per sq cm surcharge decreases markedly with both increasing molding water content and increased curing time (Figs. 8, 9, 10(b)) for both treated and untreated samples. The greatest part of the swell reduction occurred during the first 7 days of curing. It should be kept in mind, however, that the uppermost curves in Figures 8, 9, 10(b) refer to samples that started soaking immediately after compaction, and thus the samples were not allowed to cure at constant water content.

5. The strength after soaking increased with molding water content in all cases (Figs. 8, 9, 10(c)). At any water content, both the treated and untreated samples showed an increase in strength with increased curing time.

6. In general, the soaked strength correlates fairly well with the water content after soaking (Figs. 8, 9, 10(d)).

The trends exhibited by these results are reasonable in terms of the behavior of compacted soils in general. The increase in soaked strength with molding water con-

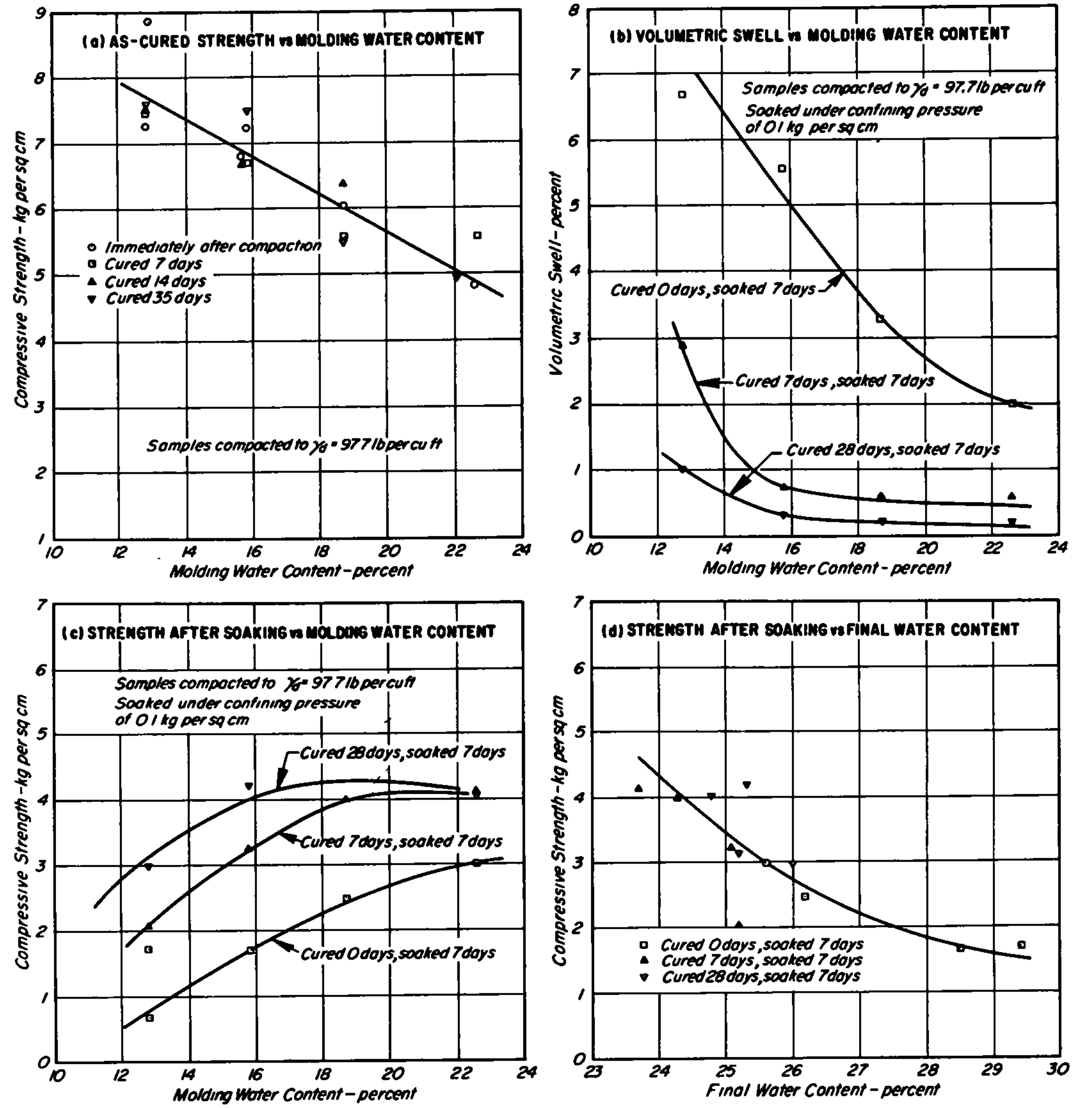


Figure 9. Strength, water content and swell relationships for samples of lime-treated expansive clay compacted 40 min after mixing.

tent is logical, because higher molding water contents lead to less swell, and, consequently, the soaked specimens retained higher densities if compacted at higher water contents. The decrease in swell and increase in as-cured and soaked strength with time for the lime-treated samples would be expected because of the relatively slow rate of the lime-soil reaction.

The increase in soaked strengths with time for the untreated samples follows from the decrease in swell with time. The explanation for the decrease in swell with time for these samples is not readily apparent, particularly because Figure 8(a) shows that the soil does not have appreciable thixotropic characteristics. Evidently some internal structural alterations—for example, in particle orientations, water structure, ion distribution, or actual chemical change—are occurring with time, which, although not appreciably altering the as-cured strength, reduce the water sensitivity of the soil. Measurements have shown the pH to increase from 5.9 to 6.15 in a period of 24 hr af-

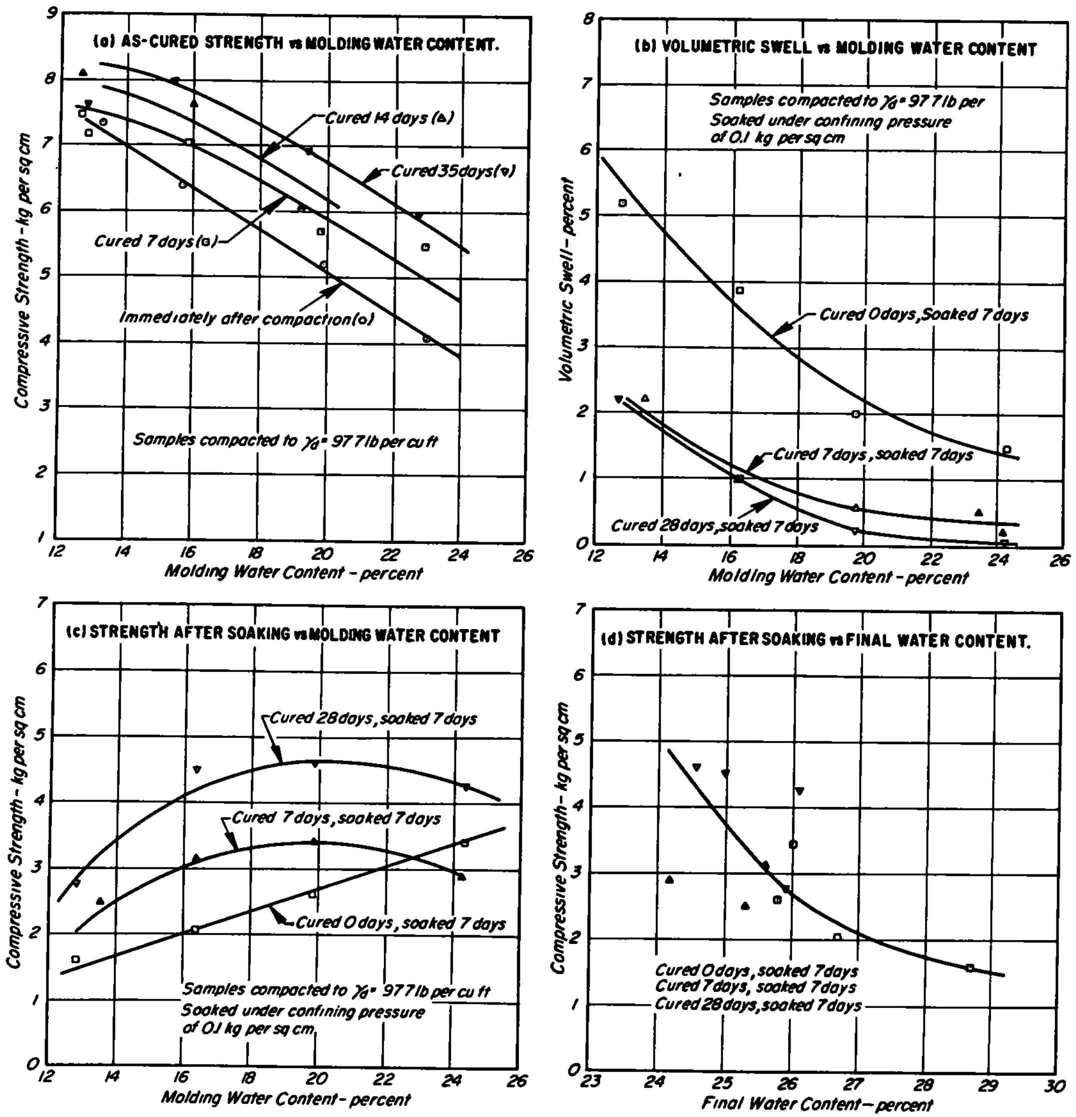


Figure 10. Strength, water content and swell relationships for samples of lime-treated expansive clay compacted 24 hr after mixing.

ter addition of water, suggesting that an actual chemical alteration may be important.

Effect of Time Between Mixing and Compaction on Properties of Lime-Treated Soil for Samples Compared at Constant Density.—Figures 9 and 10 show that essentially the same behavior is exhibited by samples compacted 40 min and 24 hr after mixing. Figure 11 compares directly the as-cured strength, volumetric swell, and soaked-strength values for samples prepared at these two time intervals after mixing. Curves are shown for samples tested 7 and 35 days after compaction. Figure 11 shows that the properties of the lime-treated soil are roughly comparable for both time intervals between mixing and compaction with perhaps some superiority exhibited by the specimens aged 24 hr between mixing and compaction. The as-cured strengths are difficult to compare due to the large scatter of the data for the 40-min samples. The swelling behavior shows a slight superiority for the 40-min samples after 7 days but after 35 days the 24-hr samples appear the better. The strengths after soaking are about the same for each case. It seems reasonable to conclude, therefore, that a time interval of 24 hr between mixing and compaction has no detrimental effect on the swell or strength of lime-treated samples of the expansive clay studied in comparison with samples prepared soon after mixing, provided samples are compacted to the same density.

These results may be contrasted with those in Figures 2 to 5 which show the behavior for samples prepared at constant compactive effort. It would appear reasonable to conclude, from a consideration of the behavior at constant density, that the decrease in strength exhibited by treated samples with increasing time between mixing and compaction, Figure 3, is for the most part due to decreased density and not cementation prior to compaction.

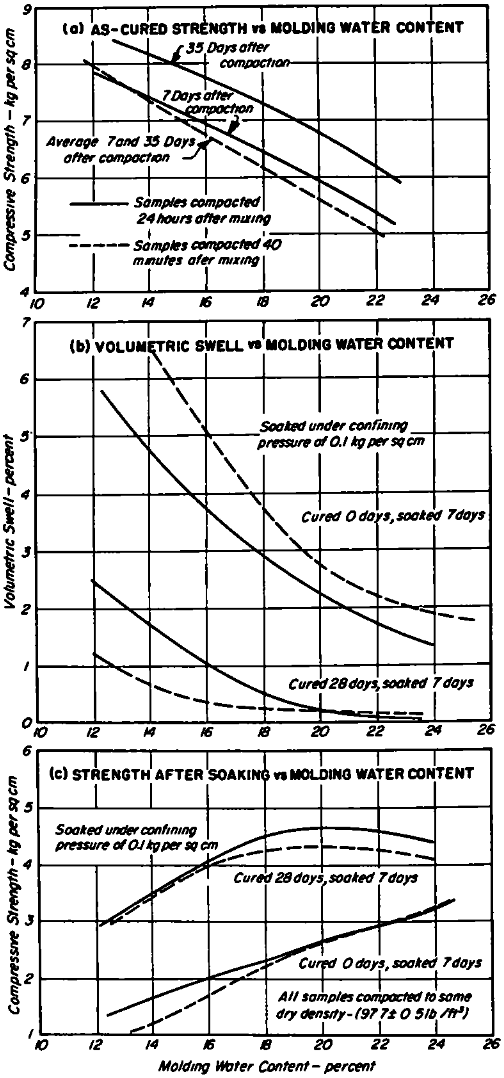


Figure 11. Comparison of strength and volume change characteristics of lime-treated samples compared at equal densities but compacted at different times after mixing.

If irreversible chemical changes were a primary factor, it would be expected that the greater the delay between mixing and compaction the poorer would be the results obtained for samples prepared at constant density. The lime involved in irreversible reactions during the tempering period would no longer be available for cementation after compaction, and the cementation during the tempering period would be largely destroyed by the compaction process. This is not meant to imply that chemical changes did not occur, but rather that any changes that did occur were insignificant from a cementation standpoint. Chemical analyses of the lime-treated soil by Goldberg (3) show a progressive decrease in undissolved calcium, increase in carbonates, and decrease in available calcium plus magnesium with time after mixing the soil with lime and water. Differential thermal analyses, also run by Goldberg, indicate that the $Ca(OH)_2$ present in the lime is reacted rapidly after addition to the soil, and disappears almost entirely by the end of a week. The $Mg(OH)_2$, however, reacts much more slowly. The relatively high (8 percent) organic matter content of the soil studied could have been responsible for the consumption of some of the lime as well. Determinations of pH indicate that the addition of lime causes an increase from 6 to 9.5 at high water contents and to 11.3 at low water contents. The pH then decreases slightly with time; the greatest decrease occurring in samples at the lowest water content and amounting to up to 1.5 pH units after 35 days.

It is suggested that the main factor responsible for the decrease in density at constant compactive effort caused by increased time between mixing and compaction is flocculation of the soil structure, which increases with time of exposure of the soil to water and lime. The effect of greater flocculation is to increase the resistance to compaction. The increase in intensity of flocculation with time after lime addition may be attributed to the greater time available for lime to dissolve and other chemical reactions to occur. Penetration of calcium and magnesium between the layers of the expansive clay minerals, with consequent reduction in swelling, and on actual attack and breakdown of the clay mineral structure could be significant factors. The uncompacted soil structure is so affected that greater compactive efforts are required to achieve a given density for greater time intervals between mixing and compaction. Flocculation would also account for the apparent slightly greater strength of the samples compacted 24 hr after mixing, as shown in Figure 11, because for two specimens compacted to the same density, the one with the more highly flocculated structure should be the stronger.

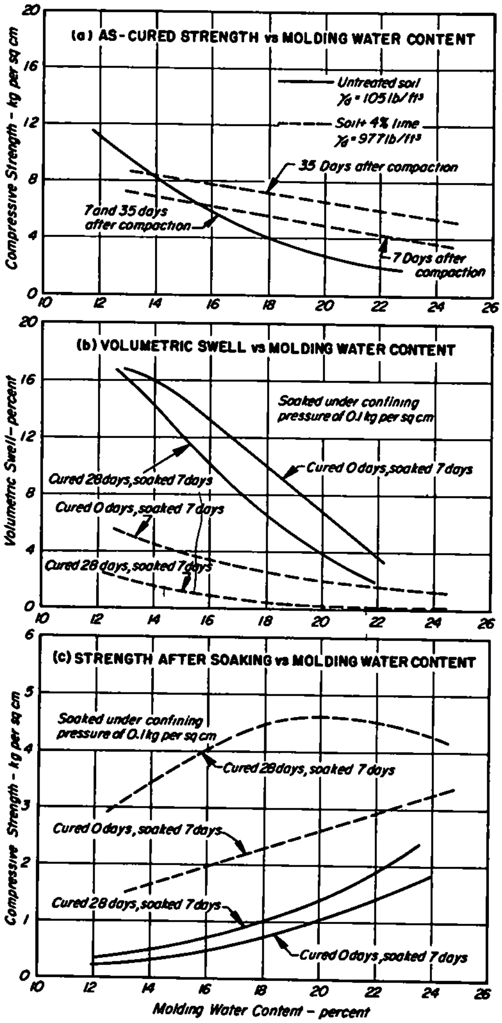


Figure 12. Strength and swell characteristic of lime-treated and untreated expansive clay prepared to 90 percent relative compaction.

Effectiveness of Lime as a Stabilizer for Expansive Clay Soil.—The over-all effectiveness of a 4 percent lime treatment of the expansive clay from the standpoint of strength improvement and swell reduction may be seen from Figure 12. Test results for untreated samples are compared with those for treated samples compacted 24 hr after mixing. Curves of as-cured strength, volumetric swell, and strength after soaking vs molding water content are shown for samples tested 7 and 35 days after compaction. It is important to keep in mind that densities of treated and untreated samples are considerably different, being 105 pcf and 97.7 pcf for the untreated and treated specimens, respectively, but that these densities represent the same degree of compaction.

It may be seen from Figure 12 that, in general, a 4 percent lime treatment of the expansive clay is quite effective. Figure 12(a) shows that in the as-cured state the strengths of the treated and untreated specimens are roughly comparable at the lower molding water contents. Lime treatment becomes more favorable with increasing water content, the strengths of the lime-treated samples tested after 35 days curing time being three times greater than those of untreated samples at 22 percent molding water content. Although this strength improvement is perhaps not as marked as might be achieved with a greater percentage of lime or with some other type of stabilizer, the as-cured strengths are fairly substantial and, furthermore, the as-cured condition is not the critical condition from the standpoint of field performance.

In the field, the critical condition will generally arise when the treated soil is exposed to water. Figures 12(b) and 12(c) indicate the performance of the lime-treated soil to be markedly superior to that of the untreated soil both from the standpoint of swell and of strength after soaking. It may be seen from Figure 12(b) that at the lowest water contents investigated lime treatment reduces volumetric swell from about 17 percent to about 5 percent for samples soaked immediately after compaction and to about 2 percent for samples permitted to cure for 28 days prior to soaking. It may be noted that the surcharge pressure of 0.1 kg per sq cm used in the tests is comparable to that afforded by a typical pavement. Because of the reduction in swell the strength after soaking remains at a high level. It may be seen from Figure 12(c) that the strength of treated samples after soaking is as much as seven times greater than the strength of untreated samples.

A further point of considerable importance from the standpoint of pavement design and performance, not shown by Figure 12, is the strain required to develop maximum compressive strength. In general, failure of soaked treated samples occurred at axial strains of less than 4 percent, whereas, 15 to 20 percent strain was required to develop the maximum deviator stress for untreated soaked samples. Because for pavement design, strength at low strain is generally the important factor, the beneficial effect of treatment is obvious.

In assessing the probable mechanism of property improvement by stabilization of this soil with lime, it is unlikely that actual cementation of particles is the major factor. Reasons for this conclusion are (a) lime treatment does not cause a large increase in the as-cured strength, (b) a delay between mixing and compaction is not detrimental provided compactive effort is increased to maintain constant density, and (c) the strength loss due to remolding treated samples cured for considerable periods at constant water content is not large. In connection with this latter point, lime-treated samples, compacted to the same density over a water content range from 12.8 to 22.6 percent, were cured at constant water content for 70 days. At the end of this period the samples were tested in triaxial compression in the usual manner. At the end of the test the samples were thoroughly remolded at constant water content, recompacted, and the strength again measured. Results of these tests are summarized in Table 3. It may be seen that the strength loss due to breaking up the structure acquired after a 70-day curing period was very small. The rigidity of the structure was lessened, however, as is indicated by the values for strain at failure.

It is suggested, therefore, that the chemical effects associated with the lime treatment of the expansive clay acted primarily to create a more flocculent and water-resistant structure, rather than to participate in significant amounts of interparticle cementation. Both cation exchange (calcium and magnesium for sodium, potassium and the 5.5 me per 100 gm of unaccounted-for exchangeable cations, Table 1), and increased electrolyte content, as well as alteration of organic matter and attack and breakdown of the expansive clay mineral structure could lead to increased flocculation and lower water sensitivity. Expansion of the montmorillonitic clay minerals would also be limited by the high calcium and magnesium content of the treated soil.

SUMMARY AND CONCLUSIONS

It has long been felt that one of the advantages of lime as a stabilizer is that the properties of the stabilized soil do not depend critically on the elapsed time between addition of lime and water to the soil and compaction of the mixture. The American Road Builders Association (1) points out that the mixed material may cure or age for 24 to 48 hr prior to compaction as long as optimum moisture conditions are maintained. Mc-

Dowell (6) states that soil-lime mixtures may be compacted any time within two days after mixing, with delays of up to 4 days permissible if heavy plastic clays are being stabilized. McDowell (2), the ARBA (1), and Dumbleton (6) point out the helpful action of lime in breaking down clay clods or "mellowing" the mixture on standing. This is an important consideration in terms of the mix uniformity of the final product, because the mellowing action greatly facilitates final mixing. A delay in compaction may reduce the effectiveness of possible cementation and carbonation reactions, however.

TABLE 3

EFFECT OF REMOLDING ON STRENGTH OF LIME-TREATED EXPANSIVE CLAY

At End of 70-Day Curing Period				After Remolding and Compaction			
Water Content (%)	Dry Density (pcf)	Compressive Strength (kg/cm²)	Strain at Failure (%)	Water Content (%)	Dry Density (pcf)	Compressive Strength (kg/cm²)	Strain at Failure (%)
12.9	97.5	6.9	1.5	12.9	97.0	7.3	3.7
16.1	97.9	7.5	1.3	16.1	96.0	6.8	5.0
19.4	97.8	6.0	3.0	19.4	97.2	5.5	6.5
21.9	98.1	5.6	4.0	21.9	97.7	5.3	12.5

The results of the present investigation have shown that for the expansive clay soil studied, a delay between mixing and compaction is definitely detrimental in terms of density, swell and strength for samples prepared using constant compactive effort. Densities were found to decrease by as much as 8 pcf, and as-cured strengths by 30 percent for a 24-hr delay between mixing and compaction. Thus, on the basis of performance of samples prepared at constant compactive effort, it might be concluded that a delay in compaction would be disadvantageous.

On the other hand, for samples prepared to a specified constant density, it was found that the properties of the treated soil after curing, and in the soaked and unsoaked condition, were about the same for samples prepared 40 min and 24 hr after mixing. This indicates, for this soil at least, that delay between mixing and compaction is not detrimental provided extra compactive effort is exerted to maintain the desired density as time between mixing and compaction increases; however, extra compactive effort means increased cost of compaction.

Consideration of probable soil-lime reactions would suggest that a flocculation of the soil structure, which increases with time after addition of lime, is the principal effect responsible for the observed behavior. Quite possibly an investigation of the effect of delay between mixing and compaction using soils in which the soil-lime reactions were predominantly cementation and carbonation would show that large delays produce irreversible detrimental effects.

In practice, the advantages of improved mix uniformity and handling characteristics that may result from allowing a delay between initial mixing and reworking prior to compaction may well offset any losses in density or strength that may result, or may justify the expenditure of more compactive effort to obtain high density.

The investigation has shown a 4 percent dolomitic hydrated lime treatment to be a very effective stabilizer of an expansive clay soil containing appreciable (8 percent) organic matter. Untreated and treated samples have been compared over a range of water contents and curing periods and compacted to dry densities corresponding to 90 percent of their respective maximum densities as determined by modified AASHO effort. This meant a comparison wherein the untreated samples were some 7 pcf denser than the treated samples.

Although the improvement in strength as a result of lime treatment was not marked for samples tested in the as-cured condition, the improvement in properties of specimens exposed to water was very marked. Swell was reduced to low levels (less than

3 percent) immediately after compaction for treated samples prepared to high water contents, and for samples cured for 28 days the swell was reduced to less than 3 percent over the whole water content range. The swell of the untreated specimens ranged from 17 percent at low water contents to 4 percent at high water contents. Soaked strengths of treated samples showed similar improvements over the untreated soil, being increased as much as sevenfold by lime treatment. The improvement in soaked strength follows directly from the decrease in swell afforded by lime treatment.

Because by lime treatment the increase in as-cured strength is not large, a delay between mixing and compaction is not detrimental provided sufficient compactive effort is used to maintain specified density, and the strength loss due to remolding and recompacting treated samples 70 days after original compaction is not large, it is concluded that the observed chemical changes on aging serve mainly to increase the flocculation and reduce the water affinity of the soil. Cementation effects are felt to be of minor importance in the over-all reaction of this soil with lime.

Finally, additional tests have shown a consistent trend for a slight decrease in strength and increase in swell to occur with both treated and untreated samples when the curing time is increased from 35 to 60 days. The cause of this behavior has not yet been established; however, the effects of the organic matter or bacteriological factors are suggested as possibilities.

ACKNOWLEDGMENTS

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Further Evaluation of Promising Chemical Additives for Accelerating Hardening Of Soil-Lime-Fly Ash Mixtures

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The results of an investigation on the effect of several amounts of 12 chemicals on the strength of a mixture of Ottawa sand-lime-fly ash are presented. The effects of four selected chemical additives on the strength of soil-lime-fly ash mixes—including four soils, two limes and three fly ashes—are also presented. An evaluation of competitive mixes of soil-lime-fly ash was made, including freeze-and-thaw studies.

●SINCE 1954 the Engineering Experiment Station Soil Research Laboratory at Iowa State University has been engaged in investigations of lime and fly ash as admixtures for soil stabilization. (Fly ash is an artificial pozzolan, by-product of the power plants burning powdered coal.) Most fly ashes give relatively low early strengths, which has raised the problem of finding an economic means of accelerating the lime-fly ash pozzolanic reaction.

Handy (9, 10), in an investigation of chemical additives in soil-cement, found that immersion of cement-treated specimens in a solution of sodium hydroxide increased their strength about 15 percent. He also discovered that the strength of soil-cement could be increased by addition of sodium hydroxide in the mixing water. In another Iowa State investigation, Goecker et al. (8) observed that calcium chloride was beneficial to the strength of compacted soil-lime-fly ash mixtures. These findings led to an evaluation of 47 chemicals as additives to accelerate the lime-fly ash reaction and the discovery that small amounts of several of the chemicals greatly increased the early and/or long-term strength of Ottawa sand-lime-fly ash mixtures (6). The results of a further investigation of these promising chemical additives is the subject of this paper.

A preliminary survey was made using 12 chemicals in varying amounts in mixtures with Ottawa sand, a calcitic hydrated lime and a selected fly ash. On the basis of the results, four chemicals (sodium hydroxide, sodium carbonate, sodium metasilicate and sodium chloride) were chosen for studies with each of four natural soils (a dune sand, a friable loess, an alluvial clay and a heavily weathered glacial till) in mixtures with a calcitic (high-calcium) hydrated lime and a dolomitic monohydrate (Type N) lime, and three different fly ashes. An evaluation of the effects of chemical additives at low curing temperatures was also made. Finally a few mixtures were selected and submitted to a freezing and thawing test.

MATERIALS USED

Soils

Ottawa sand is a natural silica sand assumed to be unreactive with lime and water at the curing temperatures used. Its gradation met the requirements for graded standard sand (ASTM Designation: C 109-58):

Sieve Size	Percent Passing
No. 16 (1,190-micron)	100
No. 30 (590-micron)	98 ± 2
No. 50 (297-micron)	28 ± 5
No. 100 (149-micron)	2 ± 2

The four natural soils were selected as being representative of important Iowa soil types. A field description of each sample is given in Table 1, and physical and chemical properties are given in Table 2.

Fly Ashes

Three fly ashes were selected to represent variations in the properties of this by-product material. Analysis of the samples are given in Table 3 and additional information on each follows:

Fly ash No. 1 was collected by multiple cyclone and electrical precipitators. The coal was from districts 3 and 8 in Ohio and from northern West Virginia, and was processed through pulverizing mills so that 70 percent passed a No. 200 mesh. The sample was sent from the St. Clair (Mich.) Power Plant of the Detroit Edison Company.

TABLE 1

DESCRIPTION OF NATURAL SOILS

Soil	Dune Sand (S-6-2)[a]	Friable Loess (20-2)	Alluvial Clay (627-1)	Kansan Gumbotil (528-8)
Location	Benton County, Iowa	Harrison County, Iowa	Harrison County, Iowa	Keokuk County, Iowa
Geological description	Wisconsin-age eolian sand, fine-grained, oxidized, leached	Wisconsin-age loess, friable, oxidized, calcareous	Recent fill, alluvial plastic, slightly calcareous	Kansan-age gumbotil, highly weathered, plastic, noncalcareous
Soil series	Carrington	Hamburg	None	Mahaska[b]
Horizon	C	C	Undefined	Fossil B
Sampling depth, ft	6-11	49-50	0-4	7.5-8.5

[a] Numbers in parentheses are those assigned by the Soil Research Laboratory of the Iowa Engineering Experiment Station.
[b] Underlies C horizon loess of Mahaska series.

Fly ash No. 2 was collected by mechanical equipment. The coal was from northern Illinois and was burned in a B and W boiler. This sample was sent from the Sixth Street Power Station in Cedar Rapids, Iowa, by the Iowa Electric Light and Power Company.

Fly ash No. 3 was collected by electrical precipitators from a dry bottom type of boiler using unwashed coal from western Kentucky. The sample was sent from the Paddy's Run Power Station at Louisville, Ky., by the Louisville Gas and Electric Company.

Limes

Calcium hydroxide (calcitic hydrated) lime, reagent grade, from Fisher Scientific Company was used in the tests with Ottawa sand. Samples of commercial calcitic (high-calcium) hydrated lime, brand-name Kemikal, and commercial dolomitic monohydrate (Type N) lime, brand-name Kemidol, obtained from U.S. Gypsum Company were used in tests with the natural soils.

Cement

The portland cement used was commercial Type I from the Penn-Dixie Cement Corporation, of Des Moines, Iowa.

Chemicals

The following chemicals used were reagent grade, except magnesium oxide which was USP grade:

Chemical	Formula
Sodium carbonate	Na_2CO_3
Sodium hydroxide	$NaOH$
Sodium metasilicate	$Na_2SiO_3 . 9H_2O$
Sodium chloride	$NaCl$
Aluminum chloride	$AlCl_3 . 6H_2O$
Calcium chloride	$CaCl_2$
Lithium carbonate	Li_2CO_3
Magnesium oxide	MgO
Manganese chloride	$MnCl_2 . 4H_2O$
Phosphoric acid	85% H_3PO_4
Potassium penmanganate	$KMnO_4$
Sodium phosphate	$Na_3PO_4 . 12H_2O$

TABLE 2

PROPERTIES OF SOILS

Soil	Dune Sand	Friable Loess	Alluvial Clay	Kansan Gumbotil
Textural composition[a], %				
Gravel (> 2 mm)	0.0	0.0	0.0	0.0
Sand (2-0.074 mm)	95.5	0.7	2.4	19.4
Silt (0.074-0.005 mm)	1.5	82.3	25.6	14.6
Clay (< 0.005 mm)	3.0	17.0	72.0	66.0
Colloids (< 0.002 mm)	2.6	14.0	61.0	63.0
Atterberg limits[b]				
Liquid limit, %	-	32	72	76
Plastic limit, %	-	25	26	26
Plasticity index	Non-plastic	7	46	50
Classification				
Textural[c]	Sand	Silty loam	Clay	Clay
Engineering (AASHO)[d]	A-3(0)	A-4(8)	A-7-6(20)	A-7-6(20)
Chemical				
Cat. exch cap[e], me/100g	1.0	14.5	44.4	39.2
pH[f]	6.6	8.4	7.7	7.4
Carbonates[g], %	0.4	10.4	3.6	2.0
Organic matter[h], %	0.1	0.1	1.6	0.1
Predominant clay mineral[i]	Montmorillonite (trace)	Montmorillonite	Montmorillonite	Montmorillonite

[a] ASTM Method D422-54T (3).
[b] ASTM Method D423-54T and D424-54T (3).
[c] Triangular chart developed by U.S. Bureau of Public Roads (11, p. 47).
[d] AASHO Method M145-49 (2).
[e] Ammonium acetate (pH = 7) method on soil fraction 0.42 mm (No. 40 sieve).
[f] Glass electrode method using suspension of 15 g soil in 30 cc distilled water.
[g] Versenate method for total calcium.
[h] Potassium bichromate method.
[i] X-ray diffraction analysis.

METHODS

Mixture Proportions

The proportions, by weight, of the Ottawa sand or soil, lime and fly ash components of mixtures were 75, 5 and 20 percent in the preliminary survey and 76.5, 6 and 17.5 percent in the tests with natural soils. The chemical additive, which was computed on a dry basis excluding the crystal water, is expressed as a percentage of the dry weight of the total Ottawa sand or soil-lime-fly ash mixture. Chemicals were added either in powder form or as a component of the mix water.

Mixing and Molding

Mixing of batches for preparing test specimens was done in a Hobart kitchen mixer, model C-100, at low speed in the following sequence of operation: The dry ingredients were mixed for 30 sec, the mix water was added and machine mixed for 1 min, the mixture was hand mixed for about 30 sec to clean the sides and bottom of the mixing bowl, and the mixture was machine mixed for 1 min. The final moisture content, based on maximum strengths after 7, 28 and 90 days curing, was on the dry side of the optimum moisture content for Standard Proctor (ASTM-AASHO) density for both sands used, and on the wet side for both clays. The optimum moisture content for maximum strength and density in the friable loess soil was the same.

TABLE 3

ANALYSIS OF FLY ASHES

Determination	Fly Ash No.		
	1	2	3
Source	Detroit Edison Co., St. Clair Station	Cedar Rapids, Iowa, Sixth St. Station	Louisville, Ky., Paddy's Run Sta.
Loss on ignition, %	3.9	7.20	2.6
Specific surface, Blaine (sq cm/g)	2,820	2,663	3,226
Specific gravity	2.58	2.39	2.60
Fineness (percent passing No. 325 sieve)	91.8	49.8	86.1
Silicon dioxide (SiO_2), %	43.54	36.68	42.5
Magnesium oxide (MgO), %	0.17	0.98	0.8
Calcium oxide (CaO), %	2.86	3.45	5.7
Aluminum oxide (Al_2O_3), %	23.25	21.29	23.4
Iron oxide (Fe_2O_3), %	24.80	24.33	20.0
Sulphur trioxide (SO_3), %	0.80	2.02	2.3

Molding of specimens was started immediately after a batch was mixed. A double plunger drop-hammer apparatus was used to mold 2-in. diameter by 2-in. high specimens. With this apparatus the equivalent of Standard Proctor compactive energy was obtained (8).

Curing

Specimens of each batch were moist cured at 70 ± 4 F and more than 90 percent relative humidity for 7 days, 28 days, and 3 or 4 months. To preserve moisture better and to reduce absorption of carbon dioxide from the air, all specimens were wrapped in wax paper and were sealed with cellophane tape before being placed in the humid room.

Strength Testing

After each curing period, specimens were unwrapped and immersed in distilled water for one day. Then they were tested for unconfined compressive strength using a load travel rate of 0.1 in. per minute. Tests were run in triplicate, and the average strengths are reported in psi.

Durability Tests

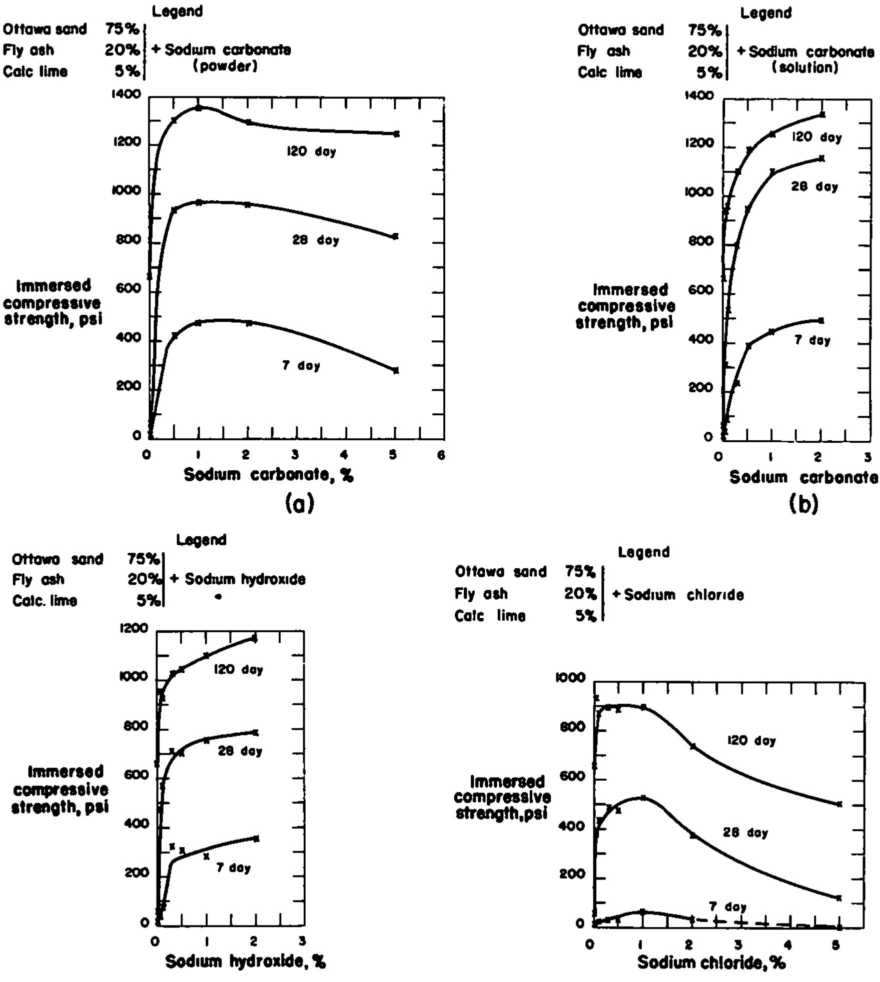
The Iowa freeze-thaw test (7) was used to evaluate the durability of selected mixtures. Four 2-in. by 2-in. specimens from each mixture were cured 28 days in the moisture room. Two specimens, designated the control specimens, were then immersed for 10 days; and the other two specimens, designated the freeze-and-thaw specimens, were exposed alternately to temperatures of 20 ± 2 F (16 hr) and 77 ± 4 F

(8 hr) for ten cycles, each cycle lasting 24 hr. A vacuum flask specimen container (4) was used to cause freezing to occur from the top down and to supply unfrozen water, kept at 35 ± 2 F by a light bulb, to the bottom of the specimen throughout the test. After these treatments, the unconfined compressive strength of the freeze-thaw specimens (p_f) and of the control specimens (p_c) were determined. These values were used to evaluate the durability of the stabilized soils. The index of resistance to the effect of freezing (R_f) was calculated from the formula:

$$R_f = \frac{100\,p_f}{p_c}\ (\%)$$

PRELIMINARY SURVEY OF CHEMICALS

The preliminary survey was made using 12 chemicals in varying amounts to determine the minimum amount of each required for substantial improvement of the lime-fly ash reaction and to serve as the basis for selecting a smaller number of chemicals for more detailed studies. Ottawa sand was used as the soil component because its gradation and monomineralic composition, silica, may make it behave as an inert ma-

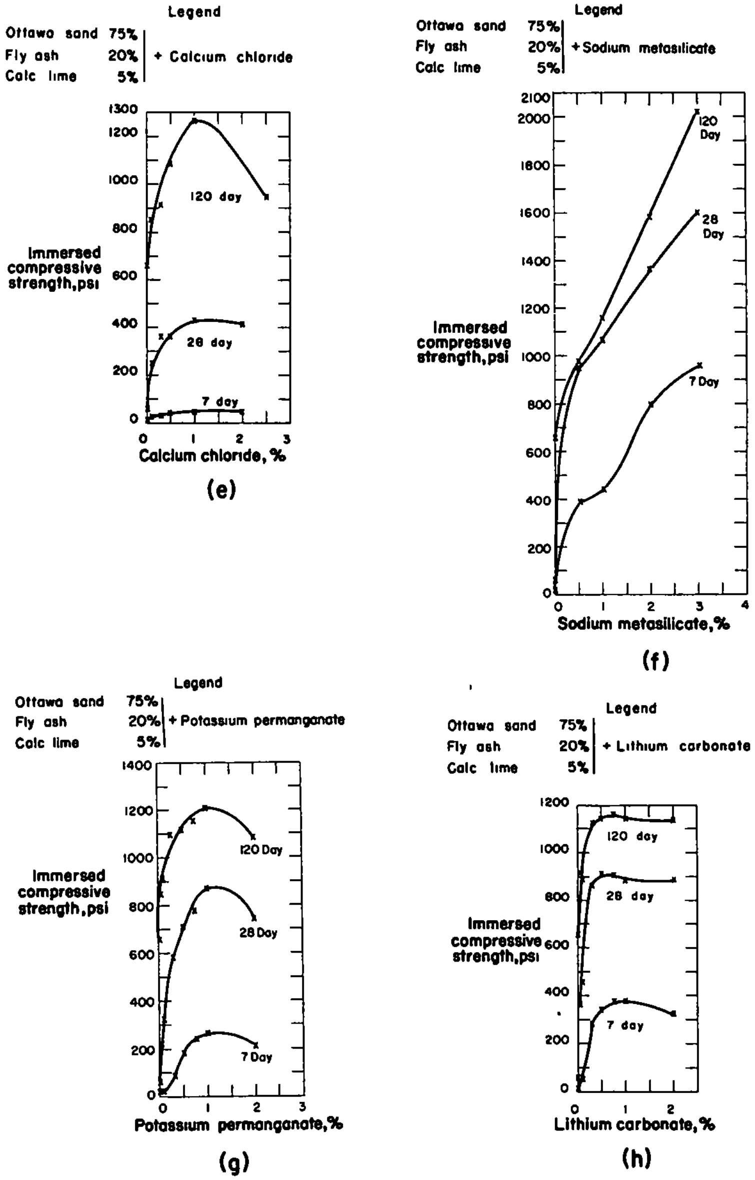


(a) (b) (c) (d)

Figure 1 (a, b, c, d). Effect of amount of chemical additive on strength of 75:5:20 Ottawa sand—calcitic hydrated lime—fly ash No. 1 mixture.

terial at the curing temperatures used, thus minimizing the effect of the soil component on the lime-fly ash reaction. A calcitic hydrated lime was chosen because, although of reagent grade, it was representative of a great amount of commercial limes produced in the U.S. A medium quality fly ash from the midwest (St. Clair Power Plant) was used as the pozzolan component. The Ottawa sand-lime-fly ash mix proportions were 75, 5, 20 percent, respectively, near optimum for these materials. Specimens were molded at optimum moisture for strength.

The test results are shown in Figure 1 (a through m). Any or certain amounts of all chemicals increased the strength of the Ottawa sand-lime-fly ash mixture. Following is an analysis of each chemical evaluated.



(e) (f)

(g) (h)

Figure 1 (e, f, g, h). Effect of amount of chemical additive on strength of 75:5:20 Ottawa sand—calcitic hydrated lime—fly ash No. 1 mixture.

Sodium Carbonate

Even the smallest amount of sodium carbonate tried, 0.05 percent, increased the strength substantially. Seven- and 28-day strengths were increased over 30 times for amounts of chemical greater than 0.5 percent. There are some differences in strength between the use of sodium carbonate in powder form or in liquid solution, but the increase in strength is great and warrants the use of the chemical in either form. The optimum amount is about 1.0 percent when used in powder form. The commercial price of this product, $35 to $65 a ton, makes it a promising additive for lime-fly ash stabilization.

Sodium Hydroxide

This chemical is also very effective. A noticeable improvement of strength started with amounts of sodium hydroxide as low as 0.03 percent. A recommended amount is about 1.0 percent. This chemical, priced at about $100 a ton, may also be an economical activator of the pozzolanic reaction.

Sodium Chloride and Calcium Chloride

The effects of these two additives are somewhat parallel. They gave little improvement to 7-day strength, but gave a substantial increase to 28-day and 4-month strengths with even small concentrations of chemical. The price difference, $20 a ton for sodium chloride and $60 for calcium chloride, and the small amounts of sodium chloride required for a maximum increase in strength, makes sodium chloride the choice when improvement of long-term strengths is the main interest. Three-tenths of a percent of sodium chloride increased the 28-day strength by about ten times, and the optimum amount was about 1.0 percent.

Sodium Metasilicate

This chemical increased the strength greatly, even in small amounts. The strength increase was more or less proportional to amount used; the optimum was above 3.0 percent. The strength of 1,000 psi was found after 7 days curing with the largest amount of sodium metasilicate tested, 3.0 percent. The commercial price of this chemical is about

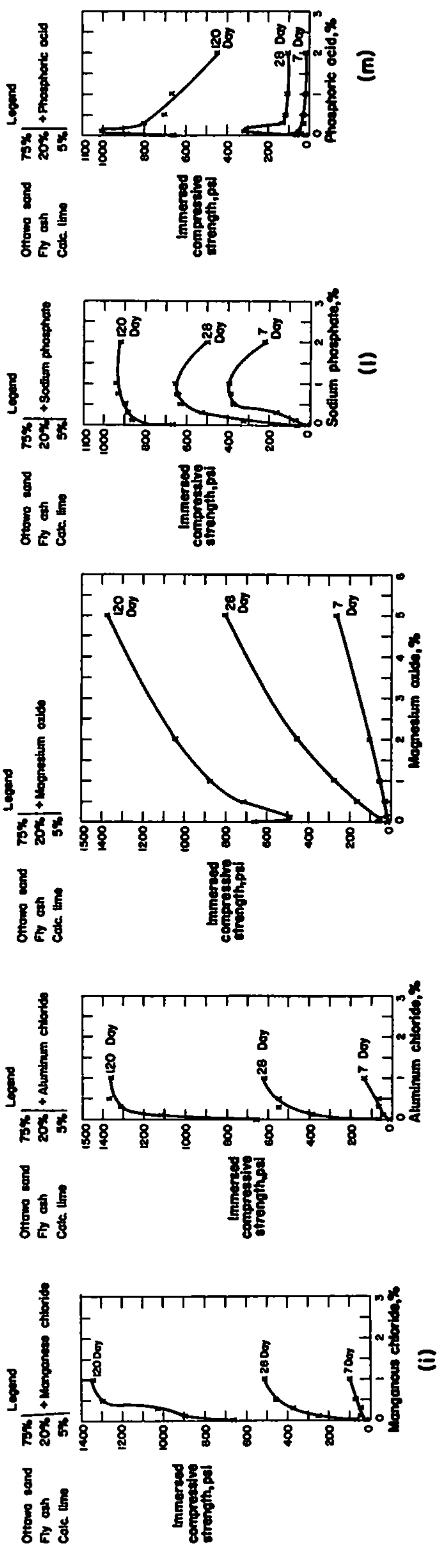


Figure 1 (i, j, k, l, m). Effect of amount of chemical additive on strength of 75:5:20 Ottawa sand—calcitic hydrated lime—fly ash No. 1 mixture.

$120 a ton on a dry basis, which makes it a promising chemical additive when used in small amounts.

Lithium Carbonate, Potassium Permanganate, Manganese Chloride, Aluminum Chloride and Sodium Phosphate

These chemicals increase strengths, but the rate of increase, amounts required, and economical considerations make them less desirable than the chemicals previously discussed.

Phosphoric Acid

Although very small amounts of phosphoric acid improved soil strength, concentrations larger than 0.03 percent caused a decrease in strength. Its use is therefore not recommended.

Magnesium Oxide

One of the components of dolomitic monohydrate (Type N) lime is magnesium oxide; consequently the effects on strength caused by addition of this chemical should give an indication on the effects of using dolomitic monohydrate lime instead of calcitic hydrated in lime-fly ash stabilization.

Small amounts, up to 0.5 percent, resulted in a slight decrease of strength, but increased amounts up to the largest amount tried, 5.0 percent, increased the strength (Fig. 1, k). The results indicate that dolomitic monohydrate limes are more effective with the fly ash used here, but they are not as effective as calcitic hydrated lime plus treatment with some of the other chemical additives. The results also warranted an investigation on the effects of chemical additives to dolomitic lime-fly ash mixtures.

EXTENDED EVALUATION

To complement the tests made with Ottawa sand, the study was extended to include four natural soils: a dune sand, a friable loess, an alluvial clay and a gumbotil (Tables 1 and 2).

The evaluation of magnesium oxide indicated that dolomitic monohydrate lime might be more effective than calcitic hydrated lime, and that the use of dolomitic lime might make unnecessary the addition of chemicals; therefore the use of both limes, calcitic hydrated and dolomitic monohydrate, was evaluated. Commercial type limes were used.

Three fly ashes were selected to include such desired variations in their properties as coarseness, carbon content, specific surface, etc.

From the preliminary studies, four chemicals warranted further evaluation based on strength improvement and economics: sodium carbonate, sodium hydroxide, sodium metasilicate and sodium chloride.

The proportions of soil, lime, and fly ash used were 76.5, 6 and 17.5 percent. The amount of chemical used was 1.0 percent in mixtures prepared with all soils, limes and fly ashes, except that 0.5 percent was also used with dune sand and fly ash No. 1. The evaluation was not intended to be an economic comparison of lime-fly ash-chemical stabilization of soils with other methods of soil stabilization, but rather to be a check on the possible beneficial effects of the selected chemicals on soil-lime-fly ash mixtures. Therefore, the mixture proportions are within the range commonly recommended for lime-fly ash stabilization, and the amount of chemical added is probably near the optimum amount, except for sodium metasilicate.

The molding moisture content for mixtures was deducted from the moisture-density and moisture-strength curves of soil-lime-fly ash mixtures without chemical additives. With friable loess, maximum density and maximum strength occurred at the same moisture content, and this was considered the optimum. The moisture requirements for maximum density and maximum strength of mixtures with sand were not the same, and as the moisture content for maximum density gave very low strengths, the moisture content for maximum strength was used as the optimum. The molding moisture of mixtures with alluvial clay and gumbotil was about two percentages above the optimum for maximum density, in order to get maximum strengths.

Dune Sand

The data of tests made with this soil and combinations of calcitic hydrated or dolomitic monohydrate lime and fly ashes Nos. 1, 2 and 3 are plotted as bar graphs in Figures 2 through 5.

Sodium carbonate, sodium metasilicate and sodium hydroxide in amounts of 1.0 percent increased 7-, 28- and 90-day strengths of all dune sand-lime-fly ash mixtures considerably. Sodium chloride increased 28- and 90-day strengths of dune sand-calcitic lime-fly ash mixtures to a great extent and also increased substantially the 90-

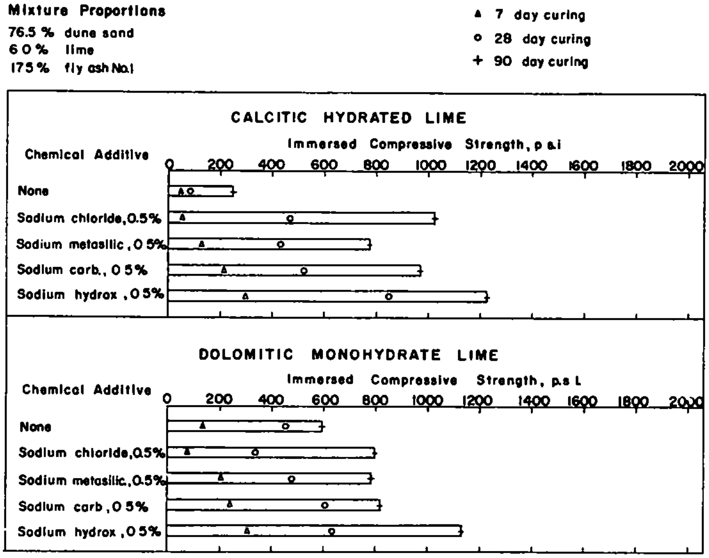


Figure 2. Effect of 0.5 percent chemical additive on strength of a 76.5:6:17.5 mixture of dune sand-lime-fly ash No. 1.

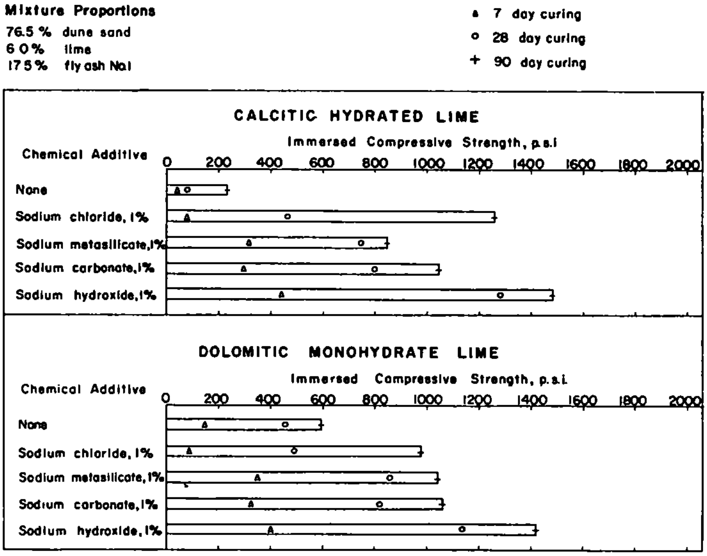


Figure 3. Effect of 1.0 percent chemical additive on strength of a 76.5:6:17.5 mixture of dune sand-lime-fly ash No. 1.

day strength of dune sand-dolomitic lime-fly ash mixtures except those made with fly ash No. 2, for which the strength increase was minor.

The strengths obtained using 0.5 percent chemical in mixtures with fly ash No. 1 are smaller than those obtained with 1.0 percent chemical additive, but the strength increases follow the same trend for both amounts.

Friable Loess

All four chemicals increased the strength of loess-calcitic lime-fly ash mixtures

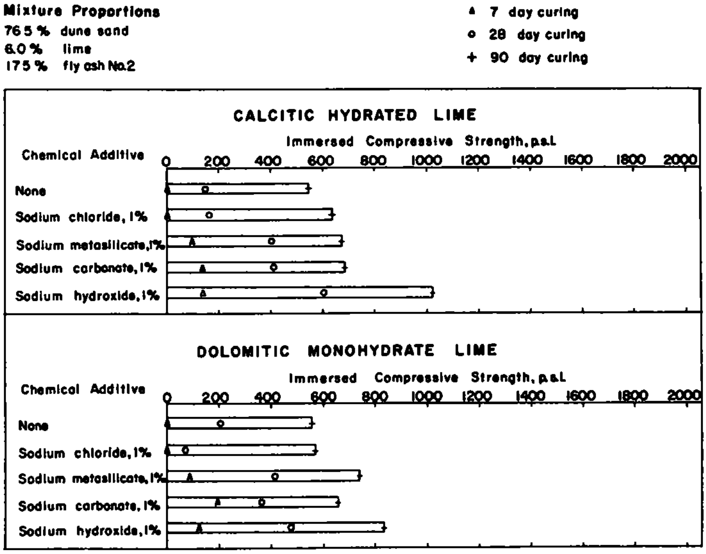


Figure 4. Effect of 1.0 percent chemical additive on strength of a 76.5:6:17.5 mixture of dune sand-lime-fly ash No. 2.

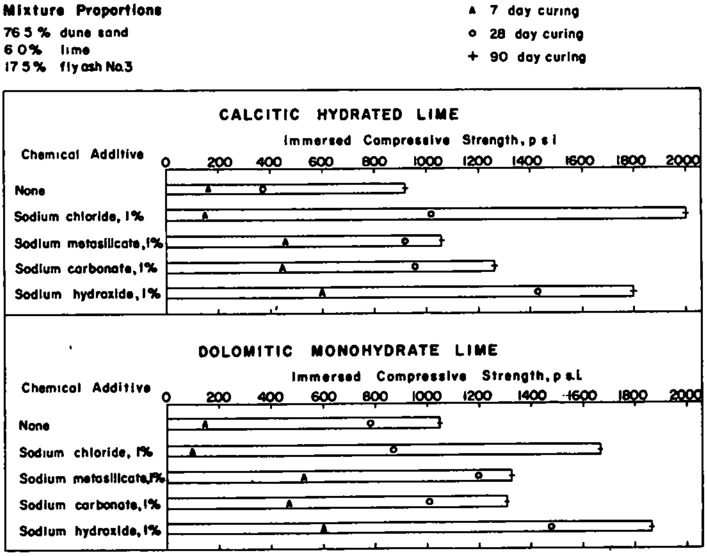


Figure 5. Effect of 1.0 percent chemical additive on strength of a 76.5:6:17.5 mixture of dune sand-lime-fly ash No. 3.

except for 90-day strength of specimens made with sodium metasilicate and fly ash No. 2 (Figs. 6 through 8). Loess-dolomitic lime-fly ash mixtures were not appreciably benefited by the addition of the chemicals.

The use of sodium chloride, sodium carbonate or sodium hydroxide in mixtures of friable loess, calcitic hydrated lime and fly ash No. 1 or No. 3 could be recommended. The strengths produced by the addition of these chemicals in mixtures containing calcitic hydrated lime surpassed that of the similarly proportioned mixtures containing dolomitic monohydrate lime, with or without chemicals.

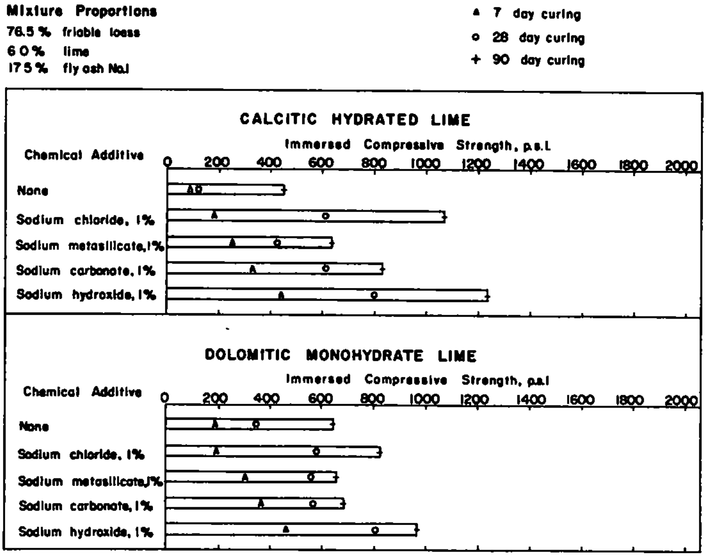


Figure 6. Effect of 1.0 percent chemical additive on strength of a 76.5:6:17.5 mixture of friable loess-lime-fly ash No. 1.

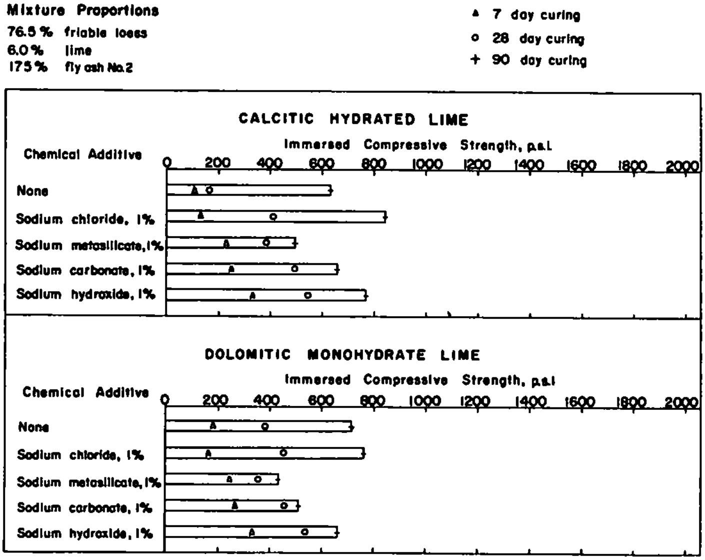


Figure 7. Effect of 1.0 percent chemical additive on strength of a 76.5:6:17.5 mixture of friable loess-lime-fly ash No. 2.

Alluvial Clay and Gumbotil

The effect of chemical additives on these clayey soils stabilized with lime and fly ash was nil and sometimes detrimental; consequently the results are not graphed. Specimens treated with sodium carbonate, sodium hydroxide or sodium metasilicate and cured for 90 days were so weakened during the 24-hr immersion period that strength testing was impossible, or strengths were much lower than the strengths of specimens made without treatment or with sodium chloride as the additive. Sodium carbonate, sodium hydroxide and sodium metasilicate are therefore not recommended for use as additives to montmorillonitic clay soils stabilized with lime and fly ash. Sodium chloride was neither harmful nor beneficial; so there appears no reason to use it as an additive.

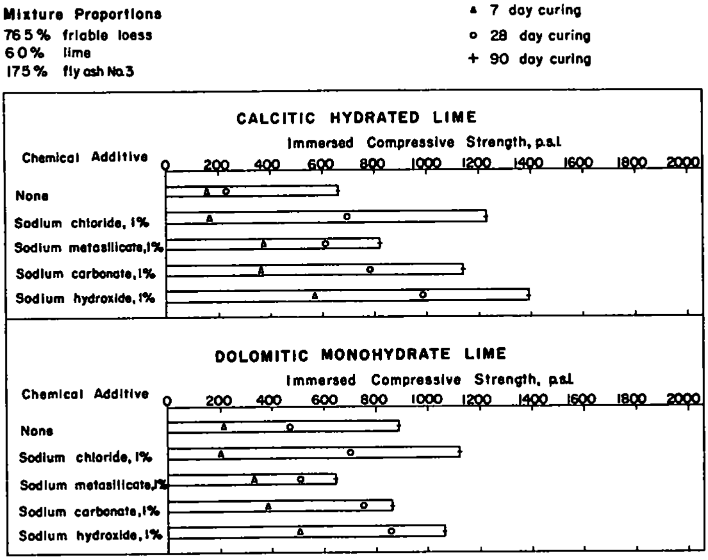


Figure 8. Effect of 1.0 percent chemical additive on strength of a 76.5:6:17.5 mixture of friable loess-lime-fly ash No. 3.

Sodium Carbonate

This chemical is very effective in the improvement of 7- and 28-day strengths of sandy soil-lime-fly ash mixtures, regardless of the kind of hydrated lime used. Ninety-day strengths are also benefited, but to a lesser extent. Sodium carbonate also improved the early strength of friable loess-lime-fly ash mixtures containing calcitic hydrated lime, but it did not improve the early strength of mixtures containing dolomitic monohydrate lime.

Owing to its relatively low cost, sodium carbonate in amounts of 0.5 to 1.0 percent is a most promising additive for sandy soils stabilized with lime and fly ash.

Neither sodium carbonate, nor sodium hydroxide or sodium metasilicate, are recommended as additives to montmorillonitic clay soil-lime-fly ash mixtures because they reduce the long-term immersed strength, and do not affect early strength.

Sodium Hydroxide

This chemical greatly improved the strength of sand and friable loess stabilized with hydrated lime and fly ash. The over-all effectiveness is greater with calcitic hydrated lime than with dolomitic monohydrate lime. As an example of the strength increases possible, dune sand stabilized with calcitic hydrated lime and fly ash No. 1

showed the following strength improvements by the addition of 1.0 percent of sodium hydroxide:

Curing Period	Untreated Mixture	Treated with 1.0% NaOH	Increase
7 days	42 psi	443 psi	10.5 times
28 days	74 psi	1,291 psi	17.4 times
90 days	241 psi	1,493 psi	6.2 times

Its use is therefore recommended with these types of soils.

Sodium Chloride

This chemical used as an additive increased the 90-day strength of dune sand-lime-fly ash mixtures, in some cases to a considerable extent. Seven-day strength was slightly reduced, and 28-day strength was sometimes greatly improved and sometimes was even reduced. All 90-day strengths were increased by the addition of sodium chloride. The same trends were observed in mixtures with friable loess as a soil. Thus sodium chloride may be a promising additive to friable soils stabilized with lime and fly ash when long-term strengths are desired. The strength of montmorillonitic clay soil-lime-fly ash mixtures was not affected by adding sodium chloride.

Sodium Metasilicate

Sodium metasilicate in the amount of 1.0 percent increased the strength of the dune sand-lime-fly ash mixtures. It can also improve friable loess-lime-fly ash mixtures containing some fly ashes. For the percentage used, this chemical rates lower than sodium carbonate or sodium hydroxide. Greater amounts are suspected to improve greatly the strength of friable soils; they were not tried here for economic reasons.

Calcitic Hydrated and Dolomitic Monohydrate Limes

The dolomitic monohydrate lime used produced better strengths than the calcitic hydrated lime when the mixtures were not treated with chemicals. However, the calcitic lime responded better to chemical treatments, surpassing in most instances the strength of mixtures made with dolomitic lime, treated or not.

EFFECTS OF ADDITIVES AT LOW CURING TEMPERATURES

The strengths obtained with lime-fly ash mixtures depend greatly on curing temperatures. When soils are stabilized with lime and fly ash in the late part of the summer in temperate climates, they may not develop sufficient strength to withstand the imposed stresses of the colder seasons. This may lead to failure of the pavement.

The effect of chemical additives at low temperatures was investigated. Dune sand and fly ash No. 1 were used with both calcitic hydrated and dolomitic monohydrate limes. The curing temperature was 43 ± 1 F. Results for 7- and 28-day strengths are given in Figure 9.

Calcitic Lime

The mixture of dune sand, calcitic hydrated lime and fly ash No. 1 without additive, cured for 7 days, failed during the period of immersion in water. The same happened with the mixture with 1.0 percent sodium chloride as additive. Additions of 1.0 percent sodium metasilicate, sodium carbonate or sodium hydroxide, however, gave strengths of about 100 psi.

After 28 days curing, the mixture without additive showed some immersed strength, 41 psi. This strength was increased five- or sixfold by additions of 1.0 percent sodium metasilicate, sodium carbonate or sodium hydroxide. Sodium chloride produced a slight strength improvement.

Dolomitic Lime

The untreated dune sand-dolomitic lime-fly ash mixture did not show any immersed strength after 7 days curing. Additions of 1.0 percent sodium metasilicate gave a 7-day strength of 107 psi; 1.0 percent sodium carbonate gave 57 psi; and 1.0 percent sodium hydroxide gave 76 psi. Sodium chloride was not beneficial.

After 28 days, the untreated mixture had a strength of 111 psi. Additions of 1.0 percent sodium metasilicate or sodium carbonate increased the strength more than two times. One percent sodium hydroxide increased the strength almost three times, to 298 psi. Specimens with sodium chloride did not show any immersed strength.

Discussion

The beneficial effects of some additives to the lime-fly ash pozzolanic reaction are very important when low temperatures are expected during the curing period. Addition of promising chemicals may lengthen the working season for stabilization of soils with lime and fly ash.

The strengths obtained with dune sand-lime-fly ash No. 1 mixtures cured at 43 ± 1 F may be of the order of 200 to 300 psi by the addition of a small amount of sodium

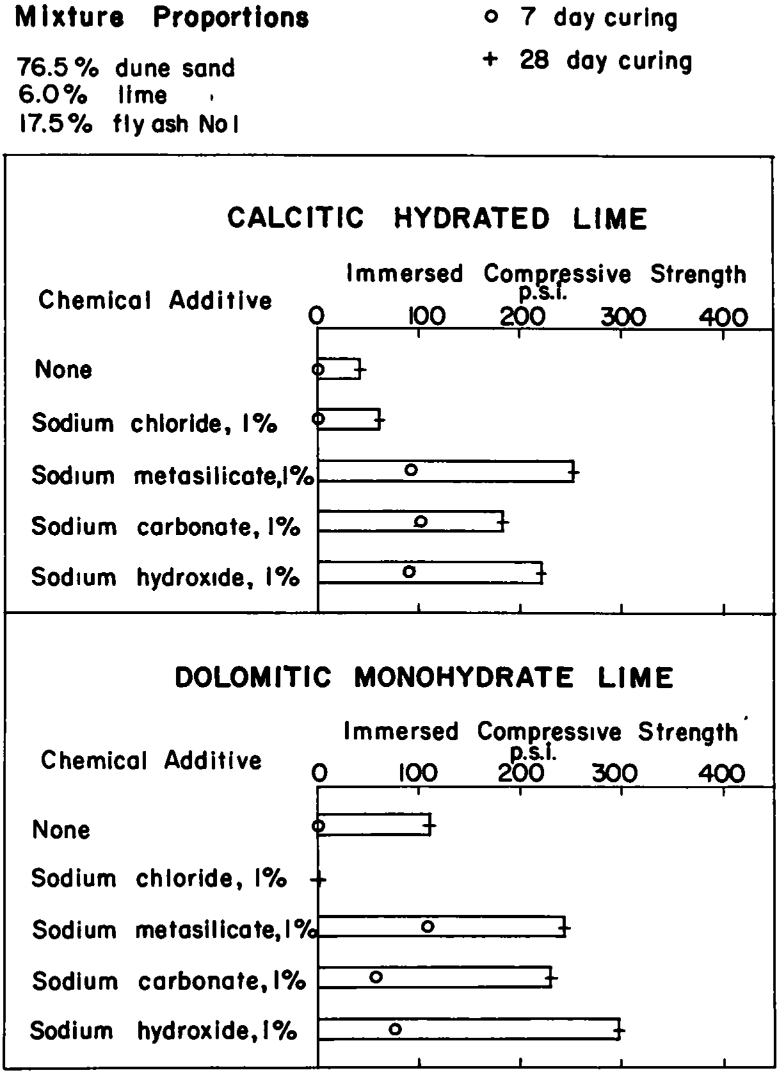


Figure 9. Effect of 1.0 percent chemical additives on strength of a 76.5:6:17.5 mixture of dune sand-lime-fly ash No. 1 cured at a temperature of 43 F.

metasilicate, sodium carbonate or sodium hydroxide. Those strengths may be sufficient in a base course to withstand the adverse effects of traffic and lower winter temperatures. Untreated sand-lime-fly ash No. 1 mixtures showed strengths of 100 psi or less after 28 days curing, which are insufficient for a base course. The same beneficial effects may be expected with other fly ashes. Sand-lime-fly ash mixtures made with either calcitic hydrated or dolomitic monohydrate lime increased in strength by the addition of sodium metasilicate, sodium carbonate or sodium hydroxide, but the data obtained herein were not sufficient to indicate which lime is more beneficial.

The chemical additives, as salts, also assist by lowering the freezing point of the free water in stabilized soil mixtures. By depressing the temperature at which the free soil water freezes, more time is allowed to gain strength; and the stabilized soil is exposed for shorter periods to the damaging effects caused by ice formation.

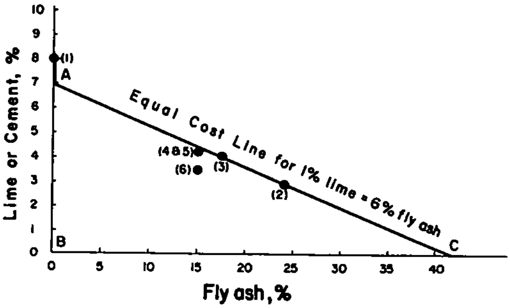


Figure 10. Equal-cost-line chart for dune sand stabilized with selected admixtures of lime-fly ash or lime-fly ash-chemical compared with dune sand-cement.

DURABILITY EVALUATION

To evaluate the effectiveness of lime-fly ash stabilization with and without chemical additives, a few mixes were prepared and compared with soil-cement mixtures. These selected mixtures were submitted to a severe freeze-thaw test.

Based on the previous investigation the selected mixes should include dune sand, fly ash No. 3, and dolomitic monohydrate lime when no chemicals are added. The lime should preferably be calcitic hydrated when chemicals are added. Sodium carbonate and sodium chloride were chosen as additives based on strength improvements, cost of the chemicals, and practicability of their use in field construction.

Given the desired components of the mixes, the proportions were calculated to compete with the required amount of cement needed to stabilize the same soil. Use was made of the Iowa State equal-cost-line method for soil-lime-pozzolan mix design (12, pp. 21-102).

It was assumed that:

1. Eight percent portland cement is required to stabilize the dune sand.
2. The cost of handling two materials (lime and fly ash) instead of one (if stabilized with cement), is equal to the cost of one percent of cement.
3. The costs of lime and cement are the same, about $22 a ton.
4. The cost of the fly ash is one-sixth that of lime or cement.
5. The cost of sodium carbonate and handling this extra material is 2.5 times that of an equal amount of cement, and that sodium chloride costs the same as cement.

From these assumptions, all the sand-lime-fly ash mixes that are within the triangle ABC of Figure 10 compete economically with soil-cement. The mixes selected (Table 4) all have the same cost or are cheaper than soil-cement.

All five selected sand-lime-fly ash mixtures gave 28-day strengths equal or greater than sand-cement for the same curing period. It has been estimated that after freezing and thawing, the stabilized soil specimens should yield a minimum of 250 psi (4). This value was surpassed by all mixtures; see column p_f of Table 4. It is desirable that soil-stabilized specimens show an index of resistance (R_f) of at least 80 percent to withstand the Iowa climatic conditions. Only mixes Nos. 4 and 6 gave an index of resistance lower than 80 percent; nevertheless they still show R_f values of 78 percent which may be adequate as the values of p_f and p_c are more than 400 psi.

Some mixtures continued gaining strength during the freezing-and-thawing tests and/

or during wetting. None of the mixtures showed any visual detriment by freezing, neither did they show any expansion.

The as-molded dry density of the several mixes changed by as much as 12 pcf, but there is no relation whatsoever between density and strength values.

The principal consequence that can be derived from these tests is that, based on 28-day strength requirements, lime and fly ash may be economically used to stabilize sandy soils. Either straight lime and fly ash mixtures or lime and fly ash mixtures with additives withstood the severity of freezing and thawing tests and had enough residual strength to be considered as good stabilizers. A good quality fly ash (No. 3) was used in these tests; these results may not be reproduced with all kinds of fly ash.

TABLE 4

DURABILITY EVALUATION OF SELECTED MIXES

Mix No.	Proportions	As-Molded Dry Density, pcf	Unconfined Compressive Strength, psi			
			28 Day[a]	P_f[b]	P_c[c]	R_f, %[d]
1	92% sand, 8% p. cement	112.6	474	507	517	98
2	73% sand, 3% dol lime, 24% fly ash No. 3	124.3	792	821	966	85
3	76% sand, 4% dol lime, 17.5% fly ash No. 3	124.4	646	634	674	94
4	82% sand, 3% dol lime, 15% fly ash No. 3 + 0.5% sodium carbonate	117.2	554	452	583	78
4A	82% sand, 3% dol lime, 15% fly ash No. 3	123.8	390	ND[e]	ND	ND
5	82% sand, 3% calc lime, 15% fly ash No. 3 + 0.5% sodium carbonate	116.1	644	596	570	104
6	82% sand, 3% calc lime, 15% fly ash No. 3 + 0.5% sodium chloride	124.1	453	414	454	78
5A-6A	82% sand, 3% calc lime, 15% fly ash No. 3	123.1	120	ND	ND	ND

[a] After 28 days curing and 24-hr immersion in distilled water.

[b] After 28 days curing, 24-hr immersion in distilled water and ten freeze-thaw cycles.

[c] After 28 days curing and 11 days immersion in distilled water.

[d] $R_f = \frac{100 P_f}{P_c}$.

[e] Not determined.

MECHANISM

A complete evaluation of the mechanism of the effects of chemical additives in lime-fly ash mixtures must involve extensive chemical analysis. Based on the strength data and on the assumption that strength is indicative of the extent of the pozzolanic reaction, an explanation of the mechanism is given herein.

The effects of chemical additives on lime-fly ash may be grouped in one or more of the three following categories:

1. Speeding up of the pozzolanic reaction,
2. Production of secondary cementitious products, and
3. Combination with the primary, or pozzolanic, cementitious products.

The first should probably be of a catalitic nature. It may show up particularly in the curve for 7-day strength versus additive content, with a sharp increase in strength for small amounts of chemical added.

In the second category, the chemicals combine or react with lime to form cementitious products like $CaCO_3$, $Ca(PO_4)_2$, $Al(OH)_3$, etc.

In the third category are included those chemicals that may combine or react with the pozzolanic cement produced, with the pozzolanic materials in fly ash or with the soil. This combination or reaction may be a complex one producing better cementitious materials or speeding up the reaction or be a reaction that activates some of the materials, increasing their pozzolanic value.

For a separate evaluation of the different chemicals, they may be grouped on the basis of their reactions—basic, neutral or acidic. Bases and basic salts, also known as alkalies and alkaline salts, produce hydroxyl ions in water solution to varying extents. Acid salts produce hydrogen ions in water solutions to varying extents. Neutral salts in water solution do not upset the natural balance of hydrogen and hydroxyl ions. Another group is formed with phosphoric acid, and magnesium oxide is in a miscellaneous group.

This evaluation is made based on the results obtained with mixtures with Ottawa sand as a soil in this and in a previous paper (6). The characteristics of this sand make it, supposedly, an inert material in the lime-fly ash or lime-fly ash-chemical reactions.

Bases and Basic Salts

Alkaline additives increase the amount of available hydroxyl ions in the moistened Ottawa sand-lime-fly ash system, and as a result the pozzolanic reaction may be accelerated by the increased solubility of the siliceous material caused by the alkalinity (9).

The base, sodium hydroxide, acts as a catalyst supposedly in the following way:

1. It first reacts with the siliceous material to produce intermediate sodium silicates.
2. The over-all reaction goes to completion when the intermediate sodium silicates subsequently react with lime (calcium hydroxide) to form sodium hydroxide and cementitious insoluble calcium silicates.
3. The sodium hydroxide is then free for further reaction with unreacted siliceous material.

In the alkaline salts, sodium carbonate very likely reacts with lime in the moist Ottawa sand-lime-fly ash mixture to form calcium carbonate and sodium hydroxide in the following way,

$$Na_2CO_3 + Ca(OH)_2 \longrightarrow CaCO_3 + 2NaOH$$

The precipitated calcium carbonate contributes cementation to the system, and, as hypothesized in the preceding paragraph, the sodium hydroxide acts as a catalyst.

The other alkaline salts used, sodium phosphate, sodium metasilicate and lithium carbonate, may act in a way similar to sodium carbonate. Sodium phosphate reacts with lime to form calcium phosphate, which may be cementitious, and sodium hydroxide, which acts as a catalyst. Sodium metasilicate forms highly cementitious calcium silicates with lime and releases also sodium hydroxide. Lithium carbonate reacts with lime and precipitates calcium carbonate releasing lithium hydroxide, an alkali that produces the same catalitic effects as sodium hydroxide in the lime-fly ash reaction.

Acid Salts

Acid salts undergo a hydrolysis reaction with the precipitation of weak bases (hydroxides). With calcium hydroxide (lime) and aluminum chloride this reaction proceeds as follows:

$$2AlCl_3 + 3Ca(OH)_2 \longrightarrow 2Al(OH)_3 + 3CaCl_2$$

The weak base formed, $Al(OH)_3$, has some cementing properties that may be beneficial. The calcium chloride formed may also benefit through complex effects of the third category.

With calcium chloride, the principal long-term strength benefits obtained are thought due to a different type of chemical mechanism than previously discussed, and that are included in the third category of effects. Calcium chloride being highly hygroscopic and deliquescent insures a relatively high concentration of calcium ions over a long period of time by providing moisture for a solution. Because lime has a low solubility and a lower ionization constant than calcium chloride, the concentration of calcium ions from lime is lower than that from calcium chloride.

The other acid salt used, manganese chloride, is suspected to produce effects analogous to those of calcium chloride.

Neutral Salts

Sodium chloride, although a neutral salt, may act similarly to calcium chloride, but it gives less benefit to long-term strength perhaps because sodium chloride is less hygroscopic and deliquescent than calcium chloride.

The mechanism of the action of potassium permanganate in lime-fly ash mixtures is also included in the third category. Potassium permanganate, a strong oxidizing agent, may oxidize the carbon in the fly ash with subsequent production of potassium carbonate and the precipitation of manganese dioxide. The potassium carbonate formed may then give rise to further reactions, of the first and second category, similar to those of sodium carbonate, previously discussed, which are beneficial to strength. Potassium permanganate may also clean the surface of fly ash by oxidation of possible organic matter present on it; this may make the fly ash more reactive with lime.

Acid

Very small amounts of phosphoric acid somewhat improved the strength. This may be brought about by the formation of complex calcium phosphates or by the activation of fly ash (1). Increased amounts of acid caused a decrease in strength, which is due to the neutralization caused by the acid which reduced the alkalinity and subsequently the silica release.

Miscellaneous Chemical

Magnesium oxide is supposed to react with lime and fly ash producing effects of the third category. It may enter into the pozzolanic reaction and form complex silicates of calcium and magnesium. The effectiveness of magnesium oxide, a component of dolomitic monohydrate lime, in calcium hydroxide-fly ash mixtures corresponds to the findings of previous research which indicated that dolomitic monohydrate lime gives better strengths than calcitic hydrated lime in soil-lime-fly ash mixtures cured at ambient temperatures.

Chemical Additives in Soil-Lime-Fly Ash Mixtures

Four chemicals were evaluated with soils: sodium carbonate, sodium hydroxide, sodium metasilicate and sodium chloride. The greater benefits were obtained with the sandy soil and the benefits decreased with the increase in the amount of clay in the soil.

With the data at hand it is difficult to evaluate the influence of the soil factor in soil-lime-fly ash-chemical mixtures. The chemical additives used were beneficial in mixtures with friable soils and detrimental in mixtures with montmorillonitic clay soils. It is supposed that the decrease in strength in the clayey soils is brought about by the excess of sodium ions and high alkalinity present in the pore fluid of the soil-lime-fly ash mixtures. Both factors introduce disruptive forces in the clay structures that are not overcome by the cementitious bond of the pozzolanic reaction.

SUMMARY AND CONCLUSIONS

Twelve chemicals were evaluated as additives to Ottawa sand-lime-fly ash mixtures. It was found that the immersed strength of the mixtures may be increased several times by the addition of small amounts of some chemicals. Sodium carbonate, sodium metasilicate and sodium hydroxide appear to be the most promising ones among those evaluated.

The immersed strength of friable soils stabilized with lime and fly ash may also be increased by the addition of small amounts of sodium carbonate, sodium metasilicate, sodium hydroxide, and in some instances sodium chloride.

The benefit in strength increase takes place at ordinary temperatures. The strength increase brought by the addition of chemicals is very critical at temperatures close to freezing because it may permit the use of soil-lime-fly ash stabilization under cold climatic conditions, thus extending the working season.

Using selected compositions of lime and fly ash, or lime, fly ash and chemicals to stabilize a dune sand, it was found that they can compete in strength, freeze-thaw resistance and costs with mixtures of the same soil stabilized with portland cement.

Sodium carbonate is the chemical recommended for use in sandy or even silty soils stabilized with lime and fly ash. Addition of 0.5 percent sodium carbonate permits a reduction of the amounts of lime and fly ash needed to obtain the same strength obtained with larger amounts of lime and fly ash.

ACKNOWLEDGMENTS

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Isolation and Investigation of a Lime-Montmorillonite Crystalline Reaction Product

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● DURING previous research by the writers it was noted that mixes of lime and a soil rich in montmorillonitic clay mixed with water above the liquid limit would within several days apparently be dried below the liquid limit with no loss in weight. To determine what was occurring, a portion of this sample was subjected to X-ray diffractometer analysis. Strong indications of a new reaction product were observed but the product could not be found under the microscope.

The object of this paper is to investigate one phase of the mechanisms producing cementation by attempting to isolate and determine the properties of a crystalline reaction product associated with the action of lime with clay.

From the results obtained in this research, other studies have developed in exploring additional crystalline products and are now being pursued at the Iowa Engineering Experiment Station Soil Research Laboratory. When this research is completed and the information is evaluated it is expected that the relationships involved in the formation of new products and the breakdown of the original clay lattice can be determined.

The research undertaken was (a) to investigate various lime-soil-water systems varying each of the three parameters one at a time to determine the best phase relationships for growing euhedral crystals of sufficient size to be observed under the microscope, (b) to isolate these crystals, and (c) to determine their physical and chemical properties.

The systems investigations were accomplished by preparing a large number of mixtures and allowing them to cure for periods from two days to eight months before examination. Isolating the crystals was done under the microscope, and the determination of their properties was undertaken primarily by microscopic and X-ray methods.

MATERIALS AND EQUIPMENT

Soils

Five soils were used in this investigation. The choice of these soils was based on the type, purity, and amount of the principal clay mineral present and the availability of the soil. Each of these soils is identified by a letter designating the principal clay mineral and a number indicating the percentage content of the soil less than 2 microns in diameter. For example, the designation, M-75, indicates a soil containing montmorillonite as the principal clay mineral with 75 percent of this soil less than 2 microns in effective diameter.

The locations from which these soil samples were taken and other pertinent information appear in Table 1. Table 2 gives the physical and chemical properties of these soils.

Clays

Two different clays were used, one a bentonite and the other a kaolinite. These particular clays were selected because they contained only small amounts of impurities. Information concerning them is given in Tables 1 and 2.

Lime

Reagent grade calcitic hydrated lime, $Ca(OH)_2$, was used to minimize compositional variables. Individual 1-lb bottles of lime were kept sealed until immediately before use to prevent carbonation of the lime by the carbon dioxide in the air.

Fly Ash

Fly ash is "the finely divided residue that results from the combustion of ground or powdered coal and is transported from the boiler by flue gasses." The fly ash used in this study was collected by a Cottrell precipitator at the St. Clair Power Plant of the Detroit Edison Co. in Detroit, Michigan.

TABLE 1

SOIL SITE CHARACTERISTICS

Sample	Location	Classification	Soil Series and Horizon	Sampling Depth (in.)
M-67	Keokuk County, Iowa	Post Kansan paleosol	Mahaska, fossil B horizon	91-101
M-51	Harris County, Texas	Coastal plane deposit, largely deltaic	Lake Charles, probably B horizon	39-144
I-44	Monroe County, Michigan	Probably Wisconsin-age glacial till	Unknown series, probably C horizon	Unknown
I-41	Livingston County, Ill.	Wisconsin-age glacial till	Clarence, C horizon	46-56
K-30	Durham County, North Carolina	Residual soil over medium-grained biotite granite	Durham, B horizon	24 in. below bottom of A horizon
Bentonite	Otay, California	Pliocene age transported altered ash	San Diego formation	120-216
Kaolinite	Bath, South Carolina	Cretaceous age	Upper Hamburg formation	40-100 ft

Optical Equipment

Microscope.—A Bausch and Lomb binocular microscope was used for general investigations of samples. With this instrument a maximum magnification of 90 times is obtained.

A Leitz research model petrographic microscope was used in examining the crystals to determine their index of refraction, interference figure, and optic sign.

X-Ray Equipment

Diffractometer.—The General Electric XRD-5 diffractometer was used for general investigations of samples for the presence of crystalline reaction products.

Debye-Scherrer Camera.—The Debye-Scherrer camera was used to determine lattic constants. The camera is 57.5 mm in diameter and 70 mm in length and is manufactured by Siemens and Halske Aktiengensellschaft, Karlsruhe, Germany. The film measuring device is made by the same company and permits an accuracy of linear measurement of ± 0.01 mm.

Weissenberg Camera.—The Weissenberg apparatus used in this study was manufactured by Otto von der Heyde Co. in Newton Highland, Mass. It was used first to obtain rotation pictures of a single crystal. Following this, Weissenberg photographs were taken to index the reflections and to aid in determining the space group to which the crystal belongs.

MIXTURE PREPARATIONS AND INVESTIGATIONS

Preliminary Work

The first indication of the presence of reaction products was found on subjecting mixtures of soil M-67, 12 percent lime, and water (above the liquid limit), which had been prepared for the determination of liquid limits, to X-ray diffractometer analysis. The results of this analysis showed new X-ray "peaks" occurring at 8.11 and 7.60 Ångstroms. However, in material from 2- by 2-in. specimens compacted at optimum moisture content no such peaks were observable.

TABLE 2

PROPERTIES OF SOILS AND CLAY

Sample	M-67	M-51	IC-44	IC-41	K-30	Bentonite
I.E.E.S. designation	528-8	AR-3	AR-4	AR-8	AR-6	-
Textural composition,[a] 2%						
Gravel (> 72mm)	0.0	0.0	0.0	0.0	0.0	-
Sand (2-0.074mm)	16.6	3.0	7.0	10.0	45.2	-
Silt (74-5μ)	15.5	36.0	36.0	38.0	18.3	-
Clay (< 5 μ)	70.5	61.0	57.0	52.0	36.5	-
Clay (< 2 μ)	67.0	51.0	44.0	41.0	30.0	-
Physical properties						
Liquid limit, %	76.6	64.6	44.0	35.5	51.0	87.0
Plastic limit, %	25.6	17.6	21.1	17.5	25.5	52.2
Plasticity index	50.0	47.0	22.9	18.0	25.5	34.8
Chemical properties						
pH	7.1	8.2	8.4	-	5.7	-
C.E.C. (soil passing No. 10 sieve, me/100gm)	-	27.5	14.5	-	8.4	-
C.E.C. (soil passing No. 40 sieve, mc/100gm)	41.0	33.1	13.4	-	13.5	-
Carbonates, %	0.8	16.1	7.2	-	0.1	-
Organic matter, %	0.2	0.1	0.6	-	0.1	-
Predominant clay mineral[b]	M	M	I and C	I and C	K	M
Classification						
Textural[c]	Clay	Clay	Clay	Clay	Clay	Clay
Unified	CH	CH	CL	CL	CH-CL	CH
BPR(AASHO)	A-7-6(20)	A-7-6(20)	A-6-6(14)	A-6-(11)	A-7-6(11)	A-7-6(20)

[a] Textural gradation tests were performed only on the soil fraction passing the No. 10 sieve. All soils used contained less than 5 percent gravel.

[b] Symbols are M-montmorillonite, I-illite, C-chlorite, and K-kaolinite. Determinations were made by X-ray diffraction analysis.

[c] U.S.D.A. textural classification was used.

Petrographic microscope examinations of the liquid limit mixtures were attempted to isolate the reaction products formed, but no new substances were observed. It then became the object of this phase of the research to attempt to grow euhedral crystals of these reaction products of at least microscopic size and determine their properties. To accomplish this, various phase relationships were investigated empirically to attempt to produce recognizable crystals.

Trial Mixtures—Lime-Water-Soil Systems

Water as the Variable.—That the clay fraction of soil M-67 was the actively participating part of the soil in the reaction with lime appeared to be a reasonable assumption. Therefore soil M-67 was shaken through a 325-mesh sieve to obtain the finest fraction of the soil possible through purely mechanical separation. To obtain the most representative sample possible the sieve pan was emptied often and the material remaining on the 325 sieve was continually repulverized. This process was repeated until only a negligible amount of soil passed through the sieve during a 30-min period. Washing the material through the sieve was not attempted in order to avoid any process which might change the nature of the clay before being mixed with lime.

This material was then mixed with 20 percent lime by weight of oven dry soil, and distilled water was added in amounts of 110, 100, 90, 80, 70, 60, 50, 40, 30, and 20 percent by dry weight of soil plus lime. The materials were mixed in soft plastic containers. After thorough mixing the containers were covered with snap-type lids and placed in a controlled temperature-humidity room (70 F and 95 percent relative humidity).

After 30 days the containers were removed and a sample of the mixture from each was subjected to X-ray diffractometer analysis, the results of which appear in Figure 1. Because the mixtures were smoothed into the sample holders by longitudinal strokes with a glass slide, preferred orientation of the particles was attained. From the diffractometer traces it is evident that the reaction products were present in greater quantities at high moisture contents. At moisture contents below 40 percent the product peaks almost disappear, which explains why specimens compacted at optimum moisture give no indication of these reaction products when subjected to X-ray analysis.

Two peaks are evident in Figure 1. The one at 8.11 Å is the second order basal reflection from montmorillonite sharpened considerably by the action of absorbed calcium and reaches maximum height at higher moisture contents. The 7.59 Å peak is that of a crystalline reaction product and is discussed in more detail later.

Optical examination of the mixtures under the binocular and petrographic microscopes disclosed the formation of euhedral crystals of hexagonal shape. Although these crystals were quite small, the larger ones being of the order of 50 μ across the flats by 5 μ thick, X-ray powder photographs of minute amounts showed them to be the crystals being sought, corresponding to the 7.59 Å peak of Figure 1.

Soil M-67 passing the 270-mesh sieve and retained on the 325-mesh sieve, 20 percent lime, and 100 percent water were mixed, cured for 30 days, and subjected to X-ray diffractometer analysis. No new peaks were formed confirming that it was the clay fraction of the soil entering into the reaction.

From the variable water investigation it appeared that it would be advantageous to mix the samples at high moisture contents and this was done with the mixtures prepared later.

Lime as the Variable.—Using the relation that mixtures should be of high moisture content, the effect of varying the amounts of lime was investigated at a constant moisture content of 105 percent. Lime was mixed with soil M-67 in quantities of 5, 10, 15, 20, 25, 30, and 50 percent by weight of dry soil. X-ray diffractometer traces were run on these mixtures after 30 days moist curing. It was found that the height of the new peaks grew successively larger as the percentage of lime was increased to 20 percent, and above this the height remained relatively constant, whereas the height of the residual calcium hydroxide peak increased. Thus it appeared that for a 30-day period the optimum lime content was about 20 percent for this soil.

Under the microscope, the same extremely small euhedral crystals were observed scattered through the matrix in those samples containing 10 percent lime or more.

Soil as the Variable.—Other soils were also investigated as to their reaction with lime. These soils included M-51, IC-44, IC-41 and K-30. Of these only the montmorillonite soil M-51 showed any significant height of new peaks. Soil K-30 gave a slight indication and the others none at all. However, these mixtures were allowed to cure for only 30 days. As will be seen later, it would be advantageous to let them cure for six months and then examine them by X-ray diffraction.

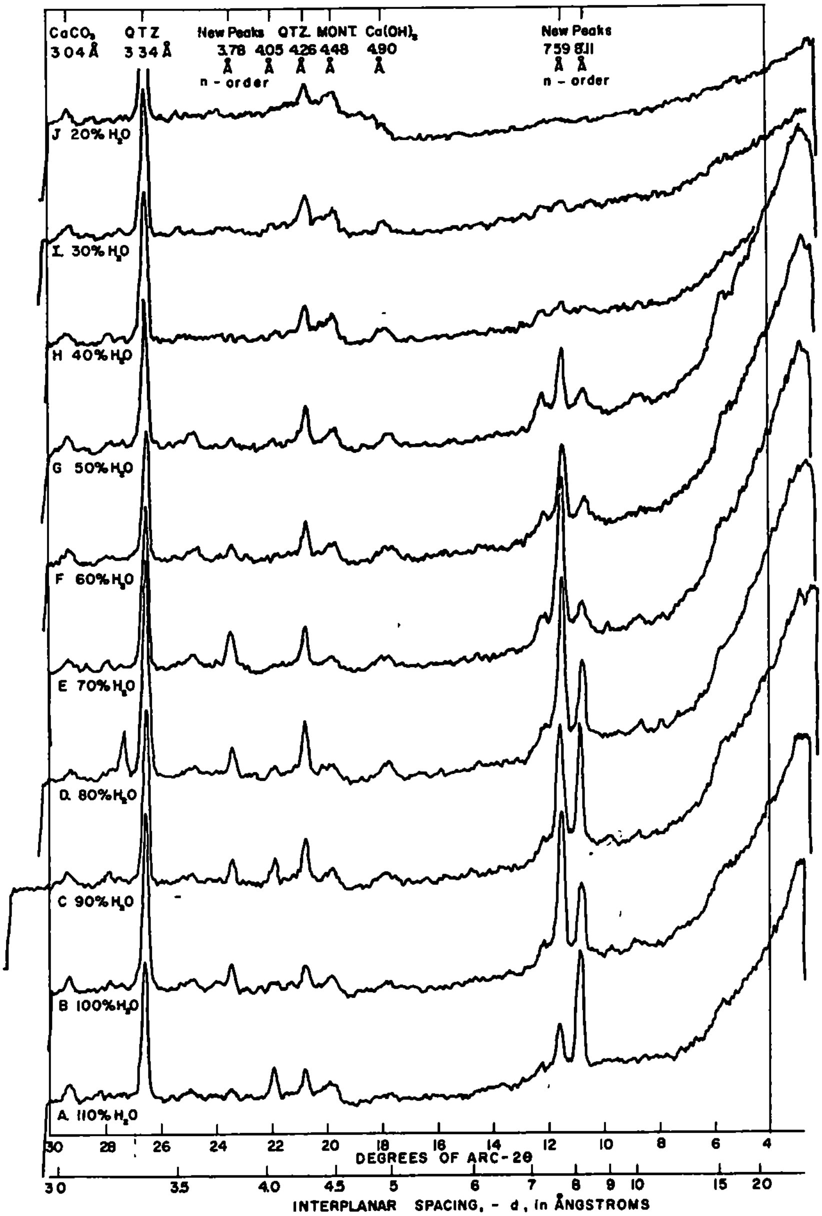

Figure 1. Diffractometer charts, mixtures of soil M-67, lime, and variable water after curing for 30 days.

Trial Mixtures—Lime-Water-Bentonite Investigation

Because the main component in soil M-67 is montmorillonite clay, and because it is the clay fraction which is reacting with the lime, it appeared logical that better results might be obtained by mixing a bentonite with lime. For this purpose, a bentonite from Otay, California, was selected because of the low amount of impurities it contains. Because the amount of montmorillonite was increased, the amount of lime added was also increased to 40 percent by dry weight of bentonite. Water was added in the amount of 110 percent by dry weight of bentonite plus lime. Several samples were made and one was tested after 30 days moist curing. The results of the X-ray analysis on this sample were disappointing in that only a small peak appeared at the expected interplanar spacing and no crystals could be located with the microscope. The other samples were allowed to remain in the humidity room for seven months before being tested.

When, after seven months, they were examined under the microscope, crystals which were large in comparison with those obtained previously were found in comparative abundance. The larger ones were about 500 μ across the flats by 20 μ thick. From these the size graded down to submicroscopic. A microscopic photograph of this crystal appears in Figure 2, A.

Because crystal formation was so much more rapid in soil M-67 than in the bentonite, the soil montmorillonite must be either less well crystallized or else the soil must contain ions which act as accelerators for the reaction.

Trial Mixtures—Lime-Water-Fly Ash Investigation

Because fly ash is well known for its pozzolanic properties and because it has been used with lime in soil stabilization, a mixture of fly ash plus 30 percent lime plus 100 percent water was placed in the humidity room. After seven months curing it was removed and investigated under the microscope. Large euhedral crystals had been formed throughout the mixture with similar geometry to those from the bentonite. A microscopic photograph of these crystals is reproduced in Figure 2, B.

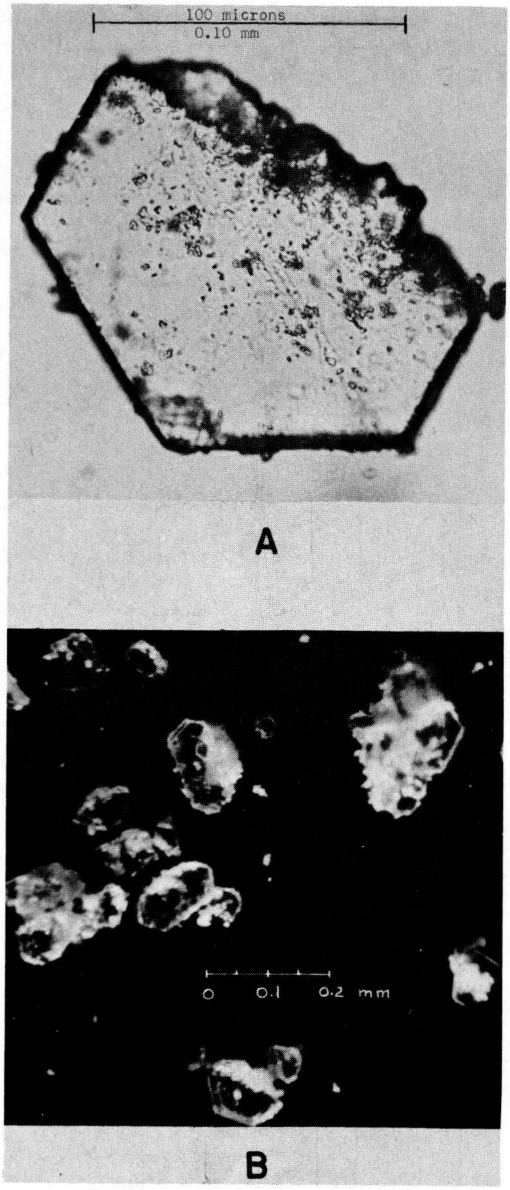

Figure 2. Microscopic photographs of reaction product crystals.

The 7.59 Å basal spacing was observed, but a Debye-Scherrer photograph was not made for several months. After several months, X-ray analysis showed that the substance had become amorphous, whereas those crystals produced from bentonite retained their crystallinity.

RESULTS

Optical Properties

Crystal Selection.—Under the binocular microscope a search was made for several of the largest perfect single crystals in the bentonite sample. These crystals were carefully removed from the matrix and placed on a glass slide. Here they were cleaned of all adhering fragments by means of a single hair. They were then transferred to a second slide and a cover glass was placed over them for examination under the petrographic microscope. Doubly polarized light was used to determine which of the crystals selected were, in fact, perfect and free from inclusions, twinning, and intergrowths.

Index of Refraction.—Those crystals determined to be acceptable were transferred to individual slides for determination of the refractive index by the method of central illumination. The crystals were mounted in various immersion media of known refractive index and examined under the microscope. By bracketing the media used, one was found which coincided almost exactly with the refractive index of the crystal; that is, no reflection or refraction occurred at the boundary between the crystal and the immersion media. By this means the index of refraction of the ordinary ray was determined to be between 1.545 and 1.550. Because of the extreme thinness of the crystals, the index of refraction perpendicular to this direction was not determined exactly; however, it was considerably less.

Interference Figure.—A crystal was mounted on a slide so that the basal pinacoid was perpendicular to the axis of the scope and was viewed under convergent doubly polarized light. On inserting the Bertrand lens an axial cross appeared. Therefore, the crystal is uniaxial and must be either in the hexagonal or tetragonal system. No birefringence was observed with the crystal in this position. However, as soon as the pinacoidal face was inclined, marked birefringent colors appeared.

Optic Sign.—A gypsum plate was inserted in the slot above the objective lens. A lens-shaped blue color appeared in the quadrants perpendicular to the direction of the slow ray in the plate thus determining the optic sign to be negative.

Physical Properties

Geometry.—In plane polarized light the crystals were transparent and colorless, platy, and hexagonally shaped with beveling observed on the edges. A measurement of the hexagonal interior angles showed them to be exactly 60 deg.

Specific Gravity.—The heavy liquid method was used to determine the specific gravity of the crystals. In this method the crystal is immersed in a liquid of known density. If the crystal is denser than the liquid it will sink to the bottom; if less dense it will float. At some density of the liquid the crystal will neither sink nor float but will be suspended in the liquid. The density of the liquid and the crystal are then the same.

In this study, mixtures of bromoform (g = 2.890) and carbon tetrachloride (g = 1.595) were varied to bracket the density of the crystal. Near the density of the crystals the mixture was varied in intervals of 0.005 gm/ml. A 5-ml sample was removed by a pipette at each point and weighed on an analytical balance to determine the exact density of the mixture. All of the crystals sank to the bottom at a liquid density of 2.060 and all were floating at 2.080 gm/ml; at intermediate densities crystals were observed in all three positions, floating, suspended, and on the bottom, establishing the specific gravity of the crystals at 2.07 ± 0.01.

X-Ray Investigations

Rotating Crystal Cylindrical Camera.—A single crystal free from inclusions was selected and was mounted on a short fiber of glass wool with Ducot cement thinned with amyl acetate in such a manner that the pinacoid (0001) was perpendicular to the axis of the glass wool fiber. The fiber was attached to a thin glass rod and the assembly was then inserted into the head of a two-circle goniometer which, in turn, was attached to the rotating spindle of the camera. The crystal was first aligned optically and then brought into exact alignment by X-ray methods.

After alignment was completed, a 15-hr exposure was taken. The diffracted rays are recorded on a cylindrical film whose axis is parallel and concentric with the rotation axis. The spots on the film formed by the diffracted rays will produce a two-dimensional pattern following a characteristic geometry. A print from this film is shown in Figure 3.

The Bragg angle θ was found by measuring the x and y coordinates of each spot on the film and using the relationship

$$\text{Cos } 2\,\theta = \text{Cos } \tau \text{ Cos } x \quad (1)$$

in which $\tau = 2x$;
$x = \text{Cotan } 2y/D$; and
D = camera diameter, in millimeters.

Then the interplanar spacing is

$$d = \frac{n\lambda}{2 \text{ Sin } \theta} \quad (2)$$

in which
λ is the wavelength of copper K_α radiation.

The d-spacings and intensities for all spots observed on the film are tabulated in columns 1 and 2 of Table 3. Because of the size of the spots these d-spacings are not so accurate as those found by other methods.

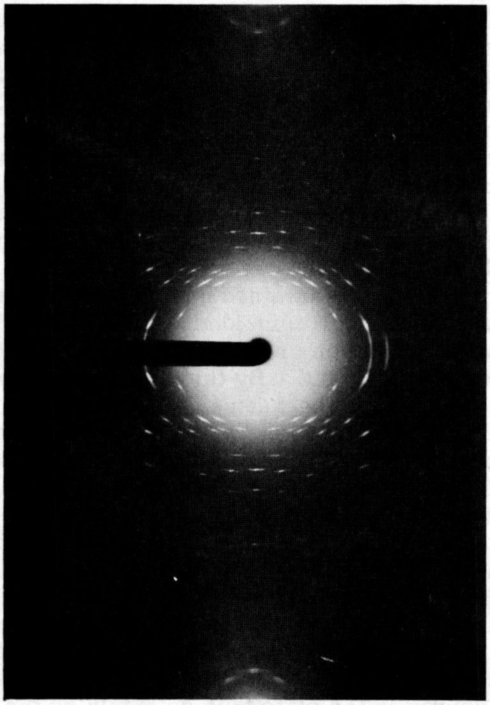

Figure 3. Single crystal rotation photograph of reaction product crystal, Cu K_α radiation.

Weissenberg Method.—The rotating crystal method has several rather serious limitations. A superposition of several reflections occurs causing difficulty in unequivocally indexing the reflections and determining their intensities. The Weissenberg method solves this difficulty by translating the film in a direction parallel to the rotation axis at a rate proportional to the angular rotation of the crystal. So that only one layer line at a time will be recorded, the diffraction cone of that layer is isolated by surrounding the crystal with a closed hollow metal cylinder containing a circular slit through which the desired layer line cone is permitted to emerge.

The equi-inclination Weissenberg method was used in which

$$\mu = -\nu \quad (3)$$

in which
μ = the complement of the angle between the direct beam and the lattice rows, and
ν = the complement of the semi-opening angle of the diffracted cone.

In this method any central layer line projects on the n-level Weissenberg photograph as a straight line of slope 2 because this method brings the crystal rotation axis exactly on the circumference of the reflecting circle. Non-central lattice lines project as ovals, the ovals assuming the same geometric pattern for each level. By combining two of these patterns separated 90 deg from each other, a template may be constructed permitting indexing of the reflections directly.

Indexing of Reflections.—Weissenberg photographs were taken of each layer line from 0 to 26. Prints of the zero level and the twentieth level are reproduced in Figures 4 and 5. Each of the 26 levels were indexed by means of the template discussed earlier and the results are tabulated in column 3 of Table 3.

Inasmuch as the central layer lines occurred at 60-deg intervals, the crystal system was confirmed as being hexagonal. Also by noting the Weissenberg projections of chains of diamond-shaped cells which occur along the position-symmetry lines of the

TABLE 3

DIFFRACTION DATA, Cu K α RADIATION

Single x-tal Rotation		Hex	Rhom	Powder Camera[a]		Back Refl. Weissenberg
d	I	hkl	hkl	d	I	d
7.75	10	00·6	222	7.59	10	-
3.83	7	01·8	332)	3.85	7	-
3.83	5	00·12	444			
3.44	4	10·10	433	3.42	4	-
2.90	8	11·0	$10\bar{1}$)	2.87	9	-
2.86	1	11·3	210			
2.72	4	11·6	321	2.71	1	-
2.54	6	10·16	655)	2.52	6	-
2.54	8	11·9	432			
2.50	1	20·2	200	-	-	-
2.46	7	02·4	220	-	-	-
2.34	9	11·12	543	2.33	6	-
2.31	6	20·8	422	2.30	8	-
2.22	6	02·10	442	2.20	4	-
2.14	10	01·20	776)	2.11	5	-
2.13	5	11·15	654			
2.08	1	12·11	542	-	-	-
2.01	6	20·14	644	2.07	1	-
1.96	1	10·22	877	-	-	-
1.94	5	11·18	765)	1.93	3	-
1.92	4	02·16	664			
1.88	4	21·4	310)	1.86	3	-
1.87	4	12·5	320			
1.81	3	12·8	431	1.79	1	
1.76	1	21·10	532	-	-	
1.73	1	21·22	976	-	-	
1.71	9	20·20	866	1.72	4	-
1.69	3	21·13	655	-	-	-
1.68	9	30·0	$11\bar{2}$)	1.66	5	1.706
1.66	2	12·14	653			
1.64	1	11·24	987)	1.63	4	-
1.64	7	30·6	411			
1.594	3	21·16	754	-	-	-
1.54	5	30·12	633	1.53	3	-
1.483	6	12·20	875	1.475	4	-
1.464	3	20·26	1088	-	-	-
1.45	6	22·0	$20\bar{2}$	1.438	1	1.465
1.43	4	22·6	420	1.416	1	-
1.396	2	13·1	$24\bar{2}$	-	-	-
1.38	1	31·5	410	-	-	-
1.36	4	22·12	642	1.355	2	-
1.301	3	12·26	1097	1.303	2	-
1.279	1	13·16	763	-	-	-
1.229	1	04·8	440		-	-
1.200	2	31·20	965	1.195	1	-
1.166	-	13·22	985	1.166	1	-
1.131	1	23·8	530	1.137	1	-
1.121	2	32·10	$73\bar{1}$	-	-	-
1.109	1	04·20	884	-	-	-
1.097	5	41·0	213)	1.089	1	1.094
1.092	3	23·14	752			
1.088	3	41·6	510	1.081	1	-
1.072	1	32·16	870	-	-	-
1.056	4	41·12	732	1.050	1	-
1.034	3	23·20	974	1.029	1	-
0.984	1	05·10	810	-	-	-
0.967	3	33·0	$30\bar{3}$	-	-	0.963
0.960	1	50·14	833	-	-	-
0.958	2	33·6	$52\bar{1}$	-	-	-
0.937	1	33·12	741	0.933	1	-
0.926	2	32·4	$41\bar{1}$	-	-	-
0.923	2	50·20	1055	-	-	-
0.876	2	42·20	1064	-	-	-
0.843	2	60·6	600	-	-	-
0.841	2	15·20	983	-	-	-
0.836	2	60·0	$22\bar{4}$	-	-	0.8314
0.818	2	52·12	831	-	-	-
0.803	5	52·0	314	-	-	0.7988[b]

[a] In addition to the powder camera data indicated, lines were also observed at 8.29, 5.52, 4.45, and 3.16 Ångstroms, which were identified as montmorillonite.

[b] Other hexagonal indices observed on Weissenberg photographs in addition to the one listed, xOz, when $z \neq 3n$, are $\bar{x}xz$, $O\bar{x}z$, $Ox\bar{z}$, $\bar{x}O\bar{z}$, and $x\bar{x}\bar{z}$.

Other indices observed in addition to Oyz when $z \neq 3n$ are $\bar{y}Oz$, $y\bar{y}z$, $yO\bar{z}$, $\bar{y}y\bar{z}$, and $O\bar{y}\bar{z}$.

Other indices observed in addition to xyz when $z \neq 3n$ are $\bar{x}$, $x + y$, z; $\bar{x} + \bar{y}$, x, z $\bar{y}$, $\bar{x}$, z, y, $\bar{x} + \bar{y}$, z, x + y, $\bar{y}$, z; y, x, $\bar{z}$, $\bar{y}$, x + y, $\bar{z}$; $\bar{x} + \bar{y}$, y, $\bar{z}$; $\bar{x}$, $\bar{y}$, $\bar{z}$, x, $\bar{x}$ + y, $\bar{z}$, x + y, $\bar{x}$, $\bar{z}$

All possible indices observed when z = 3n.

Weissenberg photographs, it is obvious that the crystals are in the rhombohedral division of the hexagonal system.

To convert the hexagonal indices given in column 3 of Table 3 to rhombohedral indices, the relationship given in Eqs. 4, 5, and 6 were used.

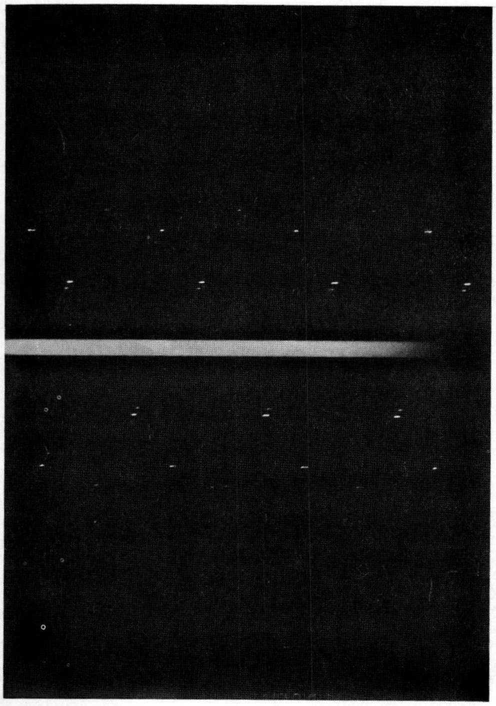

Figure 4. Zero level Weissenberg photograph of reaction product crystal, Cu radiation.

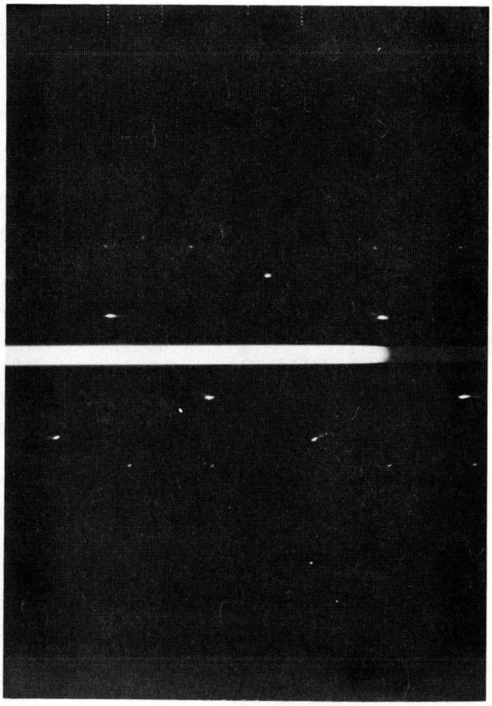

Figure 5. Twentieth level Weissenberg photograph of reaction product crystal, Cu radiation.

$$h_r = 1/3\ (2h + k + l) \tag{4}$$
$$k_r = 1/3\ (-h + k + l) \tag{5}$$
$$l_r = 1/3\ (-h - 2k + l) \tag{6}$$

The results of this conversion appear in column 4 of Table 3.

Hexagonal Cell Lattice Constants.—To determine precise lattice constants, chromium K_α radiation was used to move some of the lines observed with copper radiation into the back reflection region of the powder camera film. A fine collimator (0.7 mm) was used with a helium atmosphere and Straumanis film loading. Temperature during the exposure was maintained at 29 C.

The a_0 and c_0 constants were computed from the measured d-spacings corrected for film shrinkage and were extrapolated against the function

$$\frac{\mathrm{Cos}^2\,\theta}{\mathrm{Sin}\ \theta} + \frac{\mathrm{Cos}^2\,\theta}{\theta} \tag{7}$$

Table 4 gives the a_0 and c_0 constants for the reflections listed which represent results of the ninth a/c approximation and recalculation. Because there was no slope to the θ plot, the average value of the computed constants may be used.

The standard deviation of the values in Table 4 is 0.025 which gives 90 percent confidence limits for c_0 of 46.654 ± 0.018.

A further check on the a_0 lattice constant was made by the back reflection Weissenberg method data for which appears in column 7 of Table 3. Extrapolation of this data against the θ function, Eq. 7, gives an a_0 = 5.756 Å. Because the line breadth on the powder camera film is finer than that of the spots on the Weissenberg film, more accurate determinations are possible using the powder camera.

TABLE 4

HEXAGONAL LATTIC CONSTANTS, Cr K_α RADIATION

hkl	a_0	hkl	c_0
30.0	5.7549	20.20	46.659
22.0	5.7552	21.16	46.689
22.6	5.7549	30.12	46.657
	Av. 5.7550	12.20	46.611
		20.26	46.687
		12.26	46.640
		31.20	46.663
			46.627
	a/c = 0.12335		Av. 46.654

A rough check on the c_0 spacing is possible by using the single crystal rotation photograph. The layer line spacing, ζ, of the reciprocal lattice is given by

$$\zeta_n = \frac{y_n}{\sqrt{r_f^2 + y^2}} \qquad (8)$$

in which

r_f = film radius, and
y_n = distance from film center to any given layer line.

To find the identity period along the rotation axis (c) of the crystal, Eq. 9 is used.

$$c_0 = \frac{n\lambda}{\zeta_n} \qquad (9)$$

The average of the ζ's for the first 16 layer lines was used and a value for c_0 of 46.72 Å was computed. Because film shrinkage could not be computed for this film the c_0 spacing compares favorably with that given in Table 4.

Hexagonal Cell Volume.—The volume of the hexagonal unit cell is given by

$$V = \tfrac{1}{2}\sqrt{3a^2\,c} \qquad (10)$$

Because a_0 = 5.7550 Å and c_0 = 46.654 Å, then the volume is 1338.17 Å^3 or 133.817 x 10^{-23} cm^3.

Rhombohedral Cell Lattice Constants.—Designating the hexagonal lattice constants as a_H and c_H, the corresponding rhombohedral constants are

$$a_R = \tfrac{1}{3}\sqrt{3a_H^2 + C_H^2}$$

$$\text{Sin}\,\frac{\alpha}{2} = \frac{3a_H}{2\sqrt{3a_H^2 + C_H^2}} \qquad (11)$$

and a_R = 15.902 Å and α = 20.850°. (12)

Space Group Determination.—It has already been shown that the crystal belongs to the rhombohedral division of the hexagonal system. Other information is also available from the Weissenberg photographs to help in the space group determination.

By comparing the zero level photograph of Figure 4 (and all other 3n levels) with Figure 236 in Buerger (4), it is apparent that these levels have the characteristic appearance of plane point group C_{6l}. On the original films even more so than on the prints, the intensities of spots at the same height about a central line can be seen to be equal. The films of all other levels show the same relationship of the intensities; however, on levels other than 3n, the plane point group is C_{3l} because the period is 120 deg. Table 28, p. 474, of Buerger (4) identifies this combination as belonging to centrosymmetrical crystal class D_{3d}.

With this information it is now possible to determine the possible space groups to which the crystal may belong by observing any systematic absences of reflections. No systematic absences were observed in the general hkl reflections for the rhombohedral cell. In the hl reflections, the only l indices occurring were for those in which $l = 2n$ indicating a (110) glide plane of component c/2 (c-glide). In the h00, 0k0 and 00l, h, k, and l, respectively, were equal to 2n indicating [100] , [010] , and [001] screw axes of components $\frac{a_1}{2}$, $\frac{a_2}{2}$, and $\frac{a_3}{2}$.

Unfortunately, X-ray photographs cannot distinguish between the presence or absence of a center of symmetry. Thus, although it is known that a threefold axis is present, it is not possible to determine whether it is a rotation or an inversion axis.

Because it is known that the crystal has a threefold axis, is rhombohedral, has a c-glide, and belongs to the centrosymmetrical class D_{3d}, the space group then must be either D_{3d}^6 ($R\bar{3}c$) or C_{3v}^6 ($R3c$).

Chemical Properties

Solubility.—The crystal is soluble in most dilute acids including 0.05 normal hydrochloric acid, sulfuric acid, and acetic acid.

Chemical Analysis.—The total amount of crystals separated during the course of this investigation was about 8 mg. Although the crystals were produced in abundance, the difficulty of separating them from the matrix accounts for the small amount of pure crystals isolated. Even with these relatively pure crystals, it was exceedingly difficult to remove completely all the montmorillonite which clung to them with great tenacity. Therefore a good quantitative analysis was not obtained. Qualitatively it was determined that calcium, aluminum, silicon, and a very small amount of sodium were present; iron was absent.

X-Ray Analysis.—Comparison of the observed d-spacings and intensities with those in the X-ray powder data file (2) showed a striking resemblance between the diffraction data of this crystal and the data for two calcium aluminate hydrates: $4CaO\cdot Al_2O_3\cdot xH_2O$ and $3CaO\cdot Al_2O_3\cdot 8\text{-}12H_2O$. However, small variances in d-spacings and intensities do not permit positive identification with either of these two compounds. Wells (13) reported d-spacings for $4CaO\cdot Al_2O_3\cdot 13H_2O$ which compared almost exactly, except for the basal reflection, with those given in Table 3 for this crystal. However, intensity and indexing data differ considerably.

Others (5) have interpreted the appearance of reflections at 7.8, 3.82, and 2.87 Å to indicate the presence of $C_4A\cdot 13H_2O$.

The index of refraction of the ordinary ray of these crystals is the same as that observed by Wells (13) for compounds approaching the formula $C_4A\cdot 13H_2O$. This compound was also uniaxial negative.

However, the presence of silicon strongly observed in the qualitative analysis of the crystals leaves a measure of doubt as to the exact formula. Certainly, from the observed d-spacings, the structure of the crystal must be almost isostructural with the tetracalcium aluminate hydrates discussed previously. Deviations in intensitives may be due to incorporation of silica in the crystal structure or due to CO_2 in solid solution in the crystal.

On the basis of unit cell volume and density the $C_4A\cdot 13H_2O$ hypothesized formula would indicate a unit cell composed of 3 molecules because

$$N = \frac{\rho VA}{M} \qquad (13)$$

in which

N = number of molecules/unit cell;
ρ = density (2.07 gm/cm^3);
V = unit cell volume (133.82-10^{-23} cm^3);
A = Avogadro's number (6.025.10^{23} molecules/mol); and
M = formula weight (560.49 gm/mole).

Then $N = 2.97 \approx 3$ molecules per unit cell.

Future Investigations.—The prime objective of future research will be to produce a larger amount of these crystals and completely separate them from all contaminants to obtain an exact quantitative analysis and definitely establish their chemical composition.

CONCLUSIONS

1. In mixtures of lime, soil M-67, and water which are moist cured for 30 days, a crystalline reaction product develops. This product is produced in optimum quantities at lime contents of 20 percent by dry weight of soil and at high moisture contents. Only the clay-size portion of the soil enters into the reaction.

2. The same crystalline reaction product develops in mixes of lime, water and bentonite, but only after a considerably longer curing time is it produced in considerable quantity.

3. The crystals produced are transparent, colorless, platy, and hexagonally shaped. Their density is 2.07 ± 0.01 gm/cm^3 at 70 F. Observations under the petrographic microscope prove them to be uniaxial negative with $\omega = 1.548 \pm 0.002$.

4. X-ray investigations of the crystal structure prove that it is the rhombohedral division of the hexagonal system in space groups D_{3d}^6 ($R\bar{3}c$) or C_{3v}^6 ($R3c$). Intensities from powder camera films and indexing by Weissenberg methods establish the four strongest lines at 7.59 Å, 00.6; 2.87 Å, 110; 2.30 Å, 20.8; and 3.85 Å, 01.8 and 00.12. Lattice constants are $a_o = 5.7550$ Å and $C_o = 46.654$ Å for the hexagonal cell and a = 15.902 Å and $\alpha = 20.850°$ for the rhombohedral cell.

5. The chemical composition of the crystal could not be precisely determined; however, the d-spacings indicate a structure similar to that for $4CaO \cdot Al_2O_3 \cdot 13H_2O$ and it is possible that this is the composition of this crystal. If so, previously recorded data for indexing and unit cell dimensions are in error and should be revised.

ACKNOWLEDGMENTS

The authors wish to thank the following members of the Iowa State University faculty for their generous help: Dr. John Lemish, Geology Department, for his responsible handling of the silica analysis; Dr. R. L. Handy, Civil Engineering Department, for making the photomicrographs and powder photographs and precision controlled-temperature determinations of lattice constants, and for his continued interest and advice during the course of the research; and Dr. Robert Rundle, Physics Department, for giving freely of his time and leading the authors through to a conclusion in interpretation of the Weissenberg and single crystal photographs. Some of the research involved use of U.S. Navy property under ONR Equipment Loan Contract Nonr-2625(00).

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Relation of Strength to Composition and Density of Lime-Treated Clayey Soils

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This paper examines the effects of dolomitic monohydrate (Type N) and calcitic hydrated limes, Standard and Modified AASHO density compaction, and the predominant type of clay mineral in the soil, on the immersed strengths of soil-lime mixtures.

Dolomitic lime was found to give higher strengths in montmorillonite and illite clay soils, but only to give higher strengths to some kaolinite clay soils. This trend held at both compactive energies. Modified density compaction was found to give significantly higher strengths than Standard density compaction.

● IT IS KNOWN that small additions of lime to clayey soils may improve their consistency limits, workability and ease of pulverization, and volume change characteristics; and that use of additional amounts of lime may contribute to strength increases (3). However, much more information is needed on the relations of these property improvements to such variables as lime and soil composition, and compacted density. The purpose of this paper is to present some experimental findings concerning the relation of cured strength of lime-treated soil mixtures to predominant soil clay mineral, type and amount of lime, and compacted density.

PROPERTIES OF MATERIALS

Soils

Nine soils from various parts of the United States (Table 1) were used in the investigation. The major groups of soil clay minerals were represented in the clay fractions of these soils; three (AR-2, -3, -7) were dominated by montmorillonite, three (AR-4, -8, -9) by illite, and three (AR-5, -6, -10) by kaolinite group clay minerals. These and other property variations of the soils are given in Table 2. The montmorillonite clay soils contained some illite. One of the illite clay soils, AR-4, contained an appreciable amount of chlorite. The clay fraction of soil AR-5 was rich in halloysite, a kaolinite subgroup mineral. Soils AR-6 and AR-10 contained substantial amounts of mica.

Limes

Six commercial limes were used, three calcitic hydrated limes (A, B, C) and three dolomitic monohydrate "Type N" limes (D, E, F). Limes were used within six months of their receipt from manufacturers. When not in use, lime containers were sealed tightly to prevent carbonization of the lime. An analysis of each lime is given in Table 3.

Water

Distilled water (pH = 6 to 7) was used in all tests.

OPERATIONAL PROCEDURES

Soil Preparation

An identical procedure was used preparing each soil for all tests. As the soil was received it was spread out to air dry on brown wrapping paper placed on a concrete table. After a few days the soil was hand crushed, if necessary, and sieved through a No. 10 sieve. Material retained on the sieve was then placed in a steel bowl (mortar) for crushing. The crusher was a drill press on which was mounted a rubber pestle. A free sliding metal disk, the size of the top of the bowl, was mounted on the pestle stem to prevent loss of soil fines during crushing. The soil was crushed and sieved until the soil aggregations were completely broken down. Particles that would not pass the No. 10 sieve were discarded. Soil passing the No. 10 sieve was mixed to obtain uniformity and placed in 30-gal galvanized cans until used.

TABLE 1

SOIL SITE CHARACTERISTICS

Soil[a] Designation	Sampling Location	Geological Description	Soil Series and Horizon	Sampling Depth (in.)
AR-2	Ringgold County, Iowa	Kansan-age glacial till, calcareous	Shelby (Burchard), C horizon	54-126
AR-3	Harris County, Texas	Coastal Plain deposit, largely deltaic, calcareous	Lake Charles, probably C horizon	39-144
AR-7	Keokuk County, Iowa	Plastic loess, Wiscosin age, noncalcareous	Mahaska, C horizon	36-77
AR-4	Monroe County, Mich.	Probably Wisconsin-age glacial till, calcareous	Unknown, C horizon	Unknown
AR-8	Livingston County, Ill.	Wisconsin-age glacial till, calcareous	Clarence, C horizon	46-56
AR-9[b]	Goose Lake region, Ill.	Commercial product, noncalcareous	Unknown, probably C horizon	Unknown
AR-5	Orange, Va.	Residual soil over diorite, noncalcareous	Davidson, B horizon	Unknown
AR-6	Durham County, N. C.	Residual soil over medium grained biotite granite, noncalcareous	Durham, B horizon	24 below A horizon
AR-10[c]	N. C.	Unknown, noncalcareous	Unknown, probably C horizon	Unknown

[a] Iowa Engineering Experiment Station Soil Research Laboratory Sample Designation.
[b] Commercial product Grundite, supplied by Illinois Clay Products Co., Joliet, Ill.
[c] Supplied by Harris Clay Co., Spruce Pine, N. C.

TABLE 2

PROPERTIES OF SOILS

Soil Designation	AR-2	AR-3	AR-7	AR-4	AR-8	AR-9	AR-5	AR-6	AR-10
Textural composition[a], %									
Gravel[b] (> 2 mm)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Sand (2-0.074 mm)	20.9	3.0	0.3	7.0	10.0	6.4	11.0	45.3	38.4
Silt (0.074-0.005 mm)	40.6	36.0	60.8	36.0	38.0	18.6	37.0	18.3	34.4
Clay (< 0.005 mm)	38.5	61.0	39.0	57.0	52.0	75.0	42.0	36.5	7.0
Clay (< 0.002 mm)	33.0	51.0	-	44.0	41.0	59.3	29.5	30.0	4.0
Passing No. 10 sieve	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Passing No. 40 sieve	100.0	99.0	100.0	98.0	96.0	99.9	90.0	67.0	64.8
Atterberg limits[c], %									
Liquid limit	41.2	64.6	52.1	44.0	35.5	54.8	43.5	51.0	43.0
Plastic limit	16.7	17.6	20.0	21.1	17.5	27.1	27.0	25.5	N.P.
Plasticity index	34.5	47.0	32.1	22.9	18.0	27.7	16.5	25.5	N.P.
Chemical									
pH[d]	8.5	8.8	5.6	8.4	8.3	5.5	5.9	5.7	5.5
C.E.C.[e], me/100 g	17.5	27.3	23.5	14.5	10.8	19.1	11.0	8.4	4.6
Carbonates[f], %	7.4	16.6	1.5	7.2	22.5	1.92	0.65	0.1	0.07
Organic matter[g], %	0.06	0.13	0.2	0.64	0.7	1.54	2.62	0.1	0.02
Predominant clay mineral[h]	M	M	M	I and C	I	I[i]	H	K	K
Classification									
Textural[j]	Clay	Clay	Silty clay	Clay	Clay	Clay	Clay	Clay	Sandy loam
AASHO[k]	A-7-6(14)	A-7-6(20)	A-7-6(18)	A-7-6(14)	A-6 (11)	A-7-6(18)	A-7-6(12)	A-7-6(11)	A-5 (1)

[a] ASTM Method D 423-54T (2).

[b] Textural gradation tests were performed only on the soil fraction passing the No. 10 sieve. All soils used contained less than 5 percent gravel.

[c] ASTM Methods D 423-54T and D 424-54T (2).

[d] Glass electrode method using suspension of 15 g soil in 30 cc distilled water.

[e] Ammonium acetate (pH = 7) method on soil fraction > 2 mm (No. 10 sieve).

[f] Versenate method for total calcium.

[g] Potassium bichromate method.

[h] X-ray diffraction analysis method. Symbols mean: M, montmorillonite; I, illite; I and C, illite and chlorite; H, halloysite (kaolinite group mineral); K, kaolinite.

[i] By analysis of chemical constituents furnished by manufacturer, assuming all alkalies as potassium and determining the number of potassium ions per unit cell.

[j] From the triangular chart developed by the U.S. Bureau of Public Roads, but 0.074 mm was used as the lower limit of the sand fraction (6).

[k] AASHO Designation: M 145-49 (1).

TABLE 3

PROPERTIES AND PRODUCTION INFORMATION OF HYDRATED LIMES

	Calcitic Hydrated Lime			Monohydrate Type N Dolomitic Lime		
	A[a]	B[a]	C[a]	D[a]	E[a]	F[a]
Chemical constituent, % by wt[b]						
Calcium oxide, CaO	73.0	73.46	73.9[c]	49.1	47.52	48.3
Calcium hydroxide, $Ca(OH)_2$	96.5[d]	97.04	97.68	64.8[d]	61.81	63.7[d]
Magnesium oxide, MgO	0.3	0.93	0.64	32.0	33.50	33.2
Silica, SiO_2	0.6	0.34	0.69	0.4	0.6	0.6
Iron-alumina, R_2O_3	0.3	0.24	0.59	0.3	0.62	1.1
Total loss on ignition	24.5-27.0	24.92	24.22	17.0	17.84	16.8
Ca:Mg ratio	-	-	-	1.815:1[e]	1.682:1[e]	1.732:1[e]

[a] Designation of lime manufacturer.

[b] Data supplied by company concerned.

[c] Calculated by molecular weight ratios from amount of $Ca(OH)_2$ present.

[d] Calculated by molecular weight ratios from amount of CaO present.

[e] Calculated as ratio of calcium to magnesium by weight from amount of materials present as their oxides.

Mixing

A predetermined amount of air dry soil was weighed out on a balance sensitive to 0.1 grams and was placed in a mixing bowl. Lime additive (expressed as a percentage of the oven-dry weight of the soil), if used, was weighed and hand mixed with the soil. Additional dry mixing was accomplished for 1 min with a Hobart, Model C-100 ¼-hp mixer, at low speed. Distilled water was added, and the mixture was mixed for 2 min.

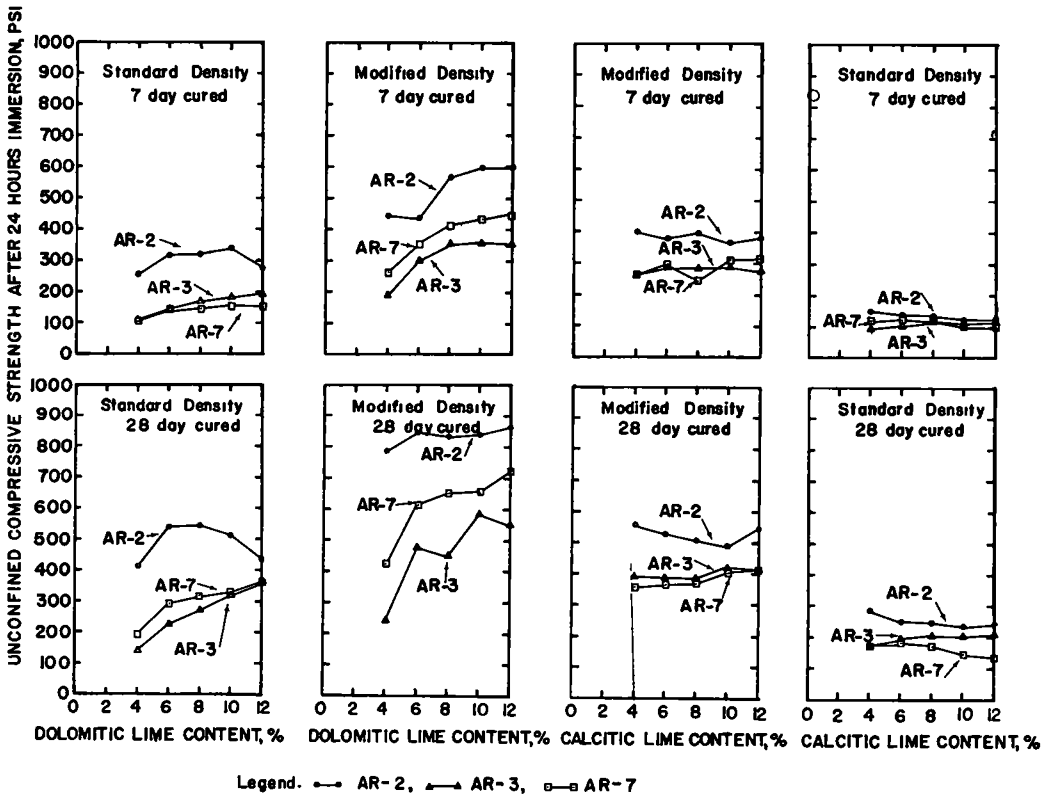

Figure 1. Immersed strength versus lime content relationships after 7- and 28-day curing for montmorillonite clay soils AR-2, AR-3 and AR-7, showing relative effects of Standard and Modified AASHO density compaction, and of dolomitic monohydrate Type N lime D and calcitic hydrated limes.

The mixture was then thoroughly stirred by hand to insure no materials were left unmixed on the sides and bottom of the bowl. The mixture was mixed again for 30 sec to complete the process.

Molding

Test specimens were prepared by use of the Iowa State University molding apparatus as described by Davidson and Bruns (8) and Viskochil, Handy and Davidson (7). The 5-lb hammer, called the standard hammer, is used to compact a predetermined amount of soil mixture in a 2-in. diameter mold to a density near Standard AASHO density (AASHO Designation:T99-57) (1). The 10-lb hammer, called the modified hammer, is used to compact a predetermined amount of soil in a 2-in. diameter mold to a density near Modified AASHO density (AASHO Designation:T180-57) (1).

After mixing and covering with a damp cloth to prevent evaporation, a predetermined amount of the mixture was placed in the compaction mold. The proper hammer was used to attain the desired density. The resultant soil cylinder was extruded from the mold with a hydraulic jack. The compacted specimen was weighed to the nearest 0.1 gram and the height measured to the nearest 0.001 in. The height of the specimen was required to be 2.000 in. ± 0.050 in. All specimens not within these limits were rejected.

Curing

Immediately after being weighed and measured, the specimen was wrapped in waxed paper and sealed to with cellulose tape to prevent loss of moisture and carbonization of lime from carbon dioxide in the air. The wrapped specimens were placed on shelves in a curing room where the relative humidity was at least 90 percent and the temperature was 75 F ± 6 deg.

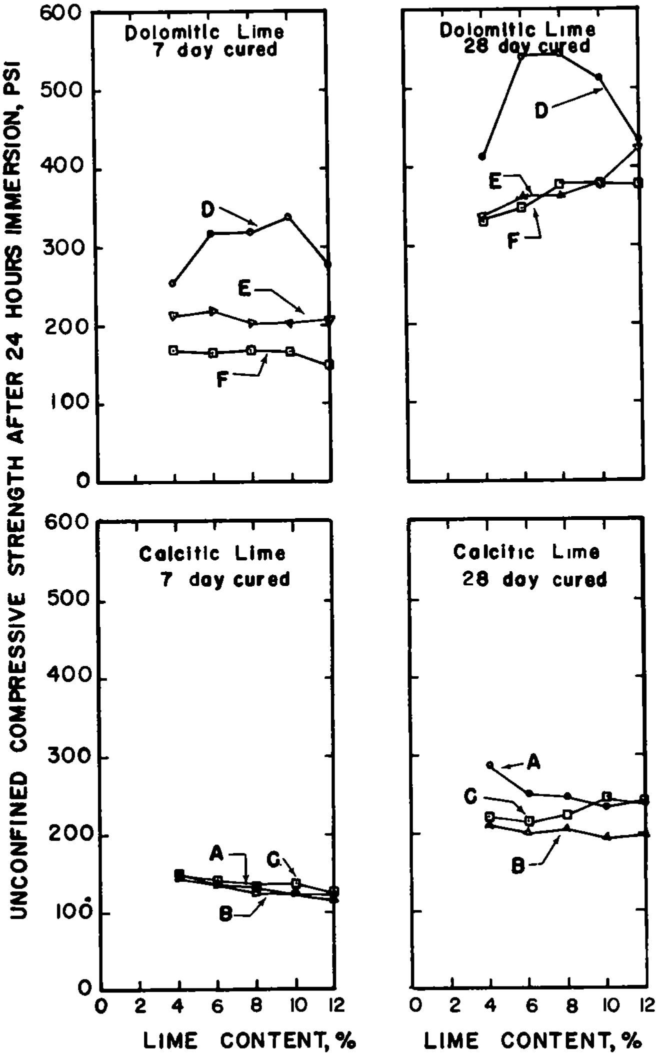

Figure 2. Immersed strength versus lime content relationships after 7- and 28-day curing for montmorillonite clay soil AR-2 and Standard AASHO density compaction, showing relative effects of three dolomitic monohydrate Type N limes and three calcitic hydrated limes.

Testing

The apparatus used for testing the strength of the specimens was a Model AP-170

Stability Testing Machine driven by a ½-hp electric motor with belt reduction. It was manufactured by Soil Test Inc., Chicago, Illinois. Loads are indicated on a sensitive, 10,000-lb capacity proving ring which is supplied with a dial indicator reading to 0.001-in. deflection. Strain was applied to the test specimen at a constant rate of 0.1 in. per min. Strain on the proving ring is related to load by means of a calibration chart.

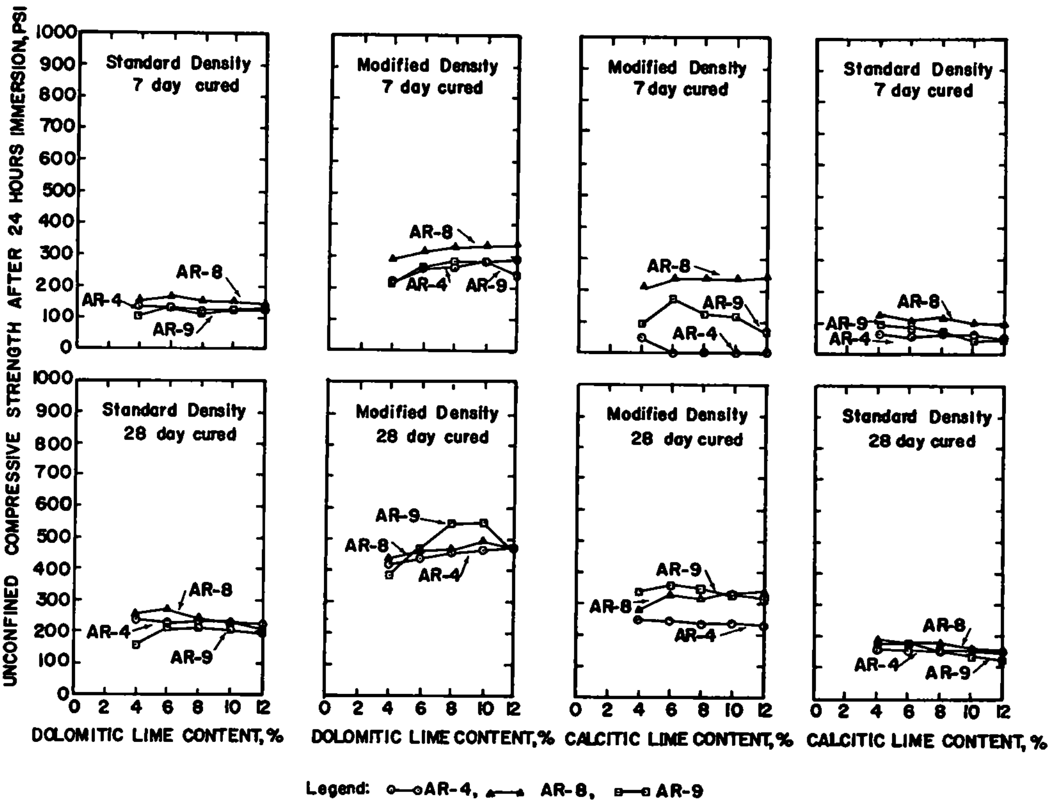

Figure 3. Immersed strength versus lime content relationships after 7-and 28-day curing for illite clay soils AR-4, AR-8 and AR-9, showing relative effects of Standard and Modified AASHO density compaction, and of dolomitic monohydrate Type N lime D and calcitic hydrated limes.

At the time of testing, specimens were removed from the curing room, unwrapped, immersed in distilled water for 24 hr ± 1 hr, and then tested to failure to determine their unconfined compressive strengths. Three identical specimens of each mixture were always tested, and strengths reported are generally the average of three specimens. If the strength of one specimen of a set fell out of the range of 10 percent of the average strength ± 3 psi, the other two samples supplied the average.

EXPERIMENTAL WORK PROCEDURES

Preliminary Study

A series of moisture-density and moisture-strength relationship tests were conducted on mixtures of each soil and 4, 8 and 12 percent of each lime, to evaluate and compare the optimum moisture contents for maximum dry density and maximum strength. These relationships were established for each compactive energy and each mixture by molding five sets of specimens at different determined moisture contents; each set contained three specimens. After being weighed and measured each specimen was moist cured for 7 days, immersed in water for 24 hr, and then tested for strength.

Graphs of dry density versus molding moisture content and of strength versus molding moisture content were plotted. A smooth curve was drawn connecting plotted points and the optimum moisture contents were extracted from the graphs. Optimum moisture contents of 6 and 10 percent lime mixtures were determined by straight-line interpolations.

Although the optimum moisture contents for maximum dry density and maximum

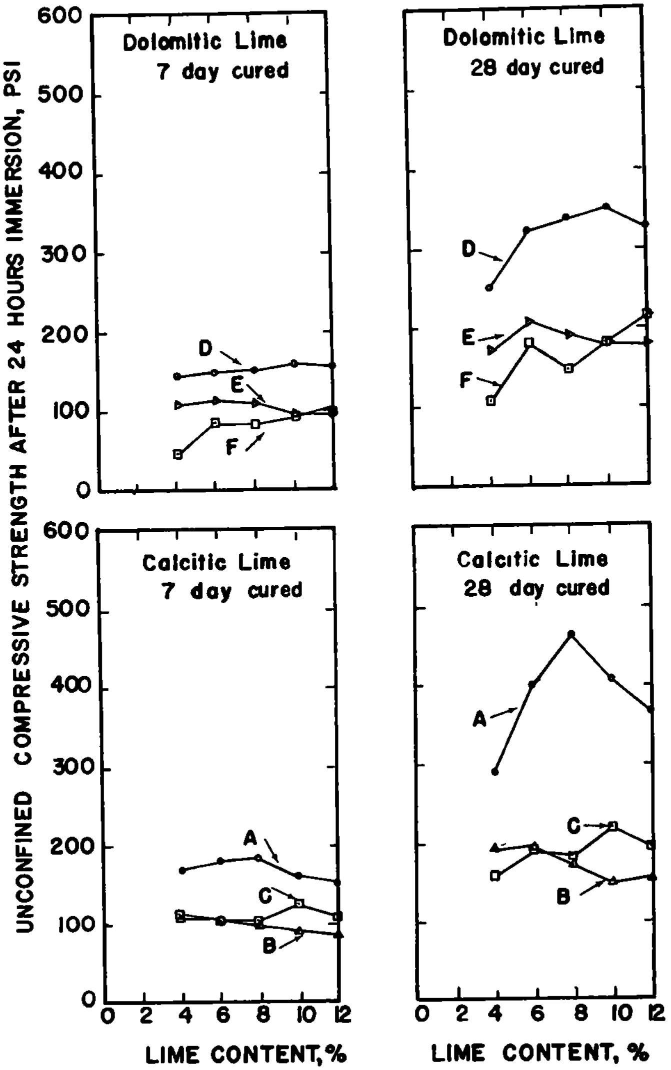

Figure 4. Immersed strength versus lime content relationships after 7- and 28-day curing for illite clay soil AR-8 and Standard AASHO density compaction, showing relative effects of three dolomitic monohydrate Type N limes and three calcitic hydrated limes.

strength were not always identical for each mixture studied, in the majority of cases they were nearly the same, and it seemed permissible and best to use optimum moisture content for maximum dry density as the molding moisture content for the mixtures evaluated in the study of strength versus lime content. This decision applied to the preparation of specimens at both Standard and Modified densities.

With few exceptions, the soils molded with Standard AASHO density compaction had their optimum moisture content for maximum dry density increased 1.7 to 6.8 percent by the lime treatments and their maximum dry density lowered 2.5 to 17.4 pcf. With the same lime treatments, but with Modified AASHO density compaction, the optimum moisture content for maximum dry density increased 0.5 to 3.4 percent and maximum dry density decreased 3.7 to 11.9 pcf.

The kind of lime used had a significant effect on the optimum moisture contents for maximum dry density and maximum strength. For treatments of a soil with equal amounts of different limes, the deviations among optimum moisture contents often exceeded 2 percent. Generally, however, the influence of kind of lime on the optimum moisture contents was least for mixtures at Modified density. Except with one soil, AR-6 at Modified density, dolomitic limes produced higher maximum dry densities than calcitic limes.

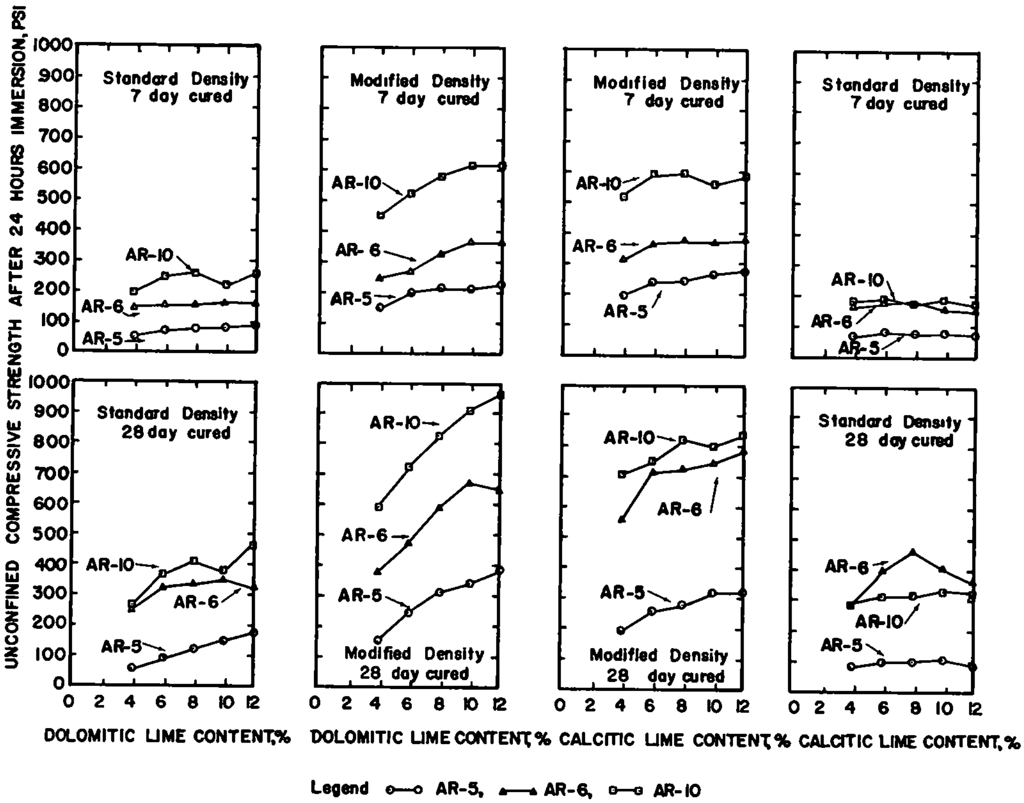

Figure 5. Immersed strength versus lime content relationships after 7- and 28-day curing for kaolinite clay soils AR-5, AR-6 and AR-10, showing relative effects of Standard and Modified AASHO density compaction, and of dolomitic monohydrate Type N lime D and calcitic hydrated limes.

Strength Versus Lime Content

Specimens of the nine soils were molded at 4, 6, 8, 10, and 12 percent lime, by dry weight of soil used, with calcitic lime A and Type N dolomitic limes. The mixtures were molded at densities near Standard and Modified AASHO densities.

Six specimens at each density were molded from each mixture. The moisture content used was the optimum moisture content for maximum dry density for the particular soil, lime content, type lime, and compactive effort used, as determined by the preliminary study. Moisture content samples were taken at the conclusion of mixing of each batch and again after the last specimen of the batch was molded. The average moisture content for the two samples taken was required to be within plus or minus 1 percent of the optimum moisture content specified. Specimens 1, 3, and 5 of each set were cured 7 days and specimens 2, 4 and 6 were cured 28 days.

After curing and immersion the samples were tested. Using average strength values for each three-specimen set, curves were constructed depicting strength versus lime content for each soil at the additional parameters of type of lime and compactive effort.

As a check to insure strengths obtained were indicative of type of lime used and not of one special lime, four other limes were used, two calcitic and two Type N dolomitic, with selected soils: montmorillonite clay soil, AR-2; illite clay soil, AR-8; and kaolinite clay soil, AR-6, were used. Mixtures were molded to near Standard AASHO density. Relative values obtained in this study could then be compared to those obtained in the main study. Using average strength values for each three-specimen set, curves were constructed depicting strength versus lime content for each lime at the additional parameter of soil type.

RESULTS

Montmorillonite Clay Soils

At both Standard and Modified AASHO density, 7- and 28-day cured unconfined compressive strengths of the three montmorillonite clay soils (AR-2, AR-3, AR-7) at lime contents at or greater than 6 percent were significantly higher for dolomitic lime D than for calcitic lime A; 150 to 250 psi higher at Standard density, and 150 to 300 psi higher at Modified density (Fig. 1). Mixtures of soil AR-2 and each of different limes, two dolomitic and two calcitic, also showed that dolomitic lime gives highest strengths (Fig. 2).

Soil-lime mixtures compacted at Modified density attained much higher strengths than when compacted at Standard density; for example, 200 to 350 psi higher for dolomitic lime mixtures and 200 to 250 psi higher for calcitic lime mixtures (Fig. 1).

There is probably an optimum lime content for maximum strength which varies for each soil (4, 5). If optimum lime content is taken to imply a strength maximum or a greatly decreased rate of strength gain with increasing lime content, Figure 1 indicates the optimum dolomitic lime content for maximum strength is at or more than 6 percent, whereas the optimum calcitic lime content is at or probably less than 4 percent.

Illite Clay Soils

At Standard density lime-treated illite clay soils (AR-R—illite-chlorite clay fraction, AR-8, AR-9) did not develop high strengths, at best only between 100 and 300 psi after 28 days of curing (Fig. 1). However, at lime contents above 6 percent, dolomitic lime gave higher strengths than calcitic lime. At Modified density, the illite clay soils did show significant strength improvements with dolomitic lime in relation to calcitic lime, with strength differences ranging from 150 to 200 psi. A comparison (Fig. 4) of strengths developed by mixtures of soil AR-8 and each of four limes, compacted at Standard density, also shows that dolomitic lime gives higher strengths than calcitic lime, particularly after 28 days of curing.

Strengths at Modified density were 150 to 250 psi higher than at Standard density for all mixtures except soil AR-4 and calcitic lime (Fig. 3). Mixtures of soil AR-4 and calcitic lime at Modified density tended to slake during immersion.

All strength versus lime content curves (Fig. 3), except for soil AR-9 and dolomitic lime, show slight or negative slopes above 6 percent lime, signifying that the optimum lime content of illite clay soils may be at or below 6 percent. Soil AR-9 with dolomitic lime had an optimum lime content of 8 percent, the higher lime requirement for maximum strength could be expected because, of the three illite clay soils used, soil AR-9 contained by far the highest percentage of clay-size material (Table 2).

Kaolinite Clay Soils

At both Standard and Modified density, the maximum strengths obtained with kaolinite clay soils AR-5 (clay mineral was predominately halloysite, kaolinite subgroup mineral) and AR-10 were at most only 130 psi higher with dolomitic lime than with calcitic lime (Fig. 5), and at lime contents of 6 to 8 percent, both limes A and D gave about equal strengths. With soil AR-6, calcitic lime A produced somewhat higher strengths than dolomitic lime D, especially in mixtures cured 28 days (Fig. 5). However, the other calcitic and dolomitic limes (Fig. 6) gave about the same strengths in mixtures with soil AR-6.

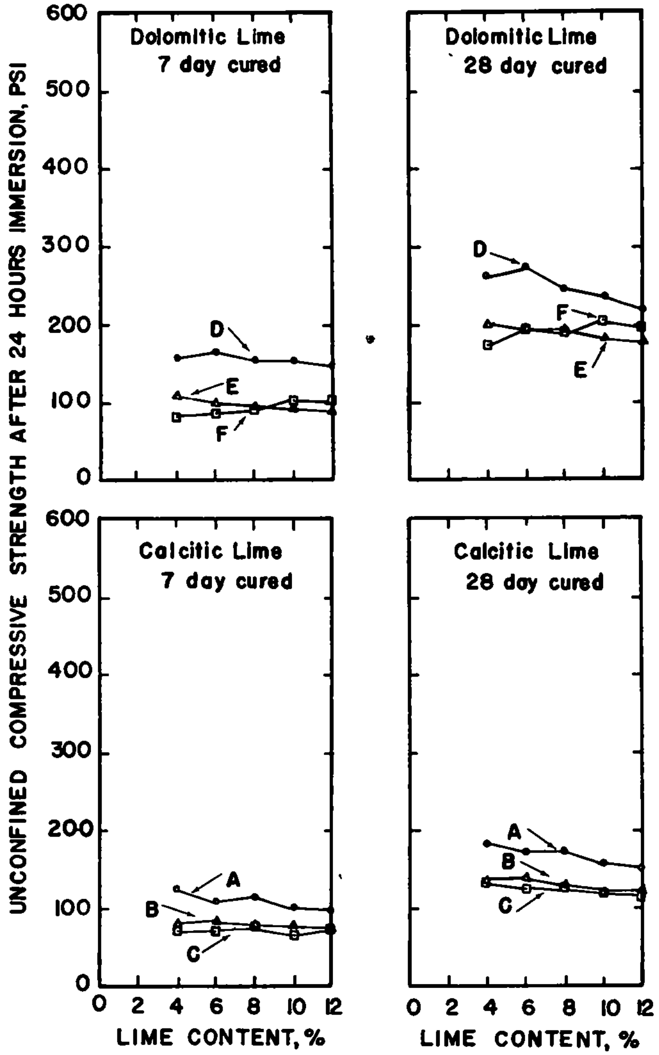

Figure 6. Immersed strength versus lime content relationships after 7- and 28-day curing for kaolinite clay soil AR-6 and Standard AASHO density compaction, showing relative effects of three dolomitic monohydrate Type N limes and three calcitic hydrated limes.

Strengths of all mixtures were significantly improved, 100 to 300 psi, by the use of Modified density compaction instead of Standard.

In general the optimum lime content for the soils was greater with dolomitic lime than with calcitic lime.

CONCLUSIONS

1. In mixtures with the montmorillonite and illite clay soils dolomitic monohydrate Type N lime produces higher immersed strengths than calcitic hydrated lime; more specifically: (a) the montmorillonite clay soils show strengths 130 to 250 psi higher at near Standard AASHO density and 150 to 300 psi higher at near Modified AASHO density, and (b) the illite clay soils show strengths 40 to 90 psi higher at near Standard AASHO density and 150 to 200 psi higher at near Modified AASHO density.
2. Neither dolomitic monohydrate Type N lime nor calcitic hydrated lime consistently produce the highest strengths in kaolinite clay soil-lime mixtures. Dolomitic lime produces the highest strengths in two of the three soils tested, and calcitic lime produces the highest strengths in the third.
3. With all soil-lime mixtures studied, Modified AASHO density compaction gives immersed strengths 100 to 350 psi higher than Standard AASHO density compaction; except for soil AR-2, strengths greater than 500 psi could be obtained only by use of Modified AASHO density compaction.
4. Optimum lime contents for maximum immersed strength are generally higher when using dolomitic monohydrate Type N lime than when using calcitic hydrated lime.

ACKNOWLEDGMENTS

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Most generous in furnishing lime for this research were the U.S. Gypsum Company, Chicago, Illinois; Marblehead Lime Company, Chicago, Illinois; Western Lime and Cement Company, Milwaukee, Wisconsin; and the Linwood Stone Products Company, Davenport, Iowa.

The Illinois Clay Products Company, Joliet, Illinois, manufacturers of Grundite, and the Harris Clay Company, Spruce Pine, North Carolina, suppliers of AR-10, were most kind in supplying clay for this research.

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Lime and Sodium Silicate Stabilization of Montmorillonite Clay Soil

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The results and interpretations of a laboratory soil stabilization investigation of combination lime and sodium silicate treatments of a montmorillonite-rich clay soil are presented. Five kinds of powdered sodium silicate were evaluated in mixtures with the soil and a calcitic hydrated lime. Mixtures with sodium sesquisilicate pentahydrate appeared least affected by elapsed time between mixing and compaction. Hence this sodium silicate was selected for studies of the effect of lime and sodium silicate mix proportions on immersed strength versus curing time and on freeze-thaw resistance.

● IN THE DEVELOPMENT of simple, economical methods of soil stabilization for use in Iowa, many chemicals and combinations of chemicals have been evaluated as stabilizing agents. This report covers an investigation of the stabilization of a clayey soil with lime and sodium silicate.

Sodium silicate and calcium chloride have long been combined as a grout to seal foundations and tunnels, but the use of this mixture in stabilizing base and subbase courses of pavements has not been very successful. However, combinations of lime and sodium silicate have given satisfactory laboratory results. This prior investigation suggested that the soil-lime-sodium silicate reaction was such that the final strength of the stabilized soil depended on the elapsed time between the mixing of the soil-lime-sodium silicate-water combination and the final compaction of the mixture (6). In field construction of base and subbase courses, several hours may elapse between the final mixing of the soil and the stabilizing agents and completion of compaction. It is of course desirable for this delay to have little or no effect on the strength of the stabilized soil.

The investigation presented in this report was conducted to determine the use of lime and sodium silicate for stabilizing a clayey soil for use as a base or subbase course of a flexible pavement, as follows:

1. The effect of elapsed time between mixing and molding on the strength and dry density of the soil treated with calcitic hydrated lime and various sodium silicates.
2. The 7-day cured strengths of the soil treated with varying percentages of calcitic hydrated lime and the sodium silicate whose strength is least affected by elapsed time.
3. The effect of curing time on the strength of the stabilized soil.
4. The resistance of the stabilized soil to freeze-thaw action.

REVIEW OF LITERATURE

Materials

Sodium silicate is a chemical compound manufactured by melting soda ash and selected silica sand (SiO_2) together at high temperatures. The resulting product is a water soluble glass. This glass is ground into various sized particles for different uses. (Fifteen different solid sodium silicates are manufactured by one company alone.) These sodium silicates vary in alkali-silica ratio, water content and particle size, depending on their proposed use. Sodium silicates are used as detergents, deflocculating agents, films and coatings, sizing, corrosion control agents, bleaching agents, adhesives, and cements (13).

Lime is strictly defined as CaO, but the term is commonly used to include quicklime and hydrated lime. These are oxides and hydroxides of calcium and calcium-magnesium. Commercial lime is manufactured by heating a crushed carbonate rock such as limestone, $CaCO_3$, causing a release of CO_2 and leaving a calcitic (high-calcium) lime or CaO. Another type of carbonate rock called dolomite is also used for lime manufacture. Because dolomitic rocks have various ratios of $CaCO_3$ and $MgCO_3$, the resulting lime, called dolomitic lime, is a mixture of CaO and MgO. Quicklime is another term for lime composed of calcium or calcium and magnesium oxides. Quicklime reacts readily with water to produce slaked lime or hydrated lime, in which all the CaO is converted to $Ca(OH)_2$. The MgO of dolomitic quicklime hydrates less rapidly, and in normal hydration processes most of the magnesium remains as MgO (11).

Previous Investigations

Sodium silicate has been used to improve building materials for more than a century. Johann van Fuchs, a pioneer in the commercial development of soluble silicates, proposed in the early Nineteenth Century that soluble silicates could be used as a hardening agent for artificial stone. He also recognized the reaction of silicates with calcium compounds. During the Nineteenth Century several patents were issued for the manufacture of artificial stones from solutions of silicates mixed with solutions of calcium compounds (20).

As early as 1910, sodium silicate is mentioned as being used as a dust palliative. The application of a solution of sodium silicate followed by an application of a soluble aluminum or calcium salt causes the precipitation of an aluminum or calcium silicate in place on the road stone (10). This led to the development of macadam roads in which the crushed stone was held in place by a cementitious matrix formed by a slurry of finely divided limestone and sodium silicate (9, 15, 20).

Around 1915, Albert Francois, a French engineer, found that the effectiveness of cement grout could be increased by preceding the grout with an injection of alum followed by sodium silicate. Apparently the resulting slippery gel (a colloidal solution of a liquid in a solid (8)) coats the granular surfaces and causes greater penetration of the cement grout (20). Later, a Dutch engineer, Jugo Joosten, injected sodium silicate as a grout into deep foundations followed by an injection of calcium chloride (16). The combination of these two chemicals forms an insoluble gel which fills the voids and prevents seepage of water or the continuance of any settlement (20). This injection method with modifications has been extensively used in foundation work and in controlling seepage, but the method is not practical in solidifying the soil within 3 ft of the surface (17). Another injection method, in which the precipitation reaction is much slower, uses a solution containing sodium silicate and sodium bicarbonate. The gel formed is not the insoluble calcium silicate and gradually deteriorates; it does not have the strength of the calcium silicate gel (14).

Sodium silicate alone was investigated as a stabilizer for various soils. The investigators came to the conclusion that although sodium silicates showed promise as stabilizing agents, the soil should be studied and evaluated prior to treatment, because some soils gave a negative response to the treatment (12).

Investigations of soil stabilization at Iowa State University indicate that combinations of sodium silicate and lime show promise, because the reaction is less rapid than that between sodium silicate and calcium chloride. The chemical reaction produces sodium hydroxide which has a desirable effect on soil stability. These incomplete investigations indicate that a wide range of soils from fine sands to montmorillonitic clay soils can be stabilized with lime and sodium silicate (7).

Mechanism of Sodium Silicate Stabilization

The exact mechanism of sodium silicate soil stabilization is not known. Laws and Page (12) found that sodium silicate reacted with the clay minerals present in the soil, but they did not make a detailed study of the nature or possible mechanism of the assumed reaction.

Sodium silicate is used in soil stabilization mainly because it reacts with soluble calcium salts in water solutions to form insoluble, gelatinous calcium silicates (9). This reaction is advantageous, because water is necessary for the proper compaction of a soil to a desired density (9).

That hydrated calcium silicates are cementing agents has been fairly well established (5). Hydration, a process of a solid combining with water (8), is the essential mechanism of "setting" or hardening of a gel (19). The exact nature of the bonding action of hydrated calcium silicates has been the subject of much investigation, but it has not been fully explained. In addition to its cementing properties, the gelatinous calcium silicate improves the stability of the soil by filling voids, thereby excluding water from the soil. Excessive moisture in ordinary soil causes loss of strength and reduces the soil stability (9).

PROPERTIES OF MATERIALS

Soil

Kansan till, comprised of approximately equal portions of sand, silt and clay-sized materials, was the soil used throughout the investigation. This soil was chosen because it is considered an in-between soil; it is not the worst soil found in Iowa for road building, and it is not the best. Another reason for this choice was that some data on Kansan till had been collected in a previous investigation of lime-sodium silicate stabilization (6). The field location of the sample is given in Table 1, and physical and chemical properties are given in Table 2.

TABLE 1

SAMPLING LOCATION OF SOIL

Location:	SW¼SE¼ of Section 23, Tier 68 North, Range 31 West, Benton Township, Ringgold County, Iowa
Geological description:	Kansan-age glacial till, plastic, calcareous
Soil series:	Shelby
Horizon:	C
Sampling depth:	4½-10½ ft
Laboratory no.:	409-12C

Lime

Calcitic hydrated lime was the only lime used in the investigation, because calcitic hydrated lime gave better results than other types of limes (6).

The properties of the commercial grade lime used are given in Table 3.

Sodium Silicates

Five powdered sodium silicates were used. These sodium silicates varied in molecular ratio of Na_2 to SiO_2 and in degree of hydration. By using sodium silicates that varied in these properties it was hoped that some indication of the influence of these variables on the final results would be obtained. Powdered sodium silicates were used because Ellis (6) had found that adding the sodium silicate in a liquid form caused the soil-lime-sodium silicate mixture to set up or solidify too rapidly to permit proper molding of specimens.

The sodium metasilicate enneahydrate was an analytical grade reagent. The other four sodium silicates—sodium metasilicate pentahydrate, sodium metasilicate anhydrous, sodium sesquisilicate pentahydrate and sodium orthosilicate—were commercial grade sodium silicates. A general description of the sodium silicates is given in Table 4. The sodium silicates are identified by their laboratory designations.

TABLE 2

PROPERTIES OF SOIL

Physical properties:	
Textural composition (% by wt):	
Gravel (above 2.0 mm)	0
Sand (2.0-0.074 mm)	33.9
Silt (0.074-0.005 mm)	27.6
Clay (below 0.005 mm)	39.5
Colloidal (below 0.001 mm)	29.0
Textural classification:[a]	Clay
Atterberg limits:	
Liquid limit, %	39.4
Plastic limit, %	16.5
Plasticity index, %	22.9
Shrinkage limit, %	8.2
Engineering classification:[b]	A-7-6(11)
Predominant clay mineral:[c]	Montmorillonite
Chemical properties:	
Cation exchange capacity (whole soil) me/100 gm	29.46
pH	8.0
Carbonates, %	6.4
Organic matter, %	0.2

[a] By the triangular chart developed by the U. S. Bureau of Public Roads (18, p. 48): 0.074 mm was used as the lower limit of sand fraction.
[b] AASHO Method M145-49 (1).
[c] Determined by X-ray analysis of the fraction passing the No. 200 sieve.

METHOD OF PROCEDURE

Preparation of Mixtures

The required amount of air dried soil passing the No. 10 sieve was dry mixed with the powdered lime and sodium silicate additive for 2 min by a Hobart Model C-100 mixer modified by attaching a scraper blade to prevent the mixture from caking in the mixing bowl. Sufficient carbon dioxide-free distilled water was added to bring the mixture to the desired moisture content, and mixing was continued for 2 min. The mixture was then hand mixed long enough to loosen any of the material stuck to the sides of the mixing container. Immediately after mixing, the mixture was placed in a covered container and allowed to age.

This order of mixing was chosen for the following reasons: In the field the probable method of mixing would be either to mix the lime with the soil, add the sodium silicate and remix, then add needed water and mix again; or to mix the lime and sodium silicate with the soil and then add the water and remix. Because the latter method seemed to be the simpliest and most likely to be employed in the field, it was chosen for this investigation. The water was not added until the lime and sodium silicate were thoroughly mixed with the soil so that the best possible

TABLE 3

PROPERTIES OF LIME

Chemical analysis, (% by wt):	
Silicon dioxide	0.28
Iron and aluminum oxide	0.6
Magnesium oxide	0.59
Sulfur trioxide	0.25
Carbon dioxide	N.D.[a]
Total calcium oxide	73.82
Available calcium oxide	70.3
Loss on ignition	24.1
Combined H_2O	N.D.[a]

[a] Not determined.

distribution of reaction products would be obtained. Carbon dioxide-free distilled water was used to reduce the number of variables involved in the investigation. The mixture was aged in a covered container to prevent evaporation of moisture and to prevent the lime from reacting with the carbon dioxide in the air.

TABLE 4

PROPERTIES OF SODIUM SILICATES

Laboratory Designation	Trade Name	Chemical Name and Formula	Molecular Ratio $Na_2O:SiO_2$	H_2O (%)	Na_2O (%)	SiO_2 (%)
Silicate A	Reagent	Sodium metasilicate enneahydrate $Na_2SiO_3 \cdot 9H_2O$	1:1	57.1	21.8	21.1
Silicate B[a]	Metso Anhydrous	Sodium metasilicate anhydrous Na_2SiO_2	1:1	0.5	51.0	45.5
Silicate C[a]	Metso Granular	Sodium metasilicate pentahydrate $Na_2SiO_3 \cdot 5H_2O$	1:1	41.7	29.5	28.7
Silicate D[a]	Metso 99	Sodium sesquisilicate pentahydrate $Na_3HSiO_4 \cdot 5H_2O$	3:2	38.1	36.7	24.2
Silicate E[a]	Metso 200	Sodium orthosilicate concentrated Na_4SiO_4	2:1	9.5	60.8	27.5

[a] Data from Philadelphia Quartz Company, Philadelphia, Pennsylvania.

All additive percentages used in this study are based on the weight of oven dry soil. Moisture contents are based on the weight of the oven dry treated soil.

Molding of Specimens

After being aged the mixture was molded into cylindrical-shaped specimens, 2 in. high and 2 in. in diameter, with a density near AASHO density (AASHO Method T99-57) (2). The molding method used is described in detail by Davidson and Bruns (4).

Curing of Specimens

Specimens made for the strength vs aging time study, and for the best lime-sodium silicate mixture study were cured for 7 days in a moist curing room. The relative humidity in the room was 95 percent or higher; the temperature was maintained at 75 ± 5 F. After being molded and before being placed in the curing room, each specimen was sealed in wax paper to reduce evaporation of moisture and to prevent carbon dioxide of the air from reacting with the lime during curing. Specimens for the strength vs curing time study were cured for 3, 7 and 28 days.

Strength Testing

All specimens except those specifically designated were immersed in distilled water for 24 hr after curing and before being tested for strength. The immersion reduces or destroys any apparent cohesion in the soil and tests the true cohesion and other stabilizing effects of the soil-lime-sodium silicate mixture. Strengths were measured by unconfined compression testing. The rate of loading was 0.1 in. per min. The maximum load causing failure was considered the compressive strength of the specimen. All of the unconfined compressive strengths were maintained within 10 percent of the average values as required by ASTM Designation: C109-54T (3), or tests were repeated.

Freeze-Thaw Testing

A modified British freeze-thaw test was used to evaluate the resistance of selected mixtures to the seasonal influence of freezing and thawing. Five specimens from each mixture were molded and cured for 7 days, then immersed in distilled water at room temperature for 24 hr. Three specimens from each mixture were then tested for unconfined compressive strength; one specimen was left immersed for 14 more days; and the last specimen was exposed alternately to temperatures of -5 C and 25 C for 14 cycles, each cycle lasting 24 hr. After these 14-day treatments the freeze-thaw specimen and the immersed specimen were tested for unconfined compressive strength. A comparison of these values was used to determine the ability of the lime-sodium silicate stabilized soil to resist freezing and thawing. The details of this test are given by Davidson and Bruns (4).

PRESENTATION AND DISCUSSION OF TEST RESULTS

Effect of Aging Time on Dry Density and Strength

This phase of the investigation had two purposes. The first was to determine the effect of aging time, defined as the elapsed time between mixing and molding, on the strength and dry density of specimens of soil treated with calcitic hydrated lime and each of the sodium silicates. The second was to determine which sodium silicate gave the highest strength and/or was least affected by aging time.

To accomplish these objectives the number of variables affecting the results were reduced to a minimum. Previous laboratory work with lime-sodium silicate soil stabilization indicated that the elapsed time between the addition of water to the soil-lime-sodium silicate mixture and the completion of molding greatly affected the density of the specimen, and suggested that strength might likewise be affected. It was also found that the lime-sodium silicate combination that gave the best results in stabilizing Kansan till was 6 percent calcitic hydrated lime and 6 percent sodium metasilicate pentahydrate (6). Because the aqueous portion of the sodium silicate does not react with the lime, although it does affect the reaction, it was decided to compare the effectiveness of the sodium silicates on an equivalent anhydrous weight basis. Therefore, combinations of 6 percent lime and an amount of each sodium silicate, such that the anhydrous portion was equal in weight to the anhydrous portion of 6 percent sodium metasilicate pentahydrate, were used in the study.

The aging time was the time, recorded to the nearest minute, between the addition of water to the dry soil-lime-sodium silicate mixture and the final compaction of the specimen. Specimens with aging times of approximately 9 min and 1, 2, 3, and 5 hr were molded. The specimens were weighed and measured immediately after molding. At the same time, samples were taken from the mixture to determine the molding moisture content of the specimen. This molding moisture content includes the water present in the sodium silicate as well as the water added. The dry density of the specimen at the time of molding was calculated from the equation:

$$\text{Dry density} = \frac{1.2125 \text{ x weight of specimen}}{(1 + \text{moisture content}) \text{ x height of specimen}}$$

To determine the influence of molding moisture content on the effect of aging time on strength and dry density, the soil-lime-sodium silicate mixtures were molded at various moisture contents. The optimum moisture content for near 100 percent Standard AASHO dry density of Kansan till stabilized with 6 percent calcitic hydrated lime was found to be 17.0 percent. For the soil-lime-sodium silicate mixtures with each of the sodium silicates, it was originally planned to use five different molding moisture contents; two below this optimum for the soil-lime mixture and three above this optimum. These planned moisture contents were 13, 15, 17, 19 and 21 percent. The amount of water needed to reach these molding moisture contents was calculated; however, because the chemical reaction between the lime and sodium silicate differed for each sodium silicate, the actual molding moisture contents differed from the planned molding moisture contents by varying amounts.

The specimens after being molded were moist cured for 7 days and then were immersed in distilled water for 24 hr before they were tested for ultimate unconfined compressive strength. Unless otherwise stated, the term strength when used in this report means unconfined compressive strength after 24-hr immersion.

Discussion

Specimens of Kansan till stabilized with 6 percent calcitic hydrated lime and varying amounts of sodium silicates were molded. The anhydrous portion of 6 percent sodium metasilicate pentahydrate used was equal in weight to the anhydrous portion of each of the other silicates used. This was done as a control. Different molding moisture contents change the effect of aging on dry densities (Fig. 1). The percentages of the different sodium silicates are based on the weight of the sodium silicate in its normal state of hydration. The points are the average of three specimens, molded at as nearly the same time as possible, plotted at the average aging time of the specimens. The molding moisture content shown is the range of the molding moisture contents of the specimens molded at the five different aging times.

In all except one of the soil-lime-sodium silicate-water mixtures, the highest dry densities were obtained at aging times near zero (Fig. 1). This one mixture showed a higher dry density after 2 hr of aging than at shorter aging times; however, this density was so close to the dry densities of the same mixture with greater and lesser aging times that it seems safe to consider that the discrepancy is due to experimental error. As the aging time increased, the dry density of the specimens molded from all the mixtures decreased, thus confirming the indications of the previous Iowa State work (6).

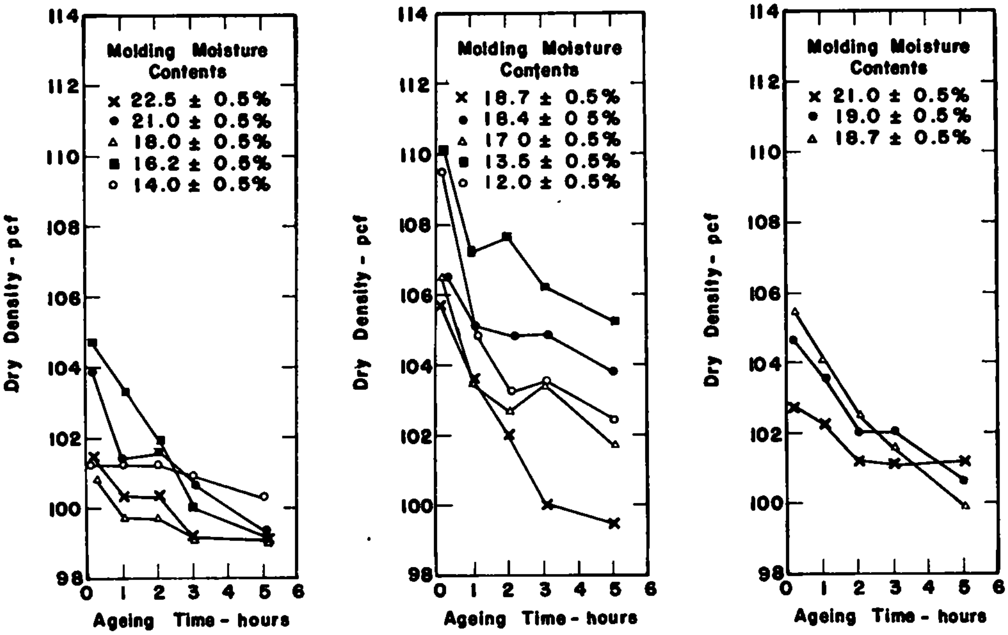
The molding moisture content influenced dry density, but the molding moisture content that gives highest dry density at the shortest aging time does not usually give highest dry density at the longest aging time. This indicates that the optimum moisture content for maximum dry density varies with aging period.

An explanation of the dependence of dry density and optimum moisture content for maximum dry density on aging time is related to the chemical reaction between lime and sodium silicate. As water comes in contact with lime and sodium silicate, the chemicals dissolve and then react to form an insoluble calcium silicate gel. Inasmuch as lime is slowly soluble, the quantity of gel formed is gradually increased as the reaction between the lime and sodium silicate continues. The gel absorbs water and at the same time begins to hydrate. As the gel hydrates, it hardens. The longer this process continues, the more gel is converted into a hydrated calcium silicate.

Both the processes of gel formation and hydration remove free water from the mixture. This free water is necessary to act as a lubricant for the compaction process (9). The gel may possibly act as a lubricant in place of the absorbed water, but further investigation is necessary to determine this. The change in the amount of lubricant present may account for the change in optimum moisture content for maximum dry density.

As the gel hydrates the character of the soil changes because the hydrated gel is cementing soil particles together, thus changing the effective gradation of the mixture and increasing its resistance to the compaction force. This not only affects the optimum moisture content for maximum dry density but it affects the maximum dry density itself. Because the amount of compactive effort applied in the standard density test is held constant, an increase in soil resistance to compaction will cause a decrease in density. The foregoing seems to be an explanation for the dependence of dry density and optimum moisture content for maximum dry density on aging time.

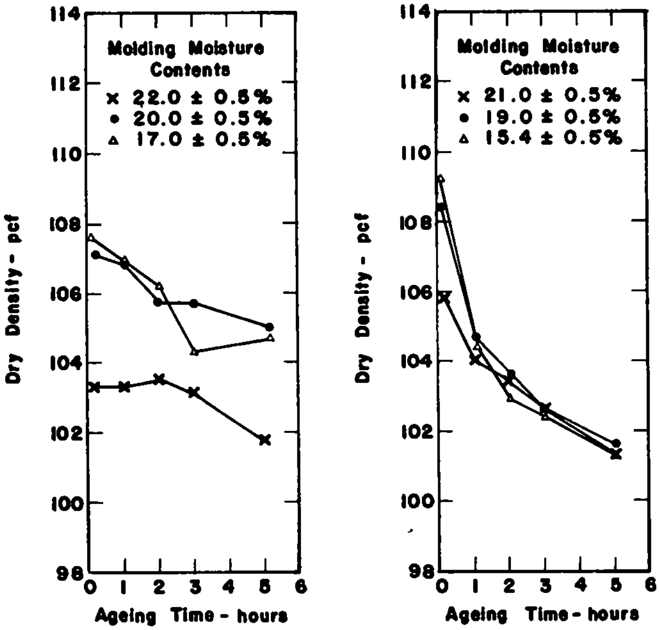
The type of sodium silicate also influenced dry density and the amount of decrease



8.0% Silicate A
6.0% Lime

3.4% Silicate B
6.0% Lime

6.0% Silicate C
6.0% Lime



5.3% Silicate D
6.0% Lime

3.8% Silicate E
6.0% Lime

Figure 1. Influence of molding moisture content on the effect of aging time on dry density at time of molding Kansan till stabilized with 6 percent calcitic hydrated lime and indicated percentages of various sodium silicates.

in dry density with aging time. Highest dry densities were obtained with Silicates B and E. But mixtures with these silicates also had the greatest decrease in dry density with increase in aging time (Fig. 1). Silicates A and C gave the lowest dry densities, and they had a smaller decrease in dry density with increase in aging time. The dry densities achieved by Silicate D mixtures were between those of the other two groups, and the decrease in dry density with aging time appeared to be smallest.

The explanation of this possibly is related to the gel-forming ability of the different sodium silicates. In the explanation given earlier for the dependence of dry density on aging time, emphasis was placed on the controlling function of the amount of gel present. A difference in the rate of gel formation by different sodium silicates would produce different rates of increase of dry density with aging time. A difference in the rate of gel formation might also account for the difference in dry densities obtained from the different sodium silicate mixtures.

Strength.—As was true with dry densities, different molding moisture contents change the effect of aging on strength (Fig. 2). The percentages of the different sodium silicates are based on the weight of the sodium silicate in its normal state of hydration. The points are the average of three specimens, molded at as nearly the same time as possible, plotted at the average aging time of the specimens. The molding moisture contents shown are the range of the molding moisture contents of the specimens molded at the five different molding times.

It is difficult to make any general statements about the effect of aging time on strength that would apply to all of the soil-lime-sodium silicate mixtures. Each mixture was affected differently by water content and aging time (Fig. 2).

Mixtures made with Silicate A showed a decrease in strength with an increase in aging time. The mixtures with the highest and lowest molding moisture contents had strengths that were least affected by aging time. At an aging time of 5 hr, the Silicate A mixture with the highest molding moisture content had the highest strength.

Silicate B mixtures showed a more marked decrease of strength with aging time than Silicate A mixtures, but the strengths of the Silicate B mixtures in general were higher than those of the Silicate A mixtures molded at approximately the same moisture content. Silicate B mixtures could not be molded at moisture contents above 19.5 percent because the material solidified in the mixer.

Based on the results obtained with Silicate A and Silicate B, it was decided to evaluate the other sodium silicate mixtures at only the three highest moisture contents.

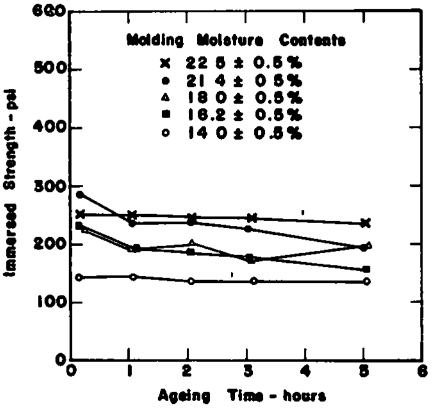
Silicate C mixtures also decreased in strength as aging time increased. Again the mixture with the highest molding moisture content showed the least effect of aging time on strength and had the highest strength with an aging time of 5 hr.

The molding moisture content had a strange effect on the strength of the Silicate D mixtures. At the lowest molding moisture content the Silicate D mixture behaved the same as the aforementioned silicate mixtures, but at the next higher molding moisture content the strength of the mixture increased between aging times of 0 and 1 hr and then decreased until at an aging time of 5 hr the strength was approximately the same as at an aging time of 0.25 hr. At the highest molding moisture content the mixture actually showed a gain in strength with an increase in aging time. However, the highest strength at an aging time of 5 hr was with the Silicate D mixture molded at the middle moisture content.

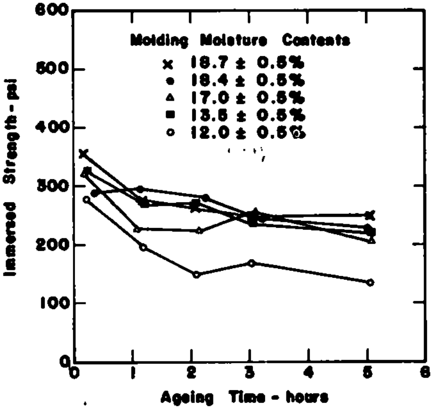
Silicate E mixture molded at the highest molding moisture content displayed the same characteristic as the Silicate D mixture molded at the middle molding moisture content. The other two molding moisture contents caused the strength of the mixture to decrease as the aging time increased. It is noteworthy that, at an aging time of 5 hr, the strength appears to be independent of molding moisture content.

The optimum moisture content for maximum strength changes with aging time and is not the same as the optimum moisture content for maximum dry density (Fig. 1 and 2). Strength is not correlated with dry density, which means that it is not possible to achieve a desired strength in the field by compacting the treated soil to a specified dry density. Nor is it possible to check field strengths by checking dry densities.

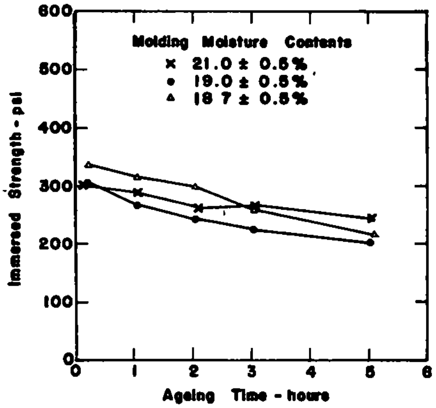
An explanation of the decrease in strength with increase in aging time may be given along the same line as the explanation for the decrease in dry density. If the amount



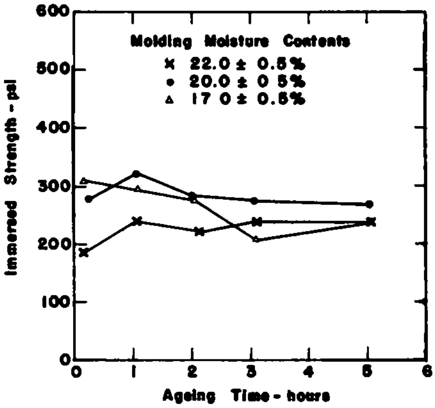
8.0% Silicate A
6.0% Lime



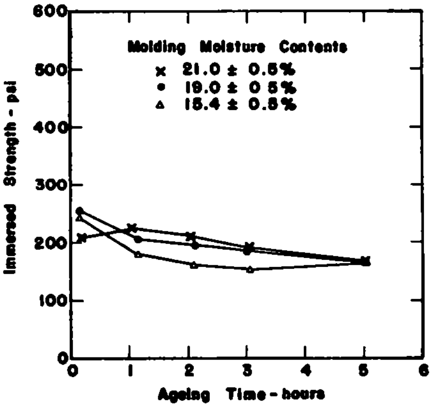
3.4% Silicate B
6.0% Lime



6.0 % Silicate C
6.0 % Lime



5.3% Silicate D
6.0% Lime



3.8% Silicate E
6.0% Lime

Figure 2. Influence of molding moisture content on the effect of aging time on 7-day cured, 1-day immersed unconfined compressive strength; Kansan till stabilized with 6 percent calcitic hydrated lime and indicated percentages of various sodium silicates.

of gel formed by the reaction between the lime and the sodium silicate increases with time, then as the aging time increases the amount of gel present at the time of molding increases. In the absence of any information to the contrary, it is assumed that the gel-forming ability of the mixture is unaffected by the compaction. The structure of this gel can be broken by compaction and will reform, but the reformed gel will not form as strong a cementing agent (19). The amount of gel present at a given time depends on the amount of water in the mixture and on the rate of gel formation (19). While gel formation rates of lime and sodium silicate mixtures have been studied (19), no investigation has been made to determine the rate of this gel formation in the soil-lime-sodium silicate-water mixture. From the results of this investigation it has not been possible to determine the exact effect of the Na_2O:SiO ratio and the degree of hydration of the sodium silicate on the strength of the lime-sodium silicate stabilized soil, but it is probable that these properties affect the rate of gel formation. This is only a possible explanation for the observed decrease of strength with increase of aging time, but it does account for some of the results of this investigation.

Selection of Sodium Silicate to Be Further Evaluated.—To assist in determining the sodium silicate whose strength was least affected by the aging time, the following criterion was adopted. The lime-sodium silicate stabilized soil should have a strength of more than 225 psi for the specimens molded after an aging time of 5 hr. This criterion recognizes that a stabilized soil must be frost resistant. Previous work had shown that materials with a 7-day cured, 1-day immersed unconfined compressive strength between 200-250 psi probably will satisfactorily resist freeze-thaw action (4). Further, the criterion recognizes that field compaction may not be completed before 5 hr after the water has been added to the lime-sodium silicate treated soil. Therefore any mixture that does not have the minimum strength at an aging time of 5 hr should be eliminated from further consideration.

TABLE 5

LIME-SODIUM SILICATE MIXTURES FOR EACH SODIUM SILICATE THAT BEST SATISFY CRITERION FOR FURTHER EVALUATION

Sodium Silicate Content		Molding Moisture Content,[a]	Strength		Amount of Decrease,[d]
Type[e]	%	%	At Approx. 0.25 Hr,[b] psi	At Approx. 5 Hr,[c] psi	%
A	8.0	22.5 ± 0.5	251	231	7.99
B	3.4	18.4 ± 0.5	289	225	22.15
C	6.0	21.0 ± 0.5	301	240	20.25
D	5.3	20.0 ± 0.5	277	267	3.61
E	3.8	21.0 ± 0.5	209	167	20.10

[a] Range of molding moisture contents of specimens molded at five different molding times.

[b] 7-day cured, 1-day immersed unconfined compressive strength with approximately 0.25-hr aging time.

[c] 7-day cured, 1-day immersed unconfined compressive strength with approximately 5 hr aging time.

[d] $\frac{(\text{str at } 0.25 \text{ hr}) - (\text{str at } 5 \text{ hr}) \times 100}{(\text{str at } 0.25 \text{ hr})}$

[e] Legend for sodium silicate type:
A - sodium metasilicate enneahydrate,
B - sodium metasilicate anhydrous,
C - sodium metasilicate pentahydrate,
D - sodium sesquisilicate pentahydrate, and
E - sodium orthosilicate.

Table 5 gives the lime-sodium silicate mixture with each sodium silicate that either best satisfies the foregoing criterion or comes closest to satisfying it. From this table it is seen that Silicate D mixture not only satisfies the criterion but also has the least over-all decrease in strength between aging times of approximately 0.25 hr and 5 hr. Therefore Silicate D was chosen as the admixture for the remaining phases of the investigation.

Because the strengths of Silicate D mixtures showed such a slight dependence on aging time, it was decided to allow the mixtures used in the remainder of the investigation to age 2 hr. This time of 2 hr was chosen because the elapsed time between mixing and compaction in the field, using a multi-pass mix-in-place procedure to mix the stabilized soil, is from 2 to 3 hr (21).

Determination of Optimum Combination of Lime and Silicate D

The purpose of this study was to determine the combination of lime and Silicate D that produced the best results in stabilizing Kansan till, and at the same time to find the optimum molding moisture content for maximum strength of this combination.

Sufficient soil for three specimens was mixed with the desired amount of water to obtain a moisture content on the dry side of optimum moisture content for maximum dry density. The mixture was allowed to age 2 hr and then specimens were molded, measured and weighed (4). Two samples of the mixture were taken for moisture content determination at the time of molding. The procedure was repeated three times with the moisture content of the mixture increased approximately 2 percent for each repetition.

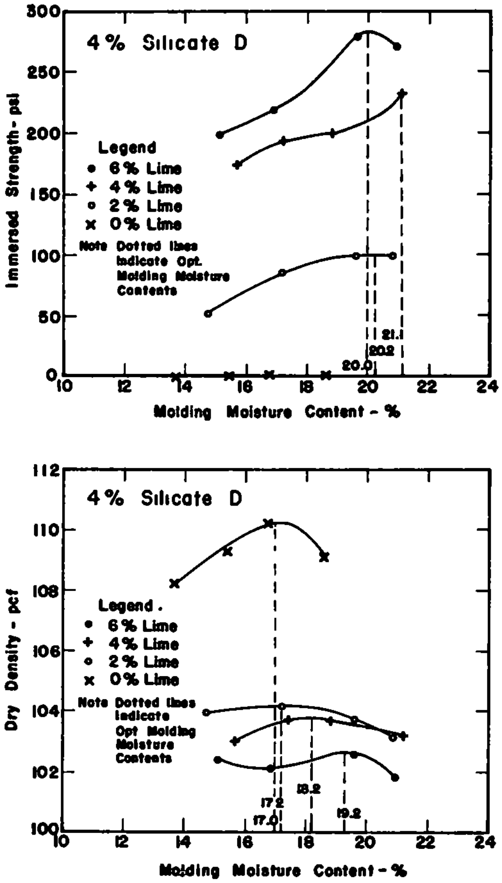


Figure 3. Typical moisture-density and moisture-strength curves for calcitic hydrated lime and Silicate D soil mixtures studied. (The molding moisture contents indicated for each curve are the selected optimums for that combination of lime and Silicate D.)

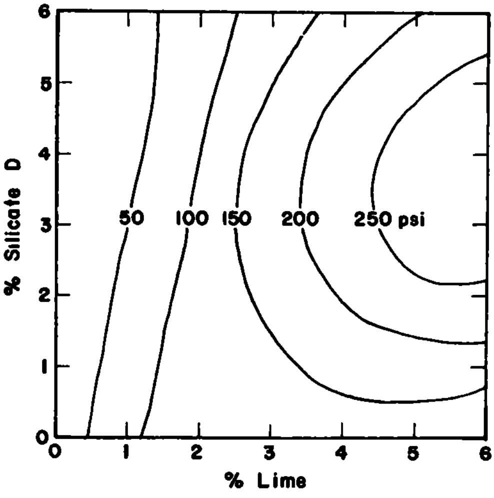


Figure 4. Iso-strength contour chart showing variation of 7-day cured, 1-day immersed unconfined compressive strength of Kansan till stabilized with calcitic lime and Silicate D.

Specimens were molded with 0, 2, 4, and 6 percent calcitic hydrated lime plus 0, 2, 4, and 6 percent Silicate D. The specimens molded with varying moisture content were cured for 7 days, immersed in distilled water for 24 hr, and then tested for unconfined compressive strength.

Average values of dry density (at time of molding) and of immersed strength were calculated for each moisture content. From these data comparisons were made of the optimum moisture requirements for maximum dry density and maximum strength obtained from curves such as those for the 4 percent Silicate D plus 0, 2, 4, and 6 percent calcitic hydrated lime-treated Kansan till mixtures (Fig. 3).

TABLE 6

SUMMARY OF TEST RESULTS FOR DETERMINATION OF OPTIMUM COMBINATION OF CALCITIC HYDRATED LIME AND SILICATE D

				Optimum Molding Moisture Content, for	
Lime (%)	Silicate D (%)	Maximum Strength[a] (psi)	Maximum Dry Density[b] (psi)	Maximum Strength[a] (%)	Maximum Dry Density[b] (%)
0	0	0	117.3	c	14.0
0	2	0	109.1	c	15.6
0	4	0	110.2	c	17.0
0	6	0	110.7	c	18.2
2	0	128	109.6	18.0	18.0
2	2	120	c	19.3	c
2	4	100	104.1	20.2	17.2
2	6	76	105.0	16.2	16.8
4	0	128	108.2	18.8	18.4
4	2	205	c	19.0	c
4	4	232	103.8	21.1	18.2
4	6	164	104.0	20.3	19.2
6	0	105	106.6	18.6	18.7
6	2	242	c	20.9	c
6	4	285	102.6	20.0	19.2
6	6	221	c	21.5	c

[a] 7-day cured, 1-day immersed unconfined compressive strength.
[b] Dry density at time of molding.
[c] Unable to determine from data.

Table 6 is a summary of the results obtained from these curves. In cases typified by the 4 percent lime and 4 percent Silicate D mixture in Figure 3, where the moisture-strength curve did not pass through a maximum, the highest molding moisture content at which specimens were molded was taken as the optimum moisture content for maximum strength.

To evaluate this data an iso-strength contour chart was used (Fig. 4).

Discussion

Silicate D used alone produced no increase in the immersed strength of the soil (Table 6). The use of amounts of Silicate D greater than 2 percent had no appreciable effect on the maximum dry density. About the only effect Silicate D had was to increase the optimum moisture content for maximum dry density. However, when calcitic hydrated lime was also added to the soil, the effect of Silicate D on maximum dry density remained the same but the effect on optimum moisture content for maximum dry density was unpredictable.

Figure 4 is an iso-strength contour chart for Kansan till stabilized with calcitic hydrated lime and Silicate D. This chart, based on the maximum strengths obtained from each of the mixtures tested, indicates that the optimum mix design for maximum strength of the combinations tested was approximately 6 percent lime and 4 percent Silicate D. For this mixture the optimum molding moisture content for maximum strength is on the wet side of optimum for maximum dry density. That this was not true for all the combinations of lime and Silicate D tested (Table 6) is further indication that strength is not correlated with dry density in lime-sodium silicate stabilized soil.

Effect of Curing Time on Strength

Based on the results obtained in the optimum lime-Silicate D study the following mixtures were selected for an investigation of the effect of length of curing time on one day immersed unconfined compressive strength: 6 percent lime, 1 percent Silicate D; 6 percent lime, 2 percent Silicate D; 6 percent lime, 3 percent Silicate D; and 6 percent lime, 4 percent Silicate D.

Specimens from each of these mixtures were molded after allowing the mix to age for 2 hr, then cured for 3, 7, and 28 days. At the end of each curing period the samples were immersed in distilled water for 24 hr and then tested for unconfined compressive strength (Fig. 5).

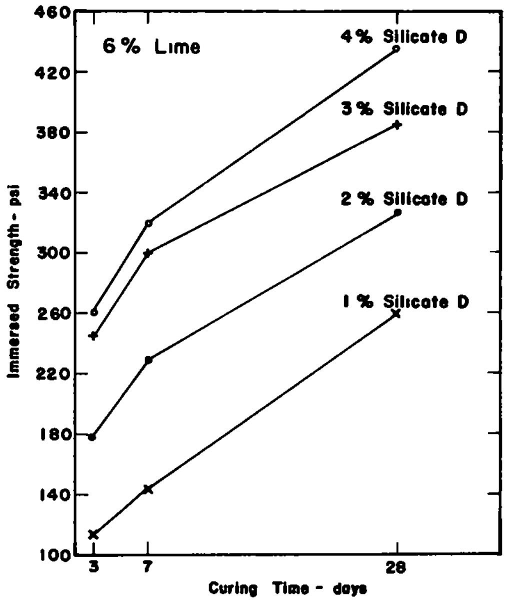


Figure 5. Effect of curing time on 1-day immersed unconfined compressive strength of Kansan till stabilized with 6 percent calcitic hydrated lime and various percentages of Silicate D.

Discussion

The effect of curing time on 1-day immersed unconfined compressive strength of Kansan till stabilized with 6 percent calcitic hydrated lime and varying percentages of Silicate D is shown in Figure 5. All four mixtures continued to gain strength throughout the entire period investigated. The average rate of strength increase for all mixtures was greatest in the period 3 to 7 days. The more Silicate D added, the higher the average rate of strength gain was for this period. For the period 7 to 28 days, the average rate of strength gain for the 1, 2 and 3 percent Silicate D mixtures was approximately the same. The addition of 4 percent Silicate D, however, increased the average rate of strength gain for this period. Further investigation of the reaction products formed by these combinations of soil, lime and Silicate D are needed before an explanation for this can be given.

Resistance to Freeze-Thaw Action

The four mixtures investigated in the curing time study were used for a study of their resistance to freezing. The four mixtures, after curing for 7 days were subjected to 14 cycles of freezing and thawing and then tested for unconfined compressive strength (Table 7).

Discussion

Tentative criteria for durable soil-cement from the data obtained in the modified British freeze-thaw test have been adopted at Iowa State University. The first criterion is the R_f value (the ratio of the unconfined strength of the immersed specimens to the unconfined compressive strength of the freeze-thaw specimen expressed to the nearest 5 percent) which must equal or exceed 80 percent for soil-cement. The second criterion is that the unconfined compressive strength of both the control specimen and the freeze-thaw specimen must equal or exceed 250 psi (4).

There is no reason to dispute the second criterion in the evaluation of the durability of the lime-Silicate D stabilized soil. However, the validity of the first criterion (the R_f value) as applied to lime-Silicate D stabilized soil is questionable.

It has been found that after an initial moist curing of 1-day specimens of lime-sodium silicate stabilized soil immersed in distilled water for 7 days yielded higher unconfined compressive strengths than specimens cured for 7 days in the moist room and immersed for one day before testing (6). This coupled with the probability that the freezing inhibits the gain of strength of the specimen during the freeze-thaw tests, seems to indicate that an R_f value of 80 percent may not be a valid criterion for freeze-thaw resistance of this type of stabilized soil. Further investigation of this criterion is needed before any final evaluation of results of the freeze-thaw tests of lime-sodium silicate stabilized soil can be made.

TABLE 7

RESULTS OF MODIFIED BRITISH FREEZE-THAW TESTS OF KANSAN TILL STABILIZED WITH 6 PERCENT CALCITIC HYDRATED LIME AND DIFFERENT PERCENTAGES OF SILICATE D

Silicate D Content (%)	P_c[a] (psi)	P_f[b] (psi)	R_f[c] (%)
1	195	6	5
2	281	116	40
3	307	179	60
4	429	218	50

[a] Unconfined compressive strength of immersed specimen.
[b] Unconfined compressive strength of freeze-thaw specimen.
[c] Index of the resistance to the effect of freezing.

None of the mixtures tested were able to pass the criterion of an unconfined compressive strength equal to or in excess of 250 psi after 14 freeze-thaw cycles (Table 7). However, the addition of Silicate D increased the strength of the specimens at the end of 14 cycles of freezing and thawing until the combination of 6 percent calcitic hydrated lime and 4 percent Silicate D came very close to meeting the criterion. The iso-strength contour chart (Fig. 4) indicates that a combination of 7 percent calcitic hydrated lime and 4 percent Silicate D would produce a stabilized soil with greater strength than the combination of 6 percent lime and 4 percent Silicate D. It is probable then that Kansan till can be made resistant to freeze-thaw action by treatment with a combination of calcitic hydrated lime and Silicate D. The exact combination of these materials needed is subject to further investigation.

SUMMARY

The investigation reported in this paper is divided into four parts.

The first part reports, discusses, and offers an explanation for the effect of aging time (elapsed time between mixing and molding) on dry density on 7-day cured, 1-day immersed unconfined compressive strength, and on optimum molding moisture content for maximum dry density and maximum strength of Kansan till stabilized with 6 percent calcitic hydrated lime and five different powdered sodium silicates. The amounts of sodium silicates used were adjusted so that the anhydrous portion of each was equal in weight to the anhydrous portion of 6 percent sodium metasilicate pentahydrate.

The second part reports the selection of the optimum combination of lime and sodium sesquisilicate pentahydrate, the sodium silicate whose strength was least affected by aging time. The effect of various combinations of lime and this sodium silicate on the optimum molding moisture content for maximum dry density and maximum strength is also commented on.

The third part reports and discusses the effect of curing time on strength of various combinations of sodium sesquisilicate and calcitic hydrated lime.

The fourth part reports and discusses the resistance of the soil stabilized with 6 percent calcitic lime and various percentages of sodium sesquisilicate to freeze-thaw action.

CONCLUSIONS

1. Kansan till treated with 6 percent calcitic hydrated lime and any one of the five powdered sodium silicates investigated displays the following characteristics:

    a. As aging time increases, dry density decreases at a rate dependent on the type of sodium silicate.
    b. Aging time affects optimum molding moisture content both for maximum dry density and for maximum strength, but the effect is not the same.
    c. As aging time increases, strength decreases. The amount of this decrease depends on the molding moisture content as well as the type of sodium silicate.
    d. Strength is not correlated with dry density.

2. Sodium sesquisilicate pentahydrate (Silicate D) when mixed with 6 percent calcitic hydrated lime and Kansan till produces a stabilized soil whose strength and dry density at time of molding are less affected by aging time than any of the other sodium silicates studied.
3. Of the combinations of lime and Silicate D studied, the optimum mix design for maximum strength was 6 percent lime plus 4 percent Silicate D.
4. For the mixtures of lime and Silicate D studied, the rate of strength increase with curing time is greater in the period 3 to 7 days than in the period 7 to 28 days.
5. In the period 3 to 7 days the rate of strength increase with curing time increases as the Silicate D content increases.
6. Three-day cured, 1-day immersed unconfined compressive strengths of more than 250 psi can be achieved by mixtures of calcitic hydrated lime and Silicate D.
7. Kansan till stabilized with 6 percent calcitic hydrated lime and amounts of between 2 and 4 percent Silicate D probably would perform satisfactorily as a pavement base course in an area with a mild climate.
8. In field construction using lime-sodium silicate stabilized soil, strict controls over moisture content and elapsed time between final mixing and compaction will be necessary to obtain the desired strength.

ACKNOWLEDGMENTS

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A Study of an Old Lime-Stabilized Gravel Base

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The life expectancy of any construction material is of interest to all engineers because of its importance in design procedures and cost analysis of structures. In the stabilizing of unsatisfactory materials in order that they may serve as a satisfactory base for highway construction, the life expectancy is of unusual interest to all engineers because these procedures will permit utilization of unsatisfactory materials at usually a considerable saving in total cost. This report covers the utilization of waste lime in stabilizing an unsatisfactory gravel base material. The lime not only provided a satisfactory stabilized base, but also proved economical to use. Now, 15 years after the base was constructed, cores have been taken and tested to determine the durability and life expectancy of this stabilized gravel. The paper gives both the results of tests on these cores and the original laboratory tests on the stabilized gravel.

●ONE of the earliest sections of lime-stabilized roads in Texas was in Williamson County, approximately 4 mi east of Round Rock, Texas. It extended from the McNutt Creek bridge, 2,168 ft west. The original base consisted of a limestone gravel with a plastic clay binder. The base material had 37 percent passing the number 40 sieve, with a liquid limit of 36 and a plasticity index of 18. Underneath the base material was a clay gravel and plastic black clay overlying the Austin chalk. The topography coupled with the porous clay gravel subgrade permitted water to seep into the base, causing it to become saturated and resulting in breaking up the pavement, making it difficult and expensive to maintain. Therefore, it was decided to use this section for experimental lime stabilization.

Waste lime was available at a very low price. This material had accumulated over a period of years from the mechanical rejections at the hydrator of the lime plant. It contained 80 to 100 percent moisture, and approximately 75 percent calcium hydroxide based on dry weight. Often large "clinkers" of partly burned limestone were found in the lime. They had a "pasty mass" of partly burned lime on the surface, and uncalcined limestone in the center (Fig. 4). These "clinkers" caused weakened areas that often resulted in the specimens breaking during the coring operation, or lowering the compressive strength.

The treatment consisted of scarifying the old gravel base to a depth of 6 in., adding 3 percent of waste lime (based on dry weight), and then mixing with a maintainer and compacting the material in two courses of 3 in. each. During the compacting operations, the material was sprinkled and rolled, first with pneumatic-wheel rollers and later finished with a three-wheel roller. The stabilized base was moist cured for a period of 4 days, then allowed to dry for 3 days prior to sealing with a triple asphalt surface treatment. Later a 2-in. asphalt concrete surface was applied. Prior to scarifying the existing base, an attempt was made to blade off the old asphalt surface. However, this was not completely successful as the scarified base contained considerable quantities of fragments of the old asphalt surfacing (Fig. 3). These particles acted similarly to the uncalcined limestone in that often they made coring difficult, and reduced the unconfined compressive strength.

Tests on the stabilized base material showed that the added lime had not changed the liquid limit in that it remained at 36. However, the plasticity index was reduced to five, and the linear shrinkage from 10 to 3.8 percent. A quantity of the stabilized base material was taken to the laboratory and used to make test specimens. It was placed in the molds with a compactive effort equivalent to twice the standard proctor value, moist cured by being placed in the saturated atmosphere for 7 days. The specimens were then dried at 140 F in a forced draft oven for one day and capillary wetted by being placed in a capillary tank 1/2 in. above the water level on porous stones, and permitted to absorb water through the porous stones. At the age of approximately 20 days they were tested in unconfined compression with the rate of strain of 0.15 in. per min. The average height of the specimens was 5.66 in., and they were 6 in. in diameter. All specimens were capped before testing. Stress-strain curves for the maximum and minimum strength specimens are shown in Figure 6. Also at this time, similar specimens made with the untreated gravel were tested and a typical stress-strain curve for this material is also shown in Figure 6.

To check the behavior of this lime-stabilized base, it was decided that cores should be taken from time to time for strength tests. The first series of cores were obtained when the stabilized base was 2 years old. These cores were taken with a ceramic coring device which used a thin brass slotted cutting barrel and carborundum abrasive. The operation was very slow and tedious, often taking well over 2 hr to cut a single core, and many times the specimens were broken during the coring operation. If any pieces of uncalcined limestone from the waste lime or asphalt surfacing materials were encountered, the cores broke in these areas. If there were any weakened planes in the specimen, it also broke. Thus, the ceramic coring operation made it difficult and time consuming to obtain the specimens. If attempts were made to core the base at any place except where perfect conditions prevail and the strongest base materal was available, it was impossible to obtain a core. While the cores taken with the ceramic apparatus gave an indication of the behavior of the base, it only gave the strongest portion of the base and no indication of any weaker areas. The same apparatus was used to take cores when the stabilized base was 7 yr old,

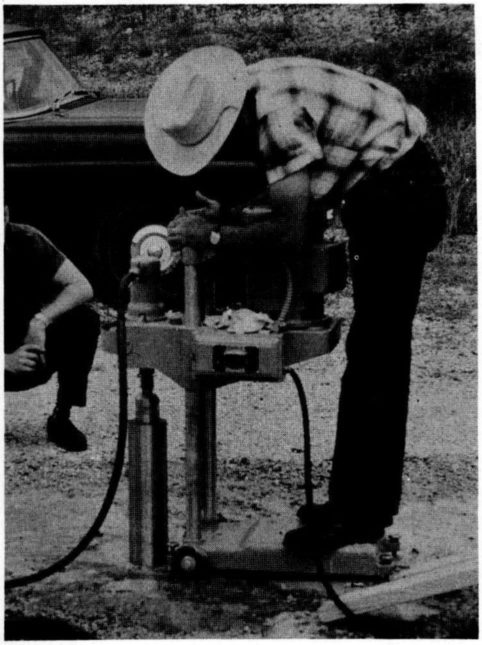

Figure 1.

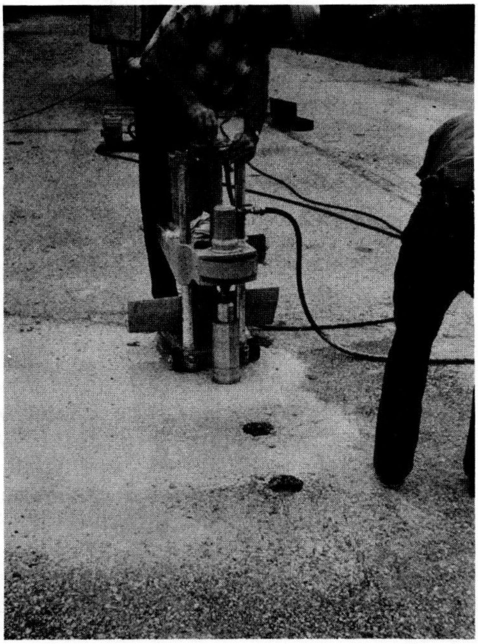

Figure 2.

and again it was possible to core only the strongest material. In Figure 7, there is a stress-strain curve for one of the 7-yr cores. Comparing this with the cores obtained later, it may readily be seen that only maximum conditions prevail.

Specimens taken from the stabilized base when it was 14 yr old were made by a Kor-it drill, using a 4-in. diamond bit and 3-horsepower electric motor. The new equipment proved highly satisfactory and cores were able to be cut through the base in approximately 5 min. Also, cores were obtained in areas of lower strength material so that an indication of the range of strength of the base could be obtained. Figures 1 and 2 show the core drill in operation. The operator soon learned to control both the amount of water and rate of feed on the drill to obtain satisfactory cores in a minimum period of time. Figure 3 shows a series of these cores with the bituminous topping, and also, the broken up bituminous material that had been mixed in the base during the scarifying operation. Figure 4 shows one of the cores cut through a "clinker" from the waste lime.

The ends of the cores were trimmed and they were capped prior to testing as shown in Figure 5. Stress-strain curves for the 14-yr old specimens are shown in Figure 6. The curves for the highest strength and the lowest strength specimen at this age are shown in this curve. Other stress-strain curves were quite similar and fell between the ones shown. At the time the 14-yr old compressive strength cores were prepared, specimens approximately 2 in. in height and 4 in. in diameter were prepared for testing in the cohesiometer.

Figure 7 shows the unconfined strengths of specimens taken from this base material from time to time. The original specimens were tested when they were approximately 20 days old and range in strength from 190 to 368 psi, the average being 276 psi. At the age of 14 yr, the minimum value had increased to 300 psi, and the maximum value to approximately 600 psi with the average being 480 psi. As explained previously, it is believed that only maximum strength specimens were obtained with the ceramic coring apparatus, and the 2- and 7-yr strengths are thus indicated on this curve. Inasmuch as it was impossible to obtain any cores from other than the highest strength areas, there was nothing to indicate the average and minimum strengths at these ages. Therefore, the dash curves are used for the average and minimum strength values. Also on this curve are shown the strengths of the untreated gravel, which was tested in the same manner as the lime-stabilized material.

The results of the cohesiometer tests are shown in Figure 8. The cohesiometer values are given in grams/inch of width corrected to a 3-in. height. To compare

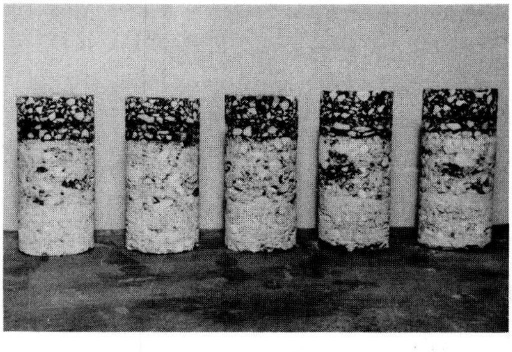

Figure 3.

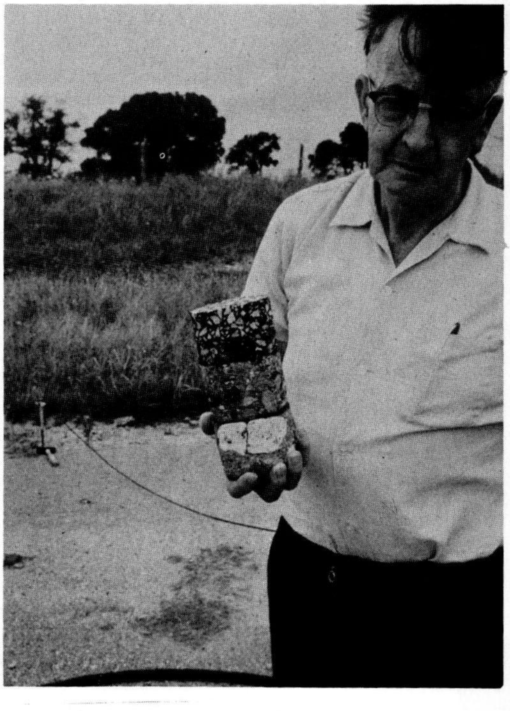

Figure 4.

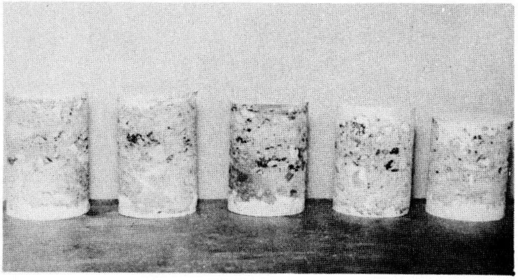

Figure 5.

the cohesiometer strengths, these values are plotted on a chart prepared by F. N. Hveem and R. M. Carmany (1). This chart also shows results of laboratory specimens of lime-treated clay gravel using 5 percent lime (2). After the specimens had been broken in the cohesiometer, they were pushed back together and a rubber band placed around the perimeter. The specimens were then placed back in the moist room and left for slightly more than 4 months, when they were again placed in the cohesiometer and broken in the same area of the original break. The specimens were carefully lined up so that the maximum stress would be at the line of the original break. By some process of autogenetic healing, the specimens developed a considerable strength in the period of recurring. The results of these breaks are also shown in Figure 8. Although no attempt is made to explain the recovery of strength in these specimens, they are reported here as a point of interest.

This section of roadway with the lime-stabilized base has remained in excellent condition during its more than 14 yr of existence with little or no maintenance required, whereas prior to the stabilization, almost continuous maintenance was necessary to keep the road in passable condition. Therefore, it can be concluded that the use of the waste lime was not only economical but highly satisfactory.

In 1947, lime stabilization was used on a section of North Main Street in Taylor, Texas. Here again a clay gravel used as a base material had given trouble and caused a continuous maintenance problem. This base was treated with 3 percent commercial lime in a manner similar to the previously described section. When it was 13 yr old, the cores were taken and unconfined compression tests were made, the results being indicated in Figure 7. It may be seen that the strength of this core was approximately equal to the maximum value obtained from the waste lime material. It was noted that an exceptionally fine core was obtained from North Main Street in Taylor, Texas, and

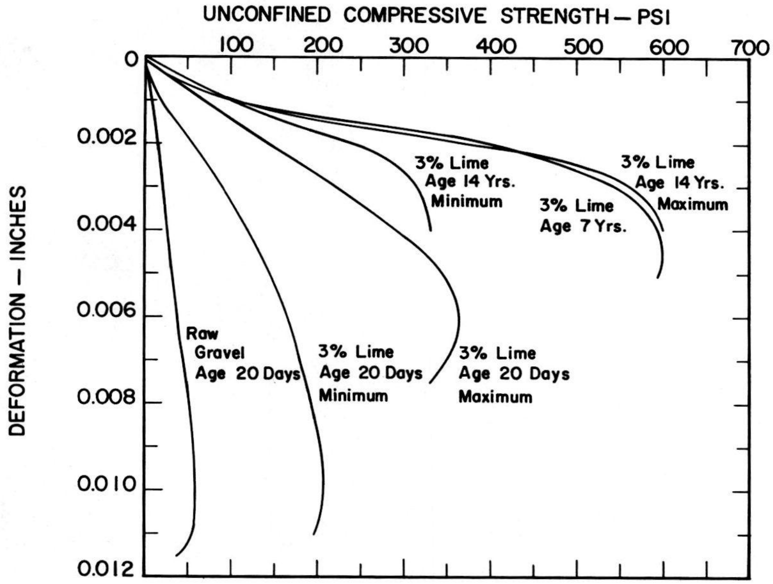

Figure 6. Typical stress— strain curves.

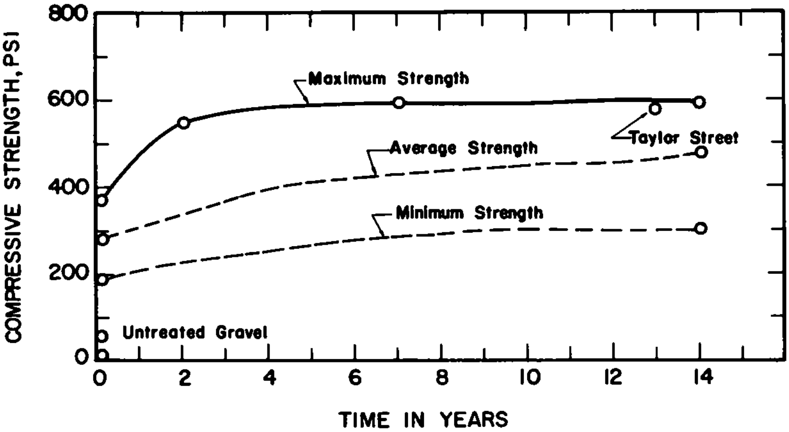

Figure 7. Unconfined compressive strengths.

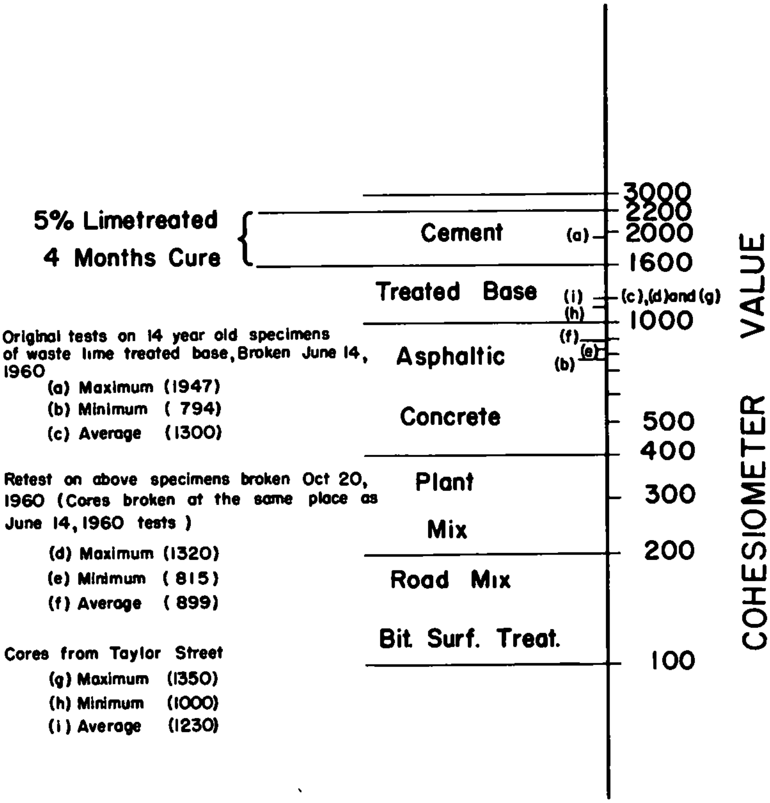

Figure 8.

it would be expected that it would give near the maximum value. Also, from this core, cohesiometer specimens were prepared and tested, similar to those from the previous section. The results are also shown in Figure 8. A rather unusual condition prevailed with these cohesiometer specimens in that after being broken they were also pushed back together and the sections held by rubber bands, and cured in the moist room for approximately 4 months; but when they were again tested in the same area, there was no regain in strength, and the specimens broke with little or no load.

The tests from the specimens from Taylor Street show that the waste lime was apparently almost as good as commercial lime. However, there was some difficulty with the "clinkers" in the waste lime which was particularly troublesome when these "clinkers" occurred within the cores cut from the base.

The success of lime stabilization on these projects as well as others throughout the state has resulted in the Texas Highway Department using more and more lime as a stabilizing agent in clay gravels and clay soils. During 1959, more than 120,000 tons of lime were used in stabilization work in the state, and the 1960 figures will exceed 160,000 tons, although the total number of contracts will probably be in the order of 30 percent less than the 1959 contracts.

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Lime-Soil Mixtures

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The over-all objective of the project described is to investigate various admixtures for chemical stabilization of soils; however, this report is limited to a summary of the published knowledge of lime-soil mixtures. It was prepared from current literature and was written to aid the highway engineer in developing a more thorough understanding of lime-soil stabilization. Other chemicals will be investigated and reported at a later date.

First, the binder material itself, lime, is discussed. The various types of limes and their chemical properties are covered rather thoroughly. Additional information is given on the methods of manufacturing lime, on the availability of different limes, and on the specifications for the control of lime to be used for stabilization purposes. Then the relatively limited data on the basic actions of lime and soil (ion exchange, cementing action and carbonation) are summarized. Other sections cover the physical changes (plasticity, density, strength, etc.) produced by the actions of limes and soils in lime-soil mixtures and lime-pozzolan-soil mixtures.

One section covers current construction practices, another section deals with the field performance of lime-soil stabilized roads. The reviewed literature continually indicated that there are certain limitations to stabilizing soils with limes. These limitations are also discussed. Finally, a number of research projects are suggested that should alleviate some of the existing lack of knowledge of lime-soil mixtures.

An annotated bibliography on lime-soil mixtures, containing all available published literature to July 1960, is included.

●SINCE the beginning of modern road construction, highway engineers have strived continuously to produce better pavements at lower costs. Although various methods have been tried, one successful method has been the stabilization of locally available soils with a binder material. Increased interest has been shown in lime lately as one of these binders. It is relatively inexpensive and can often be used to produce both a chemical change and a cementing action that will improve the soils.

Lime-soil mixtures are soils that have had their physical characteristics changed and/or the soil grains cemented together by action of lime with the aid of water. They include fine-grained soils, granular soils and even existing "gravel roads" that contain appreciable amounts of suitable clays. Lime-soil mixtures also include those soils, especially granular, which can be stabilized with lime only when another material, pozzolan, has been added so that the cementing action may take place. Lime can be used with a wide variety of soils ranging from plastic clays to pit-run gravel and has been used in the improvement of weak, undesirable subgrades as well as in the construction of subbases and bases for highway and airfield pavements.

Although there has been a growing interest in the use of lime for stabilizing soils in recent years, its use is not a new development. It is one of the oldest man-developed construction materials. Various groups of people had used it even before the Romans used it more than 2,000 yr ago in construction of subbases for their elaborate road system. Not only did the Romans develop a technology of using lime, but they even used a pozzolanic material, pozzolana, to improve the cementing action.

The history of modern road construction records little use of lime until after World War II. Primarily, this was due to misleading data and to the resulting wrong conclusions that had been drawn early in the modern construction period. McDowell reports: "Prior to 1945, field experiments were conducted in a number of states (including the midwest) without consideration of control of mixing, compacting and curing. Most of these jobs were used as open surface roads and resulting performances were disappointing. One such job in Texas probably delayed development of lime stabilization in that state for ten years." (59-8). However, as research data on lime-soil mixtures were collected and as lime-soil roads were constructed under more rigid specifications the performance of this type of road material has improved. He continues, "These same materials that failed on one job in this state (Texas), when used on another nearby job, have given splendid performance under medium heavy traffic for a period of 14 years to date." After a late beginning, lime-soil stabilization is slowly spreading in the United States and each year is being used in ever greater quantities by more states.

Considerable literature has been published on the stabilization of soils with lime. Articles have appeared in numerous technical publications in foreign languages as well as in English. These publications are listed subsequently in an annotated bibliography. Not only would it be an arduous task to read all of these articles, but also to some extent, it would be wasted effort as many articles repeat similar data and information. To conserve effort, the following sections contain a digest of these articles. They were written for the engineer who knows little or nothing about lime-soil mixtures and for the engineer who wishes to extend his knowledge of the subject.

This publication is not a treatise on lime-soil mixtures, as few exact numerical data are given. Many references are listed, though, throughout the work. When more detailed data are desired on a particular subject the reader can quickly determine which articles are most pertinent and can read them for details.

LIMES

In a restricted scientific sense, "lime" is the oxide of calcium, CaO, but rarely is the word used in such a limited way. It is generally used as an all-inclusive term, commonly referring not only to calcium oxide but also to its many derivatives. In order to identify specifically the various chemical forms of calcium oxide other terms are used. For example, the hydrate of calcium oxide is known as slaked lime. Because a large number of chemical derivatives of calcium oxide exist, cognizance should be made of the different terms and their meaning.

Types and Properties

Lime is commercially produced by "lime-burning" or calcining crushed limestone. Hot gases supplied by burning gas, coal or oil are passed over the crushed limestone to reduce the calcium carbonate in the stone to the oxide of calcium.

$$21 \text{ kcal.} + CaCO_3 \rightleftharpoons CaO + CO_2$$

The calcium oxide (CaO) produced is known as calcia or more commonly as high-calcium lime. Sometimes dolomite or dolomitic limestone, a carbonaceous rock similar to limestone but containing some magnesium carbonate ($CaCO_3 + MgCO_3$), is lime-burned. In this case the product is called dolomitic lime ($CaO + MgO$). The oxides of calcium and/or magnesium are usually known as quicklime or unslaked lime. In the presence of moisture, quicklime will slake and will also react with carbon dioxide in the air to produce a non-active powdery substance called "air-slaked lime." Therefore, it is necessary to reduce the exposure of quicklime to moisture and air.

The temperature of the lime-burning process depends on the chemical composition of the stone. Calcium carbonates are broken down at temperatures around 900 C and magnesium carbonates at temperatures about 550 C. If the temperatures are too high an overburned product will result. Overburned quicklimes hydrate slowly due to the increased crystallinity of the oxides at high temperatures. Overburning affects the reactivity of dolomitic limes more than high-calcium limes. Dolomitic limes that are overburned are called "dead-burned" dolomite. If the lime-burning temperatures are too low the quicklime will have a core of the original carbonate which renders it undesirable for stabilization purposes.

Quicklime readily reacts with water to produce calcium hydroxide which is commonly known as slaked lime or hydrated lime.

$$CaO + H_2O \longrightarrow Ca(OH)_2 + 15.3 \text{ kcal.}$$

The hydration of quicklime is generally performed by adding sufficient water to quicklime to satisfy its chemical affinity for water. Dolomitic quicklime does not hydrate as readily as high-calcium quicklime as most of the magnesium oxide (MgO) remains in a free state. The resulting lime, $Ca(OH)_2 + MgO$, is termed normal hydrated or monohydrated dolomitic lime. In recent years the hydration process of dolomitic quicklime has been improved to yield a more highly hydrated lime. (See Table 1 for a clarification of these terms.) On the basis of physical tests most high-calcium limes and monohydrated limes are classified as Type N or "normal hydrated" limes. The dihydrate lime is usually referred to as a Type S or "special hydrated" lime.

Waste lime has also been used to stabilize soils. This type of lime is usually a by-product of different manufacturing processes. One type of waste lime is collected from the draft used in the calcining process for the production of lime. This type of waste lime is usually stored in the open and may be partially hydrated and reacted with carbon dioxide from the air. Another type of waste lime is obtained as a by-product when acetylene gas is produced from calcium carbide. It is completely hydrated and may or may not be reacted with carbon dioxide. Usually, waste limes are relatively cheap but non-uniform in quality.

Commercially produced limes are not chemically pure and have slightly different properties than theoretically pure calcium and magnesium oxides and their hydrates. The properties of these limes are given in Tables 2 and 3.

TABLE 1

DEFINITIONS OF TYPES OF LIMES

Lime Produced From Limestone

CaO—calcia or high-calcium quicklime
$Ca(OH)_2$—hydrated high-calcium lime

Lime Produced From Dolomite or Dolomitic Limestone

$CaO + MgO$—dolomitic quicklime
$Ca(OH)_2 + MgO$—normal hydrated or monohydrated dolomitic lime
$Ca(OH)_2 + Mg(OH)_2$—pressure hydrated or dihydrated dolomitic lime

TABLE 2

PROPERTIES OF THEORECTICALLY PURE LIMES

	Quicklime		Hydrated Lime	
Chemical Name	Calcia or Calcium Oxide	Magnesia or Magnesium Oxide	Calcium Hydroxide	Magnesium Hydroxide
Chemical formula	CaO	MgO	$Ca(OH)_2$	$Mg(OH)_2$
Crystalline form	Cubic	Cubic	Hexagonal	Hexagonal
Melting point	2570 C	2800 C	-	-
Decomposition point	-	-	580 C	345 C
Boiling point	2850 C	3600 C	-	-
Heat of solution at 18 C	+18.33 kg-cal	-	+2.79 kg-cal	-0.0 kg-cal
Molecular weight	56.09	40.32	74.10	58.34
Specific gravity	3.40	3.65	2.34	2.4

TABLE 3

PROPERTIES OF COMMERCIAL LIMES

(a) Quicklime

Chemical Composition	High Calcium, %	Dolomitic, %
CaO	92.25 - 98.00	55.50 - 57.50
MgO	0.30 - 2.50	37.60 - 40.80
CO_2	0.40 - 1.50	0.40 - 1.50
SiO_2	0.20 - 1.50	0.10 - 1.50
Fe_2O_3	0.10 - 0.40	0.05 - 0.40
Al_2O_3	0.10 - 0.50	0.05 - 0.50
H_2O	0.10 - 0.90	0.10 - 0.90
Specific gravity	3.2 - 3.4	3.2 - 3.4
Specific heat at 100 F, Btu per lb	0.19	0.21
Bulk density (pebble lime), pcf	55 - 60	55 - 60

(b) Hydrates

	High Calcium	Monohydrated Dolomitic	Dihydrated Dolomitic
Principal chemical composition	$Ca(OH)_2$	$Ca(OH)_2 + MgO$	$Ca(OH)_2 + Mg(OH)_2$
Specific gravity	2.3 - 2.4	2.7 - 2.9	2.4 - 2.6
Specific heat at 100 F, Btu per lb	0.29	0.29	0.29
Bulk density, pcf	25 - 35	25 - 35	30 - 40

Specifications

Tentative specifications for limes to be used in soil stabilization have been written by only a few state highway departments. Usually, they are fairly brief and are based primarily on the specifications for masonry purposes as developed by the American Society for Testing Materials and also from local experience. The following ASTM Specifications have been used to specify the physical and chemical composition of limes:

ASTM C 5-26,	"Quicklime for Structural Purposes."
ASTM C 6-49,	"Normal Finishing Hydrated Lime" with Section 4 (Popping and Pitting) and Section 5 (Plasticity) omitted.
ASTM C 207-49,	"Hydrated Lime for Masonry Purposes" with Section 3 (Residue, Popping and Pitting) and Section 4 (Plasticity) omitted.
ASTM C 110-49,	"Physical Testing of Quicklime and Hydrated Lime."
ASTM C 50-27,	"Sampling, Inspection, Packing and Marking of Quicklime and Lime Products."
ASTM C 25-47	"Chemical Analysis of Limestone, Quicklime and Hydrated Lime."

Availability, Size and Method of Shipment

Lime is manufactured throughout most of the United States. However, the lime-producing deposits of limestone are somewhat more abundant in the eastern and central United States. Dolomite, on the other hand, is concentrated mainly in New England and the Middle West and in many instances occurs in the same deposit with limestone.

Commercial quicklime is available in the following standard sizes:

1. Large lump lime—quicklime as it comes from the kiln with a maximum size of 8 in. in diameter.
2. Pebble or crushed lime—sizes of particles range from 2 1/2 in. to dust.
3. Ground, screened or granular lime—sizes of particles range from 1/4 in. to fine particles.
4. Pulverized lime—a product of more intensive grinding in which substantially

all the particles pass the No. 20 sieve and 85 to 95 percent pass the No. 100 sieve.

5. Specially ground lime—quicklime more finely ground than pulverized lime that is obtainable for special application.

The normal grades of hydrated lime will have 95 percent or more of the material passing the No. 200 sieve, and for special purposes hydrated lime may be obtained as fine as 99.5 percent passing the No. 325 sieve.

Hydrated lime is generally shipped in paper bags weighing 50 lb net, but may also be shipped in the bulk. Quicklime is available in 80-lb multiwalled paper bags or in the bulk. Pebble lime, crushed lime and lump lime are generally shipped in the bulk.

Quicklime has an economical advantage over hydrated lime. Not only is the cost of quicklime per ton 15 to 20 percent less than bagged hydrated lime, but also quicklime contains about 25 percent more calcium oxide or magnesium oxide than hydrated lime. These differences, dependent on the source of the lime, result in a 50 to 65 percent lower cost of quicklime on an equivalent basis. However, in some instances the lower cost of quicklime, on an equivalent basis, may be offset by the increased cost of handling the more active quicklime.

ACTION OF LIME IN SOILS

Several types of chemical reactions take place when lime is mixed with a moist soil. Usually, a number of reactions take place at the same time in the lime-soil mixture which make it difficult to separate and analyze each of them. However, some of these reactions have been identified and are understood to a certain extent. The most important of these reactions for stabilization seem to fall into three general categories that are discussed herein. Other reactions may be important to soil stabilization but they have not been identified at this time. Basic investigations of all of these phenomena are continuing and a better understanding of the fundamental actions of lime in soil should be forthcoming.

Ion Exchange and Flocculation

When lime and a moist cohesive soil are mixed together and allowed to cure in a loose condition for a period of time (commonly referred to by many engineers as "rotting", because of the odor produced during the curing time), the soil becomes friable and attains a silty-like condition. This phenomenon is due to one of the following two conditions or possibly to a combination of them. In one, a base-exchange reaction occurs with strong calcium cations (positively charged) of the lime replacing the weaker metallic ions, such as sodium and hydrogen, on the surface of the clay particle. Another process is the crowding of additional calcium cations of the lime onto the surface of the clay. Although ions of other types exist on the clay particle, a preponderance of the desirable calcium cations will be on the particle surface. Both processes materially change the number of electrical charges on the surface of the clay particle. Because the bond between two clay particles is dependent upon the charge and the size of the ions, the preponderance of the divalent calcium ions that have replaced the univalent ions attract the soil particles together. As this reaction takes place the soil becomes more friable and the plasticity is lowered. This chemical reaction between the lime and the soil takes place rapidly when the mixture is in the loose state and is usually completed within a few days after mixing.

The lime-soil reaction that reduces the plasticity and "loosens" the soil has been used by the engineer as an aid in highway construction in two different manners. Highly plastic clays and very water-sensitive soils, such as loess, are difficult to work and compact into a desirable subgrade while wet. Where these wet soil conditions exist, either because of excess rain or naturally wet soil, some corrective measures must be undertaken to improve the workability of the soil. When lime is added to the wet soil, the soil becomes more friable, its plasticity is reduced and it drys out rapidly so that it can be satisfactorily compacted. When the lime-soil mixture is compacted, it is little affected by heavy rains. In fact, on some projects that have encountered prolonged rains, lost time has been made up through the use of lime.

This lime-soil reaction has also been used to aid the common types of binders in the stabilization of highly plastic clays. Portland cement and bituminous materials can be used effectively only with soils that can be easily pulverized and are friable; otherwise, the soil grains cannot be completely coated with the stabilizing material and poor stabilization results. Because soils with liquid limits greater than about 30 and plasticity indexes greater than approximately 12 cannot be pulverized effectively, they are usually unsuitable for stabilizing with bituminous materials or portland cement. However, if lime is first added to these plastic soils, the plasticity is reduced and the soil can be more easily pulverized. Then, the binder will adequately coat the soil grains and effectively stabilize these highly plastic soils (51-2, 52-4, 52-9, 54-3, 57-1, 59-3, 59-5).

Cementing Action

Another important lime-soil reaction produces a cementing action between the soil particles. Very little is understood of the exact reaction that takes place, but apparently the calcium in the lime reacts with certain soil minerals to form new compounds. Usually, aluminous and silicious minerals in the soil react with the lime to produce a gel of calcium silicates and aluminates that tends to cement the soil particles in a manner similar to that produced by the hydration of portland cement.

The minerals in the soil that react with lime to produce a cementing compound are known as pozzolans. The type and amount of pozzolans and thus the amount of reactivity with the lime vary from soil to soil. When desirable quantities of pozzolanic compounds are not present in the soil, pozzolanic materials such as fly ash, many times will help produce the desired reaction with the lime.

The lime-cementing action in soil is a slow reaction which requires considerably more time than would be needed for the hydration of portland cement. The rate of cementation is influenced to a great extent by the amount and type of pozzolans, as well as the type of clay mineral in the soil, and by climatic conditions. In addition, the mixture of soil and lime must be thoroughly compacted; otherwise, the desirable cementation will not take place (48-5, 51-2, 52-4, 52-9, 56-3, 57-1, 58-12, 59-6, 59-8).

Carbonation

A third important lime reaction involves the absorption of carbon dioxide (CO_2) from the air. The carbon dioxide reacts with calcium hydroxide in the lime to form calcium carbonate. In other words, this is a reversal of the lime-producing process. Not only do these carbonates form weak cements but they also deter pozzolanic action and prevent normal strength gains.

Care should be taken to prevent the lime from being carbonated by a reaction with carbon dioxide from the air. Lime must be specially protected while in storage and in shipment prior to its use with the soil. Carbonation is most active in industrial areas where the carbon dioxide content in the air is much greater than in the rural areas (52-4, 52-9, 57-1, 59-8).

PHYSICAL PROPERTIES OF LIME-SOIL MIXTURES

Through its chemical reactions, lime affects certain physical properties of soils. There is considerable lack of factual data and some disagreement exists between the available data. However, it is generally agreed that lime influences the following physical characteristics of a soil: grain size distribution, soil plasticity, volume change, field moisture equivalent, soil pressure, compaction and optimum moisture content, strength and durability.

In many instances it is difficult to evaluate the influence of lime on soil due to the lack of desirable criteria or standards for comparisons. These criteria can only be developed after a thorough knowledge of the characteristics of lime-stabilized soils is obtained. Continuing investigations in the field of lime-soil stabilization should greatly increase present day knowledge and provide the standards for economical design and construction.

Grain Size Distribution

One of the first physical changes which takes place in a fine-grained soil when lime is added is an agglomeration or flocculation of the clay particles. This produces a coarser and more friable soil. As an example (59-7), a clay with 10 percent lime changed so much in gradation after 14 days of initial curing that it was classified as a sandy loam, and after 240 days it was classified as a sand. In another instance (59-3), the amount of loess passing the No. 200 sieve was reduced from 88 percent to 58 percent in only 7 days with an addition of 5 percent lime. This phenomenon is also reflected by a change in soil classification; for example, an A-7-6 (16) soil was changed with six percent lime in 14 days to an A-4 (6) soil (59-7).

The particles that are changed the most in size by lime are the fine clay particles as they tend to react with lime more than the larger individual sandy-like materials. In most instances, fine clay particles have aggregated with lime into particles larger than those passing the No. 200 sieve and have materially increased the amount of material retained on the No. 200 sieve. There are some indications that this aggregate is not entirely water-resistant. After a period of soaking in water, a small portion of the aggregated particles tends to break down. The remaining particles are quite hydrophobic, though, and will retain their aggregation after long periods of soaking.

The amount of agglomeration is influenced by a number of different factors, of which the most important one is the type of soil. Plastic soils tend to agglomerate more than silty or sandy soils. In addition, the agglomeration appears to be influenced by the amount of lime, with more agglomeration occuring as more lime is added to the soil. Also, increased intial curing time and possibly the type of lime are factors affecting the amount of agglomeration. Quickline may be more effective than hydrated lime (58-11, 59-2, 59-7).

Soil Plasticity

Another noticeable phenomenon that is mentioned in almost every article on lime stabilization is the ability of the lime to change the plasticity of the soil. Both the plastic limit and the liquid limit of the soil are affected. The plastic limit of a soil is increased as additional amounts of lime are added to the soil. This bears out early findings in the ceramic field that hydrated lime increases the amount of water needed to develop plasticity in clay pastes.

The liquid limit normally decreases with increased amounts of lime. Although this decrease is usual, it does not occur in all soils. In some soils a distinct increase in the liquid limit is produced by adding lime. In general, but not always, the liquid limit is reduced in the more plastic clays and is increased in the less plastic soils.

Regardless of whether the liquid limit decreases or increases, the increase in plastic limit is such that the plasticity index is usually reduced with the addition of small amounts of lime. The plasticity index of many highly plastic clays may be reduced 50 to 80 percent with small quantities of lime. As an outstanding example of this (59-7), a clay with a liquid limit of 51 and a plasticity index of 30 became non-plastic in two days with only 6 percent lime. It should be noted here that in a few rare instances the plasticity indexes of slightly plastic soils have been increased a little by the addition of lime.

The amount of reduction in the plasticity index obtained in a soil as lime is added depends on many factors, but primarily on the type of soil. The plasticity indexes of highly plastic clays are usually reduced a considerable amount with only a small amount of lime. However, the less plastic soils are only slightly affected by the addition of lime; for example, the plasticity index of a loess may be reduced from five to only two with 5 percent lime. The amount of lime also influences the extent of reduction in the plasticity index. As the amount of lime is increased, the plasticity index of the soil is decreased.

The plasticity index is also influenced by the length of time the lime reacts with the soil and the type of lime. A sizable reduction is usually noticed within the first few hours after lime is added, and within 2 to 3 days almost all of the change in plasticity of the soil will have taken place. Some further reduction in the plasticity index will generally take place when longer reaction times are allowed, but it is small when compared with that which takes place in the first few days. Although little data are available, it appears that the reduction in the plasticity index is influenced some by the type of lime used. Quicklime usually brings about a reduction in plasticity faster than hydrated lime. Waste lime also reduces the plasticity index, but the time needed for reduction is substantially longer than that needed by other types of lime (48-2, 49-3, 51-2, 52-9, 57-1, 59-5, 59-7).

Volume Change

Lime tends to reduce the volume changes that take place in soils. As the lime content in the soil increases, the shrinkage limit increases and the shrinkage ratio decreases. The shrinkage limit and the plasticity index of a soil appear to be related. As the plasticity index tends to drop when the lime content increases, the shrinkage limit increases; and when there is little further change in the plasticity index there is little change in the shrinkage limit. Lime also materially influences the swelling of soils. In one test (58-8) when the specimens were subjected to a load of 1 psi with excess water present, an originally highly plastic soil (PI = 37) did not swell but actually consolidated about 1 1/2 percent.

Additional amounts of lime will produce decreasing volume changes up to an optimum lime content. Little additional reduction in volume change is produced by adding lime in amounts larger than this optimum value. Limes have a greater influence on reducing volume changes in soils that readily change volume with water than in soils that originally have small volume changes (49-3, 55-7, 58-8, 59-7).

Field Moisture Equivalent and Soil Pressure

A small amount of available data indicates that as the lime content increases in a plastic soil the field moisture equivalent increases. One experimenter (52-4) has recorded that the pressure produced by a soil swelling in restrained samples with an excess of water was materially reduced by adding lime. The swell pressure in a clay, predominantly montmorillonite, was reduced from approximately 7 psi to 1 psi with 8 percent lime (52-4, 59-7).

Compaction and Optimum Moisture Content

When compacted with the same effort, a lime-soil mixture has a lower standard AASHO T99 density than the original soil without lime; and as the lime content increases, the density tends to decrease even more. The decrease in unit weight is small and averages about 2.5 percent for most soils. In some soils, though, such as micaceous silt, the decrease in unit weight may be as much as 5 percent when approximately 5 percent lime is used. It is interesting to note that all investigators of this property except one (48-2) report that the unit weight of the mixture decreases as the lime content is increased. This one investigator reported that for two soils, of 11 he investigated, the addition of lime tended to produce an increase in the unit weight (1.2 percent increase with 5 percent lime). As the soils were derived from different rock minerals no explanation was given of this unusual phenomenon.

As lime is added to a soil the optimum moisture content tends to increase. Usually the initial increase in moisture content is rather significant even when small amounts of lime are used. In some instances there may be as much as a 25 percent increase in optimum moisture content (12 to 15 percent water content) with the addition of only 2 to 3 percent lime. Once the large initial increase in optimum moisture content takes place, additional lime produces only a slight increase in the optimum moisture content; that is, there usually is little difference in optimum moisture contents of a soil with 5 percent lime and with 7 percent lime.

In addition to the amount of lime used, compaction of lime-soil mixtures is influenced by the type of lime. Soils with quicklime usually have a slightly higher optimum moisture content than soils compacted with hydrated lime. On the other hand, the type of lime does not seem to influence the unit weight in any significant amount; that is, a soil treated with either quicklime or with hydrated lime in the same amount has about the same density. The change in density of a soil, as lime is added, apparently cannot be correlated with the type of soil, the plasticity index or with the dry density of the raw soil.

At moisture contents near optimum for the lime-soil mixtures the unit weight of a raw soil, obtained by the AASHO T99 compaction test, is usually less than the unit weight of the soil treated with lime obtained with the same compactive effort and moisture content. This indicates that lime-soil mixtures have greater compactability than the raw soils at higher moisture contents. Contrary though, lime-soil mixtures are less compactable at lower moisture contents and have a lower unit weight than the raw soil at the same moisture content.

A word of caution should be given here. Although the density of a soil tends to decrease with additional amounts of lime, it should not be assumed that the strength of lime-soil mixtures is lowered. This is not true. For example (52-8), an Illinoian drift soil with no lime had a standard proctor unit weight of about 120 pcf and a compressive strength of approximately 80 psi. The same soil with 5 percent lime had a lower standard proctor unit weight of about 114 pcf and a compressive strength of approximately 140 psi. Lime-soil mixtures are one material to which the general thought "when the density increases the strength also increases" does not always apply. It usually applies, though, if at the same lime content additional compactive effort is used to produce higher densities. When the Illinoian drift (as previously described) with 5 percent lime was compacted with a greater compactive effort to a density of approximately 119 pcf, the specimen had a compressive strength of approximately 280 psi. Lime-soil mixtures then usually have substantially higher strengths when they are compacted to a higher density with a greater compactive effort (48-2, 49-1, 49-3, 52-8, 52-9, 56-3, 59-3, 59-7).

Strength

A number of different types of tests have been used to evaluate the strength of lime-soil mixtures: unconfined compression, California bearing ratio, Hveem stabilometer, extrusion, triaxial and even the Proctor penetration needle. Of these tests, the unconfined compression test is the most popular. The trends in results obtained by these different tests usually have been similar; that is, if the unconfined compression tests indicate an increase in strength, so does the CBR test. However, the percent change in magnitude of the strength values varies considerably with the different test methods. When one strength test indicates a certain percent strength change, another test for the same material and conditions may indicate more or less of a percent change. In some instances the indicated change in strength by a particular test was so out of proportion that wrong conclusions could have been drawn.

Unlike many soil stabilizers, there appears to be no optimum lime content that will produce a maximum strength in a lime-stabilized soil under all conditions. Some data, however, have been presented that indicate the relationship between the strength and the lime content in the soil to be such that an optimum lime content does exist. Such data were usually obtained for only one particular length of curing time. If a different curing time was used a different optimum lime content would be indicated. For instance, after 7 days of moist curing a particular soil may have an optimum lime content of 5 percent; with 28 days of curing the optimum lime content may be 8 percent; and after a year there would probably be no optimum content indicated in the normal range of lime used. Strength tests have not been used primarily to obtain the desirable lime content, but they have been used to determine if a mixture hardens sufficiently and to indicate the influence of various factors on the lime-soil mixtures.

The chief factors affecting the strength of lime-soil mixtures are lime content, type of lime, type of soil, density and the time and type of curing. Most of these factors are interrelated. Except in specific cases, no one of these factors is a great deal more important than another.

Lime Content.—In general, the strength of lime-soil mixtures increases as the lime content in the soil is increased. As previously stated, there may or may not be an optimum lime content, depending primarily on the length of curing (48-5, 49-3, 52-8, 59-7, 59-8).

Types of Lime.—One group of investigators (56-3, 57-3) has indicated that the type of lime influences the strength of lime-soil mixtures. They have presented data to show that dolomitic limes produce higher strengths in lime-plastic soil mixtures than high-calcium limes. For the same soil and conditions these two types of limes produced about the same strength at low lime contents; but at the higher lime contents the dolomitic lime-soil mixtures increased in strength, whereas the high-calcium lime-soil mixtures remained about the same or decreased slightly. Quicklimes were more effective than hydrated limes in equivalent amounts for stabilizing soils. Of the hydrated-dolomitic limes, monohydrated lime (Type N) produced greater strength in soils than the dihydrated-dolomitic lime (Type S).

The foregoing evaluation of the effects of different types of limes on the strength of lime-stabilized soils did not consider the lime content as influenced by the time of curing. Inasmuch as this factor is very important, it may be possible that with optimum periods of curing there would be little difference in the strength of high-calcium lime and dolomitic lime-stabilized soils. In fact, some outstanding engineers believe that in the long run high-calcium lime will produce as high a strength as dolomitic lime and may even be more desirable for stabilizing certain soils (discussion, 56-3).

Since the exact composition of commercial limes varies among deposits, it is usually desirable to evaluate the various locally available types of lime for strengthening a particular soil prior to stabilization. In this manner, it may be determined which type of lime, if any, is best suited for the particular soil. Indications are that in some instances, by using the most desirable type of lime the same strength can be obtained with one-third as much lime as would be needed of another type of lime (56-3, 57-3).

Type of Soils.—The amount of strength increase in a soil that can be produced by adding lime is dependent on the pozzolans in the soil. When desirable pozzolans are available they react readily with the lime to improve the strength of the lime-soil mixture. However, if the soil has a small amount or no pozzolans, little improvement in strength is obtained by adding the lime. No chemical analyses are now available that will indicate the amount and type of pozzolans available in a particular soil that are suitable for reacting with lime. Because the amount of desirable compounds varies from soil to soil, each soil should be evaluated by some physical testing method(s) to determine its suitability for stabilization with lime. This is the only dependable manner now available by which the suitability of soils for stabilization with lime can be determined.

Usually, clays are more reactive with lime than other soils and are generally increased in strength materially when lime is added. In many cases only a small amount of clay is needed in the soil for reaction with lime. The strength of silts, some sands, caliche, sandy clays, plastic pit-run gravels, as well as clays, have been improved by the addition of lime. Usually higher strength, but not necessarily a larger percent increase in strength, is obtained with the larger size materials that have some clay for reaction with the lime. Lime-pit-run gravel mixtures probably have higher strengths than lime-clay mixtures.

Generally, the highly plastic soils are more reactive with lime, whereas soils with low plasticity react little with lime. This is not always true, however, as one investigator (52-8) reported that a sizeable increase in strength of a non-plastic pit-run gravel was obtained by the addition of 10 percent lime.

At least in one instance (55-7) it was reported that lime tended to decrease the strength of a couple of soils, predominantly loams. It is possible that lime has this effect on other types of soil even though it has not been reported in the literature (48-2, 48-5, 52-8, 52-9, 55-7, 57-3).

Density.—Few engineers recognize the importance of compaction in stabilizing soils with lime even though it is a very influential factor. The strength of a lime-soil mixture is increased materially when the mixture is compacted to a higher unit weight by a greater compactive effort. Strength is affected by density when the increased unit weight is produced by additional compactive effort. Strength is not related to the increased density that is produced by varying the lime content in the soil. This important relationship has been discussed to some extent in the section on "Compaction and Optimum Moisture Content," and is emphasized by McDowell (45-8) who says, "...densification is of critical importance." (48-5, 52-8, 56-1).

Time of Curing.—Lime-stabilized soils increase in strength with age in a manner similar to portland cement concrete. Usually, there is a rapid increase in strength of these mixtures at the beginning of the curing period, but as the curing progresses the rate of increase in strength becomes less and less. After considerable curing time the strengths of lime-soil mixtures still appear to be increasing very slightly. Tests (53-3) have indicated that lime-soil mixtures were increasing in strength after 4 yr of controlled laboratory curing. Even though the strength of lime-soil mixtures may increase with age indefinitely in the laboratory, it does not necessarily mean that this relationship exists when the mixtures are cured in the field under normal climatic conditions. In one instance, cores cut from a lime-soil road after 7 yr of curing had approximately the same strength as cores that were obtained when the road was only 2 yr old (53-3).

Lime-soil mixtures do not gain strength at the same rate as portland cement concrete. The gain in strength of lime-stabilized soil is slow and gradual under normal field curing conditions. Many times, 4 to 6 mo of curing are needed in the field for mixtures to obtain a major portion of their strength. In the laboratory the rate of gain in strength of lime-stabilized soils can be greatly increased by using favorable methods of curing, such as high temperatures. (See the following section.) Under favorable laboratory curing conditions strengths can be obtained in 2 wk that would require 3 mo of curing under normal field conditions (52-8, 53-3, 56-1 56-3, 59-8).

Type of Curing.—Various methods have been used to cure lime-soil mixtures. Primarily, the different methods of curing may be divided into two groups: (a) curing at varying moisture conditions or relative humidities, and (b) curing at normal or elevated temperatures. These methods of curing have been used in various manners by different groups. No one method has yet been developed that appears to be outstanding and no method has been adopted as the standard method for curing lime-soil mixtures.

Effect of Temperature. The rate of gain in strength in lime-soil mixtures is directly related to the temperature at which the compacted mixture is cured. When cured at low temperatures, the gain in strength is very slow; when cured at normal temperatures (approximately 70 F), the rate of gain in strength is greater; and when cured at high temperatures (140 F), lime-soil mixtures increase in strength quite rapidly with time. So rapid has been the strength gain in some instances (56-3), that at 140 F the same strength was obtained in 10 days as was obtained in 3 to 4 mo curing at 70 F.

Effect of Moisture. The humidity of the air during the curing of lime-soil mixtures appears to have some effect on the strength of the mixtures, but it is difficult to draw any conclusions regarding its effect. In some instances specimens cured at low humidity have higher strengths than specimens cured at high humidity. In other cases higher strengths were developed in those specimens cured at high humidity. Whatever the effect of moisture might be, it does not appear to influence the gain in strength of lime-soil mixtures as much as variations in curing temperature.

Data (48-2, 48-5) indicate that strengths higher than those produced by either moist curing or high-temperature curing can be obtained by various combinations of curing methods. If the mixtures are originally cured at normal or higher temperatures and then are moist cured for a period of time, they usually have relatively higher strengths. Some engineers believe that even greater strengths can be obtained in lime-soil mixtures by a drying and wetting cyclic curing procedure. No published data are available, though, to substantiate this thought.

In some instances it may not be desirable to cure lime-soil mixtures at high temperatures or in a cyclic manner even though very high strengths might be obtained. If the laboratory strength is greater than the strength of the mixture in the field and if the higher laboratory strength is used in designing a pavement, the pavement will be underdesigned and failure may occur (48-2, 48-5, 52-8, 56-3, 57-3, 58-8, 59-7).

Durability

Although a lime-soil mixture should have high resistance to applied stress, it is perhaps even more important that it have good durability in order to perform satisfactorily in the field. The determination of the durability properties of a lime-soil mixture is a problem because it is difficult to simulate in the laboratory the detrimental action that is produced by weathering in the field. Many different tests have been used for this purpose. Usually, weathering has been simulated by cyclic action of: (a) heating and cooling, (b) wetting and drying, or (c) freezing and thawing. The length of time of the various phases of the cycle, as well as the temperature, the method of soaking, etc., vary from investigator to investigator. So wide are these variations that no standard method of testing the durability of lime-soil mixtures has been universally accepted to date.

Not only is there disagreement as to the type of action best suited to simulate weathering in the laboratory, but also investigators are not in agreement as to the physical properties most affected by cyclic weathering. Some tests evaluate durability in terms of the loss in weight of specimens that is produced by brushing. Other tests use the percent change in compression strength as the basis for evaluation. Of the few non-destructive tests that have been investigated, perhaps the most promising test is one that uses the soniscope to measure the change in velocity of pulse propagation. Not only do some investigators believe that the change in basic properties of the mixtures, as measured by the soniscope, is a good indicator of the change in durability, but they also like the relatively few number of laboratory specimens that are required for the test. Moreover, the method is not limited to use in the laboratory. With slight modification the soniscope can be used to evaluate a change in durability of lime-soil mixtures that are in service in a road.

Generally, the durability of lime-soil mixtures in the field has been satisfactory. This is not consistent, though, with the results that are indicated by certain laboratory tests which repeatedly predict poor performance of lime-soil mixtures. For instance, the results from standard freeze-thaw and wet-dry tests using brushed samples (AASHO Designations: T-135 and T-136), when compared to criteria developed for soil-cement mixtures, indicate that lime-soil mixtures have little resistance to weathering. So rapidly did the lime-soil mixtures deteriorate when subjected to these durability tests, that one investigator (51-3) stated: "hydrated lime... (was) among the most commonly available materials found to be unsuitable (as a) soil stabilizing material." Many reports, though, indicate that numerous lime-soil projects have performed exceedingly well in the road when protected with nothing more than a bituminous seal coat (58-9). One investigator (53-3) reports that after 7 1/2 yr of service, road sections constructed with commercial lime were in "excellent" condition. It appears then, that lime-soil mixtures when normally protected from weathering by a bituminous-wearing surface are more durable than indicated by most laboratory durability tests.

Because most of the durability tests do not accurately indicate the serviceability of lime-soil mixtures under actual road use, a more accurate test is strongly needed. Investigations are now being made that will lead, it is hoped, to a universally acceptable method of testing lime-soil mixtures that will accurately predict field performance.

Even though the existing methods of evaluating durability are not the most desirable, they have strongly indicated that the resistance of lime-soil mixtures to weathering is influenced by various factors such as: amount and type of lime, age, compaction and type of soil. These are probably not all of the factors that affect the weathering resistance of this material. Additional information is definitely needed for a thorough understanding of the durability characteristics of lime-soil mixtures.

Amount of Lime.—Lime-soil mixtures with high lime contents have considerably more resistance to deterioration than mixtures with small amounts of lime. Durability, then, tends to increase as the lime content is increased. Lime content is so critical that some engineers recommend quantities of at least 5 percent even though smaller amounts might produce desirable strengths in the soil.

Aging.—The durability of lime-soil mixtures is usually related to the length of time the mixture is cured before it is subjected to detrimental weathering. Durability increases with longer curing times. It is conceivable that a mix placed late in the year could develop satisfactory strength in the short curing period before cold weather but would have such poor durability that it would fail during the first winter. Long periods of warm weather curing are most desirable for improving the durability of lime-soil mixtures.

Compaction.—At a given lime content, higher density that is produced by increased compactive effort results in greater resistance to weathering.

Types of Lime.—Only very limited data have been reported on the influence of the types of lime on the durability of lime-soil mixtures. One investigator (57-3) indicates that, at the same lime content, specimens made with dolomitic quicklime possibly have more resistance to freeze-thaw and wet-dry tests than those made with dolomitic hydrated lime. The data are so limited, though, that the relative influences of other types of limes are not known and definite conclusions cannot be drawn.

Types of Soils.—Some available data indicate that there may be a relationship between the type of soil and the resistance of the lime-soil mixture to weathering. A friable loess may possibly be more durable than glacial drift, and glacial drift may possibly be more durable than a non-plastic river terrace gravel. These are only generalities and more data are needed before a definite statement can be made (51-3, 52-8, 53-3, 57-3, 58-12, 59-8).

PHYSICAL PROPERTIES OF LIME-POZZOLAN-SOIL MIXTURES

When lime is added to many soils the mixture will not develop the desired strength even after considerable curing time. These non-reactive soils, however, can usually be stabilized satisfactorily with lime when a pozzolanic material is also added to the soil. The amount of cementation produced in a mixture by lime is related to the reactivity and amount of the pozzolanic material existing in the soil. When lime is added to soils containing little reactive pozzolanic material, not only may the rate of gain in strength be very slow, but also the increase in strength may be slight. In addition, the resistance to weathering may be virtually unchanged. These effects can usually be minimized, if not eliminated, by providing a material that will react with the lime to produce a cementing action. When a pozzolanic material such as fly ash is added to the soil an abundance of the pozzolans will be present, cementation with the lime will be assured, and the reaction will take place rapidly.

Many types of natural pozzolanic materials are available for commercial use. They are primarily siliceous materials containing certain chemical compounds that will react with lime and water at ordinary temperatures to form the necessary cementing compounds. (By themselves, natural pozzolanic materials are not cementitious.) Although a natural pozzolanic material of volcanic origin (pozzolana) has been used for more than 2,000 yr, the most common pozzolan and by far the most widely used at present in the United States is fly ash. This pozzolan is collected primarily from the flue gases produced in boilers burning powdered or ground coal. It is quite abundant in some areas and many times is considered to be a "waste product." The reactivity of fly ash varies from source to source. Tests, though, are available that will reasonably indicate the reactivity of fly ash with lime (58-2).

Materials other than fly ash are known to produce a similar pozzolanic reaction when mixed with lime, but little use has been made of them. In one instance dust from an expanded shale (58-3) acted as a satisfactory pozzolanic material in stabilizing a gravel. Finely ground raw shale, however, did not react pozzolanically. Apparently shale has to be "burned and finely ground" to react satisfactorily with lime.

The important question concerning the use of pozzolan is, "When is it necessary to use a pozzolan?" It appears that, to date, no known test on the natural soil can accurately indicate the compatibility of the soil with lime. Neither base exchange capacity, the pH of the soil, nor the plasticity characteristics seem to be satisfactory indicators (57-2). The only positive method that will indicate if lime by itself will improve a soil is to test the lime-soil mixture. If desired results are obtained, there is no need for pozzolanic materials to be added. However, if desired results are not obtained, a pozzolanic material and lime should be added to the soil and the mixture tested to determine if the pozzolanic material will aid the cementing action.

Many soils have an optimum ratio of lime to pozzolan, whereas other soils develop approximately the same strength over a wide range of lime to pozzolan ratios. In general, fly ash used in ratio of 1:9 to 2:8 by weight of lime to fly ash has been found to be satisfactory. For a specific lime-fly ash ratio, strengths tend to increase as the amount of lime-fly ash is increased. Greater strengths can be obtained, though, by correctly adjusting the lime-fly ash ratio than can be obtained by increasing the amount of lime-fly ash (56-2). The rate of reaction of the lime with the pozzolan is not an indication of the strength that will be developed when the mixture is used with a soil.

Lime-pozzolans together tend to affect the physical properties of soil in a manner similar to that of lime by itself. (See section on "Physical Properties of Lime-Soil Mixtures.") The exact magnitude of the change in strength and the rate of change in strength vary widely, and no direct relationship exists. In general, both lime-pozzolans and straight limes will reduce the magnitude of volume change of a soil, will lower the plasticity, will increase the strength, and will improve the resistance to weathering. In addition, soil stabilized with either material will gain strength with time at elevated temperature and under cyclic action. In many instances, soils containing lime and fly ash have increased in strength materially while undergoing a freeze-thaw test (58-7). In general, lime-pozzolan-soil mixtures are similar to lime-soil mixtures when the pozzolan is considered to be part of the soil (52-7, 53-4, 56-2, 58-2, 58-3, 58-7, 58-12, 58-13, 58-17).

CONSTRUCTION METHODS

Until recently little attempt has been made to standardize methods for constructing lime-soil roads. Many times the construction was haphazard, but through the joint effort of several organizations, lime-soil stabilization construction methods have become more uniform (59-12). The sequence of operations in constructing lime-stabilized roads is generally the same wherever the project, but special techniques related to local experience, available equipment and specific conditions may necessitate some variation.

Construction Techniques

The construction steps normally followed in constructing lime-soil stabilized subbases, subgrades, and bases are as follows:

Scarifying and Pulverizing the Soil.—The roadway is first brought to grade. If the natural material in the road is to be stabilized the upper portion is scarified to the depth that the lime-stabilized treatment is required. When the existing road material is not used, new soil is hauled to the road, dumped in place and pulverized. Scarification and pulverization of the soil may be accomplished by a motor grader. Where greater control of the depth and more thorough pulverization of this soil is desired a pulverizing mixer should be used.

Addition of Lime to the Soil.—Lime may be added to the soil in the form of a dry powder or wet slurry. Dry application of lime is accomplished by a mechanical spreader or by the dumping and spreading of bagged lime by hand in a controlled pattern on the roadway. When wet slurry is used, lime and water are premixed and spread together from a tank truck or distributor. Regardless of the method used for spreading the lime, uniformity of application and control of the quantity are essential.

Mixing of the Lime, Soil and Water.—Lime and soil are thoroughly mixed with a motor grader, pulverizing mixer, disc or some other type of mixing equipment. If water is needed it is added as the mixing progresses. This process of mixing and adding water is continued until the moisture content of the mixture is slightly above optimum and the lime is uniformly distributed throughout the soil.

Initial Curing.—Lime-plastic soil mixtures should cure in the loose state for 24 to 48 hr after mixing (59-12) or until the soil disintegrates easily in the hand and becomes friable. It is not intended that the mixture gain strength during this aging period, but only that the soil loses its plasticity. Initial curing is not required for nonplastic soils that are being stabilized with lime.

Final Mixing.—After the lime-soil mixture has initially aged it is remixed and repulverized. If needed, water may be added at this time to adjust the moisture content. This process is continued until optimum moisture is attained and all "clods" and "lumps" are broken down into a homogeneous mass.

Compaction and Shaping.—The lime-soil mixture is bladed to the appropriate thickness and is compacted until the desired density is attained. Satisfactory compaction may be accomplished in single or multiple lifts, depending on the thickness of treatment and the type of compaction equipment available. Pneumatic rollers and sheepsfoot rollers are primarily used during the initial compaction. These may be followed by shaping the surface of the top lift with a motor grader and then by final compaction with a steel-wheel roller. Field checks on the moisture content and the degree of compaction should be made at frequent intervals to insure adequate control.

Final Curing.—Lime-soil mixtures are cemented together only after final compaction and not during the first curing period. Ideal curing is produced by warm temperatures and by preventing the evaporation of moisture from within the compacted mixture. The latter is accomplished by applying water to the surface or by sealing the surface with an asphalt membrane. Lime-soil mixtures are usually moist cured for 5 to 7 days, but longer periods may be required if low air temperatures prevail.

Placement of the Wearing Surface.—If a lime-soil mixture is used as a base, some type of a wearing surface must be applied to prevent abrasion of the surface. Whether it be a surface treatment or a hot mix, it should not be constructed until the lime-soil base has adequately hardened and all loose material removed from the surface of the base.

Costs

The cost of lime-soil stabilization varies with the depth of treatment, quantity of lime, geographical location, familiarity with construction procedures, etc. The reported prices for lime-soil stabilization average about $0.40 per sq yd. The cost for a 6-in. depth of stabilized material on large projects may be as low as $0.27 per sq yd. On smaller projects, however, such as parking lots, the cost may be as high as $0.50 to $0.55 per sq yd (59-2).

Safety Precautions

Special safety precautions must be taken when handling lime to prevent injury to workmen. Hydrated lime is relatively safe but may cause irritations to people with sensitive skins. Quicklime, on the other hand, is quite dangerous in the presence of moisture because of its highly caustic nature. Even small amounts of perspiration on the skin will react with quicklime to cause severe skin burns. Quicklime is especially dangerous to the eyes.

Safety glasses, long sleeved shirts and relatively close fitting clothing should be worn by all personnel engaged in handling lime. If dusty conditions exist, filter masks should also be used to prevent excessive inhalation of lime dust. Special protective cream may be applied to the skin of construction personnel who are subjected to prolonged exposure of the lime dust.

All personnel should be instructed in first aid procedures for treating injuries that occur while lime is being handled. Special burn ointment, fresh water and eye wash glasses should be available at all times. All severe burns should receive immediate medical attention (52-2, 54-3, 57-9, 59-2, 59-5, 59-8, 59-12).

FIELD PERFORMANCE OF LIME-SOIL STABILIZED ROADS

Information relating to the long-range field performance of lime-soil stabilized roads is limited. This lack of information on performance may be attributed to several factors, but particularly to the fact that the stabilization of soil with lime in the United States did not flourish until the end of World War II. Because of the limited number of years that lime has been used for soil stabilization, it is difficult to evaluate long-range field performance in terms of the number of years of satisfactory service.

Until recently there has been little attempt to correlate laboratory test results and the actual field performances of lime-soil stabilized roads. The available data are limited. This is probably related to the problems encountered in obtaining cores of lime-soil mixtures from the field that can be used in the laboratory for evaluating strengths and other physical properties. Generally, the few cores that have been obtained from lime-stabilized roads were very irregularly shaped and difficult to test in the laboratory. Therefore, because of the difficulties encountered in extracting satisfactory specimens, field performance has generally been evaluated by personal observations.

The most extensive data on the field performance of lime-soil stabilized roads have come from field tests conducted in Texas and other southern states. Some limited data are also available on several test sections that were constructed in the central and north central part of the United States. The reports on the field performance of most of these projects have indicated that they performed satisfactorily.

Much of the data relating to the field performance of lime-soil stabilized bases and subbases are limited to the treatment of old existing roadbeds containing plastic to highly plastic clays and a considerable amount of granular material. Only limited information, however, has been published on the performance of lime-stabilized, highly plastic fine-grained soils that contain no granular material. Some literature is also available on the successful treatment of wet subgrades with lime as an aid to construction.

Because of the variations in construction techniques, types and amounts of lime used, types of soil stabilized, etc., it is difficult to compare field performance of lime-soil stabilized roads on an individual basis. A brief description of a few of the reports on the field performance of lime stabilized roads has been included in Table 4. If detailed information on the performance of any of these projects is desired, it is suggested that the original reference be consulted.

LIMITATIONS TO STABILIZING SOIL WITH LIME

Lime-soil mixtures have certain limitations that should be fully understood by the highway engineer before any design and construction of this type of stabilized road is undertaken. Many of these limitations are not unique with lime-soil mixtures, but are factors that must be considered in other types of soil stabilization work as well. Some of the more important limitations are briefly discussed in the following paragraphs.

Climatic Conditions

To date, there are little data available on the exact influence of climatic conditions on the use of lime-soil mixtures. Satisfactory performance of these mixtures has been obtained in areas with relatively mild temperatures. There is some question, though, as to the performance of this material in the colder climates. Only a few projects have been constructed in these areas and little performance data are available. Until more knowledge of the influence of cold weather on lime-soil mixtures is obtained, the use of this material in colder climates will probably be limited.

Some investigations have indicated that air temperatures materially influence the curing rate of lime-soil mixtures in the field. The curing rates of lime-soil mixtures are relatively fast at high temperatures, but are fairly slow at colder temperatures. Normally, during the warmest weather a minimum of 7 days moist curing is required for the lime-stabilized road to attain sufficient strength to support normal traffic. The time required for development of adequate strength and durability at colder air temperatures is considerably longer. At very cold temperatures curing may practically cease with little strength being gained until the weather becomes warm again.

Freezing of lime-soil mixtures during curing may result in a permanent reduction in strength. Swelling usually occurs because of the expansion of the frozen moisture and because of the formation of ice lenses in the mixture. In the process of swelling, the bonds between lime-soil particles are broken, the mixture becomes loose and a permanent loss in strength results. To reduce this damage and to allow sufficient strength to develop, especially during the early stages of curing, it has been

TABLE 4

FIELD PERFORMANCE OF A FEW LIME-STABILIZED ROADS

Reference No.	Location	Soil Types	Reported Field Performance
59-7	Nebraska	Highly plastic glacial clay	Satisfactory performance was observed after 2 yr service.
58-9	Perry Co., Missouri	Gravelly clays and silty clay loams	After 4 1/2 yr service, all sections had an appearance of good to excellent. The PI's increased during this time.
57-4	Mascoutah, Illinois	Highly plastic clay	Performance of this section was very good after 4 yr service.
	Engelmann Township, Illinois	Highly plastic clay	This section was in excellent condition after 5 yr service.
56-7	Mitchell Co., Kansas	Limestone gravel	The sections developed a few cracks after 1 yr of service.
53-3	Taylor, Texas	Taylor marl gravel	This section was holding up well after 5 yr use.
	Williamson Co., Texas	Granular soil	After 8 yr service, this section was in excellent condition.
	East of Taylor, Texas	Clay gravel	This road was used to serve light traffic for 5 yr. It was patched and resealed as many dry weather cracks had developed.
48-4	Texas	Old clay gravel base	After 2 yr service, the condition of the road was perfect.

recommended that lime-soil mixtures not be constructed after September 15 or after 1 mo before a probable freeze (59-12).

Some experience has indicated that soils stabilized with lime are more resistant to damage by frost action than the same untreated soils (59-2). It is thought that soils stabilized with lime form a moisture-resistant barrier which tends to obstruct the penetration of capillary water and results in the formation of fewer disruptive ice lenses (59-2). However, the exact amount of damage by frost action that will occur depends on several factors, such as air temperatures, availability of moisture, type of soil, degree of compaction, quantity of lime used, etc. Thus, it is somewhat difficult to predict whether the material is frost susceptible or not (48-4, 54-3, 57-1, 59-2, 59-12).

Permanency

A limited amount of data indicates that changes in various chemical and physical characteristics which take place originally when lime is added to a soil do not necessarily last indefinitely. Lime-soil mixtures are usually fairly effective in repelling some ground water. However, little is known of what happens when lime-soil mixtures are continually subjected to fluctuating ground water or to the percolation of water through the mixture for a long period of time. Some laboratory investigations have indicated that relatively few calcium ions are leached from a lime-soil mixture by distilled water (52-4). It is possible, however, that different reactions may take place when the mixture is leached with water containing sodium ions and other chemicals.

In many cases the plasticity indexes of lime-soil mixtures in the field have increased slightly with age (53-3). This might indicate that the original reduction in the plasticity index, gained immediately after mixing lime with the soil, may not be quite permanent. On the other hand, it may be a normal reaction. The increase in plasticity is slight and even after several years the plasticity index of most lime-soil mixtures is considerably lower than the plasticity index of the originally untreated soils. Also, the strengths of some lime-soil mixtures in the field have been found to increase for a considerable period of time and then decrease slightly (53-3). Whether these changes in physical characteristics are caused by inadequate lime, by reactions with certain chemicals in the water, or other factors, little is known at the present time (52-1, 52-4, 53-3, 58-9, 59-8, 59-10).

Thickness of Treatment

The minimum depth of treatment of thickness required for a lime-soil base, subbase or treated subgrade is difficult to determine because of the large number of factors affecting the design of this type of pavement, and especially, because of the variations in strengths attained in the field. It is recommended by some investigators that lime-soil bases be not less than 6 in. thick to prevent excessive deflection under light traffic (54-3). Attempts have been made to use only 3- and 4-in. lime-stabilized bases, but any success achieved in these experiments proved to be short lived (54-3). Where lime-soil mixtures have been used for subbase and subgrade treatments, thickness has varied from a few inches to greater than 1 ft with varying success (52-3, 54-3, 56-6, 58-16).

Resistance to Traffic Wear

Lime-soil mixtures have little or no resistance to traffic wear. Moving vehicles abrade the surface of unprotected bases, which results in the undesirable loss of material. If this is not prevented the thickness of the pavement may be considerably reduced and rapid deterioration of the pavement will take place. Lime-soil bases should be protected by an abrasion-resistant surface. Usually, a seal coat or a surface treatment is satisfactory. If additional strength is desired in the pavement an asphaltic concrete surface should be used (54-3, 58-10, 58-11).

Cracking and Fluffing of Lime-Soil Mixtures

Cracking and fluffing are two objectionable features that often occur in lime-soil stabilized bases. Cracking may be caused by volume changes in underlying untreated subgrades, by volume changes in the lime-soil mixtures or by the application of heavy loads or heavy rolling during the curing period. Fluffing or loosening of the surface of a lime-soil mixture is related primarily to the curing and in some instances may be more detrimental than cracking.

Volume changes in untreated subgrades may cause serious crack damage to the overlying stabilized bases. These excessive volume changes may be minimized by high compaction and proper moisture control in the subgrade.

Even though the volume changes of certain soils are minimized by treatment with lime, some natural shrinkage of lime-soil mixtures may still occur and a few cracks will result (57-4, 59-7). Most data indicate that these cracks usually occur during the early stages of curing. The natural shrinkage of lime-soil mixtures may be more severe in soils originally having low plasticity indexes (59-8), but the exact amount will depend on the mineralogical composition of the treated material, quantity and type of lime used and curing conditions.

Cracking of lime-stabilized bases may also be caused by the application of heavy loads from vehicles or rollers during the curing period (59-8). In a few reported instances no detrimental effects occurred when heavy traffic was not restricted during the curing period (58-10, 58-11), but this is generally not the case. Usually, no traffic is permitted on the lime-stabilized material during curing. It is interesting to note that on a few projects light pneumatic-tired rollers have been used during curing to help keep the surface knitted together (59-8).

Detrimental "fluffing" or loosening of the surface of lime-soil mixtures often occurs, especially if the mixture is cured in the absence of moisture during hot weather. This loose material on the surface may prevent a tight bond between the lime-treated base and the surface course and will contribute to peeling of the surface treatment. In one instance an asphalt curing membrane peeled and some of the loosened lime-soil mixture was blown away because of a delay in applying a surface treatment (56-6).

To minimize most of the harmful effects of cracking and fluffing it is necessary to moist cure the mixture either by applying water to the surface or by sealing the surface with a curing membrane. If cracking or fluffing does occur during curing, it may be necessary to remove the loosened material from the surface with a motor grader, or to rework the surface prior to the application of a surface treatment. To prevent fluffing on one project (59-5), base material was placed on a lime-stabilized subgrade immediately after final compaction and the treated subgrade was moist cured by permitting water to seep through the base (56-6, 57-4, 58-8, 58-10, 58-11, 59-5, 59-7, 59-8).

Construction Limitations

As with other material, construction techniques influence the performance of lime-soil mixtures. Many failures of lime-soil stabilized roads, especially those that occur along the outer edges of the pavements, have resulted from poor distribution of lime, inadequate depth control, lack of edge control and improper compaction. These could have been eliminated by proper construction techniques. In a few instances, to compensate for some of the irregularities that might occur in construction, more lime than was required for normal strength purposes was added to the soil (59-8). The use of excess quantities of lime in construction, however, should be avoided because of increased cost and the possibility that a strength reduction may occur in the mixture (56-1, 57-3). (56-1, 57-3, 59-8).

Reworking Lime-Soil Mixtures

Experience has indicated that reworking of lime-soil mixtures should be avoided after the mixture has set unless additional lime is added. Some investigators have suggested that lime-soil mixtures be compacted within 2 to 4 days after mixing and that these mixtures not be reworked after 7 to 28 days without the use of additional lime (59-2, 59-8). The amount of additional lime added is usually small (59-2), approximately 1 percent. The time limit for reworking lime-soil mixtures without adding additional lime will depend on the field conditions and the quantity of lime originally added to the soil. In one instance, a mixture was finally compacted after a 4-wk delay without the use of any additional lime (58-5). It is not known, though, whether the performance of this road was satisfactory or not (48-5, 58-5, 59-2, 59-8).

SUMMARY AND CONCLUSIONS

The reviewed literature, although not providing a comprehensive treatise on lime-soil mixtures, does provide valuable information for understanding the actions between lime and soil. The major portion of the reported data is related to the physical characteristics of the mixtures, for example, volume change, strength, durability, etc. Some of these physical properties are covered rather thoroughly, although much data are lacking. Information related to the influences of various types of lime, for example, is almost nonexistent. Also, little information is available that explains the chemical action that takes place to produce these physical changes. Almost all of the information that is available has been obtained in the laboratory. Only meager data are available concerning field tests and field performance of lime-soil mixtures. In other words, most of the reported work has been accomplished through applied research in the laboratory. Little basic research on lime-soil mixtures has been reported.

The literature indicates that lime can be used successfully to stabilize soil within limitations. Lime will modify the soil, strengthen it, and will improve its durability when properly used. Although lime is not a panacea for soil stabilization, when its limitations are considered it can be and has been used successfully in road construction.

The effectiveness of lime depends on the reactions desired and is strongly related to the type of soil that is to be stabilized. If an ionic change is desired to lower the plasticity of the soil and thus make it more friable and easier to handle, the soil originally has to be medium to highly plastic. On the other hand, if lime is to strengthen and improve the quality of the soil, the soil should originally contain the necessary chemical compounds that will react with the lime to produce the cementing effect. In general, the more plastic clays have greater reactivity with the lime and are considerably improved by adding lime. Lime is not too effective, though, in further reducing the plasticity index of a soil originally having a low plasticity. Stabilization of granular materials with lime is difficult, if not practically impossible in some instances. Sometimes, though, granular material can be successfully stabilized with lime if a suitable clay binder or a relatively small amount of pozzolanic material is added to the soil so as to improve the cementation action.

Usually, the desired improvements in the soil can be produced by any one of the commercial grades of lime and sometimes by using locally available waste limes. Although many types of lime may be used in stabilizing a particular soil, one type of lime may be more effective than another. If more than one type of lime is economically available, all limes should be checked to determine which is the most desirable for the existing conditions. The exact amount of lime needed varies not only with the type of lime but also with the type of soil and with other factors. In general, as the amount of lime in the soil is increased, the strength and quality of the mixture is also increased, although not necessarily proportionately. Regardless of the type of lime used or the other conditions, a minimum of about 3 percent lime is needed to produce the desired results.

Another major limitation in using lime as a stabilizing material is the curing temperature. Lime-soil mixtures must be cured for some time at warm or even hot temperatures to gain proper strength and durability. This necessitates early or middle season construction. Late season construction is not desirable. Many lime-soil stabilized sections that were placed late in the season have failed during the first winter because of insufficient curing during the cool fall.

Although the properties of lime-soil mixtures have been investigated rather extensively in the laboratory, little data have been obtained from field testing. So serious is this lack of information that sometimes wrong conclusions have been drawn in evaluating this stabilizing material. For example, the literature strongly indicates that the strength which can be obtained in the laboratory under ideal temperature and other curing conditions is considerably greater than that developed in the field. If the material is being improperly evaluated in this manner higher strengths will be used in design than are actually obtained in the field. In such cases the thickness of the road could be seriously underdesigned. On the other hand, it appears that current durability tests are too harsh on lime-soil specimens. Many times these laboratory tests have indicated the performance of lime-soil mixtures to be poorer than the material has actually been in the field. Until more data are obtained and laboratory and field results are thoroughly correlated, proper design of lime-soil mixtures is seriously handicapped.

In summarizing, the literature tends to indicate certain facts that allow the following conclusions to be drawn. Some of the statements are strongly supported by existing data, whereas others are inferences drawn from limited data.

1. Lime tends to modify the undesirable characteristics of the more plastic clays. It can make the clays friable, can reduce the plasticity index of the soils and can reduce the amount of volume change.
2. In most clays, lime can produce a cementing action which will result in a higher strength and greater durability than would occur in untreated soils.

3. Lime can be used to stabilize heavy clays which cannot be stabilized economically by other types of stabilizing materials.

4. Lime cannot be successfully used with all types of soils, but is limited primarily to the stabilization of medium and highly plastic soils.

5. For soils with low or no plasticity, fly ash and other pozzolanic materials may be added to the soil with the lime to aid in the cementing action.

6. Lime-pozzolan-soil mixtures and lime-soil mixtures have similar physical characteristics.

7. It appears that most of the existing methods for testing lime-soil mixtures do not accurately evaluate the strength and durability characteristics of lime-soil mixtures.

8. The amount of lime needed to modify or stabilize a soil is relatively small. Between 1 and 10 percent, but usually not less than 3 percent lime is required.

9. For the lime to react thoroughly with the soil, there should be a curing period after the lime has been mixed into the soil and before the mixture is finally compacted.

10. The lime-soil mixture must be compacted to a high degree to have high strength and high quality.

11. Because lime-soil mixtures do not set up rapidly, sufficient time usually exists for thorough mixing and compaction.

12. Subgrades that are treated with lime are less affected by rain than untreated subgrades. In some cases this enables contractors to start work after heavy and prolonged rains sooner than they could do otherwise.

13. During construction, lime-treated bases seem to be little affected by rain. Their imperviousness also prevents falling water from flowing through them to the already compacted subgrade.

14. The gain in strength and quality of lime-soil mixtures is very slow and requires rather long periods of time and warm temperatures.

15. Warm weather is almost a necessity in adequately curing lime-soil mixtures. Cool or cold weather is not desirable, and little gain in strength will be obtained at low temperatures.

16. To adequately protect the surface of a lime-stabilized base, the surface must be covered with a surfacing material such as a seal coat or asphaltic concrete.

SUGGESTED RESEARCH PROJECTS

In reviewing the existing literature on lime-soil mixtures, certain deficiencies in the technical knowledge become apparent that can only be rectified through additional research. A number of general research suggestions are listed herein that should increase this knowledge and enable these lime-soil mixtures to become more useful highway materials.

This list of research suggestions does not necessarily include all of the needed research, but probably does include the most important ones at this time. Some of these studies have been started and as results are reported, the list will need to be modified.

These research suggestions are not placed in any order, and definitely no priority rating has been established. The relative importance of each suggestion varies with specific circumstances and should be evaluated in the light of the existing conditions.

A. Basic research should be conducted to gain a more thorough knowledge of the reactions that take place when lime is added to soil. This includes, but is not limited to, the cementing actions, the chemical compounds that are formed in the soil and the influence of pozzolans on lime-soil mixtures.

B. Research is needed to develop a test or tests that will indicate if a soil is suitable for stabilization with lime.

C. The influence or effectiveness of the various types of limes for stabilizing soils should be investigated.

D. Studies should be made to determine the types of soils that are suitable for stabilization with lime. In addition, the optimum percentage of lime needed for each of these soil types should be determined.

E. The permanency of lime-stabilized soils should be thoroughly studied. Not only should this study include the permanency of the change in plastic limits and strengths of a lime-stabilized soil, but it should also include the effects of groundwater and leaching.

F. More knowledge on the methods of curing lime-soil mixtures is needed. Factors that improve curing, as well as the detrimental factors should be studied.

G. A suitable laboratory test is needed for evaluating the durability of lime-soil mixtures.

H. The effects of cold temperatures on lime-soil mixtures, primarily frost action and durability, should be determined through additional research.

I. The reactions that occur during the initial curing process should be studied, as well as the influence of this aging on the cementation process.

J. More controlled field tests of lime-soil mixtures are needed to determine the performance of this material under all climatic conditions.

K. Methods for correlating laboratory data and field data with the behavior of lime-soil mixtures under actual field conditions are needed.

L. A suitable rational design method for structurally designing semi-flexible pavements, such as lime-soil mixtures, should be derived.

M. Methods for improving the bond between lime-soil bases and bituminous surface treatments should be developed.

N. Additional investigations are needed to determine the types of materials, other than fly ash, that are suitable for use as pozzolans.

O. The effects of various chemical admixtures on the strength and durability of lime-soil mixtures should be determined.

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On the part of the University, the work covered in this report was carried out under the general administrative supervision of W. L. Everitt, Dean of the College of Engineering; R. J. Martin, Director of the Engineering Experiment Station; N. M. Newmark, Head of the Department of Civil Engineering; and Ellis Danner, Director of the Illinois Cooperative Highway Research Program and Professor of Highway Engineering.

On the part of the Division of Highways of the State of Illinois, the work was under the administrative direction of R. R. Bartelsmeyer, Chief Highway Engineer; T. F. Morf, Engineer of Research and Planning; and W. E. Chastain, Sr., Engineer of Physical Research.

Technical advice was provided by a Project Advisory Committee consisting of the following personnel:

Representing the Illinois Division of Highways—W. E. Chastain, Sr., Engineer of Physical Research; R. K. Andrews, deceased, Soils Engineer; and Dolph Hoke, Soils Engineer.

Representing the Bureau of Public Roads—H. J. Stahl, Area Engineer; and F. P. Walton, Assistant Construction and Maintenance Engineer.

Representing the University of Illinois—T. H. Thornburn, Professor of Civil Engineering; and R. E. Grim, Research Professor of Geology.

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ANNOTATED BIBLIOGRAPHY

The references in this bibliography are limited to articles related to stabilization of soils with lime. There are no references on lime specifications, lime production, lime characteristics or lime-pozzolan-aggregate mixtures.

Each article on lime-soil stabilization has a reference number assigned to it and a short annotated summary of what the article covers. The reference number is composed of two parts (59-1). The first part of the reference number indicates the year of publication (1959), and the second part refers to the arrangement within that particular year. The references are arranged chronologically according to the earliest year of publication and alphabetically according to authors within the particular year. Where the name of the author was not given, the article is referenced last in that year.

After a few of the references there are no annotations. These references are either not available at the University of Illinois Library or are written in foreign languages.

25-1. McCaustland, D. E. J., "Lime in Dirt Roads." Proc., National Lime Assn., 7: 12 (1925).

Indicates the possibilities of using lime for stabilizing road subgrades. Conclusions are based on laboratory results obtained at University of Missouri.

25-2. McKesson, C. L., "Report of the Rio Vista, California, Subgrade Treatment Experiments." HRB Proc., 5:Pt. I, 123 (1925).

A section of test road using a 1 to 20 hydrated lime mixture 12 in. deep, was constructed on adobe and silty clay soils. Transverse cracks 10 to 60 ft apart and several short irregular longitudinal cracks developed. Surface checking was also noted. Lineal shrinkage was reduced 2 percent. Conclusions of test were that hydrated lime was useless as a subgrade treatment.

26-1. Woods, H. W., Jr., "Lime in Earth Roads." Proc., National Lime Assn., 8:57 (1926).

A general article on the use of lime for stabilizing earth roads.

26-2. Anon., "Out of Mud with Lime." Bull. 317, National Lime Assn. (1926).

Field practice at this time indicated that the lime content should be between 2.5 to 4.0 percent. For troublesome clay soils 3 to 5 percent lime produced the desired results. In general, it was found that lime treated roads gave better service than untreated soils; and also that these lime treated soils provided a foundation for further improvements.

31-1. Rauterberg, E., "The Physico-Chemical Effect of Lime on the Soil." Fortschritte dev Landwirtschaft, 6:680 (1931).(In German).

32-1. Groditskaya, B. M., and Ipatova, A. I., "The Problem of Lime Stabilization of Soils for Roads." Sbornik Gdornii, Gruntodezhda, Vol. 3 (1932). (In Russian).

32-2. Ipatova, A. I., "Experiments in Stabilizing Chernozem Soils with Lime." Doroga i Avtomobil, (3) (1932). (In Russian).

32-3. Maffei, A., and Banchi, G., "The Displacement of Alkalis in Clays by the Action of Lime." Ann. Chim. Applicota, 22:93 (1932).

34-1. Eno, F. H., "Some Effects of Soil, Water and Climate upon the Construction Life and Maintenance of Highways." Eng. Exper. Sta. Bull. 85, Ohio State Univ. (1934).

35-1. Searle, A. N., "Limestone and Its Products." Earnest Benn Ltd., London, p. 588 (1935).

One section discusses the use of lime in treating road subgrades.

35-2. Volkov, M. I., and Kustiik, B. R., "Influence of Additions of Lime and Cement on Road-Building Properties of Clay Soils." Journal, Katkhov Highway Institute, 1:90 (1935).

36-1. Hogentogler, C. A., and Willis, E. A., "Stabilized Soil Roads." Public Roads, p. 45 (May 1936).

A general article on stabilizing soils encompassing all stabilizing agents used up to this time. Lime is referred to only briefly. Many good basic factors, especially those pertinent to all methods, are discussed.

37-1. Bykovski, N. I., "The Problem of Lime Treatment of Roads." Doroga i Avtomobil, Vol. 8 (1937). (In Russian).

37-2. Hogentogler, C. A., "Engineering Properties of the Soil." McGraw-Hill (1937).

The presence of lime in soils is briefly discussed. The calcium replaces hydrogen or sodium in the clay, thus converting an acid or alkaline clay into a more usable calcium clay. The action of lime also replaces the acidic hydrogen in humic acid forming the more stable so-called calcium humate or neutral humus.

40-1. Lesesne, S. D., "Stabilization of Clay Roadbeds with Lime." Bull. 325, National Lime Assn. (1940).

Stabilization of heavy gumbo clays was accomplished by adding lime and then compacting the mixture at optimum water content. Highest strength values were achieved when the lime-soil specimens were soaked in a capillary tank for a period of time. Further moist curing only maintained the peak value.

40-2. Lesesne, S. D., "Road Stabilizing Materials and Processes." Proc., National Lime Assn. (May 1940).

40-3. Willis, E. A., and Smith, P. C., "Chemical Treatment of Chert-Gravels for Use in Base Course Construction." Public Roads, 21:4, 65 (June 1940).

Primarily a description of the testing of chert-gravel stabilized with various admixes in a test track. Lime was one of the stabilizers and performed quite satisfactorily.

42-1. Visser, W. C., "Lime Status and Soil Structure." Landbouwkundig Tijdschriff, 54:791 (1942).

43-1. Li, M. C., "Research on Soil Stabilization." HRB Proc., 23:413 (1943).

This article deals with China's problem to develop a quick stable road surface by using admixtures of cinders, burnt clay, lime, etc. Various physical properties were determined, but little data were given on lime stabilization.

46-1. James, R. L., "Soil Stabilization—The Wet Sand Process." Contractors Record and Municipal Engineering (Feb. 6, 1946).

47-1. Dockery, W. D., and Manigault, D. E. H., "Lime Stabilization and Low Cost Road Construction." Roads and Streets, 90:91 (Aug. 1947).

The construction of various lime-soil stabilization projects in the Austin District of the Texas State Highway Department is reported. Primarily worn-out, clay-gravel roads were effectively stabilized with waste lime.

47-2. Freeborough, B. B., "Lime Treatment Permits Use of Substandard Flexible Base Materials." Public Works, 78:6, 17 (June 1947).

A general article on some of the advantages of stabilizing various soils with lime. Observations were made of projects in Texas.

47-3. Maiborodo, G. I., "Humus-Concrete Road Construction." Stroitel'stvo Dorog, (1) (1947). (In Russian).

47-4. Manigault, D. E. H., "Lime Stabilization." Roads and Streets, p. 94 (Aug. 1947).

Conclusions, obtained from laboratory investigations of the effects of lime in stabilizing soils, were given with a general discussion of the use of lime in road construction.

48-1. Aaron, H., "Report of Committee on Lime-Soil Stabilization." Tech. Bull. 147, American Road Builders' Assn. (1948).

In general the conclusions of the committe were: (1) More observations and studies are necessary before definite recommendations can be made relative to test methods and design procedures. (2) Small percentages of lime lower PI of gravel, disintegrated granite, and caliche, and increase stability. (3) A soil's physical reaction to lime should be thoroughly investigated before stabilization. (4) Control over proportioning of materials, mixing and compaction are essential during construction. (5) Curing is necessary to prevent too rapid drying of mixture.

48-2. Johnson, A. M., "Laboratory Experiments with Lime-Soil Mixtures." HRB Proc., 28:496 (1948).

Fine-grained soils, natural gravels, and gravel binder mixes were benefitted by the addition of lime. This was indicated by increased resistance to penetration (CBR) even though the maximum dry density was lowered.

48-3. Kuran, H., and Honnemann W., "Influence of Lime on the Mechanical Properties of Soil Colloids." Zeitschrift fur Pflanzenernährung and Dungung, 40:200 (1948). (In German).

48-4. McDowell, C., "The Use of Hydrated Lime for Stabilizing Roadway Materials." Proc., National Lime Assn. (1948).

An article describing the general use of lime for stabilizing soils. Specimens used for testing the strength of lime soil mixtures were moist cured for 7 days; air dried at 140 F for one day; subjected to 10 days of capillary wetting; and finally tested by the triaxial compression method.

48-5. McDowell, C., and Moore, W. H., "Improvement of Highway Subgrades and Flexible Bases by the Use of Hydrated Lime." Proc., Second International Conf. on Soil Mechanics and Foundation Engineering, 5:260 (1948).

Report of an exploratory investigation of lime stabilization with Texas soils. A history is given and preliminary conclusions are made.

48-6. Smith, W. H., "Stabilizing Texas Roads with Lime." Better Roads, 18:5, 23 (1948).

A brief record of early lime-soil stabilized projects carried out by the Texas Highway Department is given. Very good results were obtained when the proper quantities of lime, soil and water were adequately mixed, compacted and cured. In the few failures recorded, a mistake in one of the previous requisites was found. When the mistake was rectified, no further damage was recorded. One test section withstood heavy traffic and icy winters with only negligible maintenance.

48-7. Volkov, M. I., Gelmer, V. O., et al., "The Effect of the Addition of Lime and Cement on the Road Properties of Clay Soils." (Dorizdat), Moscow (1948). (In Russian).

48-8. Anon., "Lime Used in an Airfield Base." Roads and Streets, 91:1, 96 (1948).

During World War II an airstrip at Beeville, Texas was constructed with a base course of caliche stabilized with lime. After 4 1/2 yr of service the strip was in fine condition.

49-1. Huang, E. Y., "Effect of Quick-lime on the Compressive Strength and the Physical Constants of Fine-Grained Soils." Unpublished Thesis, Univ. of Utah (1949).

The changes in physical characteristics of various types of fine-grained soils produced by addition of quicklime were investigated. Results are similar to those produced by hydrated lime. The difference in the results obtained with the two types was not compared in this report because hydrated lime was not part of the investigation.

49-2. McDowell, C., "Hydrated Lime for Stabilizing Roadway Materials." Roads and Streets, 12:2, 81 (Feb. 1949).

General information concerning lime stabilization of soils in Texas. In addition to information concerning construction, plasticity and strength changes, results of freeze-thaw durability tests are given.

49-3. Spangler, M. G., and Patel, O. H., "Modification of a Gumbotil Soil by Lime and Portland Cement Admixtures." HRB Proc., 29:561 (1949).

This paper reports the results of a laboratory study of the effect of various percentages of unslaked lime (CaO) and portland cement on the engineering properties of gumbotil soil which is rather frequently encountered in highway construction in southwest Iowa. The results indicate that a marked and favorable modification of the soil in all the major properties studied is accomplished by adding lime.

49-4. Woods, K. B., "Lime as an Admixture for Bases and Subgrades." Paper presented at 31st Ann. Mtg., National Lime Assn. (May 1949).

Laboratory studies indicate that the degree to which subgrade soils can be improved (reduction in plasticity index and increase in strength) is dependent on the type of soil. The more plastic soils are improved to a greater degree than more silty soils. Tests indicated lime would also improve gravel-base material.

50-1. Minnick, L. J., and Miller, R. H., "Lime-Fly Ash Compositions for Use in Highway Construction." HRB Proc., 30:489 (1950).

A sand and several types of coarse aggregates were improved by stabilizing with lime and fly ash. The stabilized soils showed excellent resistance to wet-dry and freeze-thaw tests. Compressive strengths also were increased.

50-2. Mehra, S. R., and Uppal, H. L., "Use of Stabilized Soils in Engineering Construction." Journal, Indian Road Congress, 15:1 and 2 (1950).

50-3. Patel, O. H., "Stabilization of Gumbotil Soil for Highway Use." Journal, Indian Road Congress, 15:336 (1950).

51-1. Couillaud, E., "Substitute Binders." Rev. Mat'er Construction, No. 426, p. 77 (1951). (In French).

51-2. Gallaway, B. M., and Buchanan, S. J., "Lime Stabilization of Clay Soil." Texas Eng. Exper. Sta. Bull. 124, A and M College of Texas (1951).

A fairly basic investigation of the modifying effects that lime has on clays. The total base exchange capacity of a soil appears to give an indication of the reactivity of the soil and lime. Other methods, such as the X-ray, seemed not to be feasible. All results were given in terms of the plasticity change in the soil; no strength tests were run. An excellent theoretical analysis is given of the stabilizing reaction of soil and lime.

51-3. Mainfort, R. C., "A Summary Report on Soil Stabilization by the Use of Chemical Admixtures." Technical Development Report 136, U. S. Civil Aeronautics Admin. (1951).

This report covers a comprehensive long-time study of the stabilization of a number of different types of soils with various admixtures. When stabilized with lime, the soils had very little resistance to the laboratory freeze-thaw testing; however, the soils seemed more durable when subjected to actual field durability tests.

51-4. Anon., "Distribution Charts for Stabilized Materials." Roads and Streets, 94:8, 46 (Aug. 1951).

These charts were prepared by the Fourth Army Engineers for field use in their area and include cement as well as lime. Such charts are given as: (a) converting from percent lime to amount of lime per sq yd for specific depth, (b) rate of application, etc.

52-1. Carter, H. C., "Lime Stabilization in District Fourteen, Texas Highway Department." Tech. Bull. 185, American Road Builders' Assn. (1952).

This article is a short summary that deals with lime stabilization projects in a Texas Highway District. Primarily it covers construction procedures, but concludes with some pertinent observations.

52-2. Chopra, S. K., and Patwardhan, N. K., "Investigation on the Use of Lime-Sludge as a Soil Stabilizer." Journal of Scientific Industrial Research, India, 11B:10, 434 (1952).

52-3. Fuller, M. G., and Dabney, G. W., "Stabilizing Weak and Defective Bases with Hydrated Lime." Roads and Streets, 95:3, 64 (March 1952).

The Corps of Engineers built several test sections using lime to stabilize existing weak, poor quality, clay-gravel base courses. Field sections were successfully stabilized with only 3 percent lime. The plasticity index and percent of soil binder were reduced and high CBR values were obtained on all sections.

52-4. Goldberg, I., and Klein, A., "Some Effects of Treating Calcium Clays with Calcium Hydroxide." Symposium on Exchange Phenomena in Soils, Spec. Tech. Publ. 142 (with discussion), ASTM (1952).

After investigating the effects of lime on the expansion of two expansive clays, it was concluded that swelling was reduced with addition of lime. Above a given amount of lime, depending on the soil, little reduction in swelling was obtained with the addition of more lime. Detailed chemical analyses were also made to give indication of changes in lime and ions.

52-5. Gushosder, "Tentative Specifications for Methods of Constructing Soil-Cement and Lime-Stabilized Roads." Central Board of Highways, Moscow (Dorizdat) (1952). (In Russian).

52-6. Levchanovskii, G. N., "Lime Stabilized Soil Bases for Improved Road Surfacings." Saratov (Avtovefevat) (1952). (In Russian).

52-7. Minnick, L. J., and Miller, R. H., "Lime-Fly Ash-Soil Compositions in Highways." HRB Proc., 31:511 (1952).

The effect of lime and fly ash on the engineering properties of four types of soil was studied. A beneficial change was brought about in the properties investigated including durability.

52-8. Whitehurst, E. A., and Yoder, E. J., "Durability Tests on Lime-Stabilized Soils." HRB Proc., 31:529 (1952).

Studies were conducted on three soils, a Wisconsin drift soil, an Illinoian drift soil and a river terrace gravel, to determine the influence of lime on strength and durability properties (the latter as measured by the soniscope). Small amounts of lime did not help appreciably, but larger amounts (greater than 5 percent) significantly increase both strength and durability. Increased time of moist curing was beneficial. Greatest benefits were derived by the gravel and the least by the Wisconsin drift.

52-9. Woods, K. B., and Yoder, E. J., "Stabilization with Salt, Lime, or Calcium Chloride as an Admixture." Proc., Conf. on Soil Stabilization, Massachusetts Institute of Technology, p. 3 (June 1952).

In the lime stabilization section of this article, it was indicated that the addition of lime to soils brings about a reduction in the plasticity index in various ways depending on the soil type. Other properties were investigated; most of which were improved by the addition of lime. The mixtures were susceptible, however, to freezing and thawing.

52-10. Zube, E., "Experimental Use of Lime for Treatment of Highway Base Courses." Tech. Bull. 181, American Road Builders' Assn. (1952).

A description of construction procedures, materials, test results and performance of two lime stabilization projects in California. In both projects granular base course materials were stabilized with agricultural lime.

53-1. Barisova, E. G., "Theoretical Principles on the Binding of Soils by Lime." Sbornik MGU, Gruntovedenie, Vol. 3 (1953). (In Russian).

53-2. Levchanovski, G. N., "Soil Stabilization with Ground Unslaked Lime." Trudy SADI, Sbornik 12, Saratov (Saratovskoe Knizhn. izd-vo) (1953). (In Russian).

53-3. McDowell, C., "Road and Laboratory Experiments with Soil-Lime Stabilization." Proc., National Lime Assn. (1953).

Primarily this article reports the results of testing lime-soil specimens that were cured for long periods of time. Strength tended to increase for 2 to 4 yr, depending on lime content and other factors, and then changed little or even decreased slightly with more aging. The plastic limit also changed slightly with time.

53-4. Minnick, L. J., and Meyers, W. F., "Properties of Lime-Fly Ash-Soil Compositions Employed in Road Construction." HRB Bull. 69 (1953).

For a variety of soils studied, additions of small amounts of hydrated lime and fly ash helped to develop high strength at relatively early ages. Curing at elevated temperatures accelerated pozzolanic action. This material had good durability as indicated by the soniscope.

53-5. Wood, J. E., "Lime-Fly Ash-Soil Stabilization in Maryland." Tech. Bull. 199, American Road Builders' Assn. (1953).

A report of the first lime-fly ash stabilization project in Maryland. A fairly non-plastic pit-run gravel was combined with optimum proportions of lime and fly ash as determined by laboratory moisture-density test. It was allowed to cure open for 75 days and then it was sealed. After a year it was in fairly good condition.

53-6. Anon., "Texas Stabilizes Roads with Quicklime." Engineering News-Record, 150:38 (Feb. 5, 1953).

This article illustrated how the Texas Highway Department realized some savings in cost of treating road bases by substituting quicklime for slaked lime.

53-7. Anon., "Road Base in Texas Stabilized with Lime." Contractors and Engineers, 50:7 (July 1953).

A heavy clay hydraulic fill was stabilized and made into an accepted base, when slaked lime was added to improve the bearing value of the subbase.

54-1. Dougherty, J. R., "Low-Cost Dustless Surfacing for Secondary Roads." HRB Proc., 24:25 (1954).

An investigation into the economics of "low-cost" road construction was begun through the construction of a "low-cost" road test section. Various stabilizers, including lime, were used. No conclusions were given, but were to be made at a later date.

54-2. Misiaszek, E. T., "Lime-Fly Ash Stabilization Research; Abstract." Rock Products, 57:90 (June 1954).

A general review of research at Clarkson University on lime-fly ash stabilization of soils. Article is very brief with little technical data.

54-3. Anon., "Lime Stabilization of Roads." National Lime Assn., Washington, D. C. (1954).

An excellent publication on the use of lime for soil stabilization. Contains general discussion on construction procedures, recommended tests, and specifications for lime and lime stabilization. It also gives results of 30 different tests on fine-grained soils and 8 natural gravel and gravel-binder mixes as conducted at Purdue University. Several conclusions based on laboratory tests and experience are given in the bulletin. Lime was found to be most effective on clay-gravel or gravel-binder mixes in amounts from 2 to 5 percent. Five to 10 percent lime was considered necessary for most fine-grained clay soils. No definite conclusions were stated with respect to use of lime with fine-grained silty and loamy soils.

54-4. Anon., "Lime for the Stabilization of Road Bases." Roads and Engineering Construction, 92:96 (Sept. 1954).

A condensed version of "Lime Stabilization of Roads" by National Lime Assn. See 54-3 above.

54-5. Anon., "Lime Applied from Tankers on Texas Base Stabilization Project." Roads and Streets, 97:47 (Oct. 1954).

Lime stabilization of a Texas farm to market road was accomplished by the addition of hydrated lime in slurry form. No lime separation troubles were encountered. Normal construction procedures were followed except that a thin coat of emulsion was applied to the stabilized base to act as a dust palliative and to aid in curing before the protective surface treatment was placed.

55-1. Birulya, A. K., "Doragi iz Mestnykh Materialov." ("Roads from Local Materials.") Autotransizdat, Moscow (1955). (In Russian).

55-2. Chu, T. Y., Davidson, D. T., et al., "Soil Stabilization with Lime-Fly Ash Mixtures: Preliminary Studies with Silty and Clayey Soils." HRB Bull. 108 (1955).

A laboratory investigation of the stabilization of four various soils with lime and fly ash. Preliminary test results were given and a method was suggested for evaluating lime-fly ash stabilized soils.

55-3. Cooper, J. P., "Lime Stabilization of Base Material." Texas Highways, 29th Annual Highway Short Course, p. 86 (May 1955).

A fairly high plastic base material was stabilized with a lime slurry. Techniques were covered for preparing the slurry and its use.

55-4. Huff, T. S., "Use of Lime Stabilization on Roads of the Texas Highway System." Proc., AASHO, p. 157 (Dec. 1955).

This is a very general discussion of the use of lime in stabilizing soils. Characteristics of lime stabilization, both desirable and undesirable, and some basic suggestions for avoiding project failure were given.

55-5. Maclean, D. J., "Stabilization of Soils." Proc., 10th International Road Congress, Part B, Question II-Soils, Istanbul (1955).

55-6. McDowell, C., "Development of Lime Stabilization." Texas Highways, 29th Annual Highway Short Course, p. 79 (May 1955).

Primarily the past history and the development of lime stabilization are covered in this general article.

55-7. Mehra, S. R., and Chadda, L. R., "Use of Lime in Soil Stabilization." Journal, Indian Road Congress, 19:3, 483 (1955).

Results were given of a number of different laboratory tests on four typical loamy soils due to the effects of lime and lime with plaster of Paris. It was concluded that "lime improves compressive strength, reduces shrinkage, and increases resistance to softening action of water. Sandy loams react better to lime treatment than silty or silty clay loams." Also it was concluded that plaster of Paris should not be used with lime.

55-8. Misiaszek, E. T., "Effects of Lime and Pozzolanic Admixtures to Soils of New York State." Unpublished Thesis, Clarkson College of Technology (June 1955).

Laboratory investigations indicated that compressive strengths and durability characteristics of the three soils investigated were improved by stabilization with mixtures of lime and fly ash. The amount of improvement depended on the ratio and amounts of lime and fly ash used. Two of the soils were A-4 type and the other was an A-3 type.

55-9. Swain, M. S., "Lime Stabilization of Subgrades." Texas Highways, 29th Annual Highway Short Course, p. 83 (May 1955).

Construction experience on a Texas highway project where a clayey subgrade was stabilized with both hydrated lime and quicklime is summarized.

55-10. Uppal, H. L., "Laboratory Experiments in Sand Stabilization." Journal, Indian Road Congress, 19:2, 285 (1955).

Report of a laboratory investigation of the strength of a relatively clean river sand when stabilized with lime and with lime-molasses. Strength was little improved by lime alone, but lime-molasses increased the strength. There was little increase in strength when the combined mixtures were water soaked and there was even less strength when CO_2 was added to the water.

55-11. Whitehurst, E. A., "Stabilization of Tennessee Gravel and Chert Bases." HRB Bull. 108, p. 163 (1955).

Addition of lime and lime-fly ash to samples of Tennessee chert and gravel bases resulted in no improvement. Low unconfined compressive strengths and high weight loss in durability tests indicated this conclusion.

56-1. Dawson, R. F., "Special Factors in Lime Stabilization." HRB Bull. 129 (1956).

The cohesiometer was used to evaluate the strength of a clayey gravel stabilized with various amounts of lime. The cohesiometer value increased as lime content was increased to an optimum of about 5 percent lime and then decreased. In general, additional compactive effort and curing time increased the cohesiometer value.

56-2. Goecker, W. L., Moh, Z. C., et al., "Stabilization of Fine and Coarse-Grained Soils with Lime-Fly Ash Admixtures." HRB Bull. 129 (1956).

A laboratory evaluation was made of the effects of lime-fly ash on eight soils from various parts of the country. Primarily, it was a preliminary investigation dealing with mixing time, compactive effort, types of curing, time of curing and resistance to freezing-thawing and wetting-drying. The report is very comprehensive.

56-3. Laguros, J. G., Davidson, D. T., et al., "Evaluation of Lime for Stabilization of Loess." Proc., ASTM, 56:1301 (1956).

This article gives an account of research with various types of lime on loess from Iowa. Test results with lime-stabilized loess were related to the calcium:magnesium ratio of the lime and to the amount of quicklime and hydrated lime. On the basis of compressive strength, quicklime appeared to be more effective with loess than the equivalent amount of hydrated lime, and dolomitic lime was more effective than calcitic lime.

56-4. Minnick, L. J., and Williams, R., "Field Evaluation of Lime-Fly Ash-Soil Composition for Roads." HRB Bull. 129 (1956).

A number of lime-fly ash soil field projects were investigated and studied. Performance was evaluated by physical inspection and by testing of undisturbed specimens removed from the base. Results are compared with those obtained from laboratory studies that involved pozzolanic activity produced at elevated temperatures and subjected to wet-dry, freeze-thaw testing. Recommendations are made as to methods of evaluating lime-fly ash-soil mixtures.

56-5. Muse, W. W., Jr., "Lime Used to Speed Up Paving Operations." Louisiana Department of Highways (1956).

56-6. Weaver, H. C., "Lime Stabilization." Rural Roads (Nov.-Dec. 1956).

Lime was used to stabilize an inferior caliche base material. The strength of the base increased from that of a borderline base material to a good flexible base material upon stabilization with 3 percent hydrated lime.

56-7. Williamson, F., "County Builds Lime-Stabilized Roads." Better Roads, 26:6, 34 (June 1956).

To reduce the loss of gravel on county roads, lime was mixed with the gravel. Construction problems are related. After the first severe winter there was no evidence of base failure.

56-8. Anon., "Lime-Soil Stabilization; Panel Discussion." Rock Products, 59, p. 128 (June 1956).

Condensation of remarks made by members of a lime-soil stabilization panel at the annual meeting of the National Lime Association.

56-9. Anon., "Lime Stabilization Technique Makes Good Base Course in Plastic Soils." Contractors and Engineers (Nov. 1956).

Article briefly describes how lime has been used to stabilize bases on several projects in Louisiana. Highly plastic, fine-grained clays and silts have been transformed into suitable bases with the addition of 3 to 5 percent lime.

57-1. Clare, K. E., and Cruchley, A. E., "Laboratory Experiments in the Stabilization of Clays with Hydrated Lime." Geotechnique, 7:2, 97, London (1957).

A good brief resumé of the work completed on lime stabilization is covered in this article. Although ten clays were investigated, results were similar for all and data are reported for only one highly plastic clay. Effects of lime content on plasticity tests, pH value, suction/moisture and calcium hydroxide content were studied. Work covered theories of action of lime on clays and indicated that neither exchangeable base, base exchange capacity nor liquid limits provide a satisfactory means of determining suitability of soil to be stabilized with lime or what proportion of lime is needed.

57-2. Davidson, D. T., Katti, R. K., and Handy R. L., "Field Trials for Soil-Lime Flyash Paving at Detroit Edison Co., St. Clair Power Plant, St. Clair, Michigan." Unpublished report for Detroit Edison Company (1957).

57-3. Lu, L. W., Davidson, D. T., et al., "The Calcium-Magnesium Ratio in Soil-Lime Stabilization." HRB Proc., 36:794 (1957).

A laboratory study of the effects of various commercial limes and synthetic limes on various soil types. Strength (unconfined compressive and CBR) and durability properties were investigated. For the test procedure used, indications are that dolomitic quicklime gives best results for soil stabilization.

57-4. McAllister, R. W., "Report to the Mississippi Lime Company on Lime Stabilized Highway Construction." Arthur D. Little, Inc. (1957).

A general report that covers briefly the effects of lime on soils and procedures for constructing lime-soil stabilized roads. The major portion of the report deals with actual lime-stabilized soil highway projects in Illinois and Missouri.

57-5. Maclean, D. J., and Clare, K. E., "The Use of Stabilized Soil in Road Construction." Road International, 27:33 (1957/1958).

57-6. Taylor, W. H., Jr., "Stabilizing Organic Fills with Lime in Louisiana." Tech. Bull. 233, American Road Builders' Assn. (1957).

It was desired to stabilize a rather marshy soil that contained large amounts of organic matter and salt. Cement and asphalt were found to be unsatisfactory. Stabilization with lime, though, resulted in a firm subbase through which the underlying soft material did not break through on rolling.

57-7. Schofield, A. N., "Lime Stabilization of a Nodular Clayey Pea-Laterite in Nyasaland." Bull. 3, Road Research Overseas, Department of Scientific and Industrial Research, Harmondsworth, England (1957).

57-8. Viskochil, R. K., Handy, R. L., and Davidson, D. T., "Effect of Density on Strength of Lime-Flyash Stabilized Soil." HRB Bull. 183, p. 5 (1957).

The strength of lime-fly ash soils was greatly increased with increased compactive effort after 7 and 28 days of curing. The optimum lime-fly ash ratio with clays and silts resulted in a decreased percent solids for the same compactive effort and an accompanying decrease in strength.

57-9. Anon., "Lime Subgrade Stabilization on Texas Interstate Projects." Roads and Streets, 100:7, 75 (July 1957).

Lime stabilization of high PI subgrades on a Texas highway project saved 4 to 6 in. of stone usually needed to accommodate heavy wheel loads. Results of laboratory testing of soils as well as construction are covered. An unusual method of scarification aided in producing a more homogeneous mixture. Pulverization was aided by letting the lime rot in the soil for periods up to 48 hr.

57-10. Anon., "Summary Reviews of Soil Stabilization Processes, Hydrated Lime and Quicklime." Report 5, Miscellaneous Paper 3-122, Corps of Engineers, U. S. Army, Waterways Experiment Station, Vicksburg, Miss. (Aug. 1957).

The information presented in this report was obtained from available literature and thus represents the thinking of a number of investigators in the field of lime-soil stabilization. The article indicates that lime is more useful in fine-grained stabilization than in coarse-grained materials, but that reaction of lime with coarse-grained materials may be aided by "adept" material or pozzolans. Calcium chloride, zinc sterate, or salicylic acid may be added to the lime-soil mixture to increase weathering resistance. Indications are that dolomitic-type lime gives the highest strength values when used in stabilization.

57-11. Anon., "Worn-Out Road Rebuilt with Lime Stabilization; Fox Point, Wisconsin." Public Works, 88:100 (Nov. 1957).

A one-block test section of lime-stabilized road was built in Fox Point, Wisconsin. At places it was necessary to add as much as 4 in. of clay-gravel to bring the test section to grade. This material, as well as existing soil, was stabilized by the same construction procedure that is used in standard soil-cement construction except that lime was used instead of cement. Results were satisfactory.

58-1. Brand, W., "Die Bodenstabitisierung mit Kolk." ("Soil Stabilization with Lime.") Strasse und Autobahn (Nov. 1958). (In German).

58-2. Davidson, D. T., et al., "Reactivity of Four Types of Flyash with Lime." HRB Bull. 193, p. 24 (1958).

Increased lime contents resulted in increased strength of lime-fly ash mortar specimens. Time and temperature are two significant factors that influence the pozzolanic reaction. Fly ashes that possess fineness and low carbon content permit an intimate union between the lime and the fly ash and give best results.

58-3. Dawson, R. F., and McDowell, C., "Expanded Shale as an Admixture in Lime Stabilization." HRB Bull. 183, p. 33 (1958).

An expanded shale was used as a pozzolan with lime to stabilize two gravels. Only strengths (compressive and flexural) were investigated and they were improved by using the admixture.

58-4. Eades, J. L., "Progress Report No. 2—Base and Subgrade Stabilization Experiments." Virginia Council of Highway Investigation and Research (July 1958) (For summary see 59-9).

58-5. Gutschick, K. A., "Expedite Construction with Lime Stabilization." Modern Highways (June 1958).

Advantages of using lime to stabilize soils are discussed from the construction viewpoint. The addition of lime to soil has an agglomeration action and a drying out action that help expedite construction.

58-6. Hill, A. D., "Construction Report on Experimental Hydrated Lime Treated Subgrade Sections—Cloud and Jewell Counties." State Highway Commission of Kansas, Research Department (April 1958).

This report covers the construction of two experimental lime stabilization projects in Kansas. A number of conclusions were made in regard to construction techniques concerning handling of lime, mixing, etc. Comparative costs of lime-treated material and untreated base are also given.

58-7. Hoover, J. M., Handy, R. L. and Davidson, D. T., "Durability of Soil-Lime-Fly Ash Mixes Compacted Above Standard Proctor Density." HRB Bull. 193, p. 1 (1958).

Primarily the article reported that high density tends to improve the durability of soil-lime-fly ash mixtures. In addition it was noted that the strengths of these specimens were increased during the wetting-drying test.

58-8. Jones, C. W., "Stabilization of Expansive Clays with Hydrated Lime and with Portland Cement." HRB Bull. 193, p. 40 (1958).

Laboratory tests were conducted on an expansive California clay canal soil that was treated with hydrated lime and with portland cement. Expansion on wetting, shrinkage on drying, as well as strength and durability properties were checked. Lime was superior in improving some of the properties, whereas cement was superior for others.

58-9. Jones, W. G., "Lime-Stabilized Test Sections on Route 51, Perry County, Missouri." HRB Bull. 193, p. 32 (1958).

A number of road test sections were constructed with various types of limes on different types of subgrade soils. After 4 1/2 yr of service, the conditions of these sections were surveyed. This report describes the conditions of the lime sections at the end of that time and offers some tentative conclusions.

58-10. Kelly, A. R., "Low Cost Street Construction with Lime Stabilization." Public Works, 89:113 (Oct. 14, 1958).

This is a report of stabilizing an existing city street with lime. The original street had been composed of bank-run clay-gravel which exhibited high plasticity and high shrinkage. Lime reduced the detrimental effects of the clay and resulted in a greatly improved street.

58-11. Lium, E. L., "Red River Valley Experiments with Lime Stabilization." Better Roads (Nov. 1958).

Three lime stabilization projects were constructed in the Red River Valley on existing gumbo soil. Laboratory testing of the lime-treated soil indicated it would have a high CBR strength. Also, indications were that it would resist disintegration in water.

58-12. Miller, R. H., and McNichol, W. J., "Structural Properties of Lime-Fly-ash-Agregate Compositions." HRB Bull. 193, p. 12 (1958).

The strength of soils is greatly increased by addition of lime and fly ash. This combination is more beneficial than addition of lime only. Results also show that lime-fly ash-aggregate mixtures are a superior base material even before pozzolanic action takes place.

58-13. Nichols, F. P., Jr., "Virginia's Experiments with Lime Stabilization." Tech. Bull. 236, American Road Builders' Assn. (1958).

A general discussion of one of the first lime stabilization projects in Virginia. Some laboratory testing, field construction and field testing are covered.

58-14. Snyder, J. F., "Hydrated Lime—To Improve and Strengthen Subgrade and Flexible Materials." Texas Highways (Oct. 1958).

A very general article which highlights the beneficial use of hydrated lime in highway base stabilization. Specific examples are cited to bear out the claims.

58-15. Unger, A., "Verbesserung von Schluff duch Kolk." ("Improvement of Silt by Lime.") Strasse und Autobahn (Nov. 1958). (In German).

58-16. Van Dine, W. G., "Lime Stabilization Methods and Experiences at Fort Polk, Louisiana." Roads and Streets, 101:54 (Sept. 1958).

Old clay-gravel roads at Ft. Polk, La., were stabilized with hydrated lime to combat wet weather problems. Three standard sections were adopted for use depending on subjected wheel loads. These stabilized roads were surfaced with a double surface-treatment and gave excellent service in spite of regular heavy truck traffic and occasional tank traffic.

58-17. Viskochil, R. K., Handy, R. L., and Davidson, D. T., "Effect of Density on Strength of Lime-Flyash Stabilized Soils." HRB Bull. 183, p. 5 (1958).

This laboratory investigation indicated that the strengths of lime-fly ash stabilized soil are greatly improved by increasing the density and that the lime-fly ash ratio is of little consequence up to 1:9 or 2:8. If the lime content is increased above these ratios, the percentage of solids is decreased and the strength is also decreased.

58-18. Anon., "Initial Laboratory and Field Tests of Quicklime as a Soil-Stabilizing Material." Report 2, Tech. Report 3-455, Corps of Engineers, U. S. Army, Waterways Experiment Station, Vicksburg, Miss. (Aug. 1958).

Laboratory investigations indicated that a loessial soil could be effectively stabilized with quicklime in a relatively short time. However, actual test sections did not indicate this. Construction methods were probably at fault, though, and not the materials.

59-1. Arman, A., "A Study in the Use of Lime in Highway Construction." Louisiana Department of Highways, District 7 Laboratory (Feb. 1959).

59-2. Boynton, R. S., "Lime Stabilizing Airport Runways." Paper to U. S. Air Force Pavements Conf. (July 28, 1959).

The advantages of using lime for the stabilization of plastic soils were discussed. Some information is given on the changes in physical and chemical properties that occur when clayey soils are stabilized with lime. The article also lists and discusses the use of lime in stabilizing bases and subgrades of airfields and roads on several military bases in the south and southwest.

59-3. Brand, W. and Schoenburg, W., "Impact of Stabilization of Loess with Quicklime on Highway Construction." HRB Bull. 231, p. 18 (1959).

Results are given of laboratory testing water sensitive loess with quicklime and subsequent use in actual construction. Not only was strength increased, but by adding lime to the soil that had high water content, construction was able to proceed normally.

59-4. Davidson, D. T., and Katti, R. K., "Activation of the Lime-Flyash Reaction by Trace Chemicals." HRB Bull. 231, p. 67 (1959).

The results of a laboratory investigation in which the effects of small amounts of 47 different chemicals on the strength of Ottawa sand-lime-fly ash mixtures are presented. A theoretical explanation is also offered for the strength improvement produced by the different groups of chemicals studied.

59-5. Kreusel, E., "Lime Stabilization Expedites Construction of SAC Jet Runway." Tech. Bull. 239, American Road Builders' Assn. (1959).

Lime was used to stabilize a clay subgrade during wet weather as an aid to construction. Good construction procedures were developed and reported.

59-6. Leonard, R. J., and Davidson, D. T., "Pozzolanic Reactivity Study of Flyash." HRB Bull. 231, p. 1 (1959).

A basic research on the nature of pozzolanic reaction between lime and fly ash is reported. Various techniques are used. An explanation is also made of the mechanism of the pozzolanic reaction.

59-7. Lund, O. L., and Ramsey, W. J., "Experimental Lime Stabilization in Nebraska." HRB Bull. 231, p. 24 (1959).

The Nebraska Department of Roads in 1956 performed an experiment involving the use of hydrated lime in the stabilization of plastic soils, and in the upgrading of inferior base course materials. The experiment included a preliminary laboratory study and a field construction project. The report summarizes the tests performed

on the various materials before and after adding hydrated lime, and presents the results of deflection measurements at various intervals since the construction of the field project.

59-8. McDowell, C., "Stabilization of Soils with Lime, Lime-Fly Ash and Other Lime Reactive Materials." HRB Bull. 231, p. 60 (1959).

A very general article covering the past history of lime stabilization. It discusses the types of limes used, the chemical reactions that take place and a brief summary of the testing of soil-lime mixtures. It gives some general suggestions for construction procedures and concludes with the benefits of lime stabilization.

59-9. Nichols, F. P., Jr., "Progress Report No. 3—Base and Subgrade Stabilization Experiments." Virginia Council of Highway Investigation and Research (March 1959).

This report gives the progress of the testing and performance of some bases and subgrades stabilized with lime. It summarizes data reported in previous reports (58-4) and gives detailed description of a more recent project. Performance of these test sections have indicated lime and lime-fly ash are generally quite effective for treating subgrades and bases. The degree to which they improve desirable characteristics depends on the soil type. Recommendations are made concerning the use of lime and lime-fly ash as stabilizers in construction.

59-10. Parsons, C. C., "Lime Expedites Construction and Reduces Thickness of Granular Materials Required in Highway Construction." Mississippi Highways (Feb. 1959).

59-11. Anon., "Lime Stabilized Subgrade for Kansas, I Project." Roads and Streets, 102:112 (Feb. 1959).

The addition of lime to a moderately plastic, high swelling clay helped stabilize the subgrade under a portland cement concrete pavement. The stabilized subgrade also tended to aid construction speed because it readily shed water and lost little strength during a rain.

59-12. Anon., "Lime Stabilization Construction Manual." Tech. Bull. 243, American Road Builders' Assn. (1959).

This bulletin thoroughly covers the recommended procedures for construction of lime-stabilized roads. It deals with both the dry and the slurry methods of handling and spreading lime. Related factors such as maintaining traffic during construction, lime safety precautions, etc., are also covered. It is a good publication for personnel actively engaged or closely related to this type of construction.

60-1. Anday, M. C., "Progress Report No. 3—Base and Subgrade Stabilization Experiments." Virginia Council of Highway Investigation and Research (Jan. 1960).

This report presents the results of extensive laboratory and field testings on a recently constructed lime-stabilized subgrade test section in Virginia. This test section consisted of a short section of heavy clay subgrade soils stabilized with 5 percent hydrated lime.

Changes in the physical properties of the soil, as well as an increased bearing value of the subgrade, were noted immediately after construction of the section.

60-2. Davidson, D. T., Mateos, M., and Barnes, H. F., "Improvement of Lime Stabilization of Montmorillonitic Clay Soils with Chemical Additives." HRB Bull. 262 (1960).

Results of laboratory investigations of methods of improving the strength and durability of compacted lime-montmorillonitic soil mixtures were presented. For the testing procedures used dolomitic monohydrated lime improved the immersed strength of the mixture more than high calcium hydrated lime. The effects of sodium hydroxide, sodium phosphate and sodium carbonate on the physical properties of lime-soil mixtures were discussed. Addition of small amounts of sodium hydroxide were found beneficial in accelerating and in increasing the hardness of lime-soil mixture. Sodium phosphate and sodium carbonate were not particularly beneficial in improving lime-stabilized montmorillonitic soils. In addition, the effects of these chemicals on the durability of lime-soil mixtures were presented.

60-3. Eades, J. L., and Grim, R. E., "The Reaction of Hydrated Lime with Pure Clay Minerals in Soil Stabilization." HRB Bull. 262 (1960).

A laboratory study was made of the reactions that occurred when lime was added to eight soil samples of four different types of clay minerals. The study consisted of making X-ray and differential thermal analyses of untreated and lime-treated clays after various curing times. Results indicated that kaolinites, illites, montmorillonites and mixed layered clay minerals all react with lime to give greater bearing strengths and that the quantity of lime required to produce maximum strengths for the different soils varied with the types of clay minerals present. Also, the chemical changes that take place as lime is added to these minerals are recorded.

60-4. Hilt, G. W., and Davidson, D. T., "Lime Fixation in Clayey Soils." HRB Bull. 262 (1960).

Laboratory studies were conducted on lime-soil and lime-fly ash-soil mixtures that contained montmorillonite, illite-chlorite and kaolinite clay minerals. The effects of lime on the strengths and the plastic limits of various types of clay minerals were discussed. Investigations disclosed that the minimum quantity of lime required to increase the strength in some of the clays was equal to the quantity required to increase the plastic limits to a maximum extent. This correlation, however, did not exist for soils containing illite-chlorite clay minerals. The addition of a pozzolan, fly ash, to montmorillonite and kaolinite soils was found unnecessary for strength purposes.

60-5. Ladd, C. C., Moh, Z. C., and Lambe, T. W., "Recent Soil-Lime Research at the Massachusetts Institute of Technology." HRB Bull 262 (1960).

The article contains a summary of 3-yr research on lime-soil mixtures. Investigations were conducted in four fields: the use of quicklime for wet soil stabilization, the use of lime to prevent the erosion of soils, the effects of secondary additives on lime-soil mixtures and the feasibility of using lime-soil stabilization for improving soils in Honduras. Much of the data confirmed previously reported trends. However, new data indicated that the soaked strengths of some lime-soil mixtures were considerably increased by the use of sodium compound additives. Of the compounds investigated, sodium metasilicate was found to be the most effective for the soils studied.

60-6. McDowell, C., "Progress of Lime Stabilization in Texas." Paper presented at the Annual Convention of the National Lime Assn. (May 1960).

Progress in the development and use of lime for the stabilization of soils in Texas is discussed. The quantity of lime used for soil stabilization in Texas has steadily increased during the past 15 yr. In 1959 the quantity of lime used for soil stabilization purposes was in excess of the total quantity of lime produced in Texas in 1945, and the present trend is toward even greater increased use of lime for soil stabilization. Current construction procedures are reported. Also, specifications developed by the Texas Highway Department for the construction of lime-stabilized soils and for the quality of hydrated lime for soil stabilization purposes are included in the article.

60-7. Taylor, W. H., Jr., and Arman, A., "Lime Stabilization Utilizing Preconditioned Soils." HRB Bull. 262 (1960).

Investigations of a few lime-soil stabilized bases in Louisiana indicate that most of the failures have resulted from improper mixing and from delayed compaction after initial mixing (rotting) of the lime and soil. Results of laboratory and field tests indicate that improved stabilization can be obtained by applying one-half the lime to the soil initially and then waiting 30 days and applying the remainder of the lime to the soil. Compaction of the mixtures after the final application of lime was started immediately after mixing.

An Evaluation of Gravels for Use in Lime-Fly Ash-Aggregate Composition

RICHARD H. MILLER and ROBERT R. COUTURIER, Professor of Civil Engineering and Graduate Assistant in Civil Engineering, Villanova University

This report presents the results of a laboratory study of the effects of gradation, hardness, soundness and silt-clay content of gravels when the gravels are used in lime-fly ash compositions. Durability and compressive strength tests are used as the basis for evaluation.

An attempt is made to establish a correlation between the variables and the anticipated field performance of the compositions.

Standard cylindrical specimens (4 in. dia. x 4.59 in. high) were prepared for different natural gravels mixed with optimum percentages of hydrated lime and fly ash. Specimens were also molded using one gravel to which varying amounts of fines were added along with the lime and fly ash. After suitable curing periods tests for unconfined compressive strength, freezing-thawing, durability, and moisture absorption were performed on the specimens.

Seven different gravels from widely scattered geographical areas were used in the investigation. The results have shown considerable variation in the performance of these materials when used in lime-fly ash-aggregate compositions.

● IT IS well known that the percentage and character of the fines (minus No. 200 sieve) in gravels are significant factors in the field performance of gravels for highway use. It is also known that the presence of fines has an effect on the resulting composition when the gravel is used in combination with lime and fly ash. This study was intended to provide qualitative and quantitative data on the effects of fines on the strength and durability of lime-fly ash-gravel compositions. In addition, it was desired to provide more general information by studying the strength and durability of the compositions when gravels from a variety of sources were used.

To make this evaluation, the effects of the following properties of a gravel have been considered: (a) effect of fines (minus No. 200 sieve) on the strength and durability of lime-fly ash-aggregate compositions; (b) effect of particle-size gradation on durability and strength (uniform vs well graded); (c) effect of the hardness and soundness of the individual gravel particles on strength and durability; and (d) effect of particle shape on strength and durability (round vs angular).

For the most part this last effect could not be evaluated because all but one of the gravels used contained well-rounded particles.

The investigation was divided into two parts: (a) the evaluation of the effect of fines on strength and durability of lime-fly ash-gravel compositions; and (b) a general evaluation of gravels, from different geographical locations, for use in lime-fly ash-gravel compositions.

At this time seven different gravels from widely scattered geographical locations have been used.

Each gravel has been completely analyzed and described. Standard cylindrical specimens have been made and tested by standard methods after suitable curing. Unconfined compressive strength, durability, frost susceptibility, and water absorption tests were performed.

MATERIALS USED

Natural Gravel or Sand

Gravels used in this investigation came from seven different areas located in New Jersey, Michigan, Ohio, New York, Illinois and Pennsylvania.

Each gravel has been investigated and identified by means of: (a) grain size—sieve analysis and hydrometer analysis; (b) particle shape; (c) geological origin and classification; (d) Atterberg limits; (e) hardness—using the ASTM Standard Method Designation c-131-55 test, the hardness is evaluated by the total loss of the sample in percent after 500 revolutions of the Los Angeles abrasion machine; and (f) soundness—using the ASTM Standard Method for soundness of aggregates by use of a sodium sulfate saturated solution. The soundness is evaluated as the total loss of the sample in percent after 5 cycles of alternate immersion and drying.

The results are given in Table 1 and the grain size curves are shown in Figure 1. A standard fly ash and hydrated lime were used.

TABLE 1

PROPERTIES OF THE NATURAL GRAVELS

		Sieve Anal (% passing)							Hardness			
Sample Desig.	Pit Location	1 In.	No. 4	No 10	No 200	0.005 In.	Shape	Soundness, % Lost	Sample	% Lost	Uniform. Coeff , D_{60}/D_{10}	Remarks
G-101	N. J.	97	73	72	4	1	Round	8	A	48	5	Sandstone—20% Quartz—50% Quartzite—20% Conglomerate—10%
G-102	Pa.	93	71	62	23	10	⅔ round	14.3	A	54 8	320	Felspathic sandstone (arkosic)
G-103	Ill.	100	73	38	6 6	1 4	Very round	6.15	D	41 8	22	
G-104	Mich.	96	69	48	3 8	1 4	Round	7.2	B	31.5	11	
G-105	Long Island, N Y	100	88	85	3	0 5	Round	4 6	No Test		2 3	Mere sand
G-106	Ohio	96	59	40	3 2	1 4	Round	7	A	26.0	10 7	Limestone—50% Mudstone—50%
G-107	N. J	77	59	53	1.4	—	Round	2 5	A	34	25 4	Arkose—25% Sandstone—25% Quartzite—25% Quartz + silica—25%

Manufactured Gravels

The so-called "manufactured gravels" were prepared by separating a natural gravel into batches and adding to each batch a different percentage of fines (soil passing No. 200 sieve). The percentage of fines that was added was varied in increments of 5 percent. In addition, one batch of manufactured gravel was prepared by removing all the soil passing the No. 200 sieve from the natural gravel. In this manner six different gravels, which varied only in percent of fines, were produced from the one natural gravel. The properties of these gravels are given in Table 2 and the grain size curves are shown in Figure 2.

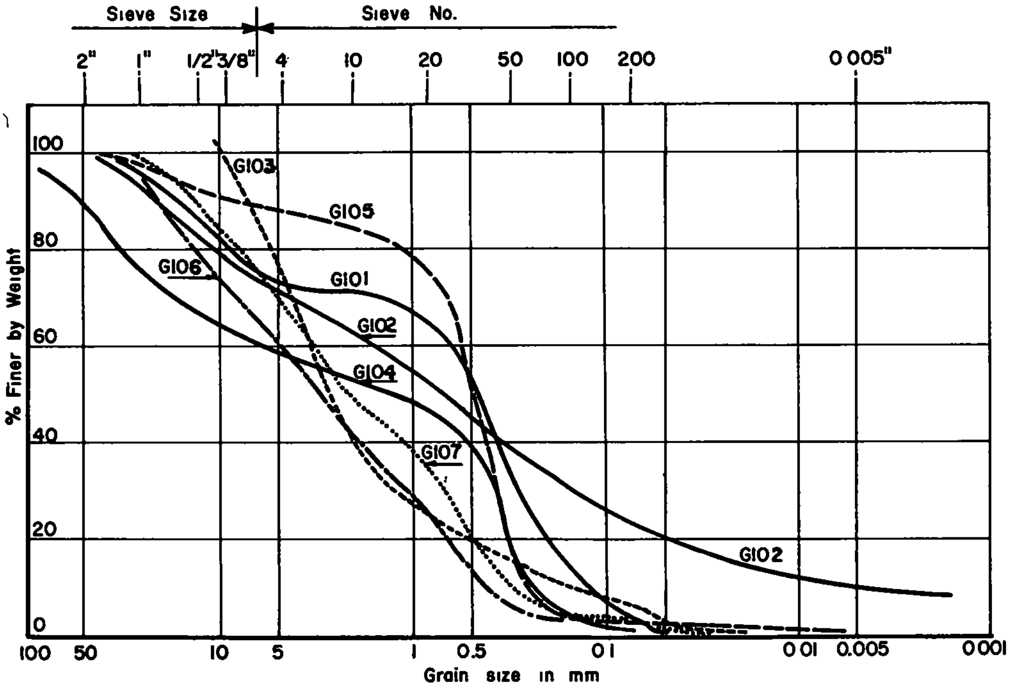


Figure 1. Grain size curve for the natural gravels.

METHODS OF TEST

Mixing

Batches of the lime-fly ash-gravel mixture, in an amount sufficient to make 3 (4 in. dia. x 4.6 in. high) specimens were prepared in the following manner. All mixing of materials was done by hand with the air dried materials (lime, fly ash, gravel) being blended first into a uniform mixture. The water was then incorporated into the mix until it appeared that the water had been evenly distributed. Due to the lack of appreciable amounts of silt and clay, it was a relatively simple matter to get a uniform mix. The amount of water added to each mixture was the one required to produce the maximum dry density under a standard Proctor compactive effort. The optimum moisture content, maximum dry density, and percentages of lime and fly ash in each mixture are given in Table 3.

Molding

Standard Proctor size samples were molded immediately after mixing in accordance generally, with ASTM Designation D-698T (Method A). Variations to this procedure were as follows:

1. Gravel sizes up to ¾ in. or 1 in., depending on the gradation of the gravel, were used in the sample.
2. Samples for the evaluation of the natural gravels were molded using different compactive efforts. One set of samples was made using standard AASHO compaction and one set was made using a compaction intermediate between standard and modified. This intermediate compaction consisted of 3 layers; 25 blows per layer of the 10 lb rammer dropped 18 inches.

Twenty-four samples were made using each natural gravel; 12 samples by standand compaction and 12 by intermediate compaction.

TABLE 2

PROPERTIES OF MANUFACTURED GRAVELS

Sample		Sieve Anal (% passing)					Uniform. Coeff , D_{60}/D_{10}	Atterberg Limits		USBPR Class.
Desig.	Manu-facturing	1 In.	No. 4	No. 10	No 200	0 005 In.		L L	P I	
N. J.	New Jersey gravel	77	59	53	1.4	–	25 4	0	N. P	A-1-b
N J 0	New Jersey sieved on No. 200	76.7	58 8	52.7	–	–	23	0	N. P.	A-1-b
N. J 5	New Jersey + 5.5% overbd	78	61 5	56	6 5	1 6	25	0	N. P.	A-1-b
N. J 10	New Jersey + 11% overbd	79.4	63.7	58 5	11 7	3 3	58	0	N P.	A-1-b
N J 15	New Jersey + 16.5% overbd	81	66	61	16 9	5	106	0	N P.	A-1-b
N. J 20	New Jersey + 22% overbd	82 3	68.1	63.5	22 1	6 6	142	0	N. P.	A-1-b
N. J 25	New Jersey + 27 5% overbd	83.7	70.2	66	27	8	146	0	N. P.	A-2-4

Note: USBPR Class. = U. S. Bureau of Public Roads Classification.
Overburden (overbd.) = selected overburden material used to add fines to the natural gravel.

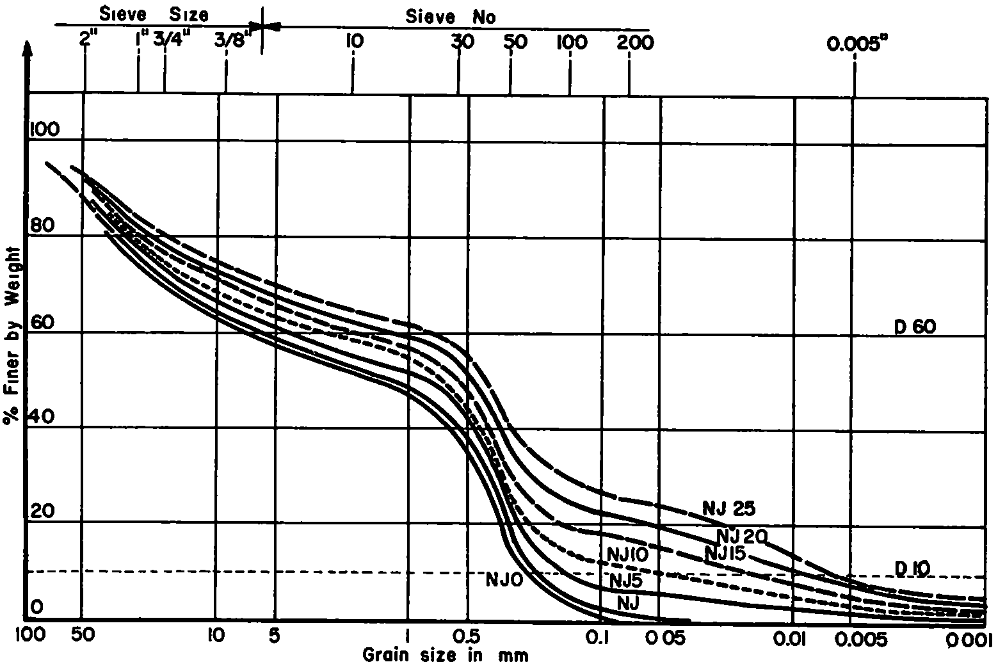


Figure 2. Grain size curve for the manufactured gravels.

Fifteen samples were made for each of the six manufactured gravels. In this case, however, only the standard AASHO compaction was used.

Curing

The following procedure was used for curing test samples:

7 Days Accelerated Cure.—Each specimen after molding was placed in a sealed container. The container was then placed for 7 days in an oven at 130 + 3 F after which period the specimen was removed from its container and submerged for 24 hr in water. The specimen was then tested within 1 hr after removal from water.

28 Days Moist Sand Curing or Beam Box Curing.—After molding, the specimens were placed for 28 days on a 1-in. layer of moist sand, surrounded by sand and covered by another 1-in. layer of moist sand. The sand was moistened daily by approximately 3/8 gal per sq yd. After the 28-day period the specimens were removed, submerged for 24 hr in water and then tested within 1 hr after removal from water.

TABLE 3

EVALUATION OF NATURAL GRAVELS — TEST RESULTS

Sample	Mixture Composition			Dry Density (pcf)						Comp Strength (psi)				Frost Suscp (% wt loss)		% Water Absorption	
				10-Lb Hammer			5 5-Lb Hammer			10-Lb Hammer		5 5-Lb Hammer				Absorbed Water/Oven Dry Wt (%)	After 12 F -T Cycles
Desig	Moist Cont (%)	Lime (%)	Fly Ash (%)	Max	Min	Avg	Max	Min	Avg	A C	B B	A C	B B	10-Lb	5 5-Lb		
G-101	8	4	10	133	130	133	125	123	124	550	180	460	165	66	52	15	13
G-102	10	4	15	120	115	117	119	113	115	570		350	330	8 0	100	15	16
G-103	6 8	4	10	140	135	136	138	133	136	1,300	820	1,300	830	1 5	3	4	7
G-104	7 7	3	11	138	135 5	137	138	133	135	1,280	710	1,380	680	2	3	4 5	7 5
G-105	8 5	4	15	128	124	126	126	120	122 5	1,125	380	1,000	340	14	17	7	11
G-106	7 5	4	12 5	138	135	136	134	129	132	1,600	810	1,700	1,100	1 4	2 8	8	7
G-107	8 1	4	12	132	129	130 5	131	129	130	1,400	475	1,340	465	7 9	8 3	5 4	7 8

Note: F -T = freezing-thawing cycle A C = 7-day accelerated cure 10-Lb or 5 5-Lb = hammer used for compaction
B B = 28-day moist sand curing (beam box) Percentages of lime and fly ash are by weight

Evaluation

Unconfined Compressive Strength.—The test specimens were tested in accordance with ASTM Designation C-39-56T, using a Universal Hydraulic Testing Machine. At least three cyclinders were used for each compressive strength value given in this report. All specimens were capped before they were broken.

Freezing-and-Thawing Test.—1. Frost Susceptibility—Those specimens that were moist cured for 28 days were subjected to 12 cycles of freezing and thawing as set forth in that porttion of ASTM Designation D-560 pertaining to the weight loss specimen. This test was performed where both the natural gravels and the manufactured gravels were used in the mixture. 2. Durability—Those specimens that were cured using the 7-day accelerated cure were subjected to 12 cycles of freezing and thawing as set forth in that portion of ASTM D-560 pertaining to the weight loss specimen. Only the mixtures containing the manufactured gravels were subjected to this test.

Water Absorption.—1. Test samples that were cured by the 7-day accelerated cure were air dried to constant weight and placed on ¼ in. thick absorbent pads. Free water was made available at the base of the pad. The specimens were allowed to absorb water until they achieved a constant weight. 2. The absorption of water during the freeze-thaw tests after 12 cycles was also observed and recorded.

TABLE 4

EVALUATION OF "MANUFACTURED" GRAVELS — TEST RESULTS
USING ONLY THE 5 5-LB HAMMER FOR COMPACTION

Sample	Mixture Composition			Dry Density (pcf)			Compress Strength (psi)				Durability (% wt loss)	Frost Suscp (% wt loss)	Absorption (%)	
									After 12 F -T Cy				Absorbed Water/Oven Dry Wt (%)	After 12 F -T Cycles
Desig	Moist Cont (%)	Lime (%)	F A (%)	Max	Min	Avg	A C	B B	A C	B B				
N J	8 1	4	12	131	129	130	1,340	465	1,130	520	3 8	8 3	5 4	7 8
N J 0	7 6	4	12	129	125 8	127 5	1,000	340	1,000	350	8 0	10	6 1	8 2
N J 5	9	4	12	132 6	130	131 3	1,100	430	875	455	4	2.7	4 5	7
N J 10	9 7	4	12	129	126	128	1,030	320	850	160	4	35	5	10 5
N J 15	10 2	4	12	129	126	126 8	950	285	800	56	4 5	70	5 8	11 6
N J 20	10 9	4	12	127	124 7	125 4	905	225	720	48	4 6	100 After 12 F - T	6 5	12 5
N J 25	11 7	4	12	123 2	121 5	122 5	820	220	740	—[a]	10 5	100 After 12 F - T	10	13 5

Note F-T-C = Freezing-thawing cycle A C = 7-day accelerated cycle
B B = 28-day beam box cure [a] = failed

Number of Tests.—Tables 5 and 6 give the type of test used for evaluation and the number of specimens involved.

TEST RESULTS

Test results pertaining to (a) unconfined compressive strength, (b) frost susceptibility, (c) curability, and (d) water absorption are summarized in Tables 3 and 4.

Table 3 contains the results of the evaluation of the natural gravels and Table 4 contains the results of the evaluation of the manufactured gravels.

Discussion

Evaluation of Natural Gravels.—An analysis of the results of this part of the investigation shows that the following variables or combination of variables affect the performance of lime-fly ash-gravel mixtures: (a) gradation of the gravel (uniformity and percent fine material), (b) hardness of the gravel, and (c) soundness of the gravel.

With the amount of data that were acquired in this study it was not possible to determine an exact relationship between the aforementioned variables and the laboratory performance of the mixtures. However, indications of their effects were rather clear.

As an example, Figure 3 shows that the highest strengths (both cures) were achieved with gravel G-106. G-106 was a well-graded sandy gravel with a low percentage of

TABLE 5

SCHEDULE OF EVALUATION TESTS — LIME-FLY ASH-NATURAL GRAVEL

Test	Curing Time (days)	Samples Tested, Each Mix.	Data Obtained
Unconf. compr. str.	7 28	3 3	Unconf. compr. str.
Frost suscept.	28	3	% Wt. loss, water abs.
Water absorp.	7	3	Water absorbed

TABLE 6

SCHEDULE OF EVALUATION TESTS — LIME-FLY ASH-MANUFACTURED GRAVEL

Test	Curing Time (days)	Samples Tested, Each Mix.	Data Obtained
Unconf. compr. str.	7 28	3 3	Unconf. compr. str.
Frost suscept.	28	3	% Wt. loss, water abs. compr. str. after freeze-thaw
Durability	7	3	% Wt. loss, water abs. compr. str. after freeze-thaw
Water absorp.	7	3	Water absorbed

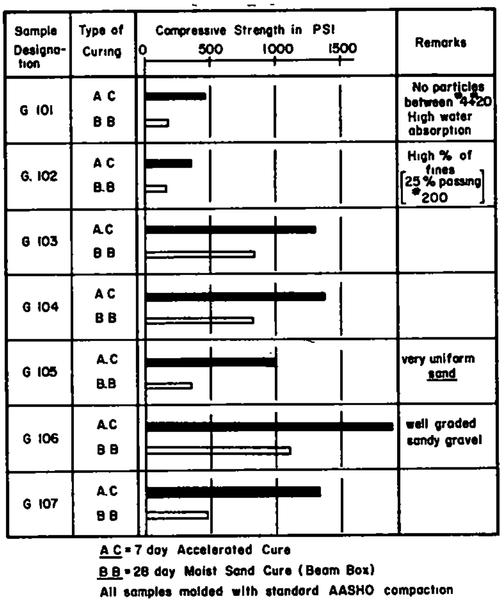


Figure 3. Compressive strength of lime-fly ash-gravel compositions using natural gravels.

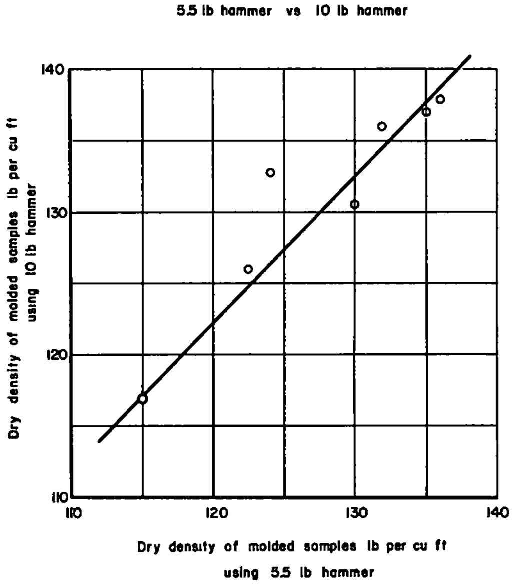


Figure 4. Variation in molded density of test sample using natural gravels.

fines (3.2 percent passing the No. 200 sieve). It also had the lowest percentage of loss during the hardness test. On the other hand, the lowest strengths were recorded for the lime-fly ash-aggregate compositions using gravel G-102. This gravel contained the highest percentage of fines (about 23 percent) and had the highest losses in the hardness and soundness tests.

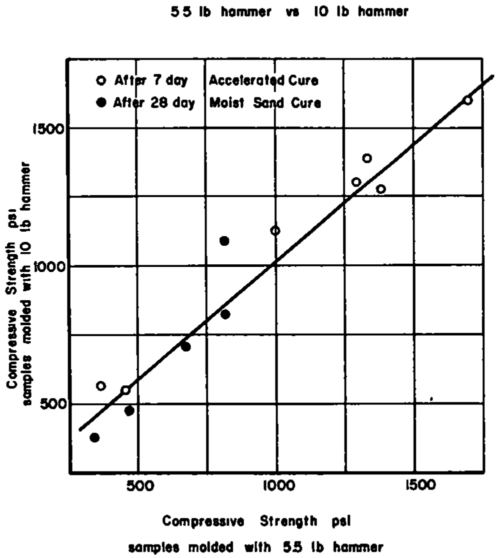


Figure 5. Variation in compressive strength of test sample using natural gravels.

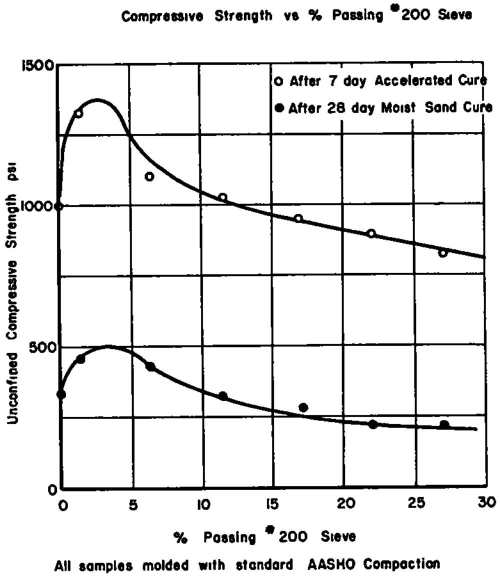


Figure 6. Variation in compressive strength of test sample using manufactured gravels.

The other evaluation tests showed results that were consistent with those of the unconfined compression test. Referring to Table 3, it can be seen that mixtures that gave high compressive strengths also had low weight losses during frost-susceptibility tests, and low water absorption. The water absorption test showed considerable promise as a "yardstick" for measuring laboratory performance. In every case where the percent water absorbed remained below the optimum or molded moisture the compressive strengths were high and the losses during frost-susceptibility tests were low. In two of the three cases (G-101, G-102, G-106) where the percent water absorbed exceeded the optimum or molded moisture content, the compressive strengths were substantially lower and the frost-susceptibility losses were high.

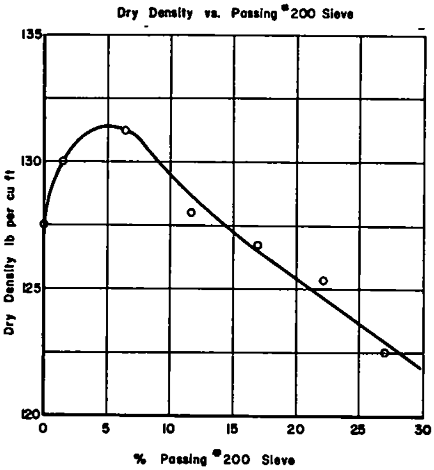
Figure 4 shows the relationship between the densities obtained by compacting with a 10-lb hammer—18-in. drop, and those obtained by compacting with a 5.5-lb hammer—12-in. drop. The greater compactive effort gave densities that ranged from 0.5 to 9 pcf higher than those with the 5.5-lb hammer. The effects of heavier compaction on compressive strength were found to be small. This was particularly true of the mixtures.

Figure 5 shows the close equivalence in compressive strengths.

Evaluation of Effect of Fines (Percent Passing No. 200 Sieve) on Lime-Fly Ash-Gravel Mixtures.—As the percentage of fines was varied in the "manufactured" gravels there was a substantial variation in the compressive strengths. This effect is shown in Figure 6. For this particular gravel there appears to be an optimum percentage of fines with regard to compressive strength. This optimum is between 3 and 5 percent of fines. As the percentage of fines in the gravel was increased beyond this optimum, the compressive strength decreased. This trend was shown in samples that were cured by the 7-day accelerated cure and by those that were cured by the 28-day moist sand cure. The unconfined compressive strengths of the samples that were cured by the moist sand cure averaged about 30 percent of the strength of the accelerated cure samples.

In Figure 7, the dry density of samples compacted with the 5.5-lb hammer is compared with the percent of fines in the gravel. This curve is similar in appearance to the curve of Figure 6. The maximum dry density was achieved with the gravel that contained about 4 percent of fines and a continuous decrease in density was obtained with an increase in fines beyond this amount.

The concept of an optimum percentage of fines appears to be substantiated by the results of the frost susceptibility tests as shown in Figure 8. The weight loss of the wire-brushed samples (shown as frost susceptibility—percent weight loss) during the freeze-thaw test is shown in this figure as a function of the percentage of fines. The



all samples molded with standard AASHO Compaction

Figure 7. Variation in molded density of test sample using manufactured gravels.

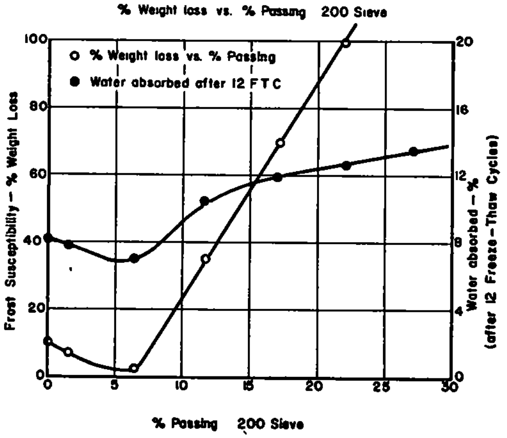


Figure 8. Frost susceptibility—percent weight loss using manufactured gravels.

smallest loss occured with the gravel that contained about 5 percent of fines and the amount of loss increased with an increase in fines content. The water absorbed vs percent passing the No. 200 sieve is also shown in Figure 8. The trend of this curve is similar to the weight loss curve. Again it can be pointed out that the water absorption test gives the same information that is obtained in the compressive strength and durability tests. Further study of the use of this test in evaluating lime-fly ash-aggregate mixtures is warranted.

CONCLUSIONS

This investigation has shown that variations in physical properties of gravel particles and gravel aggregates do affect the laboratory performance of lime-fly ash-gravel compositions. Certain generalizations can be made regarding these properties. Although these generalizations should be verified by additional data, the present study has indicated the following:

1. The compressive strengths of samples made with a 5.5-lb hammer were essentially the same as those made with a 10-lb hammer. This was true even though the densities with the lighter compaction were somewhat lower (2 to 4 pcf for the better gravels).
2. There are indications of a correlation between the performance of a gravel in a lime-fly ash composition and the properties of the gravel itself. The best results from the standpoint of compressive strength and durability were achieved with those gravels that had: (a) a low percentage of fines; (b) a relatively low weight loss in hardness and soundness tests; and (c) a good gradation of particle sizes.
3. Mixtures that had high compressive strengths in samples that were cured by the 28-day moist sand cure also showed low losses in the frost susceptibility test and a low water absorption.
4. This study has shown that, for the particular gravel that was evaluated, optimum results were obtained when the percentage of fines was approximately 4 percent. Optimum results were indicated by relatively high compressive strengths and densities, low losses in durability tests, and a low water absorption.
5. Most of the compressive strength values show a proportional increase with an increase in the dry density of the compacted sample.
6. The use of a water absorption test for evaluating lime-fly ash-aggregate compositions shows promise. The data acquired by this test show the same trend as the data from the compressive strength and durability tests. The simplicity of the water absorption test would also make it desirable.

THE NATIONAL ACADEMY OF SCIENCES—NATIONAL RESEARCH COUNCIL is a private, nonprofit organization of scientists, dedicated to the furtherance of science and to its use for the general welfare. The ACADEMY itself was established in 1863 under a congressional charter signed by President Lincoln. Empowered to provide for all activities appropriate to academies of science, it was also required by its charter to act as an adviser to the federal government in scientific matters. This provision accounts for the close ties that have always existed between the ACADEMY and the government, although the ACADEMY is not a governmental agency.

The NATIONAL RESEARCH COUNCIL was established by the ACADEMY in 1916, at the request of President Wilson, to enable scientists generally to associate their efforts with those of the limited membership of the ACADEMY in service to the nation, to society, and to science at home and abroad. Members of the NATIONAL RESEARCH COUNCIL receive their appointments from the president of the ACADEMY. They include representatives nominated by the major scientific and technical societies, representatives of the federal government, and a number of members at large. In addition, several thousand scientists and engineers take part in the activities of the research council through membership on its various boards and committees.

Receiving funds from both public and private sources, by contribution, grant, or contract, the ACADEMY and its RESEARCH COUNCIL thus work to stimulate research and its applications, to survey the broad possibilities of science, to promote effective utilization of the scientific and technical resources of the country, to serve the government, and to further the general interests of science.

The HIGHWAY RESEARCH BOARD was organized November 11, 1920, as an agency of the Division of Engineering and Industrial Research, one of the eight functional divisions of the NATIONAL RESEARCH COUNCIL. The BOARD is a cooperative organization of the highway technologists of America operating under the auspices of the ACADEMY–COUNCIL and with the support of the several highway departments, the Bureau of Public Roads, and many other organizations interested in the development of highway transportation. The purposes of the BOARD are to encourage research and to provide a national clearinghouse and correlation service for research activities and information on highway administration and technology.