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(As of December 31, 1963)

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Foreword

Highway engineers have long searched for materials and methods to treat clayey subgrade soils to improve their strength and to lessen their susceptibility to change in volume and strength during cycles of wetting and drying and freezing and thawing. This publication, composed of three technical papers from the Board's 43rd Annual Meeting and four papers from the 44th Annual Meeting, reports on field and laboratory studies involving the use of lime and cement as means for improving the properties of clayey soils. One report also includes experience with phosphoric acid.

The lead-off paper reports on the effect of freezing and thawing on the compressive strength of soil-lime mixtures. Especially interesting are the results of freeze-thaw cycles as they affect compressive strength and the modulus of deformation (stress ÷ strain at maximum stress) for different lime contents.

Another paper discusses an investigation of the effects of hydrated lime on a highly plastic clay (with and without the further addition of small proportions of sodium hydroxide, sodium orthosilicate and calcium chloride) on the strength, consolidation, durability, permeability, and elastic properties of the mixtures. It compares the results with those obtained on the same soil treated with portland cement.

Experiences with an experimental road test that employed Type 1 portland cement, lime and cement, and lime-fly ash to treat plastic Iowa soils of the A-6 and A-7 groups are outlined in another report. It describes prescribed construction controls and evaluates the controls in terms of how well each was satisfied during construction. In addition, the effect of some of the controls on the behavior of the stabilized areas is discussed.

Missouri's first experience with lime stabilization was in 1952. Ten years later extensive studies on full-scale field experimental projects were conducted in two counties of the state. These field projects, though, were preceded by extensive laboratory testing on mixtures of soil with lime and phosphoric acid. During this time, a short pilot section was constructed to determine the practicability of the materials' use. A two-part comprehensive report on the experiments is given in this Record. The first discusses the lime stabilization projects; the second views the design, construction and performance phases of the 16 test sections that comprised one project.

One of the most difficult problems regarding soil stabilization involves the use of laboratory tests that will evaluate laboratory results in terms of the properties inherent in a stabilized soil. Because tensile strength is one such property of significance in pavement design, another paper published here is important because it explores the tensile strength of lime-stabilized soils by means of the "split test." The author develops the theoretical solution for the test, evaluates the method and correlates it with other test methods.

A material may be used successfully in engineered construction of stabilization when only its mechanical properties are understood. When knowledge is expanded to include understanding of the physico-chemical and chemical mechanisms that result in the stabilizing effect, progressive improvements in stabilization usually require much less time. One paper reviews current knowledge and explains the mechanisms that provide the stabilizing effect on soils in terms that should be of interest to engineers as well as chemists.

Any soil stabilizer that is to be effective must be thoroughly mixed with the soil. And thorough mixing is dependent in large measure on the degree of pulverization of the soil before mixing and compaction. In the final paper, the authors discuss their experiments to determine the influence of different degrees of preliminary pulverization on the properties of soil-lime mixtures resulting from migration of calcium and magnesium ions into lumps of soil Their results are presented in terms of depth of penetration of calcitic lime into the soil-water system and the relation of that depth of penetration to time.

Contents

Effect of Freezing and Thawing on Unconfined Compressive Strength of Lime-Stabilized Soils

RICHARD D. WALKER and CETIN KARABULUT
Respectively, Associate Professor of Civil Engineering and Graduate Student, Virginia Polytechnic Institute

The effect of a closed-system freezing-and-thawing test on the confined compressive strength of a lime-stabilized silt and a clay soil was the object of this study. The clay soil was classified as A-7-5(18) and the silt soil as A-5(8). The percentages of hydrated lime used were 0, 5 and 10 by dry weight of soil. All specimens were wrapped in aluminum foil, sealed with paraffin, and cured for 48 hr at 120 F.

In one series of tests, the specimens were frozen in a standard deep-freeze unit to a temperature of 0 F and then thawed in a 100 percent humidity room at 70 F. All specimens were kept sealed in aluminum foil until tested in unconfined compression. The compression tests were run at the end of 0, 5, and 10 cycles of freezing and thawing. In a second series of tests, automatic equipment was used to freeze and thaw specimens alternately between 0 and 40 F. These specimens were tested at the end of 29 cycles of freezing and thawing.

Results showed that the strength of clay soil was greatly increased by the addition of lime but suffered large decreases in strength when exposed to freezing and thawing. The strength of the silt soil was affected very little by the addition of lime or by alternate cycles of freezing and thawing. With the untreated soils, the clay lost most of its strength after freezing and thawing, and the silt lost little of its strength.

Attention is given to the possible role of an activity similar to that described by Powers' hypothesis for concrete, taking place in the stabilized soil during freezing and thawing. A discussion of closed-system and open-system types of freezing-and-thawing tests is also included.

•IN VIRGINIA, as well as in other parts of the country, increasing quantities of hydrated lime are being used to improve subgrade materials under primary highways. Improvement of subgrade strength is becoming recognized as one of the most important factors in a successful pavement design.

Although results of previous investigations have shown that lime changes the plasticity and increases the unconfined compressive strength, the durability of lime-soil mixtures when exposed to freezing and thawing has not been studied to a very great degree. Those previous studies that have been accomplished have indicated that lime-soil mixtures may be susceptible to deterioration due to freezing and thawing. Thus, if a pavement design is to be based on an improved strength value, the expected loss in strength due to freezing and thawing must be known.

The effect of a closed-system freezing-and-thawing test on the unconfined compressive strength of a lime-stabilized silt and clay soil was the main object of this study.

Paper sponsored by Committee on Lime and Lime-Fly Ash Stabilization and presented at the 43rd Annual Meeting.

A closed-system test prevents the ingress and egress of moisture and differs from the more common freezing-and-thawing tests where the specimen is permitted to draw up water by capillarity and, thus, often fails by ice segregation within the specimen. The closed-system test might be more appropriate, especially for plastic clays of low permeability, where the amount of water that could be brought up by capillarity would be very small.

An attempt is made to show how the freezing-and-thawing process in lime-stabilized soils might be similar to that in concrete as described by Powers' hypothesis (8). This would be a destructive process caused by hydraulic pressures rather than by ice segregation.

FREEZING AND THAWING TESTS

Open and Closed Systems

It is generally believed that detrimental effects of freezing and thawing on soils are caused by the formation of segregated ice or ice lenses. Since the belief has been prevalent that soils in the field generally behave as open systems, most freezing-and-thawing studies have been made with specimens in contact with water and with moisture absorption allowed.

Soil systems can be classified as open or closed systems. An open system is the one which has a water supply from outside. A closed system has only the water within itself and water is not gained or lost. Tabor (10), thinking that very impermeable soils behave as closed systems, conducted laboratory tests on clays containing bentonite. He found that segregation of ice does not occur in closed systems.

Freezing and Thawing of Concrete

Powers (8) developed a theory to explain the freezing-and-thawing action in concrete. Concrete is believed to act somewhat as a closed system. Powers' hypothesis rests mainly on the premise that the destruction of concrete by freezing is caused by hydraulic pressure generated by the expansion accompanying freezing of water rather than by direct crystal pressure developed through growth of bodies of ice crystals. If the destructive action of freezing is due to hydraulic pressure, the resistance to movement of water must be the primary source of pressure, since all concrete contains enough air-filled space to accommodate water-to-ice expansion. The intensity of the hydraulic pressure developed during the freezing depends on degree of saturation, pore size, and permeability characteristics.

By using the test data of this study and by deductive reasoning, an attempt is made to rationalize this concept to lime-stabilized soils.

Testing of Stabilized Soils Exposed to Freezing and Thawing

The most commonly used tests fall in one of the following three categories:

1. Loss in weight of specimens which is produced by brushing;
2. Percent change in unconfined compressive strength; and
3. Change in velocity of pulse propagation.

There are several objections to the brushing test. It can generally be used only on well-stabilized granular materials. Also, the results depend somewhat on the amount of pressure applied by the brusher and on the condition of the brush. The unconfined compression test has the advantage of being relatively free from operator influence and has been used extensively by Davidson (4), Whitehurst and Yoder (13), and several others. Even though pulse velocity measurements have been used for many years, the method is still in the experimental stage and is not as accurate as the compression test. However, it has the advantage of being a nondestructive test.

The unconfined compression test is used exclusively in this study.

Freezing-and-Thawing Durability of Lime-Stabilized Soils

Probably most of the work on the freeze-thaw durability of lime-stabilized soils has been done at the Iowa Engineering Experiment Station (4, 5, 7). Yoder, at Purdue University, has also done considerable work in this field (13, 14). General conclusions of this past work can be listed as follows:

1. Soil texture appreciably affects durability;
2. Increase in percent lime generally increases soil durability; and
3. Durability is increased by longer curing before freeze-and-thaw cycles.

Importance of Freeze-Thaw Durability in Southwest Virginia Area

Southwestern Virginia is generally located in the Ridge and Valley and Appalachian Mountain physiographic provinces. The climate is severe from the standpoint of the number of alternate cycles of freezing and thawing, and plastic clay residual soils are in abundance. The Virginia Department of Highways is using lime to stabilize these soils on all important highway projects and, thus, it would seem to be important to discover how permanent the strength gains are.

All of the previous research cited dealt with open-system tests on granular to silty-clay soil textures. Little or no attention has been given to the more impermeable plastic clay which may act more as a closed system.

MATERIALS

Soil

The clay soil was reddish-brown in color with a liquid limit (L. L.) of 77, a plastic limit (P. L.) of 53, and a plasticity index (P. I.) of 24. It had about 59 percent finer than 0.002 mm and was classified according to the AASHO system as A-7-5(18). The silt soil was brown with plasticity characteristics as follows: L. L. = 41, P. L. = 32, and P. I. = 9. It had about 31 percent finer than 0.002 mm and was classified according to the AASHO system as A-5(8).

Lime

The hydrated lime used in all tests was manufactured by the Ripplemead Lime Co. X-ray analysis of the lime from the bag used after all tests were completed showed that the $CaCO_3$ content was as much as 60 percent. Since all the tests were run within a relatively short period of time, this should not have had significant effect on the variability of the results. However, it does point out the importance of checking the lime source for percent of calcium hydroxide.

LABORATORY TEST PROCEDURES

Density Tests

Density tests were performed on air-dry samples with the Harvard miniature compaction apparatus. Experience had shown that this device loaded with a 40-lb spring with 25 blows on each of three layers of soil gave similar results to those obtained by the standard AASHO procedure. Moisture-density curves were obtained for both soils, containing 0, 5 and 10 percent of lime, to determine optimum moisture content for each condition.

Fabrication of Specimens

All specimens were compacted using the Harvard miniature apparatus. The samples were molded at the optimum moisture contents of the soils for 0, 5 and 10 percent of lime. All specimens after compaction were extruded from the mold, immediately wrapped with aluminum foil, and coated with paraffin to preserve the molding moisture content during curing and freezing and thawing.

Curing

All specimens were cured for 2 days at 120 F. This procedure was developed by Anday (1) and is roughly equivalent to 60 to 65 days of simulated field curing during a summer at Charlottesville, Va.

Freezing and Thawing

Two different methods of freezing and thawing were tried.

1. Four specimens were prepared for each of the 0, 5 and 10 percent combinations of lime with silt and clay, giving a total of 24 specimens. All specimens after 2 days of curing were placed in a copper container (3 by 3 by 15 in.) with about ⅛ in. of water in the bottom and the container was placed on a freezing plate. (The purpose of the water was merely to aid in conducting heat in and out of the specimens.) The specimens then underwent 29 cycles of freezing and thawing at a rate of 8 cycles per day. The temperature in adjacent cans which contained concrete specimens alternated from 0 and 40 F. The automatic equipment was designed for the freezing and thawing of concrete specimens according to ASTM Designation: C 290 and is described in an article by Cordon (3).
2. Four specimens were prepared for each combination of lime with silt and clay. After 2 days of curing, the specimens were frozen in a deep-freeze unit for 8 hr and thawed in a moist room at 70 F and 100 percent RH for 16 hr. (The main purpose of the moist room was to afford constant temperature.) Half of the specimens were subjected to 5 cycles of freezing and thawing and the other half to 10 cycles.

Twenty-four additional specimens were fabricated as controls and were tested after the 2-day curing period without undergoing freezing and thawing.

Unconfined Compressive Strength Tests

The unconfined compressive strength of all specimens was determined using a loading rate of 0.05 in./min. Altogether, 72 specimens were prepared for testing. The maximum compressive strength was determined, as well as a secant modulus which was determined by dividing the strain at the time maximum stress was reached into maximum stress.

RESULTS

Standard Atterberg limit tests and Harvard miniature compaction tests were run on both soils with lime at 0, 5 and 10 percent. The results of these tests are given in Table 1. The results obtained are similar to those obtained by other researchers in that the P.I. of the plastic soil was greatly reduced with the addition of lime and that of the silt was not. Also, both soils showed a reduction in optimum density and an increase in optimum moisture content with an increase in lime content.

Figures 1 and 2 and Tables 2 and 3 present the results of the unconfined compressive strength tests for all the specimens in this durability study.

TABLE 1

ATTERBERG LIMIT AND DENSITY TEST RESULTS

Soil	Lime (%)	L. L.	P. L.	P. I.	Opt. Moist. Cont. (%)	Opt. Dry Density (pcf)
Clay	0	77	53	24	32	89
	5	70	55	15	35	85
	10	67	56	11	36	83
Silt	0	41	32	9	17	103
	5	42	33	9	22	99
	10	43	34	9	23	97

DISCUSSION OF RESULTS

Effect of Lime on Soil Strength

Figure 1 and Tables 2 and 3 illustrate the great increase in compressive strength of the plastic clay by addition of lime. With the silt, the compressive strength was slightly decreased. However, Table 3 indicates that the secant modulus did increase when lime was added to the silt. Thus, although total capacity is not increased, the slope of the stress-strain

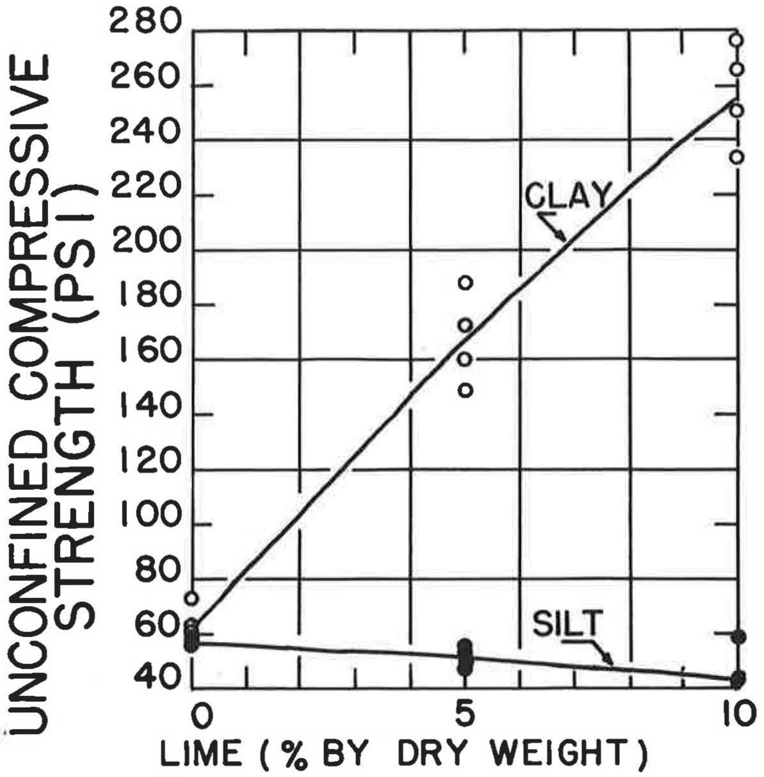

Figure 1. Strength vs percent lime in clay and silt.

curve was made steeper. Apparently the cementing action generated in the silt was broken down at a relatively low compressive stress. Both the secant moduli and compressive strengths were considerably higher for the clay-lime than for the silt-lime mixtures.

The significance of the increase in secant modulus for the silt-lime mixtures cannot be overemphasized. Whatever the cause of the increase (perhaps a weak $CaCO_3$ cement), it can result in lower pavement deflections, providing a certain stress is not exceeded. Heretofore, most silts were not considered likely subjects for lime stabilization, but actually great benefits may be derived. Anything that can reduce deflections in a flexible pavement is bound to be beneficial.

Effect of Freezing and Thawing on Soil-Lime Mixtures

The comparison of the unconfined strengths after freezing and thawing among the various combinations of soil-lime mixtures can be made from Figure 2 and Table 2. In the clay-lime mixtures, all specimens showed a drastic decrease in strength after being exposed to 5 cycles of freezing and thawing. There was no appreciable difference between the strengths after 5 and 10 cycles, indicating that the loss in strength mainly occurred in the first few cycles. In the silt-lime mixtures, initial strength was low and freezing and thawing produced at the most only slight decreases in strength. In some cases, strength was higher after 10 cycles of freezing and thawing; this could be due to additional curing of the specimens.

The effectiveness of lime in resisting the effects of freezing and thawing can be compared by using a resistance value similar to that proposed by Davidson et al. (4):

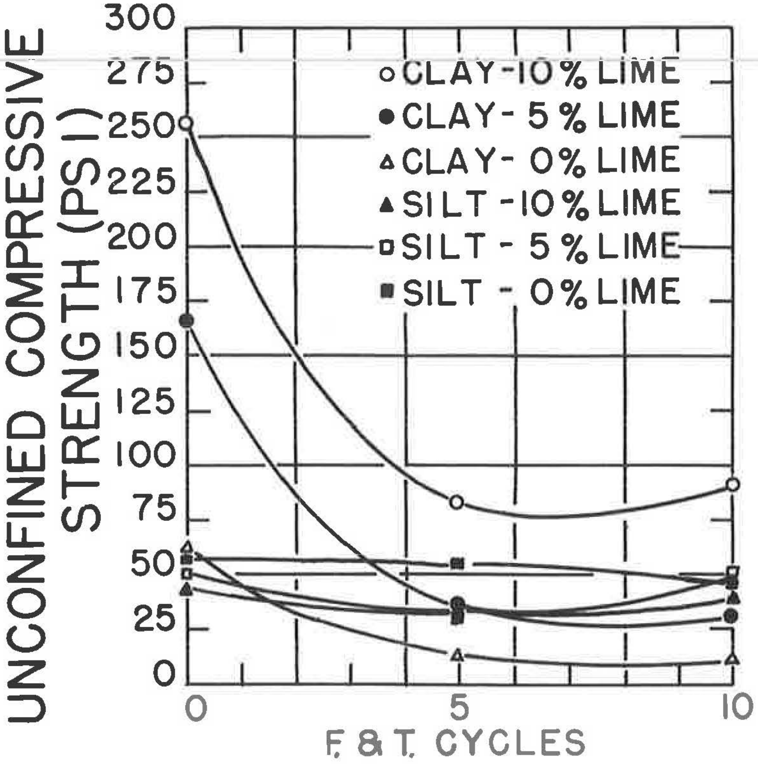

Figure 2. Strength vs freezing-and-thawing cycles, lime-soil mixtures.

$$R_f = p_f/p_o \tag{1}$$

where

R_f = resistance factor,
p_o = compressive strength at 0 cycles of freezing and thawing, and
p_f = compressive strength after 5 or 10 cycles of freezing and thawing.

Resistance values calculated from this equation and given in Table 4 indicate that with a larger percentage of lime, a larger percentage of original strength can be retained.

Although lime-treated clays lost strength after freezing and thawing, they still had higher strengths than the untreated soil at a given number of cycles. In fact, clay soil with 10 percent lime, after freezing and thawing, had more strength than untreated clay soil not subjected to freezing and thawing.

The effect of freezing and thawing on the secant modulus of the silt soil (Table 3) should be noted. As with the control specimens, increases in lime content did bring about an increase in strength. More significantly, relatively large decreases in secant moduli occurred with the untreated silts when subjected to freezing and thawing, but little if any decrease occurred in the lime-treated silts. This would indicate that the durability of the lime-silt mixtures was improved by the addition of lime. Therefore, the deflection-resistance properties of the lime-stabilized soils would not be impaired by freezing and thawing.

It should also be noted that the cycles of freezing and thawing produced no visible distortion of the specimens.

TABLE 2

UNCONFINED COMPRESSIVE STRENGTHS OF CLAY-LIME AND SILT-LIME MIXTURES BEFORE AND AFTER FREEZING AND THAWING

Soil	Lime (%)	Specimen No.	Strength (psi)			
			Cycles of Freezing and Thawing			
			0	5	10	29[a]
Clay	0	1	72.1	13.1	--	--[b]
		2	62.7	15.2	--	--[b]
		3	59.3	--	11.0	--[b]
		4	56.0	--	11.7	--[b]
		Avg	62.5	14.2	11.4	
	5	1	160.0	38.0	--	19.3
		2	148.2	32.7	--	19.1
		3	171.9	--	29.8	20.2
		4	188.0	--	31.1	--[b]
		Avg	167.0	35.4	30.5	19.6
	10	1	250.5	90.5	--	65.8
		2	234.0	79.1	--	71.9
		3	277.0	--	94.5	--[b]
		4	266.0	--	84.3	--[b]
		Avg	256.9	84.8	89.4	68.9
Silt	0	1	57.0	58.3	--	66.0
		2	62.6	50.6	--	63.1
		3	56.4	--	45.0	--[b]
		4	52.0	--	45.8	--[b]
		Avg	57.0	54.5	45.4	64.6
	5	1	55.0	33.7	--	46.7
		2	47.0	32.5	--	43.2
		3	53.0	--	51.1	54.5
		4	47.6	--	43.6	--[b]
		Avg	50.5	33.1	47.4	48.1
	10	1	49.1	29.6	--	33.3
		2	40.2	35.6	--	35.5
		3	40.7	--	37.2	37.9
		4	42.8	--	41.8	62.6
		Avg	43.2	32.6	39.5	42.3

[a] Specimen in automatic freeze-thaw apparatus.

[b] Sample destroyed during freezing and thawing because of entrance of water to wrapped specimen.

TABLE 3

SECANT MODULUS OF CLAY-LIME AND SILT-LIME MIXTURES BEFORE AND AFTER FREEZING AND THAWING

Soil	Lime (%)	Specimen No.	Secant Modulus (psi)		
			Cycles of Freezing and Thawing		
			0	5	10
Clay	0	1	1,100	316	--
		2	1,067	338	--
		3	1,710	--	318
		4	1,350	--	275
		Avg	1,350	327	347
	5	1	9,250	2,440	--
		2	7,140	2,100	--
		3	5,250	--	1,720
		4	7,780	--	1,495
		Avg	8,105	2,270	1,608
	10	1	12,050	7,450	--
		2	15,000	3,270	--
		3	13,300	--	6,050
		4	11,000	--	3,480
		Avg	12,840	5,360	4,765
Silt	0	1	1,180	800	--
		2	1,645	697	--
		3	1,250	--	810
		4	1,500	--	825
		Avg	1,394	749	818
	5	1	3,180	1,085	--
		2	2,260	1,343	--
		3	1,915	--	2,460
		4	2,760	--	1,800
		Avg	2,524	1,214	2,130
	10	1	2,820	1,224	--
		2	2,330	2,940	--
		3	1,960	--	2,150
		4	1,545	--	3,020
		Avg	2,164	2,082	2,585

Mechanism of Disintegration Due to Freezing and Thawing

No water was lost or gained during the curing process or the freezing-and-thawing tests, as indicated by weighing the specimen just after molding and just before compression testing. Therefore, it is considered that the specimens behaved as a closed system, similar to that of concrete. If the destructive action of freezing is due to hydraulic pressure, as it is with concrete, then the resistance to movement of water is the primary source of pressure. The intensity of the hydraulic pressure depends on the pore size and rate of freezing. Resistance to freezing and thawing is largely affected by degree of saturation and pore characteristics.

It is known that the durability of concrete is greatly affected by pore characteristics of both the mortar and the aggregate it surrounds. The problem of concrete durability in a freezing-and-thawing environment is summarized by Powers (8, 9) and Verbeck and Landgren (12). Water permeability has been used as a measure of pore characteristics of the mortar and the aggregates that make up concrete. It is interesting to compare these permeabilities to values generally applied to different soils. In Table 5 are given typical permeability values for some chert and dolomite aggregates that present durability problems. Also given are typical ranges of values for concrete mortar and for silt and clay soils. It may be seen that clay soils fall in the general range of permeabilities for the components of concrete. If, by the addition of lime, sufficient strength was available to supply resistance to the flow of water, the loss of strength of the specimen in this study may well have been due to excessive hydraulic pressures rather than to crystal growth.

TABLE 4

RESISTANCE VALUES OF MIXTURES OF CLAY AND LIME[a]

Lime (%)	Freeze-Thaw Cycles			
	0	5	10	29
0	100	22.7	18.3	--
5	100	21.2	18.2	11.7
10	100	31.5	34.9	27.0

[a] Calculated from Eq. 1.

TABLE 5

TYPICAL PERMEABILITY VALUES

Material	Permeability (cm/sec)
Concrete mortar[a]	1 to 300 x 10^{-10}
Dolomite[a]	300 x 10^{-10}
Chert[a]	1 x 10^{-10}
Clay[b]	< 1,000 x 10^{-10}
Silt[b]	≤ 10,000 x 10^{-10}

[a] From Verbeck and Landgren (12).
[b] From Terzaghi and Peck (11).

Studies have shown that the critical degree of saturation for concrete exposed to freezing and thawing is about 0.85 (6). The clay-lime specimens in this study had degrees of saturation ranging from 0.86 to 0.89. This also supports the possibility of hydraulic pressure being in action.

Both clay- and silt-lime mixtures showed a decrease in strength after 5 cycles of freezing and thawing. The decreases in clay-lime mixtures were much greater, but then they had more strength to lose and offered greater resistance to water flow, making possible greater hydraulic pressures. In these tests the bond between particles must have been broken in the first few cycles, since beyond 5 cycles no further loss in strength was produced.

The results presented here do not constitute a proof that Powers' hypothesis operates in clay-lime mixtures, but it is hoped that further study of the problem can lead to better understanding of the deterioration or loss of strength in lime-stabilized soils exposed to freezing and thawing.

SUMMARY

The most important results of this study are as follows:

1. A smaller percentage of strength was lost during freezing and thawing when lime content was increased up to 10 percent.
2. Decrease in strength due to freezing and thawing occurred in the first 5 cycles for all the soil-lime mixtures.
3. Decreases in unconfined compressive strength due to freezing and thawing are much greater in clay-lime mixtures than in silt-lime mixtures.
4. In the closed systems used in this study, disintegration in lime-stabilized soils due to freezing and thawing might be caused by hydraulic pressures generated by the movement of unfrozen water in the pores as a result of expansion of water by freezing.
5. If the hydraulic pressure theory was operating, then greater pressures probably developed in the clay-lime than in the silt-lime mixtures due to smaller pore sizes and much better cementation in the clay-lime mixtures.

REFERENCES

1. Anday, M.C. Curing Lime Stabilized Soils. Va. Council of Highway Invest. and Res., Prog. Rept. No. 2, 1962.
2. Anday, M.C. Accelerated Curing for Lime-Stabilized Soil. Highway Research Board Bull. 304, pp. 1-13, 1961.
3. Cordon, W.A. Automatic Freezing-and-Thawing Equipment for a Small Laboratory. Highway Research Board Bull. 259, pp. 1-6, 1960.
4. Davidson, D.T., Mateos, M., and Barnes, H.F. Improvement of Lime Stabilization of Montmorillonitic Clay Soils with Chemical Additives. Highway Research Board Bull. 262, pp. 33-50, 1960.

5. Hoover, J.M., Handy, R.L., and Davidson, D.T. Durability of Soil-Lime-Flyash Mixes Compacted Above Standard Proctor Density. Highway Research Board Bull. 193, pp. 1-11, 1958.
6. Lewis, D.W., Dolch, W.L., and Woods, K.B. Porosity Determinations and the Significance of Pore Characteristics of Aggregates. Proc. ASTM, Vol. 53, 1953.
7. Lu, L.W., Davidson, D.T., Handy, R.L., and Laguros, J.G. The Calcium-Magnesium Ratio in Soil-Lime Stabilization. Highway Research Board Proc., Vol. 36, pp. 794-805, 1957.
8. Powers, T.C. A Working Hypothesis for Further Studies of Frost Resistance of Concrete. Proc. ACI, Vol. 41, pp. 245-272, 1945.
9. Powers, T.C. Basic Considerations Pertaining to Freezing and Thawing Tests. Proc. ASTM, Vol. 55, p. 1132, 1955.
10. Tabor, S. Freezing and Thawing of Soils as Factors in the Destruction of Road Pavements. Public Roads, Vol. 2, pp. 113-132, 1930.
11. Terzaghi, K., and Peck, R.B. Soil Mechanics in Engineering Practice. New York, John Wiley and Sons, 1948.
12. Verbeck, George, and Landgren, Robert. Influence of Physical Characteristics of Aggregates on Frost Resistance of Concrete. Proc. ASTM, Vol. 60, p. 1063, 1960.
13. Whitehurst, E.A., and Yoder, E.J. Durability Tests on Lime-Stabilized Soils. Highway Research Board Proc., Vol. 31, pp. 529-540, 1952.
14. Yoder, E.J. Freezing-and-Thawing Tests on Mixtures of Soil and Calcium Chloride. Highway Research Board Bull. 100, pp. 1-16, 1954.

Discussion

MANUEL MATEOS, Torán y Compañía, Madrid, Spain.—The writer was impressed by the very small number of specimens used in this work which, together with the miniature size of the specimens, the accelerated curing, and the accelerated freezing and thawing, makes him wonder about the reliability of the results.

The writer has used "open systems" in the freeze-thaw tests of lime-stabilized soils (15) to submit the specimens to the worst conditions which could be found in the field. Since it appears that the prevalent belief among soil engineers is that the open system reproduces the worst conditions likely to occur in the field, the paper would have been of greater value if it had had a comparative study of open and closed systems.

The established philosophy in research work is that when one of the materials used is spoiled or adulterated, all work affected by these materials should be repeated. The authors used a lime which contained 60 percent calcium carbonate. This is a very high degree of carbonation and, consequently, this lime should not have been used in research work.

Reference

15. Mateos, Manuel. Soil-Lime Research at Iowa State University. Proc. ASCE, Jour. Soil Mech., 1964.

M.C. ANDAY, Highway Research Engineer, Virginia Council of Highway Investigation and Research.—The authors are to be complimented for pointing out that some of the

TABLE 6

SOILS TESTED

Soil	Description	HRB Class.	Passing Sieve (%)				
			No. 10	No. 40	No. 200	L. L.	P.I.
A	Clay from Valley and Ridge	A-7-5(16)	99.2	93.0	83.1	55	19
B	Sand clay from Piedmont	A-4(4)	97.6	81.5	53.1	26	9
C	Micaceous silt from Piedmont	A-5(8)	97.9	93.1	70.2	45	N. P.

properties imparted to soil by treatment with lime are subject to detrimental change by the forces of nature. They have chosen freezing and thawing as a test of durability, since this is one of the two most commonly used methods of simulating the forces of nature.

The studies conducted at the Virginia Council of Highway Investigation and Research since 1962 have been, by choice, based on the wetting of stabilized soils after treatment and curing. It is believed that though the ground surface temperatures might indicate several cycles of freezing and thawing, the stabilized layers might not freeze or thaw for long periods of time. Therefore, at least in Virginia, the number of freeze-thaw cycles at the ground surfaces might constitute too severe a criterion of durability.

It is interesting to note, however, that some of the results obtained by the authors are very similar to the ones obtained by the writer. With the hope that the readers might have a combined view of the effects of the detrimental forces of nature, the following data are offered in support of Messrs. Walker and Karabulut's findings.

The studies at the Virginia Council of Highway Investigation and Research involve soils from 13 projects in Virginia, most of which have been under traffic for several years. Since the overall data are very similar, only those for a few soils are illustrated here. A complete report will be available in the near future.

TABLE 7

EFFECT OF SOAKING

Soil	Lime (%)	Laboratory Cured Strength (psi)[a]				Secant Modulus (psi)		R_f (%)[c]
		As Cured	V[b]	Cured and Soaked for 5 Days	V[b]	As Cured	Cured and Soaked for 5 Days	
A	8.0	320 ± 10	5.7	201 + 5	4.6	28,700	20,330	63
B	5.0	230 ± 20	15.7	153 ± 8	9.4	18,170	15,000	67
C	5.0	91 ± 2	4.6	66 ± 3	7.0	3,017	2,266	73

[a] Cured for 2 days at 120 F.

[b] Coefficient of variation in percent (10 samples per test).

[c] Resistance factor (R_f) = $\dfrac{\text{compressive strength after soaking}}{\text{compressive strength as cured}}$.

It might be beneficial to note that the procedures used by the authors and myself were very similar. The main difference was that in my work the Harvard mold-size specimens, after curing for 2 days at 120 F, were soaked for 5 days instead of being subjected to freezing and thawing.

Some of the physical properties and the classification of the soils are given in Table 6. The strength of specimens before and after soaking, their secant moduli and their resistance factors are given in Table 7. Comparison of the resistance factors given in Table 7 with those presented by Walker and Karabulut might indicate that soaking is a less severe test than freezing and thawing and that caution should be exercised in the choice of the durability test; prevailing climatic conditions should be considered.

RICHARD D. WALKER and CETIN KARABULUT, Closure.—This discussion deals with points brought up by Mr. Mateos. It is true that the results of this study were based on a small number of specimens, i.e., as few as two specimens (one specimen from each of two different mixes) for each design consideration. However, the trend shown by these few specimens is unmistakably clear. An example of this can be seen by looking at Table 2. By taking the 5 percent lime case for the clay soil as an example and comparing the lowest zero-cycle strength to the highest 5-cycle strength, a range of 110.2 psi is found (148.2 psi for zero cycles vs 38.0 psi for 5 cycles). Similar comparisons can be made in any direction in the table. Although we emphatically agree that it is always desirable to have a statistically designed experiment, lack of such a design does not negate the large differences exhibited by the data shown. It is agreed, however, that in order to say that anything was significant concerning the silt soil, an adequate statistically designed experiment would have to be made.

The authors agree with Mr. Mateos that a comparative study of open- and closed-system tests would have been desirable. Such a project is being planned for the near future.

Concerning the fact that the hydrated lime was partially carbonized, it should be restated that this should not have affected the variability of the results. Perhaps it should have been stated with greater emphasis that the results were obtained with a lime that has undergone carbonation. We believe the effect of such carbonation should be more thoroughly investigated because there is no doubt that there are many cases in the field where lime is used which contains significant quantities of calcium carbonate.

Lime-Stabilized Soil Properties and the Beam Action Hypothesis

JOAKIM G. LAGUROS, Associate Professor of Civil Engineering, University of Oklahoma

The stabilizing effects of hydrated lime and chemical additives (sodium hydroxide, sodium orthosilicate, and calcium chloride) on a Texas montmorillonitic clayey soil were studied and measured by means of compressive strength, durability, consistency, consolidation and permeability tests. The strength beneficiation obtained from the addition of lime to the soil was substantial. Although further improvement with chemical additives was minimal for strength and consistency, it was considerable when viewed on the basis of consolidation, durability, and permeability test results. Also, the modulus of elasticity of the lime-established soil was determined and compared with that of the same soil stabilized with cement. Based on Winkler's model, a hypothesis was advanced to calculate the deflection and stresses in a stabilized soil layer, wherever such a layer is used as a pavement component.

•STABILIZATION OF in-place soils by admixtures has become an increasingly accepted method for improving the bearing capacity of either the substructure or some other component of the pavement. To measure the amount of beneficiation which weak soils derive from the incorporation of admixtures, the consistency limits, the unconfined compressive strength and the durability tests have been widely used. Of great importance to load-sensitive soils are other properties such as compressibility, permeability, and confined compressive strength. The extent of the stabilizing effect may also be evaluated by measuring the static modulus of elasticity, which is an important parameter in the determination of load-stress-deflection relationship, when it is hypothesized that a stabilized soil base course behaves as a semirigid finite beam resting on an elastic foundation.

In this paper the properties of a montmorillonitic clayey soil stabilized with hydrated lime and with lime and chemicals are evaluated. In addition, the modulus of elasticity of the lime-stabilized soil is compared with that of the cement-stabilized soil and, based on Winkler's model, the finite beam approach is suggested.

MATERIALS

The stabilizing agents used for the poorly reactive soil, whose properties are given in Table 1, were a high-calcium hydrated lime, sodium hydroxide, sodium orthosilicate, and calcium chloride. A preliminary investigation (10) indicated that these chemicals were most effective.

PREPARATION OF MIXES

The soil was air dried, pulverized, and screened through a U. S. No. 10 sieve. Each batch of soil was first hand mixed with lime, then part of the compaction water

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Paper sponsored by Committee on Lime and Lime-Fly Ash Stabilization and presented at the 43rd Annual Meeting.

TABLE 1

PROPERTIES OF SOIL

Property	Value
Sample designation	Texas clay
Sampling location	Harris Co., Texas
Parent material	Coastal Plain deposit, largely deltaic
Great soil group	Grumusol
Soil series	Lake Charles
Horizon	C
Sampling depth, in.	39-144
Textural composition:[1]	
Gravel (> 2 mm), %	0.0
Sand (2 - 0.074 mm), %	3.0
Silt (0.074 - 0.005 mm), %	36.0
Clay (< 0.005 mm), %	61.0
Clay (< 0.002 mm), %	40.0
L.L., %[2]	65
P.L., %[3]	18
P.I.	47
CEC, meq/100 g[4]	27.30
Carbonates, %[5]	16.20
pH[6]	8.2
Organic matter, %[7]	0.13
Non-clay minerals[8]	Quartz, feldspar
Predominant clay mineral[8]	Montmorillonite
Classification:	
Textural (Triangular chart, BPR)	Clay
Engineering (AASHO M 145-491)	A-7-6(20)
Unified	CH

[1] Textural gradation tests performed on soil fraction passing No. 10 sieve; ASTM Method 422-54T.
[2] ASTM Method D 423-54T.
[3] ASTM Method D 424-54T.
[4] Cation exchange capacity determined by ammonium acetate (pH = 7) method on soil fraction less than 0.42 mm.
[5] Versenate method for total 1 N HCl-soluble calcium.
[6] Glass electrode method using suspension of 15 g soil in 30 cc distilled water.
[7] Potassium dichromate method.
[8] X-ray diffraction analysis.

(that amount of water to give optimum moisture content for near standard Proctor density) was added and the mixture was hand mixed. Whenever chemicals were used, they were dissolved or dispersed in the compaction water.

METHODS OF EVALUATION

Preliminary Investigation

From mixes containing various amounts of lime and from those with various amounts of lime and chemicals, and on the basis of 28-day unconfined compressive strength, the optimum amounts (by ovendry weight of soil) of lime and chemicals were established as:

Lime,	6 percent;
Sodium hydroxide,	0.25 percent;
Sodium orthosilicate,	0.50 percent; and
Calcium chloride,	0.50 percent.

Consistency Tests

When mixing was completed, 2-in. high by 2-in. diameter specimens were molded in a drop hammer molding apparatus to near standard Proctor density, wrapped well in waxed paper, and cured in a chamber at 95 ± 5 percent RH and 70 ± 5 F for 28 days. At the end of the curing period the specimens were pulverized and their consistency limits were determined by the ASTM standard methods. The results are given in Table 2.

Unconfined Compressive Strength

Following the procedure explained under the consistency tests, specimens were molded and cured for 28 days. Then they were unwrapped and immersed in distilled water for 24 hr, after which they were tested to failure to determine their unconfined compressive strength. These results are also given in Table 2.

Durability Tests

The natural and stabilized soils were evaluated by the Iowa freeze-thaw (6) and the standard test (1). In both methods the specimens were cured for 7 days. Results are given in Tables 3 and 4.

TABLE 2

CONSISTENCY LIMITS AND UNCONFINED COMPRESSIVE STRENGTH OF NATURAL AND STABILIZED SOIL

Specimen	L.L. (%)	P.L. (%)	P.I.	Comp. Strength (psi)
Natural soil	65	18	47	0
Soil + 6% lime	30	15	15	300
Soil + 6% lime + 0.25% NaOH	28	15	13	320
Soil + 6% lime + 0.50% Na_4SiO_4	30	15	15	300
Soil + 6% lime + 0.50% $CaCl_2$	28	15	13	330

Triaxial Compression Tests

From each mix batch, nine cylindrical specimens were molded in the Harvard miniature compaction apparatus which gives a compacted sample at approximately standard Proctor density. All specimens were cured for 28 days as previously explained. At the end of the curing period the specimens were immersed for 24 hr in distilled water, drained naturally in the curing chamber, and tested in a triaxial compression machine under undrained conditions. Three specimens were used for each of the three lateral pressures of 10, 20, and 30 psi. The cohesion (c) and angle of internal friction (ϕ) of any particular mix were determined graphically using the Coulomb-Mohr theory given by the formula:

$$s = c + p \tan \phi \qquad (1)$$

Results are given in Table 5.

TABLE 3

DURABILITY INDICES FOR NATURAL AND STABILIZED SOIL BY IOWA TEST[a]

Specimen	P_f (psi)	P_{c_f} (psi)	R_f (%)	P_i (psi)	P_{c_i} (psi)	R_i (%)
Natural soil	0	0	-	0	0	-
Soil + 6% lime	0	396	0	360	448	75
Soil + 6% lime + 0.25% NaOH	320	400	80	382	490	78
Soil + 6% lime + 0.50% Na_4SiO_4	284	379	75	336	412	89
Soil + 6% lime + 0.50% $CaCl_2$	383	488	78	396	531	75

[a] Symbols used are as follows:

P_f = unconfined compressive strength of freeze-thaw specimen;
P_{c_f} = unconfined compressive strength of control specimen;
$R_f = \left(R_f/P_{c_f}\right) \times 100$;
P_i = unconfined compressive strength of immersed specimen;
P_{c_i} = unconfined compressive strength of control specimen; and
$R_i = \left(P_i/P_{c_i}\right) \times 100$.

TABLE 4

SUMMARY OF STANDARD FREEZE-THAW TESTS

Specimen	Init. Density (pcf)	Init. Moisture (%)	No. Cycles[a]	After Cycling			
				Density (pcf)	Moisture (%)	Soil Volume	
						Change (%)	Loss (%)
Natural soil	102.9	22.3	0	129.4	35.1	1.1	10.2
Soil + 6% lime	118	23.0	3	124.1	30.9	1.0	9.2
Soil + 6% lime + 0.25% NaOH	118.5	23.5	5	123.1	31.0	1.0	9.7
Soil + 6% lime + 0.50% Na_4SiO_4	118.5	23.5	5	123.2	31.1	1.0	9.8
Soil + 6% lime + 0.50% $CaCl_2$	118.5	23.5	5	123.2	31.0	1.0	9.7

[a] Source: ASTM Standards (1, p. 34).

Permeability Tests

The coefficient of permeability (k) for the five different mixtures (Table 5) was determined by using a falling-head miniature permeameter (Soiltest Model K-620). The test specimens, identical in form and method of preparation with those for the triaxial compression test, were cured for 28 days in their molds and then saturated and tested.

Consolidation Tests

The apparatus used in this phase of the study was the fixed type consolidometer (Soiltest Model C-252). Through trial tests the correct amount of soil to give a 1-in. high specimen and the static load required to give standard Proctor density were established. The specimens, 4⁷⁄₁₆ in. in diameter and 1 in. high, were compacted by the static load in the specimen ring, and the assembly was cured for 28 days. The two-way drainage test was run after saturating the specimen in the ring. Figure 1 shows the consolidation characteristics of the natural and stabilized soil through the e-log p curves and c_v values of the test specimens. For the soil-lime-chemical mixtures, the differences in the e-log p and c_v values were extremely small and, therefore, a typical curve was drawn.

TABLE 5

RESULTS OF TRIAXIAL COMPRESSION TESTS AND k VALUES

Specimen	c (psi)	ϕ (deg)	$s = c + p \tan \phi$	k (cm/sec)
Natural soil	25	20	$s = 25 + 0.36p$	8.6×10^{-6}
Soil + 6% lime	28	49	$s = 28 + 1.15p$	6.8×10^{-5}
Soil + 6% lime + 0.25% NaOH	43	42	$s = 43 + 0.90p$	3.9×10^{-5}
Soil + 6% lime + 0.50% Na_4SiO_4	42	41	$s = 42 + 0.87p$	4.8×10^{-5}
Soil + 6% lime + 0.50% $CaCl_2$	43	44	$s = 43 + 0.97p$	3.5×10^{-5}

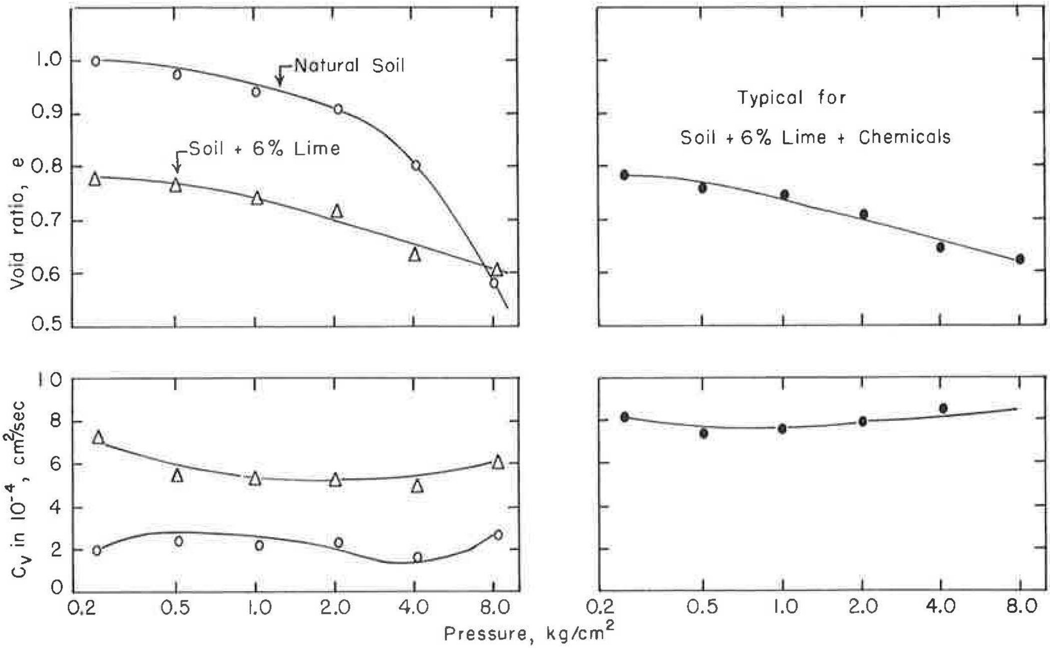

Figure 1. Consolidation characteristics of test specimens.

Static Modulus of Elasticity

Specimens were molded in the Harvard miniature compaction apparatus and cured as for the triaxial compression tests. At the end of a 7-day curing period, they were unsealed, immersed in water for 24 hr, and tested in unconfined compression to failure at a rate of 0.05-in./min strain. For reasons of comparison, specimens stabilized with 12 percent portland cement and with 12 percent cement plus 0.25 percent sodium hydroxide were prepared and cured in the same manner. The lime-stabilized specimens showed so little strength gain at the end of the 7-day curing period that their modulus of elasticity could not be well defined. Therefore, the curing period was extended to 28 days. However, the cement-stabilized specimens showed marked gain in strength at the end of the 7-day curing period. This is supported by previous experience (11, 13) which indicates that the 7-day unconfined wet compressive strength is a fairly reliable measure for soil-cement. Table 6 presents a summary of pertinent data rela-

TABLE 6

STRENGTH—E_{SC} RESULTS

Specimen	Max. Dry Density (pcf)	Opt. Moisture Content (%)	After Curing and 24-Hr Immersion			
			Density (pcf)	Moisture Content (%)	Unconfined Compressive Strength (psi)	Avg. E_{SC}, (psi)
Texas clay + 6% lime	118	23.0	127.2	24.8	300	8.8×10^3
Texas clay + 6% lime + 0.25% NaOH	118.5	23.5	127.8	25.1	320	8.9×10^3
Texas clay + 12% cement	102.5	22.6	122.1	26.3	350	9.4×10^3
Texas clay + 12% cement + 0.25% NaOH	102.5	22.6	123.2	26.6	420	1.9×10^4

tive to the test specimens. The E_{sc} (static modulus of elasticity under compression) values were calculated from the stress-strain measurements of each specimen by fitting a tangent to the straight portion of the stress-strain curve. This straight portion extended over a section approximately 30 to 60 percent of the maximum strength.

In investigating the elastic behavior of soil-cement, triaxial loading has been used (2). Although this type of testing is desirable in many instances, for cohesive materials such as the ones used in this study, it does not seem to be absolutely essential. Under triaxial loading the E_{sc} values are expected to be higher. It is questionable, however, if at the shoulder pavement interface, where conditions are critical, the confinement pressure will be of a magnitude to increase significantly the supporting power of a base course.

Since it is proposed to use the stabilized soil layer as a beam, the use of the unconfined compression test in lieu of a beam test may invite some argument. It must be remembered, however, that because of inherent structural discontinuities, if not for reasons of unpredictable crack formation in the stabilized soil, the static modulus of elasticity in flexure may lead to unsafe high values of design, whereas the same modulus in compression is more conservative and does not appear to lead to uneconomical design since both moduli are of the same order (5).

DISCUSSION OF RESULTS

It is an established fact that the addition of hydrated lime to clay soils produces a notable improvement in their workability as a result of the predominant interaction between the clay particles and the calcium ions (9). In view of this aggregating influence of lime, a comparison of the consistency, strength, compressibility, durability, and permeability properties of the natural soil with those of the lime-stabilized soil leads to the conclusion that, in the stabilized soil, these properties are greatly improved. Thus, the lime-stabilized soil gains strength, attains resistance to weathering, and becomes more permanently compressible in a much shorter time. A reduction in the measured permeability of the lime-stabilized soil is also an indication of the improvement effected by lime.

Further modifications in consistency and unconfined compressive strength resulting from the addition of sodium hydroxide, sodium orthosilicate, and calcium chloride to the soil-lime mixture are obtained. Although the data indicate small changes (about 7 percent), they prove that the montmorillonitic clayey soil responded favorably to the additions of sodium and calcium ions—at least in the form in which they were used in this study. That sodium hydroxide was more effective than sodium orthosilicate may be explained in terms of the additional effect of hydroxide which interacts with the silica surfaces of the quartz in the soil, producing more numerous bonds. Consistency test results do not show any difference between the effectiveness of sodium hydroxide and calcium chloride, and the difference in the unconfined compressive strength is too small to provide a definite conclusion. Although this is true at 6 percent lime, a study (10) with 3 percent lime has indicated that unconfined compressive strength with sodium hydroxide is 50 percent more (280 psi) than with calcium chloride (180 psi). This difference in response may be explained by the lime fixation or retention point theory (8) whereby at low Ca^{++}-ion concentration, resulting from 3 percent lime and 0.50 percent $CaCl_2$, very little, if any, lime is available for pozzolanic reactions; at 6 percent lime, a sufficient amount of lime is available for such reactions and, therefore, relatively high strengths are obtained. On the other hand, sodium hydroxide, both at low and high lime concentrations, seems to promote the reaction of calcium and silica either by making more silica available or by providing suitable alkaline conditions for the reaction to take place.

The durability tests produced similar results as the unconfined compressive strength test and, therefore, similar conditions of stabilization mechanism may be assumed as controlling. Qualitatively, however, a beneficial divergence is noted in the behavior of the lime-stabilized soil when chemical additives are used. On the basis of the R_f values of the Iowa freeze-thaw test, the weather resistance of the soil is substantially increased from zero to about 80 percent; the standard freeze-thaw test indicates an increase from 3 to 5 cycles on the addition of the chemicals. These increases can only be attributed to the effect of the added chemicals, which lower the freezing point of water and also increase the cohesion of the mix.

When lime was added to the soil, its cohesive strength was improved simultaneously with an increase in the angle of internal friction as evidenced from the results of the triaxial compression test. When the chemicals were added to the soil-lime mix, a gain in the overall shearing resistance was recorded, but the increase in the cohesive strength was somewhat masked by a slight decrease in the angle of internal friction. Similar trends have been reported for soil-cement (11), and it is reasonable to expect that the same explanation for this phenomenon holds true. The increase in cohesion has already been accounted for. The decrease in the internal friction could not be traced to a possible reduced agglomeration of particles, at least not as far as the plastic properties of the mix were concerned. The soil-lime mixes did not seem to have plasticity indices very different from those of soil-lime modified with chemicals. The addition of the chemical, then, gives greater cohesion resulting from greater interparticle attraction, but the ions possibly increase the interparticle distance slightly as they take their places between the already aggregated particles, with the result that interparticle contact is reduced and, therefore, internal friction is slightly reduced.

The compressibility tendency of a soil, which has been compacted to a maximum standard Proctor density, presents a problem only when traffic compaction in excess of that anticipated takes place. This, coupled with the saturation of the soil mass, is a condition that cannot be excluded as a possibility during the life of a highway pavement. Therefore, the consolidation properties and, associated with this, the coefficient of permeability, k, were separately determined for the soil and all the mixtures.

Solely in terms of the order of k, it may be said that the fat clayey soil attains the permeability properties of a loam when it is stabilized with lime and a further reduction in the k value occurs on the addition of the chemicals. This modification is reflected in the coefficient of consolidation, c_v, and it is attributable to the already discussed flocculating character of lime and the chemicals. Also, the comparatively flat e-log p curves for the stabilized soil suggest that large applied pressures will not cause high settlements as appears to be the case for the natural soil.

A comment deserves to be made regarding the apparent ineffectiveness of chemicals in the lime-stabilized clayey soil when measured by the unconfined compressive strength and the consistency limit tests. In both tests, the values obtained are indications of the shear strength of the mixture, but the role of hygroscopic or gravitational water is not distinguished from that necessary to give cohesion to the mix. It has been argued (14) that the water acts as a filler separating particles and resisting close approach and that shearing possibly occurs along the water space. Had the effective values been measured in the triaxial test, it would not have been surprising to record lower values of cohesion and slightly greater values of internal friction than the ones observed. For the durability test results, it may be said that the continuous moist environment was the factor which enhanced the effectiveness of chemicals. The permeability and consolidation test results, where neutral stresses are of no bearing, prove the beneficial effect of the chemicals on the lime-stabilized soil.

The E_{sc} values tend to reflect the same beneficial trend due to the chemicals both for the lime-stabilized and the cement-stabilized soil (11). Although the data suggest a relationship between unconfined compressive strength and E_{sc} values, unreported work on different soils by the author indicates that this may not be true. Furthermore, this limited study seems to indicate that the E_{sc} values may heavily depend on either the clay content or the type of clay mineral, since they manifest a propensity for a kaolin-illite-montmorillonite order.

FINITE BEAM CONCEPT APPLIED TO SOIL-LIME AND SOIL-CEMENT

The stresses induced in the constituent layers of a pavement by surface loads have been the subject of experimental and theoretical research for many years. The Boussinesq approach and the layered system solution (15) have been used in various forms which necessarily include simplifying assumptions when applied to compacted or modified soil masses.

Finite Beams

Lime- or cement-stabilized soil manifests such properties of strength that it is reasonable to classify it as a stiff, but not altogether rigid, construction material possessing an adequate modulus of elasticity, especially in its free-of-cracks condition. Therefore, it seems equally reasonable to analyze the stresses in such a pavement component by a deflection beam method, the simplest of which is the one given by the traditional Winkler model (7). In drawing such a similarity, it is assumed that:

1. The soil-lime or soil-cement base, which has a thin asphaltic overlay, rests on a subgrade which constitutes the elastic foundation;
2. Since the Winkler model idealizes the foundation with an infinite number of springs, the stabilized soil "beam" is made up of independent beams of length equal to the width of the pavement, of width of unit length, and of height equal to the base course thickness;
3. Since the Winkler approach provides a good engineering approximation for all but rigid beams (4), it can be used in our case; and
4. By the method of superposition (end conditioning principle) an infinite beam is transformed to a finite beam, such as the stabilized soil beam, for the particular load(s) under consideration.

The Winkler model requires the solution of the equation:

$$EI \frac{d^4 y}{dx^4} + ky = w(x) \tag{2}$$

where

EI = flexural rigidity of the beam;
k = a constant;
$w(x)$ = applied loading;
$y = e^{\lambda x}(C_1 \cos \lambda x + C_2 \sin \lambda x) + e^{-\lambda x}(C_3 \cos \lambda x + C_4 \sin \lambda x)$;
C_1, C_2, C_3, C_4 = constants of integration; and
x = distance from load to any section.

The expression y gives the deflection and, from its derivatives, the slope, moment, and shear of the beam may be obtained.

Whichever solution is used (3), a common phase is that of selecting certain constants, including the modulus of elasticity of the beam.

Evaluation of λ

In the equation for the deflection, y, the important term λ appears. This is the "characteristic" of the system, and it is evaluated from the relationship;

$$\lambda = \sqrt[4]{\frac{kb}{4EI}} \tag{3}$$

where

E = modulus of elasticity of beam;
I = moment of inertia of beam;
k = subgrade reaction coefficient; and
b = width of beam.

For the stabilized soil base courses, it is proposed to use the E_{SC} values for E, to calculate I assuming a 1-ft wide solid beam and to evaluate the usable value of k of the subgrade by the plate loading test modified accordingly (16).

Loads

Loads on pavements, the form of area through which they are applied and, consequently, their intensity have raised formidable engineering problems. Simplifications and approximations have been used successfully.

An examination of the form and size of area of tire imprints (15, 17) reveals that the actual elliptical form, which has been approximated to a circular area with equivalent area, may as well be simplified to an equivalent rectangular area having a width of 1 ft, so that it extends over the entire width of the proposed beam, and a length to be calculated from the equivalent area. In fact, in recent studies (15) tire imprints have been accepted as having a form similar to a rectangle with semicircles attached at the two opposite ends. Thus, a 9 x 20 truck tire transmitting a load of 9,000 lb at a pressure of 86 psi will be equivalent to a uniformly distributed load of about 1,000 psi over a 10-in. section of the beam. Since also more than one wheel will be applied along a transverse line on a pavement, the problem assumes the form of a beam with free ends carrying discontinuous uniform loads. The total effect of the various uniform loads is the summation of the effects of each load. The numerical problem resulting from this analysis is expedited by the use of coefficients or influence lines or some other method (3, 4).

The preceding treatment of stresses and deflections at a stabilized soil base-subsoil interface is suggested as a possible method of analysis. Although oversimplified, at least it may prompt questions and answers which will lead to more refined and sophisticated approaches.

CONCLUSIONS

1. Lime stabilization of the Texas montmorillonitic clayey soil offers some promise which becomes considerable with the use of chemical additives.
2. The strength beneficiation of the soil-lime and the soil-lime chemical mixtures seems to be small, judging from the unconfined compression test results; triaxial test results indicate greater improvement.
3. The addition of the chemicals does not improve the consistency properties of the soil-lime mixtures once lime is used as the main stabilizing agent.
4. The resistance of the soil-lime mixtures to weathering, as measured by the Iowa freeze-thaw and the standard test, is greatly increased with the use of the chemicals.
5. Similarly, the chemicals are effective in promoting the compressibility properties of the soil-lime by increasing its permeability.
6. The static modulus of elasticity in compression, E_{SC}, reflects the beneficial effect of sodium hydroxide, but it is not necessarily related to the unconfined compressive strength.
7. A lime- or cement-stabilized soil base course may be treated structurally as a finite beam resting on an elastic foundation, where stresses and deflections are calculated by employing the Winkler model and the Hetenyi approach, the suggested method being regarded as tentative until actual measurements can be made.

ACKNOWLEDGMENTS

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REFERENCES

1. ASTM Standards, 1958, Part 4. ASTM, 1958.
2. Balmer, G. C. Shear Strength and Elastic Properties of Soil-Cement Mixtures Under Triaxial Loading. Proc. ASTM, Vol. 58, 1958.

3. Barden, L. Finite Beams Resting on an Elastic Soil. Civil Eng. and Public Works Rev., Vol. 57, Nov. 1962.
4. Barden, L. Influence Coefficients for Beams Resting on the Soil. Civil Eng. and Public Works Rev., Vol. 58, May 1963.
5. Felt, E. J., and Abrams, J. S. Strength and Elastic Properties of Compacted Soil-Cement Mixtures. ASTM STP No. 206, 1957.
6. George, K. P., and Davidson, D. T. Development of a Freeze-Thaw Test for the Design of Soil-Cement. Highway Research Record No. 36, pp. 77-96, 1963.
7. Hetenyi, M. Beams on Elastic Foundation. Univ. of Michigan, 1946.
8. Ho, C., and Handy, R. L. Characteristics of Lime Retention by Montmorillonitic Clays. Highway Research Record No. 29, pp. 55-69, 1963.
9. Ladd, C. C., Moh, Z. C., and Lambe, W. T. Recent Soil-Lime Research at the Massachusetts Institute of Technology. Highway Research Board Bull. 262, pp. 64-85, 1960.
10. Laguros, J. G., and Davidson, D. T. Effect of Chemicals on Soil-Cement Stabilization. Iowa Eng. Exp. Sta., Final Rept., April 1962.
11. Laguros, J. G., and Davidson, D. T. Effect of Chemicals on Soil-Cement Stabilization. Highway Research Record No. 36, pp. 172-203, 1963.
12. Soil-Cement Laboratory Handbook. Portland Cement Assoc., 1959.
13. Road Research Laboratory. Soil Mechanics for Road Engineers. London, Her Majesty's Stationery Off., 1954.
14. Seed, H. B., et al. The Strength of Compacted Cohesive Soils. In Research Conference on Shear Strength of Cohesive Soils. New York, ASCE, 1960.
15. Sowers, G. F., and Vesic, A. B. Vertical Stresses in Subgrades Beneath Statically Loaded Flexible Pavement. Highway Research Board Bull. 342, pp. 90-123, 1962.
16. Terzaghi, K. Evaluation of Coefficients of Subgrade Reaction. Geotechnique, Vol. 5, 1955.
17. Yoder, E. J. Principles of Pavement Design. New York, John Wiley and Sons, 1959.

Evaluation of Experimental Stabilized Soil Base Construction, Webster County, Iowa

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This paper presents a portion of the results of an experimental stabilized soil road base program initiated by the Iowa State Highway Commission and the Webster County Engineer's Office. The 8.058-mi long site chosen is typical of the Clarion-Nicollet-Webster soil association area materials found in hundreds of miles of farm-to-market roads in the north-central third of Iowa. The Webster series, a black, heavy-textured, poorly drained clayey soil, dominates.

Variable thickness base sections were constructed by using the in-place soil materials stabilized with Type I portland cement, lime, lime-fly ash and a combination of lime and portland cement. The surface course was a double bituminous armor coat using $^3/_8$-in. crushed stone chips.

The experimental features of the project were divided into two primary objectives, both directly related: (a) evaluation of conventional construction procedures, existing construction-inspection specifications and techniques, and recommendations for establishment and/or changes to each of the above areas; and (b) evaluation of the constructed material by field and laboratory tests for determination of stability requirements in the development of design criteria for low-cost stabilized soil base roads. This paper deals principally with the area of the first objective.

Presented are an evaluation of the construction techniques, gradation specifications preceding introduction of the stabilizing agent(s); discussion of use of lime as a pretreating agent for reduction of plasticity and increase of friability, comparison of laboratory and field standard Proctor moisture-density relationships, in-place field density determinations using oil density and standard Proctor penetrometer methods, and variation of unconfined compressive strength of 7-day moist-cure specimens with variations in moisture and density.

•THE ECONOMIC and areal lack of high-quality aggregates and soils suitable for highway bases and subbases in Iowa has prompted the use of more locally available or on-site materials. As a result, sponsored research programs at the Iowa Engineering Experiment Station, Soil Research Laboratory, have in the past 15 years evaluated innumerable types of stabilization additives for use with problem soils in highway construction. Soil additives have included inorganic and organic products, by-products, waste products and even a few so-called magic ingredients. Iowa soils are unique in that they range from old to young glacial materials, thin to thick loess deposits (150 ft and over), residual to alluvial, and even include stable sand dunes. Two major problem soils of the state, in terms of highway construction, are the relatively flat, heavy textured, poorly drained topsoils of the northern areas of the state, and the loess of western and southern counties.

Field trials of stabilized soils have been undertaken only after laboratory tests have firmly established the reliability of the newer stabilizing materials. In 1957, 6,000 ft of experimental subbase and base course were laid on Iowa Primary Highway 117 north

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of Colfax, Iowa, using lime, lime-fly ash, Arquad 2HT (Armour Industrial Chemical Co.), and lime-fly ash-activating agents in comparison with soil-cement and soil-aggregate types of stabilization (1). The major purpose of the project was to evaluate the field stability of the soil-stabilizing agent(s) mixtures. The project clearly indicated the positive effectiveness of some of the materials for use in stabilized soil base and subbase usage in Iowa.

In 1959, an additional experimental program was initiated by the Research Department of the Iowa State Highway Commission and the Webster County Engineer's office. The major purpose of the program was to produce and evaluate a low-cost stabilized base course made from the existing gravel-treated surface of the road embankment. The experimental features of the 8.058-mi long stabilized soil base project were divided into two primary objectives, both directly related: (a) evaluation of conventional construction procedures, existing construction-inspection specifications and techniques, and recommendations for establishment and/or changes to each of these areas; and (b) evaluation of the constructed material by field and laboratory tests for determination of stability requirements in the development of design criteria for low-cost stabilized soil base roads in Iowa. This paper deals only with the area of the first objective. No attempt will be made to analyze the mix design, thickness or stability of the stabilized materials.

Four types of stabilization treatment were used on the project: (a) 8 percent Type I portland cement; (b) 3 percent monohydrate dolomitic lime (used as a pretreatment for reduction of plasticity and increase of friability of the soil material) followed with 5 percent Type I portland cement; (c) 4 percent monohydrate dolomitic lime and 20 percent fly ash; and (d) 8 percent monohydrate dolomitic lime of which 3 percent was used as a pretreatment. All percentages of stabilizing agents are expressed on the basis of percent of dry soil weight at standard AASHO density of the soil-agent mixture.

Since construction, the roadway has withstood considerably increased traffic including use as a contractor's haul road in 1961, farm implements including corn drills and spiked tooth and disc harrows, and two of the severest winters in 30 years.

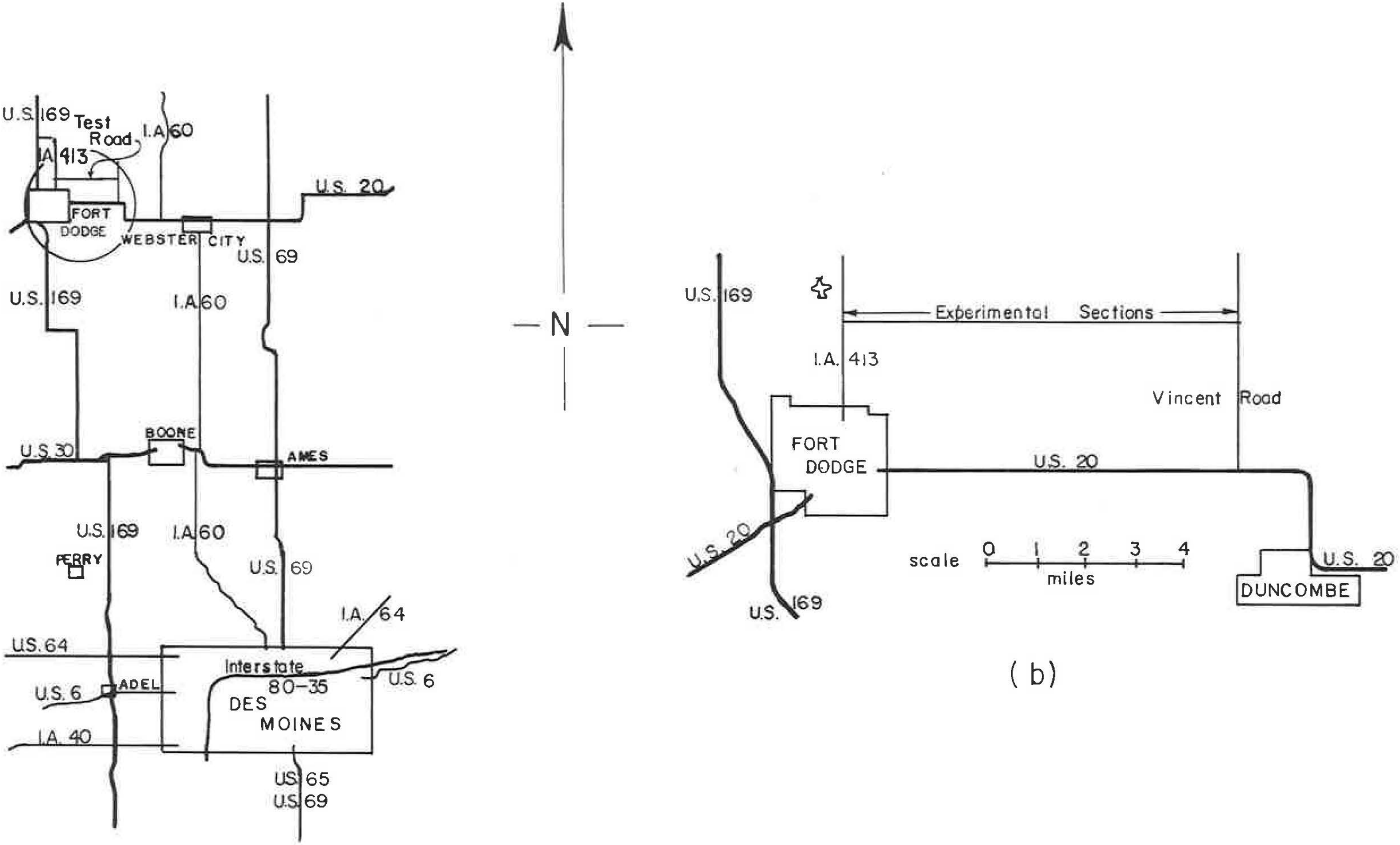
MATERIALS

Soil

The test site chosen (Fig. 1) is typical of the Clarion-Nicollet-Webster soil association area materials found in the subgrades of hundreds of miles of farm-to-market roads in the north-central third of Iowa. Since the major topography is flat to very gently undulating, most farm-to-market and secondary roads of this area are basically of ditch cleanout construction. This results in subgrades which often are predominantly topsoil (high organic matter contents) or B-horizon materials (high clay contents). Gravel or crushed stone surfacing are extremely common but seldom penetrate more than 2 to 3 in., 1 to 2 in. being more common. Frost heave and boiling are common occurrences in winter and spring, as are dusty and washboard surfaces in summer and fall.

The existing gravel-treated surface of the road embankment was used for construction of the stabilized soil base course without the addition of any material other than the stabilizing agent(s). Initial inspection and sampling of the roadway showed an average thickness of gravel of about ½ in. underlain by soil material typical of the aforementioned soil association area. As shown in Figure 2, compacted thicknesses of the stabilized base sections were 6 and 8 in. Scarification depths were approximately 5 and 7 in. with respect to compacted thicknesses, depending on the quantity and type of agent added and the density of the in-place material. Therefore, sampling of the proposed base material was made to scarification depth in 18 locations (Fig. 2, Table 1). Soil samples generally fell into classifications A-2-4(0) to A-7-6(8) over the 8-mi length. The soil material in the base had an average of 88.7 percent passing the No. 4 and 48.2 percent passing the No. 200 U. S. standard sieves. The average moisture content over the scarified depth of material was 10.7 percent at the time of sampling in August 1959. The average organic matter and carbonate contents were 1.4 and 10.2 percent, respectively, with an accompanying alkaline soil pH averaging 7.7.

The subgrade material was also sampled in each of the base sample locations, covering the thickness from bottom of scarification to a depth of 18 in. This material ranged



(a)

(b)

Figure 1. Site of Webster County, Iowa, experimental stabilization project in relation to (a) Des Moines and (b) Fort Dodge.

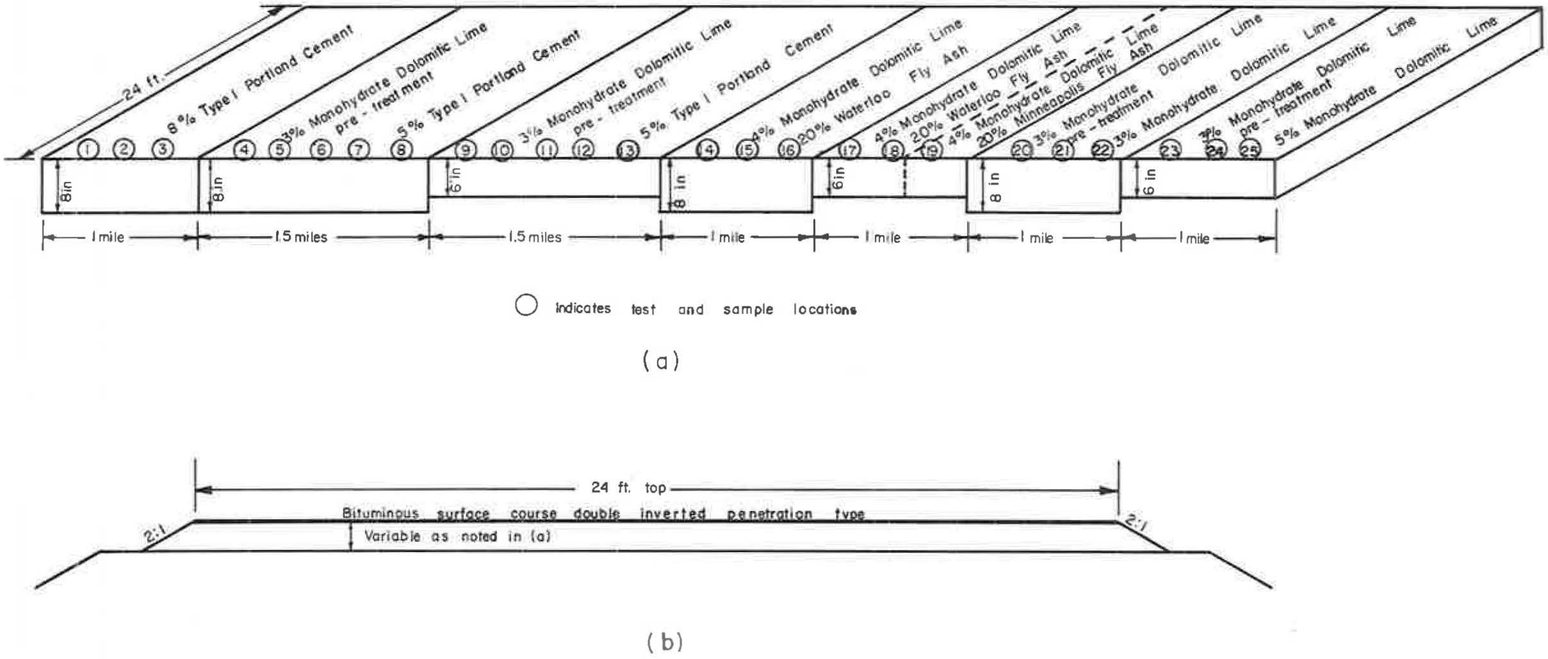


Figure 2. Experimental road sections, Webster County, Iowa: (a) general layout and location; and (b) cross-section.

TABLE 1

PROPERTIES OF BASE AND SUBGRADE SOILS AT FIELD SAMPLE AND TEST LOCATIONS[a]

Sample Location[b]	L. L. (%)	P. I.	Moisture Content (%)	Eng. Classification	CEC (meq/100g)	Organic Matter Content (%)	Carbonate Content (%)	pH
(a) Base Materials								
1	33.1	13.9	10.7	A-6(5)	--[d]	--	--	--
2	37.6	18.0	11.7	A-6(8)	--	--	--	--
3	38.0	15.7	14.4	A-6(7)	--	--	--	--
5	31.1	7.6	12.3	A-4(3)	26.5	2.3	4.5	7.2
6	38.6	14.5	12.7	A-6(6)	--	--	--	--
8	36.9	14.8	11.5	A-6(2)	17.0	1.3	8.7	7.1
9	41.4	18.3	16.6	A-7-6(8)	27.0	2.1	9.7	8.2
11	33.3	10.5	8.0	A-6(2)	--	--	--	--
13	38.4	16.5	13.6	A-6(8)	--	--	--	--
14	35.2	13.1	8.8	A-6(1)	16.1	1.6	11.1	7.8
15	26.6	8.0	8.5	A-2-4(0)	--	--	--	--
16	39.1	19.4	13.1	A-6(7)	--	--	--	--
17	40.4	16.6	14.1	A-4(7)	--	--	--	--
19	40.1	20.1	9.4	A-4(4)	19.9	0.9	16.7	--
20	24.2	7.4	6.0	A-2-4(0)	10.3	0.8	15.8	8.0
22	27.2	9.8	4.6	A-2-4(0)	13.2	0.7	12.3	8.0
23	31.5	12.2	7.2	A-6(2)	--	--	--	--
25	38.5	19.9	8.6	A-6(10)	22.3	1.7	2.6	7.6
(b) Subgrade Materials								
1	36.5	14.7	19.0	A-6(7)	--	--	--	--
2	41.5	17.2	22.9	A-7-6(10)	26.8	2.4	2.6	7.5
3	40.3	17.4	22.0	A-6(10)	26.5	2.3	2.4	6.9
5	37.4	14.5	14.9	A-6(4)	--	--	--	--
6	46.2	19.7	24.7	A-7-6(13)	--	--	--	--
8	37.6	13.9	18.8	A-6(9)	--	--	--	--
9	45.4	21.0	21.8	A-6(11)	26.4	3.1	8.3	7.4
11	45.8	18.1	25.1	A-7-6(12)	--	--	--	--
13	45.1	24.1	24.6	A-7-6(16)	31.2	3.4	4.2	7.6
14	42.5	17.3	24.2	A-7-6(10)	29.1	2.8	2.9	7.5
15	29.4	12.7	11.5	A-6(4)	--	--	--	--
16	47.2	22.8	22.2	A-7-6(14)	--	--	--	--
17	37.4	11.1	19.7	A-6(2)	19.5	1.9	3.4	7.4
19	47.5	26.3	18.1	A-7-6(16)	29.4	2.3	6.3	7.8
20	40.0	15.4	21.1	A-6(9)	--	--	--	--
22	45.6	17.4	25.9	A-7-6(11)	--	--	--	--
23	43.5	19.5	20.0	A-7-6(12)	--	--	--	--
25	49.1	21.8	20.5	A-7-6(12)	--	--	--	--

[a] Samples taken prior to construction in August 1959.
[b] See Figure 2a for relation of stabilized section and sample location number.
[c] Of dry soil weight.
[d] Not determined.

TABLE 2

REPRESENTATIVE LABORATORY ANALYSES OF FLY ASH SAMPLES[a]

Property	Percent	
	Waterloo	Minneapolis
SiO_2	37.70	44.52
Al_2O_3	16.88	21.08
Fe_2O_3	19.88	17.92
MgO	0.87	0.58
SO_3	0.41	1.55
Loss of ignition	17.29	9.05
Passing No. 325 sieve[b]	69.2	76.3

[a] Samples removed from trucks dumped and spread at site.
[b] Samples sieved through a No. 20 U. S. standard sieve before test; amount given in percent by weight.

in classification from A-6(2) to A-7-6(16), with an average of 66.7 percent material passing the No. 200 U. S. standard sieve (Table 1). The average moisture content of the full depth at time of sampling in August 1959 was 20.9 percent by dry soil weight. The average organic matter and carbonate contents were 2.6 and 4.3 percent, respectively, with an accompanying alkaline soil pH averaging 7.4.

Stabilizing Agents

Figure 2 shows the various types of stabilization treatments. Type I portland cement was obtained by the contractor from two sources: Dewey Cement Co. of Linwood, Iowa, and Lehigh Cement Co. of Mason City, Iowa. These cements are hereafter referred to as Dewey and Lehigh, respectively.

Monohydrate dolomitic lime was selected for use in the project as previous investigations had shown it to be superior to other limes for soil-lime stabilization in Iowa, where the dominant clay mineral is montmorillonite (2, 9). The lime was obtained by the contractor from two sources: Dewey Cement Co. and Rockwell Lime Co. of Rockwell, Wis. These limes are hereafter referred to as Linwood and Rockwell, respectively. Samples of the bulk lime were removed from railroad cars in which they had been shipped and were tested by the Iowa State Highway Commission Materials Department. A representative analysis of the Rockwell lime indicated 0 percent retained on No. 30 sieve, 5.0 percent retained on No. 200 sieve and a magnesium oxide content of 33.3 percent. The lime was tested in accordance with ASTM designation: C 207-49.

The fly ash used in the experimental sections was obtained from the Iowa Public Service Co. in Waterloo and the Northern States Power Co. in Minneapolis, Minn. Analyses of fly ash samples (Table 2) obtained at the job site were made by the Robert W. Hunt Co. of Chicago, Ill.

CONSTRUCTION

All supervision and inspection of construction was under the direct control of the Webster County Engineer and/or his official representatives. Personnel of the Soil Research Laboratory, Iowa Engineering Experiment Station, collected and compiled the data pertinent to the objectives of the research, and acted as consultants to the Research Department, Iowa State Highway Commission, and the Webster County Engineer.

The base course construction consisted of scarification of the existing surface to the required depth(s), followed by pulverization of the scarified material with multipass mixers to produce the desired gradation as set forth in Iowa Highway Commission Standard Specifications, i.e., 80 percent of the soil material (on a total soil-moisture weight basis) to pass the No. 4 U. S. standard sieve before addition of any stabilizing agent. It was soon noted that this specification was economically impossible without lime pretreatment, due to the high amount of resistant clay aggregates of approximately ½- to 1½-in. size. The specification was therefore reduced to 65 percent passing the No. 4 sieve; this was still difficult to achieve without the assistance of the lime pretreatment.

Following pulverization, each stabilizing agent was spread by a conventional truck-pulled cement spreader and thoroughly mixed dry. Water was then added through the spray bar of the multipass mixer to bring the mix to optimum moisture content for maximum density, and the materials were thoroughly mixed again. Initial compaction was accomplished by a tapered tamping foot roller with a contact pressure of about 300 psi. Final compaction was with a segmented grid roller filled with lead dust with contact pressure of about 465 psi. Each compactor worked in conjunction with a spiked tooth harrow for removal of tractor tread and compactor marks. A vibratory steel drum roller was tried in one section but created 1- to 1½-in. deep surface cracks spaced a few inches apart. These were apparently due to the fineness of the soil material coupled with rolling movement of the material ahead of the drum.

A 4-in. crown was cut by a blade grader. This was followed by light rubber-tired rolling to reseal any surface hair cracking caused by the cut of the blade. Edges were cut on a 2:1 slope, as shown on the cross-section in Figure 2b. The surface of the base

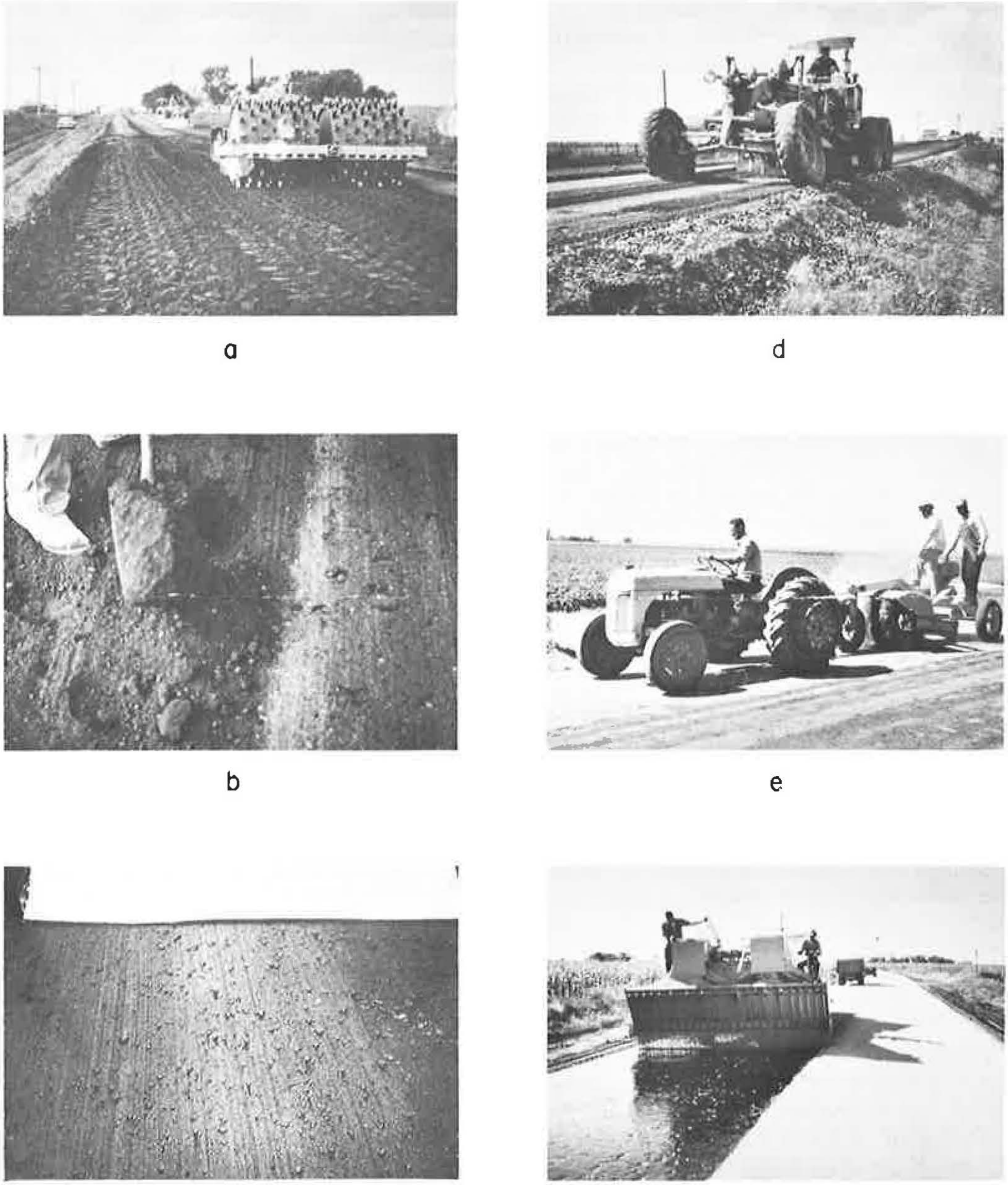


Figure 3. Construction: (a) general construction sequence following pulverization of in-place soil; (b) results of one pass of multipass mixer over lime-pretreated soil; (c) appearance following third pass of multipass mixer over same location; (d) cutting of 4-in. crown and 2:1 sloped edges; (e) brooming to remove loose material from surface of finished base; and (f) spreading limestone chips for double inverted penetration type surfacing, using a medium cutback asphalt as binder.

was then broomed for removal of any loose material and sprayed with a light coat of water to assist in prevention of penetration of the bituminous prime coat. Each section was primed with MC-0 cutback asphalt and closed to traffic for at least 7 days to allow an initial set of the stabilized material. Surfacing was of a double inverted penetration type (double-chip coat) using MC-4 cutback asphalt and $^3/_8$-in. limestone chips. Figure 3 shows some scenes of the construction.

Lime pretreatment of various sections noted in Figure 2 created an additional construction sequence. Following scarification of the roadbed, lime was applied and lightly mixed, bladed and shaped to crown, rubber-tire rolled to seal the surface lightly, and allowed to react for a minimum of 48 to 72 hr. In some of the drier sections a light application of water was mixed into the material before sealing of the surface. Following the pretreatment period, the material was again scarified and mixed with the multipass mixer until gradation requirements were met. Construction then proceeded as outlined previously.

Also following scarification and light blading of the roadbed, moistened fly ash, where used, was dumped from uncovered trucks and spread with a blade grader to a uniform loose thickness and width. Multipass mixers were then used to obtain an intimate mix of fly ash and soil before addition of lime.

Construction was started in August 1959; only 4½ mi were completed before cold weather because of unusually wet weather conditions in September. Construction was halted in Oct. 30, 1959. Completed that year were: (a) base courses in the west 2½ mi and east 2 mi; (b) lime pretreatment in the 6-in. thick section of lime and cement; (c) double-chip surfacing in the west 2½ mi of the project and the east ½ mi of the 6-in. lime section; (d) single-chip surfacing of the 8-in. lime section and the west ½ mi of the 6-in. lime section; and (e) spreading and mixing all fly ash with the soil in the 2 mi of lime-fly ash section.

During the winter of 1959-60, Iowa had unusually severe weather. The 3½ mi containing only the lime pretreatment and the fly ash were at times nearly impassable, even to tractors. The Webster County maintenance crews spread crushed pit-run gravel at a rate of approximately 1,000 tons/mi over the length of the previously lime-pretreated section. This provided only a light crust as a surface but assisted farmers in the area in getting to and from their farms, fields and livestock.

Construction finally resumed in the first week of August 1960 and the project was completed about mid-October. Procedures of construction were the same as used in 1959.

EVALUATION OF CONSTRUCTION CONTROL

Gradation

Originally, the project was to consist of two 1-mi long sections of 6- and 8-in. compacted thicknesses for each type of stabilizing agent(s) used. Due to the gradation difficulties during the first part of the construction, only the 8-in. section of cement-treated material was constructed, and the two sections of lime pretreatment followed by cement treatment were increased in length by ½-mi each. This decision was based on several factors.

1. Gradations before adding the stabilizing agent continuously ran well under the 80 percent passing the No. 4 U. S. standard sieve no matter how much the contractor aerated and pulverized with the multipass mixers. For example, one 1,200-ft section increased from 63 to 67 percent passing the No. 4 sieve during one full day of mixing and aeration. After 5 to 6 hr of mixing the following day, gradation was still at 67 percent passing the No. 4, yet the clods of material when examined were quite friable though very moist. It was also noted that each of the clods had a rounded or balled appearance after going through the mixer. The combination of moisture, high organic matter content of the soil, and the balling action of the mixer probably prevented the reduction of moisture content and decrease of surface tensile effects between individual particles.
2. A pulverization procedure of first surface air drying, then mixing by machine

and by spiked tooth harrow provided no additional percentages of gradation passing the No. 4 sieve. Appearance of the material remained similar to that noted previously.

3. Four percent cement was added as a conditioner to an 800-ft length of previously scarified, lightly mixed and pulverized base soil. Following dry mixing, the mix was brought to approximate optimum moisture content, mixed, compacted, shaped to approximate crown, and lightly rubber-tire rolled. After 7 days the material was rescarified and pulverized with some degree of success; the maximum pulverization obtainable was about 72 percent of the material passing the No. 4 sieve. This was still not considered satisfactory.

Pulverization difficulties were further shown in later construction. A short section of base already containing fly ash would not pulverize to more than 24.1 percent passing the No. 4 sieve at a moisture content of 22.3 percent. After addition of the 4 percent lime with just over 1 hr of total mixing time, gradation increased to 67.2 percent passing the No. 4 sieve at a moisture content of 16.2 percent. A 500-ft section of base containing fly ash had 17.0 percent passing the No. 4 sieve at a moisture content of 20.6 percent following 4 hr of pulverization. The addition of 2 percent lime changed the gradation to 62.6 percent passing at 19.0 percent moisture after 2 hr additional mixing. An additional 2 percent lime further changed the gradation to 71.0 percent passing at 17.6 percent moisture. In another example, a large frost boil developed at the extreme east end of the project during the winter of 1959-60. The subgrade was removed to a depth of 2 ft and was replaced with soil with 8 percent lime added as a frost treatment. No gradation tests were possible before addition of the lime, though the soil material contained only 20.5 percent moisture. Just before recompaction, 56.1 percent of the mix passed the No. 4 sieve. No moisture content was taken at this point, however.

Thus, gradation requirements of 80 percent of the total material passing the No. 4 U. S. standard sieve were impossible to meet. Help was provided through the use of lime and cement. The average gradation passing the No. 4 sieve of all lime-pretreated sections constructed in 1959 was 73 percent following the 48- to 72-hr pretreat period. The maximum gradation obtainable without pretreatment appeared to be 60 to 70 percent passing the No. 4 sieve and also appeared dependent on the moisture content of the material.

Quantity of Admixture

With the exception of the fly ash, all stabilization agents were spread by a conventional truck-pulled cement spreader. Some difficulty was encountered in controlling the rate of feed of the spreader. Table 3 presents a comparison of the specified and spread quantities of stabilizing agents at each sample location. The calculated spread quantities ranged from 97 percent to 112.5 percent of the specified plan quantities.

Density

The effect of length of time of mixing on the moisture-density relations of cement-treated soils has long been recognized (10). Also, the effect of laboratory vs field determination of moisture-density relations has been noted by most highway materials personnel. The Webster County, Iowa, project is of interest for comparing the density effects with several stabilizing agents. Table 4 gives materials in the various mixes.

Table 5 shows the effect of mixing conditions on optimum moisture-maximum density relations and 7-day moist-cure strength of samples molded in accordance with ASTM Method D 698-58T (basically in accordance with Method C). Spot checks indicated an occasional particle larger than ¾ in., which was then removed from the mix. Samples were obtained as follows:

1. Laboratory mixed and molded—Representative samples of the various field materials were proportioned, mixed and molded in the Soil Research Laboratory, Iowa Engineering Experiment Station.

2. Field dry mixed, laboratory wet mixed, and molded—Immediately following field dry mixing and before application of water, representative samples were removed from

the construction site and returned to a field laboratory for wet mixing and molding.

3. Field dry and wet mixed, laboratory molded—Immediately following field wet mixing and before field compaction, samples were removed from the construction site and molded at the field compaction moisture content.

For each laboratory and field condition of mixing and molding all samples were extruded from the Proctor molds, securely wrapped, and returned to and stored in a humidity room of the Soil Research Laboratory at a constant temperature of 70 F and approximately 100 percent RH. After 7-day moist curing they were tested for unconfined compressive strength. For conditions 1 and 2, combined plots of moisture vs density and moisture (at time of molding) vs unconfined compressive strength were made. The strength results presented in Table 5 are those at optimum moisture content and maximum density obtained from the graphs.

In general, field dry mixing caused an increase in optimum moisture content and a decrease in maximum dry density from that obtained under laboratory mixing conditions. Though field dry and wet mixing showed little change in moisture content from that noted for condition 2, the dry density was decreased even further from that obtained by the field dry mixing only.

Also of interest is the general decrease in unconfined compressive strength due to the variation of mixing conditions on density; the field dry- and wet-mixed specimens had only about 50 to 60 percent of the strength of the laboratory-mixed specimens.

Table 6 presents a comparison of standard maximum density values obtained by the research personnel and those determined and used by the county inspectors at 12 sample locations constructed in 1960. Also presented are depth of base, moisture content and in-place density obtained by county inspection, and the in-place density expressed as a percentage of the values determined by county inspectors and research personnel. The values determined by research personnel were obtained on samples taken from the construction site at each sample location immediately following dry mixing and before application of water. The values determined by county inspection personnel were similarly obtained or were obtained from composite representative samples of the total test sections as noted in Figure 2 and seen in the continuity of the specified densities in Table 6.

TABLE 3

COMPARISON OF SPECIFIED AND CALCULATED SPREAD QUANTITIES OF STABILIZING AGENTS AT EACH SAMPLE LOCATION

Sample Location No.	Specified % dry soil wt.	Spread % dry soil wt.[a]
1	8% portland cement	8.8
2		9.2
3		8.0
4	3% lime pretreatment + 5% portland cement	3.1 + 5.2
5		3.2 + 5.3
6		3.2 + 5.3
7		3.1 + 5.7
8		3.1 + 5.7
9	3% lime pretreatment + 5% portland cement	3.2 + 5.5
10		3.1 + 5.6
11		3.1 + 5.2
12		3.1 + 5.4
13		3.1 + 5.7
14	20% fly ash + 4% lime	--[b] + 4.1
15		-- + 4.0
16		-- + 3.9
17	20% fly ash + 4% lime	-- + 4.0
18		-- + 4.0
19		-- + 3.9
20	3% lime pretreatment + 5% lime	3.1 + 5.4
21		2.9 + 5.5
22		2.9 + 5.4
23	3% lime pretreatment + 5% lime	3.2 + 5.8
24		3.1 + 5.2
25		3.1 + 5.9

[a] Calculated from control densities as determined by Webster County inspectors, spread tonnage of stabilizing agent available to author, and standard plan cross-section of finished area.

[b] Not determined.

TABLE 4

DESIGNATIONS OF VARIOUS LABORATORY- AND FIELD-STABILIZED SOIL MIXTURES[a]

Mix	Field In-Place Materials	Additives
A	In-place gravel-treated surface of road embankment.	8% Dewey Type I portland cement.
B	Same as A.	3% Linwood lime pretreatment plus 5% Dewey Type I portland cement.
C	Same as A but containing 3% Linwood lime as pretreatment, field mixed.	5% Lehigh Type I portland cement.
D	Same as A but containing 20% Waterloo fly ash, field mixed.	4% Rockwell lime.
E	Same as A but containing 20% Minneapolis fly ash, field mixed.	4% Rockwell lime.
F	Same as A.	3% Linwood lime pretreatment + 5% Linwood lime.

[a] All percentages by weight.

TABLE 5

EFFECT OF MIXING CONDITIONS ON OPTIMUM MOISTURE-MAXIMUM DENSITY RELATIONS AND 7-DAY MOIST-CURE UNCONFINED COMPRESSIVE STRENGTH OF MIXES USED IN FIELD TEST

Mix	Lab. Mixed and Molded[a]			Field Dry Mixed, Lab. Wet Mixed and Molded[b]			Field Dry and Wet Mixed, Lab. Molded[b]		
	Opt. Moisture Content (%)	Dry Density (pcf)	7-Day Strength (psi)	Opt. Moisture Content (%)	Dry Density (pcf)	7-Day Strength (psi)	Moisture Content (%)	Dry Density (pcf)	7-Day Strength (psi)
A	14.6	112.9	409	--	--	--	--	--	--
B	15.2	112.2	390	--	--	--	18.6	97.2	156
C	13.8	114.7	485	16.5	108.5	257	16.3	101.9	160
D	15.0	109.8	245	16.0	106.7	175	15.8	101.9	136
E	14.5	112.6	370	17.7	107.4	225	16.2	106.3	238
F	16.2	108.1	230	20.4	99.1	138	18.7	99.8	105

[a] Data are averages of minimum of two samples of each mix.

[b] Data are averages of two to six samples of each mix.

TABLE 6

COMPARISON OF MOISTURE-DENSITY DETERMINATIONS OF 12 SAMPLE LOCATIONS DURING CONSTRUCTION, 1960

Sample Location No.	Opt. Moist.-Dens. Relations, Res. Team		Specified Dry Dens., County Inspectors (pcf)[a]	In-Place Dens., County Inspectors				In-Place Dens., County Inspectors, as Percent of Standard Used by:	
	Opt. Moisture Content (%)	Dry Density (pcf)		Depth of Base (in.)		Moisture Content (%)	In-Place Density (pcf)[b]	County Inspectors	Res. Team
				Const.	Plan				
9	--	--	102.4	6¼	6	19.3	93.3	91.1	--
10	--	--	102.4	7	6	14.7	98.5	96.2	--
11	14.5	112.4	102.4	5¾	6	18.5	101.6	99.2	90.4
12	16.6	106.5	102.4	6¼	6	18.5	100.3	97.9	94.2
13	18.5	106.7	102.4	6	6	15.5	103.9	101.4	97.4
14	17.0	106.4	102.4	8	8	16.9	93.0	90.8	87.4
15	16.0	109.0	104.0	7¼	8	16.8	104.3	101.3	95.7
16	16.8	99.9	107.0	6¾	8	14.9	99.3	92.8	99.4
17	14.2	111.3	114.4	6	6	19.9	92.8[c]	81.1[c]	83.4[c]
18	9.6	124.2	114.4	5½	6	13.4	107.3	93.8	86.4
19	17.7	107.4	114.4	5½	6	18.2	97.5	85.2	90.8
20	18.2	102.5	101.0	8	8	16.1	104.4	103.4	101.9

[a] Optimum moisture contents for maximum dry densities used by county inspectors were not obtained by author and are not included.

[b] Determined by oil method, AASHO Designation: T 147-54.

[c] This section was not accepted on basis of density by county inspectors and was reconstructed.

Comparison of the density values determined by the research team and those specified by the inspectors in Table 6 indicates a degree of variation of density similar to that noted from Table 5. Of significance, however, is the variation in maximum density determined at each sample location by the research personnel. A comparison of these values with the engineering classification of the base course soils of Table 1 indicates a high degree of correlation, i.e., the higher the density the better the quality of the base course soil, and vice-versa. The in-place density follows the same general pattern of higher density for better quality soil materials.

The wide variation of in-place density, expressed as a percentage of the standard used for construction control, is also felt to be related, at least in part, to variation of classification of base course soils. The in-place density expressed as a percentage of standard used for construction control by the county inspectors varied by 18.2 percent and that expressed as a percentage of the density determined by the research team varied by 15.5 percent, exclusive of sample location 17 in each case.

Thus, the variation of density with respect to engineering classification of base soil materials points up the necessity for determination of density of stabilized mixes at every suspected change of soil type. This would also produce a more uniform control of construction compaction as regards normal specifications requiring a contractor to compact a base to a minimum percentage of a standard density.

TABLE 7

RELATION OF MOISTURE-DENSITY DATA OF CORE AND BLOCK SPECIMENS REMOVED FROM BASE TO CONDITION 3 SPECIMENS

Mix[a]	Core and Block Samples[b]		Condition 3 Samples[c]	
	Moisture Content (%)	Dry Density (pcf)	Moisture Content (%)	Dry Density (pcf)
A	19.0	101.1	—	—
B	19.6	101.0	18.6	97.2
D	15.5	103.0	15.8	101.9
F	19.8	104.8	18.7	99.8

[a] Test sections of mixes A, B and F constructed in 1959; test section of mix D constructed in 1960.

[b] Data presented is average of 5 to 10 core or block specimens from each mix (with the exception of mix D which is the average of only 2 specimens for sample location 17) removed from the base in October 1960.

[c] Data presented is average of 2 to 6 samples for each mix and taken from table 5.

The thickness of constructed base noted in Table 6 was measured during the conduct of in-place density determinations and indicates a fairly high degree of performance in construction and inspection to attain the specified compacted thickness of base section.

In October 1960, core and block specimens were removed from the base course sections which were a minimum of 1 mo old from time of construction. In addition to other tests, moisture contents and densities were determined on all specimens, the average values of which are given in Table 7. As an approximate means of comparison to the mixes as originally constructed, Table 7 also gives the moisture contents and densities of the lab-molded specimens of field dry- and wet-mixed samples (condition 3) previously shown in Table 5. It will be noted that the moisture contents and densities of the core and block samples are slightly higher than the condition 3 specimens. The reader might be led to the speculation that the high variations of in-place densities as a percent of standard, presented in Table 6, are not as critical as a first glance might indicate.

In-Place Density by Proctor Penetrometer

The Proctor penetrometer and test procedure were developed for control of field compaction of fine-grained soils in embankments and other earth structures. Curves are normally established for the moisture-density and moisture-penetration resistance relationships on the same graph. Thus, the penetrometer and the curves can be used to control moisture content and/or density.

ASTM Method D 1558-63 sets forth the standard procedure of penetrometer testing for moisture-density relationships. In the test a needle penetrates the soil at a rate of 0.5 in./sec for a distance of 3 in., and the maximum pressure exerted is read off a scale on the stem of the penetrometer handle.

Penetration resistance determinations cannot be obtained on very dry or very granular soils as even the smallest needle size ($1/20$ sq. in. cross-section) will not penetrate. However, it was felt that a revised penetrometer method held some promise for determination of in-place density with stabilized materials, and the Webster County experimental project presented an opportunity to analyze the procedure.

In the standard procedure, the total length of time of test is 6 sec. In the procedure adopted, if the needle did not penetrate at a rate of 0.5 in./sec within the first 3 sec, it would be pushed to its maximum pressure reading (110 lb on the penetrometer used), held for an additional 3 sec, and the depth of penetration measured and recorded. A Penetration Resistance Factor (PRF, psi/in. or pci as assumed herein) was then calculated by the formula

$$\text{PRF} = \frac{(\text{L})\ (\text{S})}{\text{D}} \qquad (1)$$

where L is load in pound, S is reciprocal of needle size in square inches, and D is depth of penetration in inches. Graphical representation of PRF for various needle cross-sections could be produced for rapid determination of PRF knowing these values.

Penetration tests were thus run on three soil-additive mixture test conditions for the sections constructed in 1960:

TABLE 8

RELATION OF MOISTURE CONTENTS, DENSITIES AND PENETRATION RESISTANCE FACTORS AT ALL SAMPLE LOCATIONS CONSTRUCTED IN 1960

				Standard Proctor Penetrometer			
Sample Location	Test Conditions[a]	Moisture Content (%)	Dry Density (pcf)	Needle size (sq in.)	Avg. Penetration (in.)	Avg. Pressure (lb)	PRF (pci)
9	A	13.8	114.7	$^1/_{20}$	--	--	1,650
	C	19.3	93.3	$^1/_{20}$	2.60	97	750
	A	13.8	114.7	$^1/_{20}$	--	--	1,650
10	B	13.3	106.3	$^1/_{20}$	0.37	110	6,200
	C	14.7	98.5	$^1/_{20}$	2.57	97	760
	A	13.8	114.7	$^1/_{20}$	--	--	1,650
11	B	16.3	101.6	$^1/_{20}$	1.25	110	1,760
	C	18.5	101.6	$^1/_{20}$	2.73	107	790
	A	13.8	114.7	$^1/_{20}$	--	--	1,650
12	B	15.3	101.9	$^1/_{20}$	0.93	110	2,400
	C	18.5	100.3	$^1/_{20}$	1.70	106	1,260
	A	13.8	114.7	$^1/_{20}$	--	--	1,650
13	B	18.2	98.7	$^1/_{20}$	1.14	110	1,940
	C	15.5	103.9	$^1/_{20}$	1.77	105	1,190
	A	15.0	109.8	$^1/_{20}$	--	--	1,100
14	B	14.0	102.3	$^1/_{20}$	0.29	110	7,500
	C	16.9	93.0	$^1/_{20}$	1.13	110	1,960
	A	15.0	109.8	$^1/_{20}$	--	--	1,100
15	B	14.8	105.3	$^1/_{20}$	0.53	110	4,200
	C	16.8	104.3	$^1/_{20}$	1.23	110	1,790
	A	15.0	109.8	$^1/_{20}$	--	--	1,100
16	B	17.2	97.8	$^1/_{20}$	1.42	110	1,550
	C	14.9	99.3	$^1/_{20}$	1.20	110	1,830
	A	15.0	109.8	$^1/_{20}$	--	--	1,100
17	B	16.7	99.3	$^1/_{20}$	0.43	110	5,000
	C	16.9	92.8	$^1/_{20}$	2.78	98	710
	A	14.5	112.6	$^1/_{20}$	--	--	1,250
18	B	9.4	123.1	$^1/_{20}$	0.43	110	5,000
	C	13.4	107.3	$^1/_{20}$	1.82	109	1,200
	A	14.5	112.6	$^1/_{20}$	--	--	1,250
19	B	16.2	107.5	$^1/_{20}$	1.00	110	2,250
	C	18.2	97.5	$^1/_{20}$	2.63	99	760

[a] A = Laboratory mixed, molded and tested on representative sample of full road test section; PRF determined from moisture-density-PRF curves.

[b] B = Field dry and wet mixed, laboratory molded and tested.

[c] C = Road base tested; indicated moisture contents and in-place densities determined by Webster County inspectors at or near indicated sample locations.

A. Standard density specimens produced under controlled laboratory conditions using representative lab samples of full test section soils and additives;

B. Standard density specimens on field mixes removed from each sample location during construction immediately following wet mixing and before field compaction; and

C. The finished road base immediately following rubber tire compaction.

A minimum of 5 tests per specimen were run for conditions A and B and 15 for condition C. Three penetration tests were performed 1 ft from each edge, three at each quarter point of cross-section and three at the centerline of the base in each sample location.

Moisture-density and moisture-average PRF graphs were drawn for each mix in test condition A. Average PRF values were determined for each specimen of condition B and for each sample location for condition C. All PRF values for each individual test were read from a graph produced for the $\frac{1}{20}$-sq in. cross-section needle used throughout the tests. Table 8 presents the moisture contents, densities, and penetration results for each test condition at the sample locations of the sections constructed in 1960.

A wide variation of PRF for each test condition may be noted in Table 8. Comparison of moisture contents, densities and PRF values also indicated a widespread variation for each test condition. It was felt that one additional test condition was needed, i. e., PRF values for all specimens laboratory wet-mixed and molded from field dry-mixed samples of each location. This test condition had been considered, but due to extreme time limitations and lack of personnel for all the testing being conducted on the project it was decided to use only the values from condition B for evaluating the field mixing aspect. This decision appears justified on the basis of the moisture-density comparisons of Table 9 for mixes C and D, but does not adequately take into account the variations in field mixing-laboratory molding and laboratory mixing-molded moisture-density variations previously noted.

In general, it was concluded that the results of the testing were of little significance in the development of the practical application of the standard Proctor penetrometer test for use with those stabilized materials.

Variation of Moisture Content with Depth in Base

Continual inspection of the field mixing operations had noted discontinuities with depth of stabilization additives and moisture content. The general appearance was of pockets of unmixed additives and slightly dryer material in the lower depths of the loose mixes. The contractor was required to mix each section further until the loose material had a more uniform appearance and feel before compaction.

Though no check was run on quantity of additives vs depth, moisture contents vs depth in base were performed at sample location 15 immediately following compaction. Table 10 presents the results. It will be noted that there was an average variation between the top and bottom of the base of almost 5 percent moisture.

Results are rather conclusive that the wet mixing had been inadequate at this single sample location. Although it is not known what moisture content conditions existed at other sample locations, it may be speculated that moisture variations with depth also occurred elsewhere. It may also be speculated that the multipass mixers used for this project were not completely capable of uniform mixing procedures. This latter statement is also based on visual observations of the difficulties encountered in reaching full depth of loose base thickness.

TABLE 9

COMPARISON OF AVERAGE MOISTURE-DENSITY DETERMINATIONS OF IN-PLACE BASE MATERIAL AND SPECIMENS MOLDED FROM FIELD MIXES IMMEDIATELY BEFORE FIELD COMPACTION

Mix	In-Place Specimens		Condition 3 Specimens[a]	
	Moisture Content (%)	Dry Density (pcf)	Moisture Content (%)	Dry Density (pcf)
C	17.3[b]	99.5[b]	16.3	101.9
D	15.8[c]	101.0[c]	15.8	101.9

[a] From Table 5.
[b] Includes sample location 9, 10, 11, 13 from Table 6.
[c] Includes sample location 14, 15, 16, 18 from Table 6.

TABLE 10

MOISTURE CONTENT VS DEPTH IN BASE IMMEDIATELY FOLLOWING CONSTRUCTION AT SAMPLE LOCATION 15

Depth Sampled (in.)	Avg. Moisture Content 4 Ft Right of Centerline (%)	Avg. Moisture Content 4 Ft Left of Centerline (%)	Avg. Moisture Content from Both Holes (%)
0-1	15.7	17.1	16.4
1-2	15.8	15.8	15.8
2-4	16.0	15.2	15.6
4-6	14.0	13.1	13.5
6-8	13.0	10.2	11.6

Adequacy of Length of Lime Pretreatment Time

As previously indicated, all sections pretreated with lime were allowed to react for a minimum of 48 to 72 hr before addition of stabilizing agent. As a means of checking the adequacy of this specification, sieve analyses, liquid limits (L. L.) and plasticity indexes (P. I.) were run on samples of lime-pretreated soil following 0 (untreated soil), 1, 6, 16, 24, 48 and 72 hr of curing. The soil used was a representative sample from full depth of base soils taken at several locations over the 6-in. section of lime pretreatment plus cement treatment immediately following field scarification and before addition of lime. With the exception of several stones, which were discarded, all the sample passed the 1-in. screen as removed from the field site. Average moisture content of the sample was 17.1 percent. The following laboratory test procedure was used in an attempt to simulate field pulverization and lime pretreatment construction conditions:

1. A 7,000-g sample was quartered from the total sample.
2. A 1,000-g portion was quartered out as the untreated sample and 3 percent lime, by dry soil weight, was then added to the remaining portion.
3. Both the untreated and treated samples were machine mixed for 10 to 5 min, respectively. This constituted zero time.
4. Sieve analysis of the untreated sample was immediately run using the 1-, $^3/_4$-, and $^1/_2$-in. and Nos. 4, 10, and 40 sieves; the treated sample was divided into six equal portions. Each treated portion was sealed and placed in the humidity room until time of testing.
5. At the end of each treatment period, the samples were machine mixed for an additional 5 min before sieve analysis as in step 4.
6. L. L. and P. L. were determined for all material passing the No. 40 U. S. standard sieve immediately following sieving.
7. All sieve analyses were expressed as a percentage of the weight of the total sample. All L. L., P. L. and P. I. data were expressed as a percentage of the oven-dry weight of the material.

Results of the testing are shown graphically in Figure 4. After treatment with lime, it was impossible to obtain a P. L. determination since the material would change rapidly from a thread of about $^1/_8$-in. diameter to a friable mass of crumbs. In 1 hr, the lime treatment reduced the L. L. of the soil from 40 to 36 percent with little additional change during the period of 1 to 72 hr.

Results of the sieve analysis were somewhat more erratic. The most signficant increase in particle size reduction will be noted with the No. 10 sieve. Percentage passing the No. 10 sieve increased from 33.8 percent for the untreated sample to 46.4 percent with 1 hr of lime pretreatment and averaged 43.3 percent for the remainder of the 72-hr period.

Since time, additional construction costs, and established construction sequences did not permit a similar field evaluation of length of pretreatment period, the minimum 48- to 72-hr curing was maintained throughout construction. However, the results of the laboratory study strongly indicated that a field curing period of 24 hr or less might be sufficient for future lime-soil pretreatment construction operations for reduction of plasticity and increase of pulverization-gradation requirements. It was judged that field pretreatment lasting longer than 72 hr would provide little or no additional benefits to pulverization requirements to offset any additional delay of construction. This conclusion was further shown by sections pretreated with lime in 1959 and finished in 1960 which showed an average gradation passing the No. 4 U. S. standard sieve of 74.1 percent—a rather small increase from the 73.0 percent average following the 48- to 72-hr pretreat period shown previously during 1959 construction.

Base Surface Raveling

During construction in 1960, some of the primed base course sections remained without surfacing for as long as several weeks. Though barricades were maintained, traffic still managed to drive on the base surface. Visual examination revealed a sur-

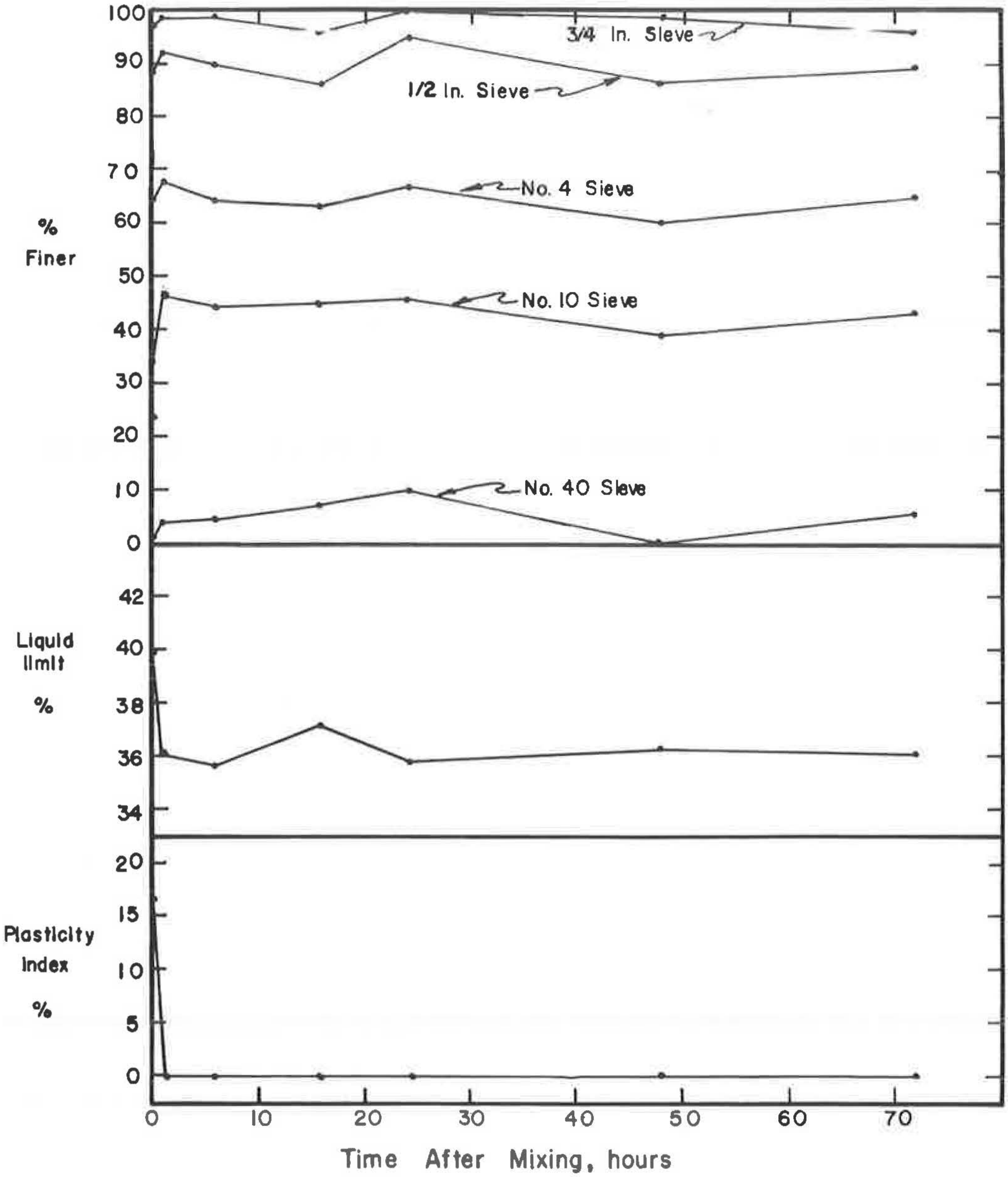
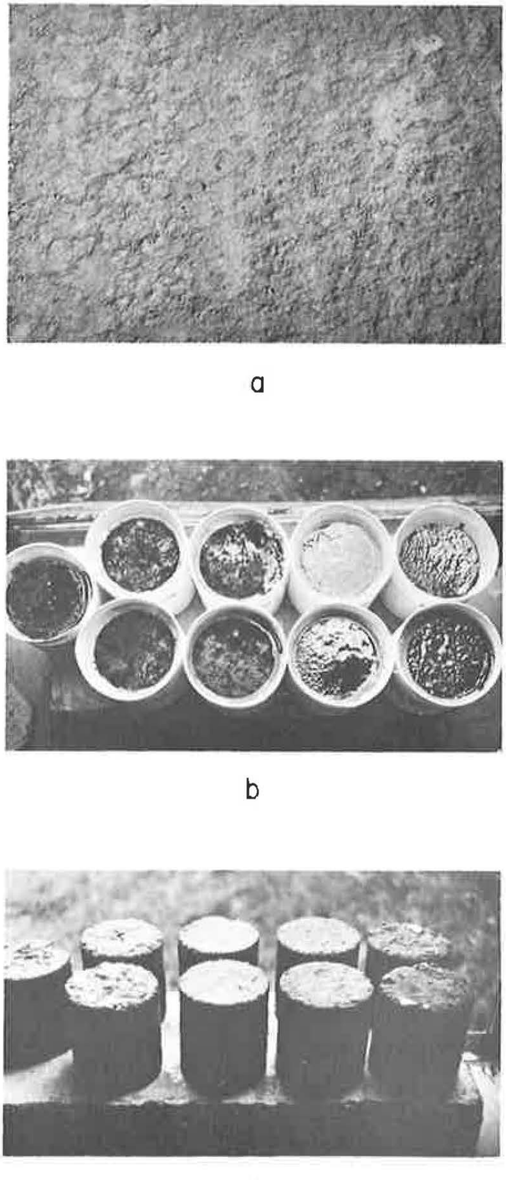


Figure 4. Effect of curing time on sieve analysis, L.L., and P.I. of laboratory-mixed lime-pretreated base course soil.

face checking, slight raveling and, in some cases, softening of the upper $\frac{1}{2}$ to $\frac{3}{4}$ in. of the base sections (Fig. 5a). It also appeared that the MC-0 prime had penetrated the base surface to a similar depth. X-ray diffraction studies of samples removed from the upper 1 in. of base revealed nothing as to the cause of the checking or softening in relation to loss or ineffectiveness of stabilization additive(s).

A laboratory study was undertaken in an attempt to duplicate field finishing methods of blading, moistening, priming and curing. Duplicate standard density specimens were molded from laboratory samples of representative materials below, near, and

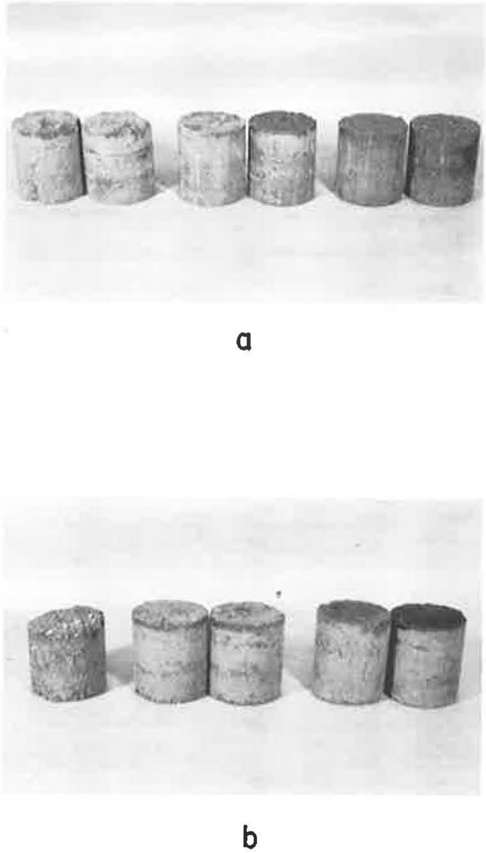


a

b

c

Figure 5. Base course raveling study: (a) primed base left without surfacing for several weeks during 1960 construction showed surface checking, raveling and some softening after limited traffic application; (b) specimens molded at varying moisture content increasing from left to right, with near optimum moisture content in two specimens second from right; each specimen shown as it appeared following designated curing period and just before removal from carton for brushing; and (c) specimens immediately following brushing.



a

b

Figure 6. Examples of specimens tested in base course raveling study following brushing: (a) specimens of mix A (specimen second from left cracked on compaction plane during brushing); and (b) specimens of mix F.

above optimum moisture content of each mix. One field-mixed sample included in the testing was removed from near location 13 in the 6-in. lime-pretreated, cement-treated section immediately following dry mixing. The procedure was as follows:

1. Duplicate standard density specimens were molded below, near, and above optimum moisture content for each laboratory mix designation. Duplicate specimens were molded from the field-mixed sample for each point of a standard moisture-density curve.
2. After compaction, the collar of the mold was removed and the specimen was trimmed even with the top of the mold with

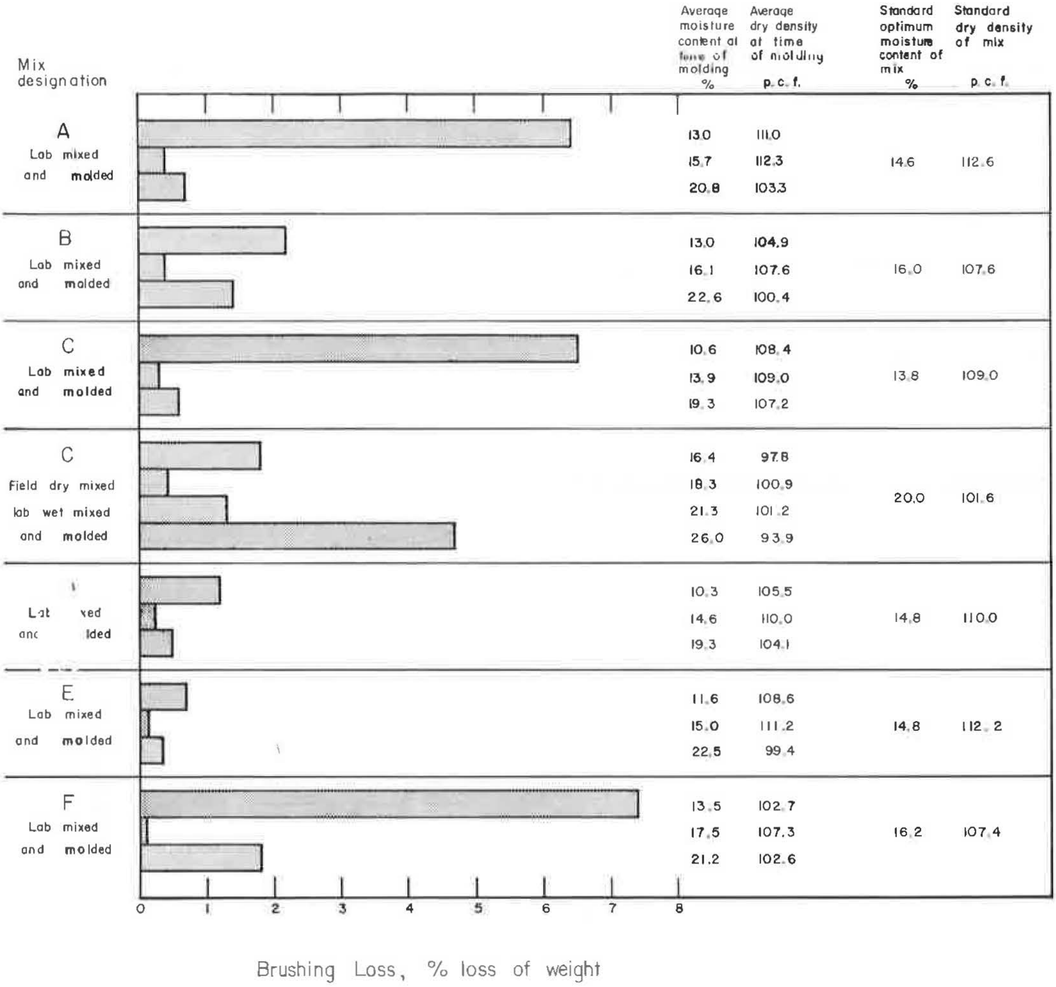


Figure 7. Effect of average moisture content and density of laboratory compacted specimens on percentage loss of weight during brushing.

a straight edge applied in only one direction. Any holes developed in the specimen surface by removal of coarse material or surface blading cracks were not compacted or patched as in ASTM Designation: D 698-58T.

3. Each specimen was extruded from the mold by pushing from the bottom up, weighed, and placed in an ice cream carton of diameter about 1-in. larger than that of the specimen and in which paraffin had previously been placed to a depth of about ½ in. in the bottom. Moisture tests were made on soil from the mixing bowl.

4. Melted paraffin was slowly poured in the space between carton and specimen (so that no air bubbles would collect) until level with the specimen surface. Each container was then allowed to cool to room temperature.

5. Water was added to the specimen surface at a calculated quantity of 2 percent additional moisture for the upper 1 in. as based on standard optimum moisture content at maximum density. This was assumed to be approximately the same quantity used in the field construction before priming.

6. About 15 min later, MC-0 cutback asphalt heated to 50 C was applied at a rate of 0.25 gal/sq yd, i.e., at the same rate and approximately the same time lapse as in the field construction.

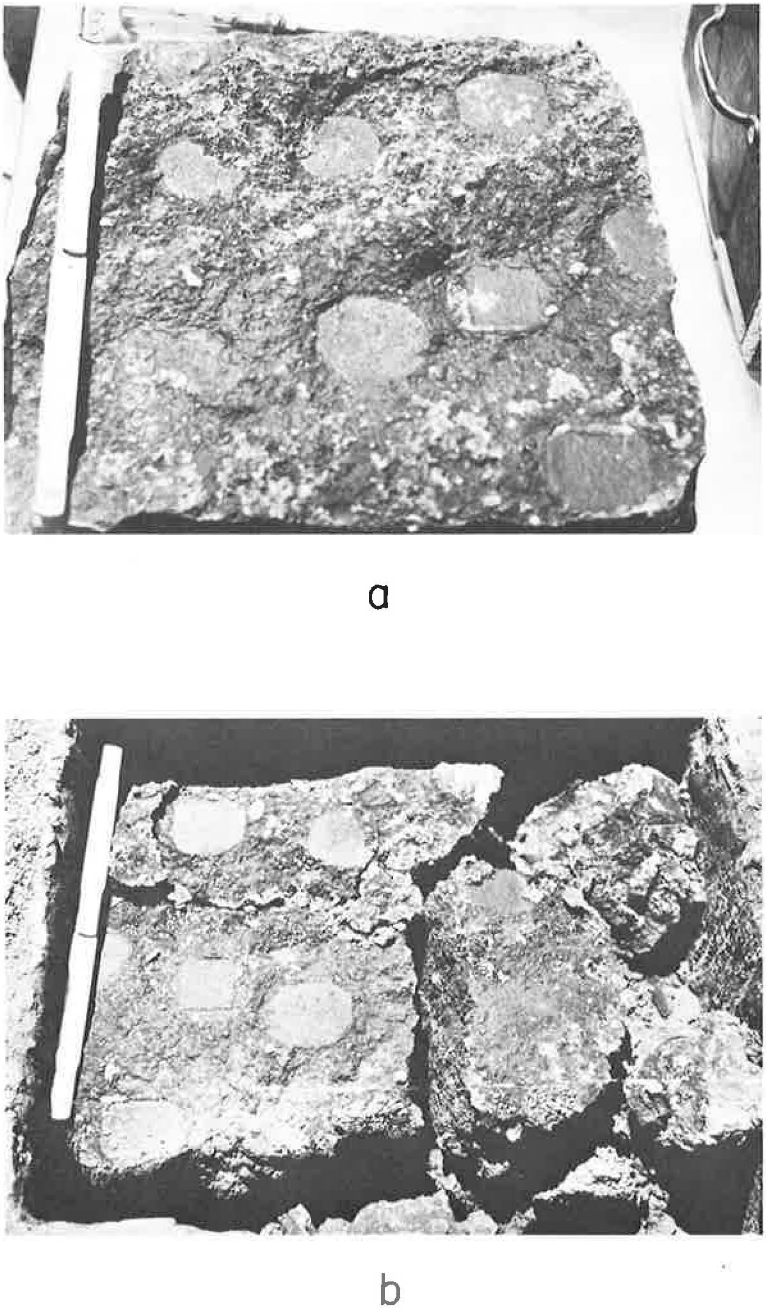


Figure 8. Effect of inadequate removal of compactor footprints during compaction: (a) upper portion of a block sample removed from lime-pretreated, lime-stabilized 6-in. base section; and (b) lower portion of the same block sample. Cracking occurred when lower portion of block dropped back in hole during removal.

7. The specimens were then allowed to cure in a room atmosphere for 7 days.
8. Following curing each specimen was carefully removed, cleaned, weighed, and the top surface was firmly steel wire brushed 100 times while the specimen was constantly turned.
9. After brushing, each specimen was again weighed and the loss due to brushing was computed as a percentage of the weight immediately before brushing.

Examples of the test are shown in Figures 5b, 5c, 6a and 6b. Results of testing are shown in Figure 7.

In all mixes the brushing loss was higher at moisture contents either lower or higher than standard optimum. However, the loss at moisture contents higher than optimum were considerably less than the loss at those lower than optimum, disregarding the loss at 26.0 percent moisture in the field-mixed Mix C.

One hr after application, it was estimated that 75 to 80 percent of the prime had soaked into the samples having less than optimum moisture and maximum density. Surface checking was visible after 72 hr in all specimens on the low side of the moisture-density curve.

After 7-days curing, dry surfaces, checking and cracking were noted on all low moisture-density specimens. An estimated 50 percent or more of the prime remained on the surfaces, and there was some visible but very slight surface checking on all specimens at or near standard optimum moisture-maximum density. An estimated 50 to 100 percent of the prime remained on the surfaces, with the checking ranging from none visible to slight, on all high-moisture specimens. Following brushing, the specimens molded near and above optimum moisture content still had some asphalt adhering tightly to them. It was apparent that much of the brushing loss of these specimens could be attributed to chipping of the surface edges during brushing.

Field observations tended to corroborate the observations made on the tested specimens. Patches of the base surface seemed to soak up the primer and within a short time would appear dry and somewhat checked. Other areas of the base surface after several days from priming were still very tacky with asphalt, though some surface checking was visible.

Evaluation of the tests and laboratory and field observations made indicate that the surface checking and, in some cases, softening of the base is due at least in part to the absorption of the primer by the base material at somewhat less than optimum moisture content and maximum standard density. Traffic on the unsurfaced roadway undoubtedly aggravated the checking and created some raveling.

Removal of Compactor Feet Marks During Compaction

During construction, initial compaction was accomplished with a tamping foot roller and final compaction was done with a segmented grid roller, each working in conjunction with a spiked tooth harrow for removal of tractor tread and roller grid marks. During removal of core samples in 1959, it was noted that an occasional sample would transversely split about mid-height of its longitudinal axis either during drilling or following removal from the core barrel. Visual examination of these broken cores indicated a flat surface about the size of the tamping foot marks, but the cause was not verified.

In early spring 1960, three block samples approximately 1 ft square in cross-section were randomly cut from the full depth of the 6-in. compacted thickness lime section. During removal, one of the blocks split over its full cross-section and about midway of its full depth, exposing the pattern of the tamping foot compactor (Fig. 8). Each tamping foot mark had a shiny, almost glassy, appearance, much in contrast with the broken appearance of the areas in between.

Since 1960 an occasional sample removed by coring has also indicated evidence of breakage along a tamping foot mark; no visible evidence of grid marking has been noted to date. The spiked tooth harrow was apparently inadequate to reach deep enough for removal of tamping feet marks but was adequate in the upper area of the base.

It should not be assumed that the entire base course is in distress due to possible planes of weakness created by nonremoval of tamping feet marks, as there is much evidence (though not a part of this paper) to show the adequacy of the supporting capacity of all the base sections. However, it may be assumed that there are random limited areas where a heavier and longer toothed drag would have assisted in removal of all compactor foot markings and produced a more durable base course.

SUMMARY AND CONCLUSIONS

The major purpose of the Webster County, Iowa, experimental project was to produce and evaluate a low-cost stabilized soil base course made from an existing gravel-treated surface of road embankment using conventional construction equipment. The road has now given more than 3 years of quality service and has sustained severe freezing and moisture conditions during two of the worst winters in over 30 years in Iowa. This in itself is a tribute to the quality of construction control measures used in the experimental project.

Following is a summary of the major evaluations regarding the construction and the control measures used to produce the stabilized soil base course:

1. Pulverization-gradation requirements of 80 percent of the total material to pass the No. 4 U. S. standard sieve before introduction of stabilizing agent were impossible to meet. Additional laboratory and field studies are needed to adequately define and specify minimum gradation requirements of Iowa soils before introduction of soil stabilizing agents. However, 60 to 70 percent passing the No. 4 sieve appears the maximum gradation obtainable without lime or cement pretreatment for the type of soil material encountered in this project.
2. Introduction of 3 percent lime as a pretreatment agent for the soil materials assisted in a reduction of plasticity, increase of friability and increase in pulverization gradations obtainable. The 48- to 72-hr lime pretreatment period appeared adequate to produce the maximum practical field benefits. Laboratory pretreatment studies indicated maximum benefits in about 1 hr. With further field evaluation it appears possible to reduce the length of pretreatment curing time to 24 hr or less.
3. Specified spread quantities of stabilizing agents were not obtained with the conventional cement spreader used during construction but in general were within a normal 10 percent tolerance.
4. The variation of control and in-place densities with respect to the engineering classification of the various base soils encountered illustrated the need for an optimum moisture-maximum density test of field-mixed stabilized materials at every suspected soil change during construction.
5. The use of a standard Proctor penetrometer as a rapid means of in-place moisture and/or density measurements appeared of little practical value without further testing and development of test procedure.
6. As a means of comparison of uniformity of dry and wet mixing of the materials, trials of additional types of mixing equipment would have been desirable. The multi-pass mixers used on the project appeared to have difficulty in obtaining a uniform mixture of base material and in reaching the loose mixing depth required in the 8-in. compacted thickness sections.
7. For prevention of surface checking and softening due to penetration of the MC-0 primer during the minimum 7-day curing period, the constructed base material should be of a moisture content at or about 1 to 2 percent above optimum moisture of the field-mixed materials.
8. Spiked tooth drags used in removal of tractor tread and compactor foot marks should follow the first pass of the compactor and be of a weight and length adequate to reach through the uncompacted material and scratch the surface of the compacted layer.

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A special remembrance is also made of the late Dr. D. T. Davidson, Iowa State University, who gave so much of himself to the present-day knowledge of soil stabilization and who counseled the author in the research on this project, not only as a colleague but also as a friend.

REFERENCES

1. Hoover, J. M., Huffman, R. T., and Davidson, D. T. Soil Stabilization Field Trials, Primary Highway 117, Jasper County, Iowa. Highway Research Board Bull. 357, pp. 41-68, 1962.
2. Pietsch, Paul E., and Davidson, D. T. Effects of Lime on Plasticity and Compressive Strength of Representative Iowa Soils. Highway Research Board Bull. 335, pp. 11-30, 1962.
3. Handy, R. L., and Davidson, D. T. Cement Requirements of Selected Soil Series in Iowa. Highway Research Board Bull. 267, pp. 14-27, 1960.
4. Davidson, D. T., and Bruns, B. W. Comparison of Type I and Type III Portland Cements for Soil Stabilization. Highway Research Board Bull. 267, pp. 28-45, 1960.
5. Soil Stabilization with Portland Cement. Highway Research Board Bull. 292, 1961.
6. Goecker, W. L., Moh, Z. C., Davidson, D. T., and Chu, T. Y. Stabilization of Fine and Coarse-Grained Soils with Lime-Flyash Mixtures. Highway Research Board Bull. 129, pp. 63-82, 1956.
7. Hoover, J. M., Handy, R. L., and Davidson, D. T. Durability of Soil-Lime-Flyash Mixes Compacted Above Standard Proctor Density. Highway Research Board Bull. 193, pp. 1-11, 1958.
8. George, K. P., and Davidson, D. T. Development of a Freeze-Thaw Test for the Design of Soil-Cement. Highway Research Record No. 36, pp. 77-96, 1963.
9. Lu, L. W., Davidson, D. T., Handy, R. L., and Laguros, J. G. The Calcium-Magnesium Ratio in Soil-Lime Stabilization. Highway Research Board Proc., Vol. 36, pp. 94-805, 1957.
10. Felt, Earl J. Factors Influencing Physical Properties of Soil-Cement Mixtures. Highway Research Board Bull. 108, pp. 138-162, 1955.

Lime and Phosphoric Acid Stabilization in Missouri

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This report presents information accumulated since 1958 on a 17.37-mi chemical stabilization project in Worth and Gentry Counties in northwest Missouri. The results of field investigations and laboratory research tests, preliminary to construction in 1961-62, are reviewed. Explanation of the design procedure, based on Missouri's Flexible Pavement Thickness Chart, is included. Also reported are results of tests during construction, and those of subsequent investigations by Benkelman beam, rut gage, roughometer and core drill.

The first section of the report is limited to details of the lime stabilization on all three projects, F-297(7) and F-297(9) in Worth County and F-524(2) in Gentry County. The second section reviews the design, construction and performance of the 16 test sections composing 2 mi of Project F-524(2), Gentry County, in which various combinations of 4- and 8-in. thicknesses of rolled stone base were constructed on untreated glacial clay subgrade, on a 5-in. subbase of that soil mixed with lime or with phosphoric acid, or on a 5-in. rolled stone subbase.

•EARLY IN the 1920's, shortly after becoming Missouri's first Materials Engineer, F. V. Reagel started experimenting with methods and materials which might be used to process native soils and adapt them to more effective use in highway construction. Throughout the following years Missouri's highway engineers have diligently attacked the problem while applying different principles of road-building science to various types of soil.

Experience with lime stabilization was primarily confined to a project constructed in 1952 by maintenance forces (1). Two separate sections on Route 51, Perry County, totaling approximately 2 mi, were treated with hydrated lime, quicklime, a combination of the two materials, and a stackdust waste product. Periodic tests since that time have indicated that permanent improvement of the soil resulted from the treatment.

The most recent and most extensive study was constructed in 1961-62 in Worth and Gentry Counties, although preliminary work started as early as February 1958 when a soil survey was made and routine laboratory tests were completed.

A further step, preceding the design stage, involved extensive laboratory research on mixtures of the soil with lime and phosphoric acid. A short pilot section was constructed in another area, with phosphoric acid as the stabilizing agent, to determine the practicability of field processing such materials. With the fundamental preliminary work completed, and with due consideration for unknown influencing factors such as changes in soil or projected traffic, the thickness was determined in conformity with Missouri's method of flexible pavement design and to the standards current in 1959.

A contract was awarded to the Howard Construction Co. of Sedalia, Mo., in February 1960 for Project F-297(7), Worth Co., from the Iowa line south for 10.328 mi. Early

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in 1961, the same contractor was awarded Project F-297(9), continuing 2.497 mi south across Worth Co., and an adjacent 4.545-mi project, F-524(2), in Gentry Co. All work was completed by October 1962.

The study herewith reported consists of four principal phases:

1. Selection of a location where reasonably uniform soil conditions prevailed and the predominant soil reacted favorably when mixed with lime or phosphoric acid (and of only slightly secondary importance, a location where aggregate deposits were scarce);
2. Extensive laboratory research to investigate the effects of adding the stabilizing agents and to determine the optimum amounts to use;
3. Design and construction, guided and controlled by application of the knowledge gained in the laboratory; and
4. Studying the performance of the completed project.

LIME STABILIZATION

Purpose

This experimental project was designed and constructed to determine the performance of a road built with a 5-in. by 30-ft trenched subbase of natural soil modified by mixing 6 percent hydrated lime into the glacial clay, covering the subbase with an 8-in. compacted thickness of rolled stone base from inslope to inslope, and surfacing a 24-ft width with 3 in. of asphaltic concrete in two courses. These dimensions apply to the total length of Projects F-297(7) and F-297(9) in Worth Co. and to more than half of the 4.545 mi of Project F-524(2) in Gentry Co.

The remaining 2 mi in Gentry County were designated as the site of a complete factorial experiment of sixteen 660-ft sections. Included were duplicate sections with 4- or 8-in. thicknesses of rolled stone base over a 5-in. subbase of rolled stone or a 5-in. subbase of native soil modified by lime or phosphoric acid. Four sections had no subbase. All test sections were surfaced with 3 in. of asphaltic concrete. Three sections were treated with acid at 50 percent strength and one at 75 percent, in quantities ranging from 2.8 to 4.3 percent, on a dry soil basis. Six percent lime was used in the lime sections. Some test sections were deliberately underdesigned.

Scope

This study, as stated previously, began with an early routine soil survey in which the soils were identified and sampled, and their extents were determined. Normal procedures were followed in the laboratory testing of these samples to determine mechanical analysis, plastic index (P. I.), group index, maximum density at optimum moisture (AASHO Designation: T 99, Missouri modification), and A-group classification.

The uniform clayey soil conditions and the scarcity of aggregate deposits prompted the suggestion that lime stabilization should be applicable, beneficial, and economical in this area. Tentative approval incited further sampling and more exhaustive testing, which involved the foregoing series of tests, plus slaking and unconfined compression tests. All tests were made on specimens of Shelby soil mixed with 4, 5 and 6 percent lime.

The testing program during construction was elaborate and thorough. The subgrade, lime subbase and base were, in general, all sampled at the same place, at 2,000-ft intervals. The following tests were performed on subgrade, lime subbase and base: liquid limit (L. L.), plastic limit (P. L.), optimum moisture, maximum density, field moisture, field density, and laboratory CBR. Additional tests on subgrade and subbase were shrinkage limit, free swell, and compressive strength; on subgrade, mechanical analysis; on subbase, passing No. 200 sieve, pulverization, and thickness, and on base, sieve analysis, thickness, and L. A. abrasion.

Rutting and deflection measurements were made in conjunction with crack surveys on various sections of these projects in November 1961 and May 1962 (before opening to all traffic), June 1963, fall of 1963, and spring of 1964. In August 1963, an attempt was made to measure deflections in each component of the complete structure. Permeability tests were made on the asphaltic concrete in December 1962. Distress has twice been reported and investigated.

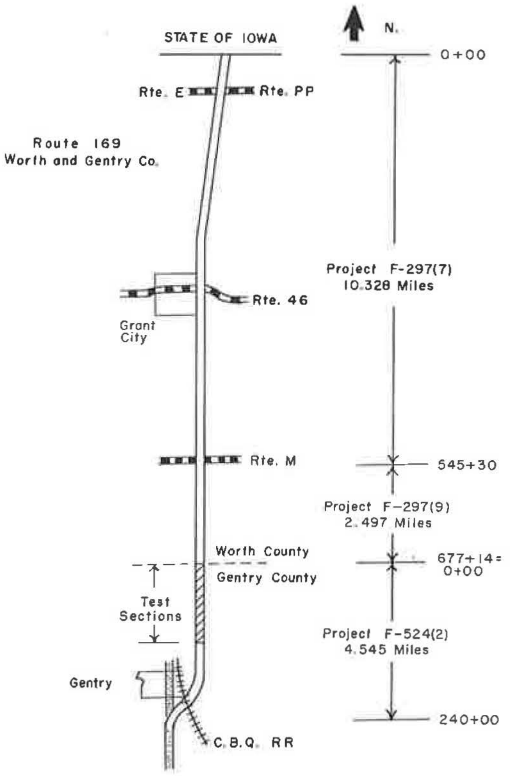


Figure 1. Study site.

Location

The experimental section is located about 60 mi northeast of St. Joseph, Mo. The length of each of the three projects is shown in Figure 1.

Topography

In general, Route 169 traverses heavily rolling country with narrow valleys and steep-sided, deeply eroded gullies. The grade consists mainly of a series of cuts and fills, of which many exceed 20 ft and a few are more than 40 ft.

Soils

The C horizon of the Shelby soil series predominates throughout this project. Glacial in origin, the C horizon is usually a yellow clay, classified A-7-6(16), and averages about 30 percent glacial sand and gravel. The clay mineral fraction is a calcium montmorillonite composed of approximately 58 percent montmorillonite, 20 percent illite, and 22 percent kaolinite.

From Station 200 (Gentry Co.) to the south end of Project F-524(2) at Station 240, the soil is Wabash clay, a black, organic, alluvial deposit classified as Group A-7-6(18-20). Since it is a derivative of the Shelby, it is considered also to be primarily montmorillonitic.

Preliminary Testing

Samples representative of the C horizon Shelby were obtained in 1958 while making a routine soil survey. Laboratory tests showed the following average results: L. L., 48; P. I., 35; S and C, 65 percent; M. D., 108 lb; O. M., 17.7 percent; and classification A-7-6(17) clay.

TABLE 1

PRELIMINARY LIME-SOIL TESTS, WORTH COUNTY

Station	Lime (%)	L. L.	P. L.[a]	P. I.[a]	S. L.[a]	Passing No. 200 Sieve (%)[a]	Silt (%)[a]	Clay < 5 μ (%)[a]	Group	Comp. Strength (psi)[b]	Slaking[b]
23 + 00	0	55	31	24	12.9	98.8	48	43	A-7-5(17)	—	—
	4	56	38	18	23.6	23.0	—	—	A-2-7(1)	39	Very slight
	5	54	36	18	23.7	23.3	—	—	A-2-7(1)	33	Slight
	6	51	39	12	27.8	20.0	—	—	A-2-7(0)	39	Minor
263 + 00	0	40	20	20	14.2	67.5	27	33	A-6(10)	—	—
	4	37	28	9	29.9	18.7	—	—	A-2-4(0)	44	None
	5	38	28	10	25.9	23.2	—	—	A-2-4(0)	44	None
	6	37	29	8	24.4	20.3	—	—	A-2-4(0)	52	None
352 + 00	0	46	18	28	9.8	75.4	27	40	A-7-6(16)	—	—
	4	39	30	9	22.7	6.8	—	—	A-2-4(0)	41	3/16 in.
	5	39	29	10	25.1	4.5	—	—	A-2-4(0)	82	1/8 in.
	6	40	32	8	26.6	2.9	—	—	A-2-4(0)	92	Small loss

[a] Made on loose material after 24-hr moist curing.

[b] Made on 4- by 4.6-in. specimens compacted at optimum moisture and to maximum density, moist cured 7 days, dried 8 hr at 140 F, and soaked for 16 hr.

To permit research into the effects of mixing lime with the soil, more samples were obtained. The material from Station 352 was considered to represent the job's predominant soil. The results of tests on this and other soils, as well as on mixtures of each with 4, 5 and 6 percent lime, are given in Table 1. Based on these results, a lime content of 6 percent by weight of dry soil was recommended because:

1. All such mixtures showed the greatest reduction in plasticity (probably even greater if cured compacted specimens rather than loose material had been tested);
2. Two of the three soils showed higher shrinkage limits, indicating less volume change;
3. The same two soils showed more agglomeration, as indicated by the lower minus No. 200 content;
4. The compressive strength was highest with two of three soils, and as good with the other soil; and
5. Slaking was negligible with all the 6 percent mixtures, but was rather severe for the 4 and 5 percent mixtures with soil from Station 352.

Design

The Missouri Flexible Pavement Thickness Design Chart was used to determine the total roadbed thickness. At the time, no equivalencies had been determined for rolled stone base vs other types of base construction. Total thickness was governed by the 20-yr anticipated heavy axle traffic and the group index of the subgrade. On this basis a 16-in. total thickness was recommended, consisting of a 5-in. lime-treated subbase, 8 in. of compacted crushed stone base, and 3 in. of asphaltic concrete, as shown in the typical section (Fig. 2).

Specifications. —The following specifications were to apply to all lime-stabilized sections covered by this section of the report:

1. Hydrated lime—to comply with requirements of ASTM C 207-49, omitting Sections 3a and 5b;
2. Subgrade—top 18-in. to be compacted to a minimum of 95 percent of T 99 maximum density (Missouri method, using four layers when P. I. exceeds 25);
3. Subbase—5-in. depth, trenched 30 ft wide, to be compacted to a minimum of 95 percent of standard T 99 density;
4. Base—8-in. compacted rolled stone base to be placed in two 4-in. lifts, each compacted to a minimum of 95 percent AASHO T 99 density;
5. Surface—3-in. of asphaltic concrete, 24 ft wide, consisting of $1\frac{3}{4}$ in. of Type B base course and $1\frac{1}{4}$ in. of Type C surface course;
6. Shoulders—9 ft wide where guardrails were specified, elsewhere 7 ft wide; on the high side of superelevated curves surfacing as on the 24-ft riding surface, elsewhere paved with Type B asphaltic concrete tapered from a thickness of about 2.5 in.

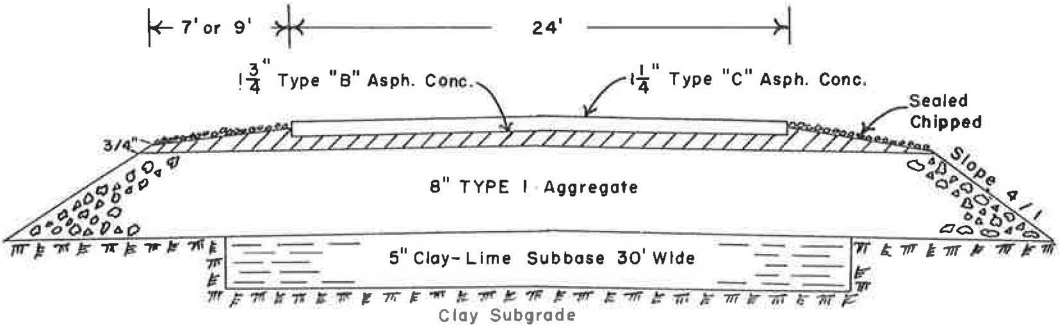


Figure 2. Typical section, lime subbase.

at the riding surface edge to $^3/_4$ in. at the edge of the shoulder, and sealed with 200-250 penetration asphalt or RC-4 cutback asphalt, and white stone chips;

7. Pulverization—on Project F-297(7), the first constructed, the only requirement was that 100 percent pass the $1^1/_2$-in. sieve after lime was added. On Projects F-297(9) and F-524(2), specifications were changed to require that 100 percent pass the 1-in. sieve and at least 60 percent pass the No. 4 sieve. Moisture was to be within 2 percent of optimum at the completion of mixing and during compacting.

Construction

Equipment. —The contractor chose to use bulk lime and mix in place with the soil. For these projects the following equipment was found to be satisfactory and sufficient: one 10-ft P & H one-pass stabilizer, one Seaman lime spreader, three lime-hauling dump trucks, two water trucks (5, 000 and 1, 500 gal), one motor grader, and one 12-ton pneumatic roller.

Procedure. —Construction procedure involved spreading lime, processing soil, compacting the mixture, finishing and sealing.

Lime Application. —Lime, furnished by the Marblehead Lime Co. of Hannibal, Mo., was distributed on the unscarified finished subgrade by the Seaman spreader. Because of the varying air content and fluffiness of the bulk lime it was sometimes necessary to make two passes of the spreader to apply the proper quantity.

Processing the full 30-ft width in three passes with the stabilizer removed some of the subgrade crown and diluted the lime. The crown could be protected by making four passes, but progress would be considerably reduced by such a procedure. Therefore, it was decided to compensate for the additional soil by increasing the lime quantity slightly and processing in three strips. The lime spread was increased from 72 to 75 lb/lin ft, a quantity which was generally maintained throughout the remainder of the lime stabilization work on all three projects (Table 2).

Processing. —With the exception of perhaps 10 percent of the project's length, satisfactory pulverization (100 percent pass the $1^1/_2$-in. sieve) was obtained on Project F-297(7) after one pass of the P & H machine at its slowest speed, 5. 5 ft/min, and no rotting period or remixing was necessary. An average of screening tests indicated that all material passed the $1^1/_2$-in. sieve, approximately 97 percent passed the 1-in. sieve, and the minus No. 4 content was 43 percent.

To comply with gradation specifications on Projects F-297(9) and F-524(2) it was essential to allow a seasoning period during which the initial soil-lime reaction occurred and the breakdown of clay lumps began. Although a few short sections showed satisfactory pulverization after only a 6-hr rotting period, 24 hr was usually allowed before remixing. In no instance was the specified 72-hr maximum exceeded.

Because the soil-lime mixture was much more friable after seasoning, the remixing pass of the P & H could usually be made in second gear at 10. 1 ft/min. Occasional trials prove that any speed faster than that did not permit satisfactory pulverization.

TABLE 2

SUMMARY OF LIME SPREAD DATA[a]

Project	Tot. Ft. Proc.	Tot. Lime (tons)	Avg. Appl. Rate (lb/lin ft)	Tot. Time for Appl. & Proc. (days)	Avg. Daily Prod. (ft)
F-297(7)	56, 773. 8	2, 127	74. 9	45	1, 261. 6
F-297(9)	13, 183. 6	511. 5	75. 4	19	693. 9
F-524(2)	12, 488	460. 37	73. 7	16	780. 5

[a] Detailed project data from which this summary was derived are available from the Highway Research Board, Supplement XS-5 (Highway Research Record No. 92), 29 pp.

Because the optimum moisture of the soil-lime mixture was higher than that of the natural soil, water had to be added through the P & H during the initial pass. Light rolling immediately followed to reduce evaporation and, by somewhat densifying the mix, to promote the reaction of the soil-lime-water system. During remixing it was again necessary to add water.

Spray nozzles in the P & H sometimes became plugged, causing variations in the amount of water added. Soil moisture also varied because of the frequent rains. Satisfactory moisture control was nevertheless maintained by the squeeze test method when used by competent inspectors.

Erratic glacial boulders sometimes stripped teeth off the P & H cylinder, and paddles had to be changed often because the sand and gravel quickly wore them out. When properly maintained, however, the machine did an excellent job of processing the soil and lime.

Compaction. —Compacting equipment closely followed the P & H as it mixed the subbase materials. As stated elsewhere, this was done to seal the surface partially and prevent entrance of excess water during frequent rains, as well as to force the soil and lime into more intimate contact and hasten the base exchange reaction. The initial relatively light rolling was applied to all sections, whether or not they were to be remixed.

Early tests proved that density requirements could be met by about three passes of the pneumatic roller alone, and the sheepsfoot tamper was not used after a short trial on the first day of construction.

Testing During Construction. —Construction forces performed many routine tests in the normal course of their work, the results of which are not included in this report. The experimental research nature of the project, however, demanded sampling and testing far beyond their capacity. The Materials Division had been instrumental in promoting the experiment and had recommended the design thickness and percentage lime to be used. As a result of this special interest, a testing crew of Geology and Soils personnel was assigned to the project, with a mobile laboratory to supplement the usual facilities available in the field. Some tests, impossible for the testing crew to make because of time or equipment limitations, were performed by the Jefferson City central laboratory.

Test Methods. —The procedure set forth in AASHO Designation: T 99, Method A, was followed to determine moisture-density relationships in both subgrade and subbase, with one modification. Because the P. I. of the soil exceeded 25, those specimens were compacted in four layers, as is standard procedure in Missouri. Lime-soil specimens were molded in three layers. Densities of subgrade and subbase were determined by the sand-cone method. AASHO Designation: T 88 procedure was followed to determine gradation of the soil.

Field tests to determine the minus No. 200 fraction of the lime-soil mixture were made on processed material from immediately behind the P & H. The loose mixture was first oven-dried, then thoroughly soaked and, accompanied by light rubbing, washed through the sieve.

Compressive strength tests of soil were made on unsoaked, triplicate 2- by 2-in. specimens molded at optimum moisture to maximum density and stored overnight in the humid room.

Lime-soil specimens were usually 2 by 2 in., although the dimensions of a few were 4 by 4.6 in. Triplicate test specimens were molded to maximum density at the field moisture content, wrapped in plastic or foil, and cured for 28 days at 72 F and 95 percent RH. Most were broken without soaking, although in a few instances comparative strengths were obtained by breaking soaked and unsoaked sets.

Laboratory CBR was determined on raw soil specimens molded in the laboratory under static load to T 99 maximum density and optimum moisture and tested after soaking until swelling became negligible.

The laboratory CBR test was made on lime-soil specimens molded in the field at field moisture with dynamic compaction (T 99). The specimens were moist cured for 28 days and tested after soaking until swelling had ceased.

Pulverization Tests. —The results of tests to determine the amount of pulverization obtained on each project are given in Table 3. The results for Project F-297(9) seem

TABLE 3

RESULTS OF PULVERIZATION TESTS

Project	Station	Rotting Time (hr)	Percent Passing Sieve		
			1½ In.	1 In.	No. 4
F-297(7)	350	0	100	100	48
	287	0	100	98	35
	227	0	100	100	44
	207	0	100	92	44
	114 + 50	0	100	99	53
	107	0	100	100	30
	67	0	100	90	38
	687 (Rt. 46)	0	100	98	51
Avg.			100	97	43
F-297(9)	587	6	—	100	67
	598 + 50	6	—	100	61
	625 + 10	18	—	100	61
	627 + 25	18	—	100	61
	647	48	—	100	60
	664	48	—	100	60
Avg.			—	100	62
F-524(2)	107	24	—	100	58
	127	72	—	100	69
	147	24	—	100	64
	167	24	—	100	61
	174	24	—	100	71
	204	24	—	100	56
	217	24	—	100	71
	232	24	—	100	67
Avg.			—	100	65

TABLE 4

AVERAGE EFFECT OF LIME ON SOILS, SUBGRADE AND SUBBASE[a]

Property	Amount (¢)		
	F-297(7)	F-297(9)	F-524(2)
P. I., decrease	86	90	92
Shrinkage limit, increase	141	132	152
Shrinkage ratio, decrease	25	24	25
Free swell, decrease	27	45	53
Opt. moisture, increase	44	50	57
Max. density, decrease	13	14	13
Minus No. 200 sieve material, decrease	56	69	62
Compressive strength, increase	406	609	276[b]
	—	—	190[c]

[a] See footnote, Table 2.
[b] Both unsoaked.
[c] Lime soaked.

to indicate that a long rotting time has little or no effect on soil breakdown. In ten additional tests on Project F-524(2), after 24 hr rotting, the minus No. 4 content ranged from 60 to 81, with 6 of the 10 below 64 percent. This may indicate that, for this soil and equipment, a higher minus No. 4 requirement might be achieved only through additional pulverizing effort, or by using a greater percentage of lime, since the effect of tripling the rotting period was almost negligible.

Summation of Test Results. —Numerous other tests were made on both subgrade and subbase. Analysis of the results on the three projects, summarized in Table 4, permits the following statements to be made.

1. A reduction in L. L. is shown from 46 in the raw soil to 40 after mixing in lime.
2. An average P. L. of 16 in the clay was increased to 31 in those samples of the mixture on which the test could be made.
3. Average P. I. for native soil was 30. After lime processing, 18 tests averaged 9, 23 were nonplastic and the overall average was 4.
4. The shrinkage limit increased from 11 in the soil to 27 in the mixture.
5. The shrinkage ratio changed from 2.03 to 1.52.
6. Natural soil displayed a free swell of 75 percent, and the mixture showed 50 percent in this indicator test.
7. The average optimum moisture of 15.5 for soil was increased in the mixture to 22.8 percent.
8. Addition of 6 percent lime reduced maximum density from 113.3 to 98.7 pcf.
9. The average content of minus No. 200 material for untreated soil was 73 percent. Processing with lime reduced this to 30 percent.
10. The average group index of 16 for the soil was reduced to an average of 1 in the mixture.
11. Based on averages of all tests on the three projects, the classification shows a change from A-7-6(16) to A-2-4(0), a very remarkable improvement.
12. Although exhibiting rather wide variations in unconfined compression tests, the following averages are indicative of the improvement in compressive strength from untreated to treated soil: (a) Project 297(7), from 41 to 207 psi; (b) Project 297(9), from 25 to 292 psi; and (c) Project 524(2), from 49 to 184 psi. Specimens were cured for 28 days and broken without soaking. Twelve additional pairs of specimens were molded and cured in similar manner. One of each set was soaked overnight before breaking; the other was not. Average strength was reduced, by soaking, from 170 to 135 psi.

TABLE 5

Project	Subgrade		Subbase	
	From O. M. (%)	% M. D.	From O. M. (%)	% M. D.
F-297(7)	+1. 1	99. 5	-0. 1	101. 1
F-297(9)	+0. 9	99. 2	+0. 7	98. 5
F-524(2)	+5. 4	93	+0. 7	95. 8

13. Although only four CBR test were made on soils and four on mixtures from the same locations, the limited data are indicative of the improvement caused by addition of the lime. Average CBR of the soil was 3. 8, which was increased in the mixture to 66. 5.

14. The average percent compaction on F-297(7) was 101. 1, as compared to 98. 5 on F-297(9) and 97. 8 on F-524(2). All projects received approximately the same compactive effort and density tests were made on the mixture after processing and finishing. No explanation can be given for the fact that the unrotted material showed a higher percentage of compaction than the rotted sections.

15. Table 5 indicates that, except for the subgrade on F-524(2), compliance with moisture and density requirements was excellent.

16. The thickness was measured at all sampling locations, at approximately 2, 000-ft intervals. Measurements were made at edges, quarter points and centerline. Averages show that the depth of subbase exceeded the 5-in. designed thickness on all projects. On F-297(7) it was 5. 44 in., whereas F-297(9) and F-524(2) showed 5. 45 and 5. 08 in., respectively.

Finishing. —The day after compaction was completed the subbase was trimmed by blade to proper grade and cross-section, sprinkled, and finished with a pneumatic roller. A considerable quantity of material was shifted by the blade in evening up ridges and depressions left by the stabilizer and pneumatic. The probability of developing compaction planes during this phase of the work was not considered to be of critical importance at a depth of 11 or more in. below the surface.

Curing. —All lime subbase was cured by sprinkling daily until the first 4-in. layer of base material was placed. On F-297(7) a 20-day limit was specified between completing the subbase and placing the base. The difficulty of maintaining the proper moisture content on this project influenced the curing procedure on the other projects. Although no time limit was set, the contractor found it expedient to protect the subbase within a few days to avoid frequent applications of water. It is of interest to note that placing stone base was sometimes delayed after a rain because of the soft untreated shoulders, but the lime-treated subbase was in excellent condition and supported traffic without deformation of any sort.

Production. —Crushed rock for the rolled stone base was produced at a nearby local quarry from the Stanton formation of shaly Pennsylvanian limestone. Specifications applicable at that time were for Project F-297(7): pass 1½-in. sieve, 100 percent; pass 1-in. sieve, 95 to 100 percent; pass ½-in. sieve, 60 to 85 percent; pass No. sieve, 40 to 60 percent; pass No. 40 sieve, 15 to 35 percent; L. L., 25 max. and P. I., 6 max. Specifications for Projects F-297(9) and F-524(2) were pass 1-in. sieve, 100 percent; pass ½-in. sieve, 60 to 90 percent; pass No. 4 sieve, 40 to 60 percent; pass No. 40 sieve, 15 to 35 percent; L. L., 25 max., and P. I., 6 max.

The material was inspected during production and all that complied with specifications was stockpiled. The average P. I. was well below the allowable 6, having been reduced by the addition, during production, of 5 to 10 percent nonplastic fine sand when early tests indicated the possibility of P. I. trouble.

Moisture. —Water added to the pug mill mixer at the plant was usually adequate to assure satisfactory compaction. Critical points were about 0. 5 percent below and 2 percent above optimum. Beyond those limits density requirements were difficult to meet without additional work to bring the moisture within that range.

Test Methods. —AASHO Designation: T 99, Method A, was used to determine the moisture-density relations in the minus No. 4 portion of the stone base. The maximum density and optimum moisture of the total base were computed from this moisture density relation, the percent passing the No. 4 sieve, and the specific gravity and absorption of the plus No. 4 material. Gradations were determined by washing.

TABLE 6

SUMMARY OF ROLLED STONE BASE TEST RESULTS[a]

Course	L. L.	P. L.	P. I.	Opt. Moist.	Max. Dens.	Field Moist.[b]	Field Dens.	Comp. (%)	Percent Passing Sieve							
									1½ In.	1 In.	¾ In.	½ In.	No. 4	No. 10	No. 40	No. 100
(a) Project F-297(7)																
1, avg.	18	14	4	7.6	138.7	7.9	136.5	98.4	100	97.7	92.6	83.4	59.3	45.2	29.2	17.3
2, avg.	19	14	5	7.3	137.8	7.4	135.7	98.5	100	96.0	88.6	77.8	54.1	43.0	38.9	16.9
Both:																
Avg.	19	14	5	7.4	138.2	7.7	136.1	98.5	100	96.8	90.5	80.5	56.6	44.1	29.0	17.1
Max.	21.8	16.5	6.6	8.3	141.0	9.2	146.3	104.5	100	100	98.4	96.2	70.2	53.0	37.5	25.5
Min.	15.1	11.6	NP	7.0	134.5	5.7	129.3	93.3	100	87.2	74.3	60.2	39.3	30.5	21.2	12.6
Specification	≤ 25	—	≤ 6	—	—	—	—	≥ 95	100	95-100	—	60-85	40-60	—	15-35	—
(b) Project F-297(9)																
1, avg.	21	14	7	7.2	137.4	8.1	138.7	100.9	—	100	96.9	87.7	64.7	50.2	30.0	18.8
2, avg.	21	15	7	7.2	137.8	8.0	142.3	103.2	—	100	95.5	86.4	63.6	49.2	31.5	20.5
Both:																
Avg.	21	14	7	7.2	137.6	8.1	140.5	102.1	—	100	96.2	87.0	64.1	49.7	30.8	19.7
Max.	23	16	9	7.2	139.5	8.7	144.9	105.6	—	100	98.4	89.7	68.4	53.2	34.2	22.5
Min.	20	13	5	7.2	136.0	7.6	134.0	97.8	—	100	86.8	85.0	60.3	43.7	27.8	16.2
Specification	≤ 25	—	≤ 6	—	—	—	—	≥ 95	—	100	—	60-90	40-60	—	15-35	—
(c) Project F-524(2)																
1, avg.	21	15	6	7.8	138.8	7.7	136.5	98.3	—	100	96.3	84.9	60.0	45.3	28.6	18.8
2, avg.	21	15	6	7.9	137.9	7.9	138.5	100.4	—	100	96.5	84.6	58.3	44.4	29.5	19.8
Both:																
Avg.	21	15	6	7.9	138.4	7.8	137.5	99.4	—	100	96.4	84.8	59.2	44.9	29.1	19.3
Max.	22	16	8	7.9	139.4	8.4	143.4	103.7	—	100	98.3	88.0	63.0	47.7	32.0	22.0
Min.	20	13	5	7.7	137.3	7.3	132.0	95.4	—	100	95.2	80.4	54.3	42.0	26.4	17.4
Specification	≤ 25	—	≤ 6	—	—	—	—	≥ 95	—	100	—	60-90	40-60	—	15-35	—

[a] See footnote, Table 2.

[b] Results for Project F-297(7) at time of compaction.

Construction Procedure. —The 8-in. designed thickness of crushed stone was compacted in two layers by pneumatic roller and a 12-ton tandem flat wheel to a density of 95 percent or more of the maximum weight.

Test Results. —A summary of the results of tests for P. I., moisture, density and gradation in both lifts, at about 2,000-ft intervals, is given in Table 6.

TABLE 7

BASE THICKNESS MEASUREMENTS

Project	Max.	Min.	No. Tests	Avg.
F-297(7)	9	7	250	8.02
F-297(9)	10	7¾	29	8.23
F-524(2)	8⅜	7¼	26	8.03

Base thickness was measured at quarter points and centerline. Results are given in Table 7.

Tests on plus No. 4 material showed average specific gravity of 2.44, 2.45 and 2.47 for the three projects in the usual order; absorption was 4.4, 3.5 and 4.3 percent.

One CBR test showed 100 percent and one Los Angeles abrasion test (Gradation B) showed a wear of 36.3 percent.

The following conclusions may be drawn from an analysis of the data.

1. Average liquid limits were from 18 to 21, and none reached specified maximum of 25.
2. The P. I. of the first lift on F-297(7) averaged four, with none higher than 6, the specified maximum. The second lift, on which the average was 5, showed 4 of 29 tests above 6. Results were not as good for Project F-297(9), where an average for both courses was 7, and 7 out of 9 tests exceeded 6. Both courses on F-524(2) averaged 6, and 6 of 14 individual tests were above 6.
3. At the time of compaction, base moisture on all projects was within less than 1 percent of optimum.
4. Densities ranged from 98.5 to 102.1 percent of the maximum.
5. As far as gradation was concerned, complying with the No. 4 sieve specification gave the only trouble. The crushed material, as produced, was in the upper register of the 40 to 60 percent specified band and, at the same time, near the maximum allowable P. I. Addition of fine sand to lower the P. I., plus some probable degradation during stockpiling, caused quite a few samples to exceed the upper limit of 60 percent minus No. 4.

Curing. —Following completion of compacting operations the base was properly shaped and cured to meet specified moisture content before priming. On Project F-297(7) it was required that the 8-in. base contain no more than 50 percent of optimum moisture at the time of application of the 0.35 gal/sq yd MC-0 prime. Due to the change in specifications referred to previously, requirements were less stringent on the other two projects, where it was necessary that the moisture in the top 2 in. of the base be no more than two-thirds of optimum.

Asphaltic Concrete

Compaction of the two courses of asphaltic concrete was accomplished with tandem steel-wheel and pneumatic rollers. Job mix limits and average results of tests made on the finished asphaltic concrete for Project F-297(7) are as follows:

Limits—Type B (%)	
1½ in.-½ in.	29.1-39.1
½ in.-No. 10	38.4-38.4
No. 10-No. 200	21.6-27.6
-No. 200	2.4-4.4
AC	4.0-5.0

Limits—Type C (%)	
¾ in.-No. 10	47.3-57.3
No. 10-No. 200	31.5-37.5
-No. 200	4.8-6.8
AC	4.7-5.7

Finished product, Type B—spec. gr. = 2.341, % voids = 3.9,
% compaction = 99.9, % bitumen
(by extraction) = 4.2 %.

Finished product, Type C—spec. gr. = 2.295, % voids = 5.0, % compaction = 98.2, % bitumen = 4.7 %

For Projects F-297(9) and F-524(2), these are as follows:

Limits—Type B (%)	
1 in. -½ in.	21.6-31.6
½in. -No. 10	31.3-46.3
No. 10-No. 200	26.9-32.9
-No. 200	2.0-3.7
AC	4.0-5.0

Limits—Type C (%)	
½ in. -No. 10	50.3-60.3
No. 10-No. 200	31.5-37.5
-No. 200	4.2-6.2
AC	4.5-5.5

Finished product, Type B—spec. gr. 2.355, % voids = 4.1, % compaction = 100.7, % bitumen = 4.4.

Finished product, Type C—spec. gr. 2.325, % voids = 4.9, % compaction = 99.7, % bitumen = 4.91.

Construction Costs. —Listed in the following are costs, from final plan quantities, of pavement construction on the various projects.

Project F-297(7):

2089.0 tons lime at $30	$62,670.00
10.76 mi lime processing at $2500, 5 in. × 30 ft	26,900.00
110,500 tons base stone at $2.00, 8 in. × 41 ft avg.	221,000.00
554,000 gal water at 60/100 gal for subgrade and base maintenance	3,324.00
10.76 mi at $2500 for spreading, shaping and compacting stone	26,900.00
79,908 gal primer at $0.18 for stone base	14,383.44
22,813 tons Type B asphaltic concrete at $7.02	160,147.26
10,664 tons Type C asphaltic concrete at $7.31	77,953.84
33,447 gal 200-250 penetration asphalt at $0.18 for shoulders	6,020.46
1538 tons cover aggregate at $6.00 for shoulders	9,228.00
Total	$608,527.00

Cost per mile:	
Lime subbase	$8,324.35
Stone base	23,347.96
Asphaltic concrete surface, prime, shoulder surfacing	24,882.24
Total	$56,554.55

Project F-297(9):

197,600 gal water at $0.30/100 gal for lime-soil mixture	$592.80
501.6 tons lime at $30	15,048.00
2.5 mi lime processing at $2500, 5 in. × 30 ft	6,250.00
24,833 tons base stone at $2.20, 8 in. × 41 ft	54,632.60
25,000 gal water at $0.30/100 gal for subgrade and base maintenance	75.00
2.5 mi at $2500 for spreading, shaping and compacting stone	6,250.00
19,881 gal primer at $0.17 for stone base	3,379.77
5185 tons Type B asphaltic concrete at $7.50	38,887.50
2430 tons Type C asphaltic concrete at $7.50	18,225.00
7080 gal RC-4 cutback asphalt at $0.18 for shoulders	1,274.40

Project F-297(9) (Cont'd):

249 tons cover aggregate at $6.00 for shoulders	1,494.00
Total	$146.109.07

Cost per mile:	
Lime subbase	$8,756.32
Stone base	24,383.04
Asphaltic concrete surface, prime, shoulder surfacing	25,304.26
Total	$58,443.62

Project F-524(2), Station 105+60 to 234+77, minus bridge:

154,740 gal water at $0.30/100 gal for lime-soil mixture	$464.22
451.00 tons lime at $30	13,530.00
2.36 mi lime processing at $3000, 5 in. × 30 ft	7,080.00
24,363 tons base stone at $2.20, 8 in. × 41 ft	53,598.60
24,000 gal water at $0.30/100 gal for subgrade and base maintenance	72.00
2.36 mi at $2500 for spreading, shaping, compacting stone	5,900.00
20,742 gal primer at $0.17 for stone base	3,526.14
5037 tons Type B asphaltic concrete at $7.50	37,777.50
2516 tons Type C asphaltic concrete at $7.50	18,870.00
6552 gal RC-4 cutback asphalt at $0.18 for shoulders	1,179.36
326 tons cover aggregate at $6.00 for shoulders	1,956.00
Total	$143,953.82

Cost per mile:	
Lime subbase	$8,929.76
Stone base	25,241.77
Asphaltic concrete surface, prime, shoulder surfacing	26,825.85
Total	$60,997.38

By prorating costs of the stone base, the following comparisons of costs per mile of 5 in. by 30-ft stone base vs the same thickness and width of lime subbase can be made. (Calculations are based on the assumption 5 in. by 30-ft stone base is 44.3+ percent of total base material used and, therefore, is that percentage of cost of stone base per mile.)

Project	F-297(7)	F-297(9)	F-524(2)
Lime	$8,324.35	$8,756.32	$8,929.76
Stone	$10,352.48	$10,811.43	$11,192.20
Additional costs, stone over lime	24.4%	23.5%	25.3%

These comparisons show that, under these contracts, 4 in. of rolled stone base 30 ft wide costs approximately the same as 5 in. of lime-soil.

Test of Finished Pavement

Deflection. —Table 8 is a sample sheet of results of Benkelman beam deflections made on each project at the test station locations. The deflections were obtained soon after completion of each section of pavement. The test results indicate no apparent correlation, at this time, between deflection and voids, density, moisture, etc., of the subgrade and the various components of the pavements when constructed.

TABLE 8

SAMPLE SHEET, DEFLECTION TEST RESULTS[a]
Project F-297(7), Sta. 545+30 to 308+75
Construction Completed and Traffic Started, October 1961

Station	Test Date	Deflection (in.)				
		Northbound		Southbound		Avg.
		OWP	IWP	IWP	OWP	
540+50	Nov 1961	0.018	0.013	0.013	0.015	0.015
	May 1962	0.034	0.037	0.039	0.042	0.038
	Nov 1962	0.038	0.032	0.035	0.035	0.035
	May 1963	0.037	0.042	0.049	0.037	0.041
527+00	Nov 1961	0.023	0.021	0.018	0.017	0.020
	May 1962	0.044	0.042	0.030	0.044	0.040
	Nov 1962	0.040	0.032	0.040	0.032	0.036
	May 1963	0.049	0.039	0.042	0.039	0.042
508+06	Nov 1961	0.021	0.018	0.022	0.022	0.021
	May 1962	0.034	0.025	0.025	0.037	0.030
	Nov 1962	0.038	0.021	0.027	0.035	0.030
	May 1963	0.037	0.022	0.022	0.039	0.030
487+00	Nov 1961	0.015	0.010	0.013	0.010	0.012
	May 1962	0.037	0.017	0.012	0.027	0.023
	Nov 1962	0.027	0.013	0.016	0.019	0.019
	May 1963	0.037	0.017	0.017	0.034	0.026
467+00	Nov 1961	0.021	0.021	0.021	0.022	0.021
	May 1962	0.042	0.044	0.030	0.049	0.041
	Nov 1962	0.032	0.032	0.032	0.032	0.032
	May 1963	0.052	0.052	0.037	0.044	0.046
447+00	Nov 1961	0.019	0.021	0.013	0.018	0.018
	May 1962	0.030	0.025	0.015	0.047	0.029
	Nov 1962	0.030	0.030	0.019	0.024	0.026
	May 1963	0.042	0.039	0.032	0.030	0.036
427+00	Nov 1961	0.017	0.018	0.015	0.017	0.017
	May 1962	0.030	0.030	0.022	0.054	0.034
	Nov 1962	0.024	0.016	0.021	0.032	0.023
	May 1963	0.027	0.020	0.025	0.044	0.029
407+00	Nov 1961	0.015	0.018	0.018	0.023	0.019
	May 1962	0.032	0.020	0.042	0.059	0.038
	Nov 1962	0.026	0.023	0.031	0.026	0.027
	May 1963	0.030	0.030	0.034	0.052	0.037

[a] See footnote, Table 2.

Permeability. —In December 1962, permeability tests were made of the asphalt surfacing by applying water through a meter to a 4-in. diameter area under a falling head, the maximum head being about 15 in. The test showed that the newer pavement, which had been completed about 2 months before, had a permeability ranging from 1.4 to 12.8 ml/min and averaged 5.7. The older pavement, subjected to more traffic, showed a range in permeability from 0 to 7.1 ml/min and averaged 0.2. All figures represent an extremely low permeability.

Rutting. —Rutting measurements were made at the test locations in the spring of 1963.

Station 545+30 to 308+75, Worth Co., Completed in October 1961. —Rutting was negligible in the inner wheelpaths. In the outer wheelpath of the northbound lane, ruts varied from 0 to 0.3 in. and averaged 0.1 in. In the southbound outer wheelpaths, ruts ranged from 0 to 0.6 in. and averaged 0.25 in.

Station 308+75 to 223+16, Worth Co., Completed in November 1961. —No appreciable ruts were measured at the test stations except in the outer wheelpath of the northbound lane. Here they ranged from 0 to 0.4 in. and averaged 0.25 in.

Station 223+16 to 0+00, Worth Co., Completed in June 1962. —An average rut of about 0.1 in. was measured in the outer wheelpath of the northbound lane. Other wheelpaths showed negligible rutting.

Station 545+30 to 677+14, Worth Co. and Station 105+60 to 234+77, Gentry Co., Completed in October 1962. —Rutting was negligible on both projects.

Roughness. —All the projects showed approximately 68 in./mi roughness soon after completion. This is about 7 in. less than the average roughness for newly constructed flexible pavements in Missouri. There has been very little change in roughness except for the first constructed sections from Station 545+30 to Station 223+16 in Worth Co., which are also showing greater deflection and more rutting. Through these sections the roughness has increased on the average about 11 in./mi.

Investigations of Distress

In June 1962 cracking, rutting and failure occurred in the outer wheelpath of one section. An investigation was made by excavating a trench into the subgrade through the outer 7 ft of the roadway. Results of the examination and measurements indicated that the asphaltic concrete was 0.25 in. thinner in the failed part than in adjacent unfailed areas, which was probably caused by abnormal truck traffic.

The stone base in the failed area showed greater density than that generally found during construction, which may indicate some base densification under traffic. Also there was a faint indication of slippage between the two courses of stone. Moisture content of the base was below optimum. The lime subbase was hard and showed no deformation. The moisture content was about optimum and the density was about the same as during construction. The density in the subgrade was 99.5 percent of standard and the moisture content was 2 percent above optimum. It was concluded that rutting, which led to cracking, was caused by densification of the asphaltic concrete and stone base and possibly some lateral movement of the base. The area was repaired by the contractor.

A limited investigation of localized areas was made in April 1963. Both distressed and nearby unaffected areas were examined, tested and measured. But in March and April 1964 the latest, most thorough and productive investigation was completed on a portion of Project F-297(7). Fourteen cores were removed from the asphalt surface for laboratory tests. Thickness of the various layers was determined; density and moisture measurements were made in base, subbase and subgrade; cores were removed from the lime subbase; and field and laboratory tests were performed on samples from all layers at all locations.

The investigation was restricted largely to a section about 2 mi long where, although no failures had occurred, considerable rutting and alligator cracking had developed. For the most part this was the section which was finished late in 1961, barricaded against all traffic (including snow removal equipment), and subjected to extremely harsh treatment early the following spring when used by loaded northbound base trucks and southbound sand trucks. Failed areas of asphalt which quickly developed in the northbound lane were removed and rebuilt by the contractor in June 1962.

In January 1964, alligator cracking averaged about 5 sq ft/1,000 sq ft of 24-ft roadway, and was confined mainly to the inner wheelpath of the southbound lane. By April spot sealing had covered most of the cracked areas and the increase in cracking, if any, could not be determined. No significant distress was noted at this time in the areas which had been repaired by the contractor.

Consideration of test results, thickness measurements and general conditions at the 14 test locations indicated the following at the time of this survey and on this section.

1. Surface—Asphaltic concrete thickness varied from $2\frac{5}{8}$ to $3\frac{1}{4}$ in., with no apparent correlation between distress and thickness. Asphalt recovered from cores 2 and $2\frac{1}{2}$ yr old showed penetration anticipated, respectively, in 5- and 10-yr old asphalt. Ductility at 45 F of less than 8 is considered by some to indicate asphaltic concrete of low serviceability.

2. Base—Thickness of rolled stone base averaged 7.2 in., compared to the 8 in. specified. Density in the wheelpaths had increased 2.5 percent since construction, which could account for about 0.2-in. base settlement in those areas. In general, the prime had functioned satisfactorily, adhering to base and asphalt surfacing. Cracked areas occurred over both thin and thick base. Averages showed that moisture at this time was 1 percent below optimum and 1.6 percent less than water of saturation. Only one P. I. equaled the specified maximum of 6; none exceeded it. Gradation was somewhat finer than during production and CBR testing indicated very satisfactory base material.

3. Subbase—Lime subbase averaged 1 in. more than the 5-in. designed thickness. At various locations a thin clay-like layer was noted at the top of the course. This was believed to have resulted from repeated applications of water while curing, and was not considered to be detrimental to the performance of the subbase. Average P. I. was 5.8, which would indicate some rebound since construction if credence can be placed in the accuracy of the test. The subbase moisture, density and shrinkage limit had changed little or not at all since construction. Unconfined strengths of cores submerged before breaking averaged 190 psi and indicated a gain of about 14 percent in slightly more than 2 yr. No relation could be seen between strength and distress. Average pH of about 10.4 indicated good lime distribution. There was no indication, at any site, of displacement or deformation of the lime subbase.

4. Subgrade—Average subgrade moisture was 3.8 percent above that during construction and 4.8 percent over optimum, which might be expected to reduce the strength and stability of overlying layers. No relationship could be established between higher moisture and distressed areas.

5. Cracks—Some longitudinal shoulder cracks in both cut and fill were believed to have been caused by desiccation, shrinkage and some fill subsidence. Transverse cracks which extended half to full width, and in some places into the shoulders, are

believed to have originated in the asphalt and progressed downward. They may also have been associated with shrinkage.

6. Evaluation—The upheavals, depressions and general edge distortion which commonly signal subgrade failure were in no place evident here, nor were there signs of incipient failure in the lime subbase. By process of elimination, then, substantiated by increased base density and rutting of the asphalt in the wheelpaths, and some earlier noted signs of lateral movement between the two base courses, it was decided that the source of the distress lay in the rolled stone base. Densification of the 8-in. base under traffic demanded corresponding flexure of the surface course. Since the asphaltic concrete was apparently of inferior quality it cracked instead of deflecting.

Summary

1. The Missouri Highway Department cannot lay claim to having made any discoveries in connection with soil-lime stabilization as a result of this experimental project.
2. Many data were accumulated to support the already well-known fact that hydrated lime, when intimately mixed with the glacial clay of northern Missouri, greatly improves the engineering properties of the native soil.
3. The P. I. and group index, the volume change and the minus No. 200 content all decreased and the compressive strength increased. Optimum moisture increased and the maximum density decreased with addition of lime. The consequent bulking meant, on this project, that about 4.33 in. of compacted soil expanded with addition of 6 percent lime to give 5 in. of compacted mixture.
4. Intimately mixing hydrated lime with wet clay tends to reduce moisture in the clay. The agglomeration of clay-size particles improves the drainability. In combination, the two develop an unyielding, relatively dry working platform soon after a rain, which permits lime processing to continue in what would otherwise be adverse conditions. The importance of this quick-drying feature was emphasized on these projects when frequent rains softened the untreated shoulders and, in some locations, made it impossible for trucks to cross. Thus, the base-laying operations were sometimes delayed in certain areas when they might have been in progress.
5. Construction of lime-soil subbase was relatively trouble-free. By its quick reaction with the clay, lime aided in pulverization, and density was obtained after only a few passes of a pneumatic roller.
6. Tests made on the first project seemed to substantiate the idea that a rotting period was not essential. Only a few stretches had to be remixed to meet pulverization requirements. Yet the quality of subbase was, to all appearances, as good as that on the remixed sections, and the average density was higher. Therefore, since some initial cementing action might be destroyed by remixing, and since the lime-clay reaction is of diffusive nature, rotting and remixing might not be necessary. Pulverization requirements could then be less restrictive than were applied to the second and third projects on this job.
7. Where remixing was unnecessary, average daily progress was increased by about 40 percent.
8. It was impossible to maintain a uniform moisture content while water-curing the subbase. Because wetting-drying cycles may cause damage it is believed that application of a bituminous seal would be a better curing method.
9. During the short time this road has been in service no evidence has been adduced to indicate that a crushed stone base with P. I. slightly higher than 6 is detrimental.
10. Certain sections of this job were detrimentally affected by abnormal construction traffic at an early age. As should have been anticipated, the results of such treatment were quickly reflected by the development of more and greater distress in those areas than where the roadway carried normal traffic.
11. Even including the sections mentioned above, the lime subbase portion of these three projects is giving satisfactory performance and showing only minor distress in limited areas.
12. One peculiarity, being studied with interest but not yet correlated with any feature of design or construction, is the cracking which occurs to a greater extent in the inner wheelpaths than in the outer paths, as is usual.

13. No correlation has yet been established between deflections and the moisture or density of any increment of the structure. Nor does there appear to be any relation between the age of subbase when asphaltic concrete was placed and the performance of the road.

14. Missouri's previous experience with relatively thin asphaltic concrete surface on crushed stone bases has not always been all that could be desired. Data thus far accumulated in Worth and Gentry Co. lead to the belief that lime subbase below an 11-in. depth, as here, adequately replaces the same thickness of crushed rock. In such an area, where aggregates are scarce, replacement may effect approximately a 20 percent reduction in cost at no sacrifice in service.

15. It is believed that such distress as has already appeared can be accounted for by consideration of several factors touched on in this report and that, in general, this lime subbase experimental project has been satisfactory in every respect.

TEST SECTIONS

(Project F-524(2), Gentry Co.)

As shown in Figure 1, 16 test sections were located on U. S. 169, Gentry Co., beginning at the Worth Co. line and extending south for 2 mi. This segment of the report describes the various features of design, construction and performance of these sections.

Purpose

This complete factorial experiment was proposed to try to determine the effect on a 3-in. asphaltic concrete surface course of various combinations of subgrade, three types of subbase and two thicknesses of rolled stone base.

Scope

Two factors of the design are under study. The first factor (thickness of base) contains two levels: (a) 8-in. Type I (rolled stone) aggregate base; and (b) 4-in. Type I (rolled stone) aggregate base. The second factor (type of subbase) contains four levels: (a) 5 in. of lime-stabilized subbase; (b) 5 in. of phosphoric acid-stabilized subbase; (c) 5 in. of Type I aggregate subbase; and (d) no subbase. The subbase sections are $^{1}/_{8}$ mi in length and are replicated under each thickness of base (Table 9).

Topography

The topography is very similar to that described for the preceding projects—heavily rolling country with deeply eroded gullies and narrow valleys.

TABLE 9

SUBBASE SECTIONS

Test Section	Station Limits	A. C. (in.)	Base (in.)	Subbase
1	0+00-6+60	3	8	5-in. lime
2	6+60-13+20	3	8	5-in. acid
3	13+20-19+80	3	8	5-in. lime
4	19+80-26+40	3	8	5-in. Type I
5	26+40-33+00	3	8	None
6	33+00-39+60	3	8	5-in. Type I
7	39+60-46+20	3	8	None
8	46+20-52+80	3	8	5-in. acid
9	52+80-59+40	3	4	5-in. lime
10	59+40-66+00	3	4	5-in. Type I
11	66+00-72+60	3	4	5-in. lime
12	72+60-79+20	3	4	None
13	79+20-85+80	3	4	None
14	85+80-92+40	3	4	5-in. acid
15	92+40-99+00	3	4	5-in. acid
16	99+00-105+60	3	4	5-in. Type I

Soils

The glacial Shelby was identified as the soil comprising the subgrade throughout these 2 mi. However, borrow soil was used in three of the acid sections.

General

Construction and testing of the lime sections developed little information not already determined on adjacent projects with similar subbase. The same was true concerning rolled stone base material, whether used as base or subbase. Experiences with these two types have been reported in detail in the previous section, and, to avoid repetition, additional data will not be included here. Therefore,

TABLE 10

SOIL PROPERTIES

Property	Shelby	Grundy	Till
L. L.	48	48	55
P. L.	16	17	18
P. I.	32	31	37
Shrinkage limit	12	10	12
Vol. change at FME	51	53	42
Minus No. 200	77	82	82
Silt	30	31	30
Clay	41	42	45
Colloids	30	31	37
Max. density	113	109	108
Opt. moisture	16	17	18
Psi at O. M. - M. D.	51	45	44
Free swell (%)	75	80	85
Class.	A-7-6(17)	A-7-6(18)	A-7-6(19)
Texture	Clay	Clay	Clay

this part of the report will consist largely of information having to do with phosphoric acid stabilization.

Soil Replacement

Calcareous concentrations occur frequently in the Shelby, and tests of the soil with hydrochloric acid showed considerable effervescence on Sections 2, 14 and 15. To eliminate the need for addition of excess acid to neutralize carbonates, noneffervescing soil was imported for subbase material on those three sections. Therefore, phosphoric acid was mixed on Section 2 with B horizon Grundy, on Sections 14 and 15 with nonreactive B and C Shelby, and on Section 8 with glacial till in place. Test results given in Table 10 indicate the general similarity of the three soils.

Preliminary Testing—Acid

Investigation of the stabilizing effect of phosphoric acid was started by Missouri in 1957, and much cooperative laboratory research work was completed. Preliminary work was confined to mixtures of acid in various percentages with highly plastic Putnam soil with extremely high volume change. A test section on Putnam soil was built in Callaway Co. by maintenance personnel under the supervision of the Missouri State Highway Commission and in cooperation with research personnel of the Monsanto Chemical Co. This pilot section was too short to provide much definite and reliable information, but it did focus attention on some of the factors that would probably influence construction of longer projects.

The combined laboratory and field work indicated that the potential of phosphoric acid as a stabilizing agent for heavy clays would justify a larger scale experiment. A location for such a project, with suitable soil and traffic and fitting the scheduled work program, was selected in conjunction with the extensive study of lime stabilization reported in the previous part of this paper.

Representative samples of the predominant Shelby soil were tested in the laboratory, with results as shown in Figures 3 and 4. Based on these data and reinforced by information from other sources, the recommendation was made that 2 to 3 percent acid be added for optimum results.

Design

Because soils and traffic were very nearly the same on these 2 mi as on adjacent sections, the same total thickness of 16 in. was recommended for the test sections. The half section in Figure 5 illustrates the various combinations of subgrade, subbase and base. The sections were chosen by random selection and each combination was duplicated.

Specifications.

1. Subgrade—top 18 in. to be compacted to minimum of 95 percent of T 99 standard (Missouri modification, four layers if P. I. is 25 or more).
2. Subbase-Lime—trenched 30 ft wide, 5 in. thick, 6 percent lime (dry soil basis), compacted to 95 percent or more of T 99 standard; pulverization requirement after mixing lime with soil, 100 percent passing 1-in. sieve and at least 60 percent passing No. 4 sieve.
3. Subbase-Acid—same trenched section, 2 to 3 percent of 50 percent acid (diluted from 75 percent), compacted to at least 100 percent of T 99 standard, determined by Missouri's procedure (four layers); pulverization, 100 percent passing the 1-in. sieve and a minimum of 80 percent passing the No. 4 before acid is added.

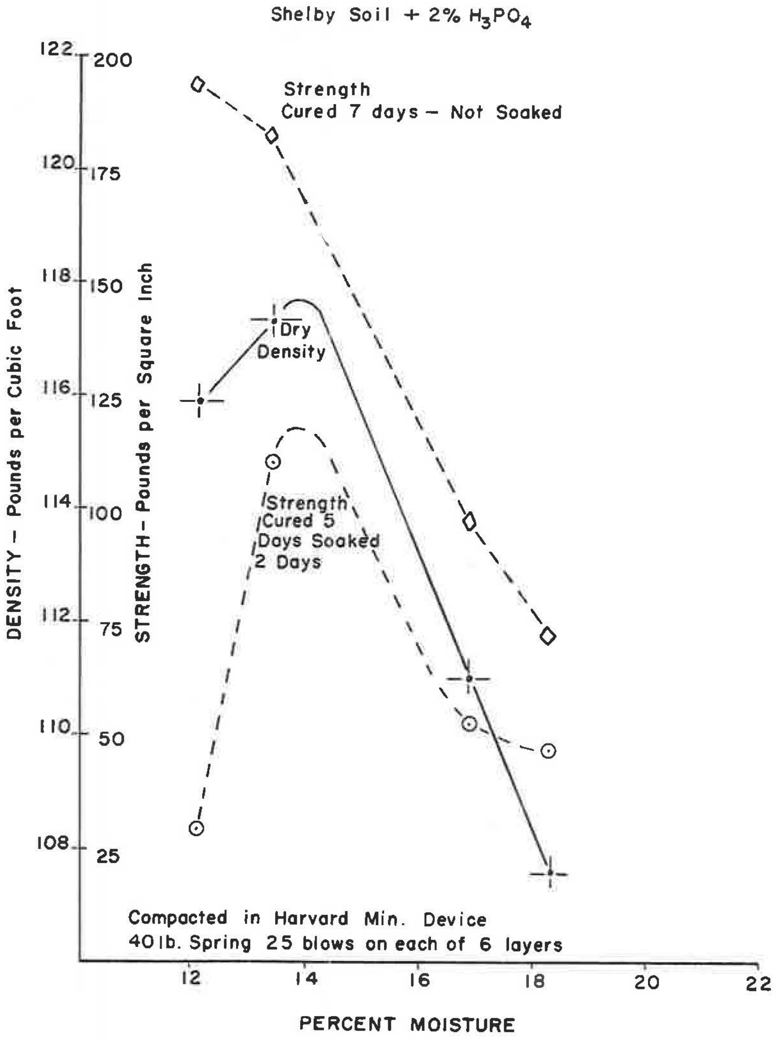


Figure 3. Shelby soil + 2 percent H_3PO_4.

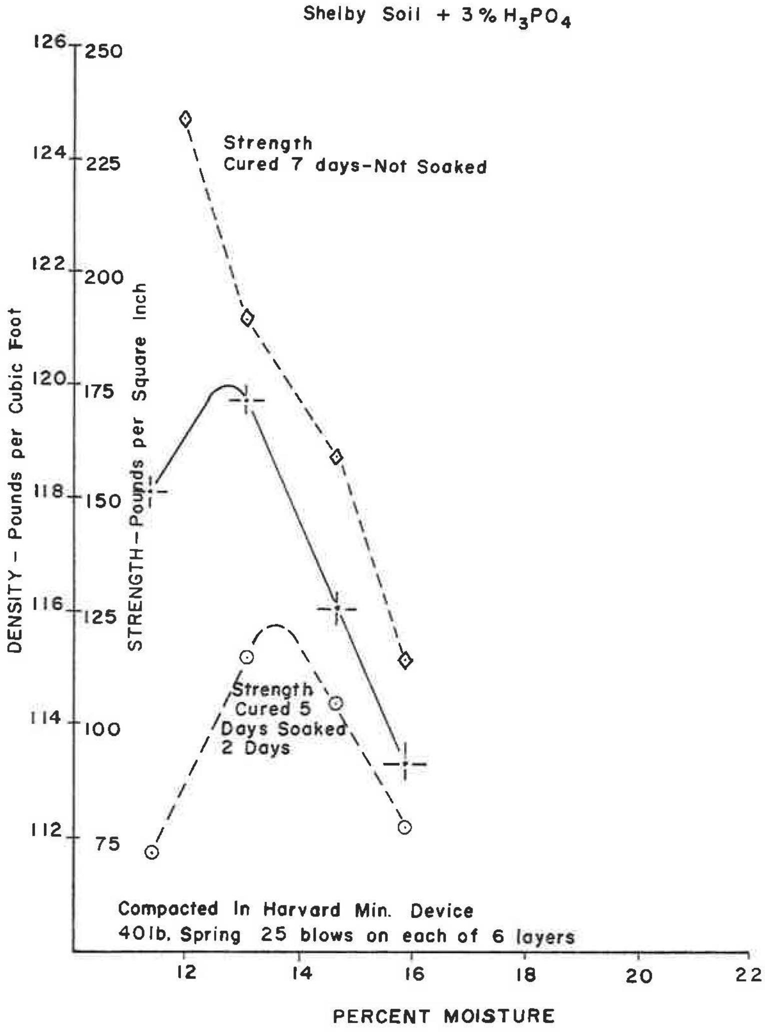
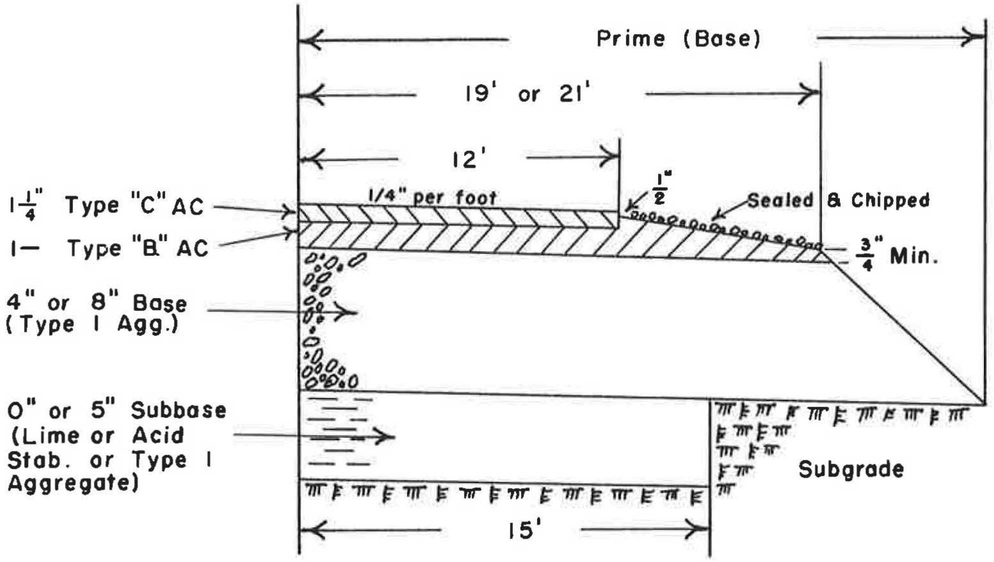


Figure 4. Shelby soil + 3 percent H_3PO_4.



Asphaltic Concrete:

1. Type "B" AC 19' wide unless guard rail is present.

Shoulders:

1. Sealed with 0.35 gal./sq. yd. of RC-4 cutback asphalt.
2. Chipped with 35 lb./sq. yd. of crushed stone.

Figure 5. Half section on tangent test sections.

4. Rolled Stone Base—dimensions shown in Figure 5; compaction to be minimum of 95 percent of standard determined on minus No. 4 fraction by T 99, and computed for total sample, P. I. to be 6 or less.

Construction—Acid Subbase

Pulverization. —It was not possible to meet the specification that 80 percent of the soil pass the No. 4 sieve before incorporating acid. Low speed passes with the P & H were supplemented by repeated processing with tiller, disc, blade and tamper. The final product only emphasized the fact that best laboratory results may be obtained by procedures that are not necessarily practicable when transferred to field operations.

In this instance, high humidity and frequent rains usually prevented proper pulverization. Continuous manipulation during the rare drying periods caused the formation of clay balls with dry hulls and wet cores. These were windrowed and sheepsfooted in one section before being processed by a tiller. At the end of this day's work, 73 to 75 percent passed the No. 4 sieve. And that night it rained again.

Prior to processing Section 14, 1 percent of the 75 percent undiluted acid was added, primarily to facilitate pulverization and secondarily to neutralize carbonates. Every care had been taken in selecting borrow soil, and the acid test and been applied frequently to avoid carbonates. Oddly enough, after repeated pulverizing passes, it was almost impossible to find a noneffervescing soil sample.

Most effective pulverization was accomplished on other sections when the moisture was reduced to near the shrinkage limit. This section could not be dried to that extent, and the average of several tests showed moisture at about optimum. As far as is known, the extra acid did neutralize most of the carbonates.

TABLE 11

RESULTS OF PULVERIZATION TESTS

Section	Station	Percent Passing Sieve			Moisture (%)	O. M.
		1 In.	½ In.	No. 4		
2	8+50	99	62	19	12.1	14.4
2	11+00	100	93	60	9.2	14.4
8	48+00	97	65	21	14.7	20.0
8	51+00	97	75	41	11.1	20.0
8	52+50	97	70	27	13.1	20.0
14	86+00[a]	95	58	19	16.3	17.6
14	87+50[a]	94	53	18	17.1	17.6
14	89+50[a]	97	45	15	18.0	17.6
15	94+50	100	84	50	11.4	18.4
15	97+00	100	87	51	11.6	18.4

[a] One percent extra acid added.

Results of pulverization tests made just before addition of the acid are given in Table 11. Little definite information can be extracted from this tabulation. Although not substantiated by all tests, the trend is toward more pulverization the further below optimum the moisture is reduced. In clay soil such a result would be expected. Wet weather, high humidity, and soil moisture cause greatest trouble in the pulverization process, but slight changes in soil characteristics may also contribute. Since neither factor is readily controllable, it seems probable that only a less stringent pulverization specification would have made phosphoric acid stabilization practicable in the conditions that prevailed in Gentry Co. in 1962.

Pulverization costs. —The cost per square yard for pulverizing varied widely for the four acid sections, indicating to some degree the time and effort devoted to this portion of the processing. All sections were 660 ft long, 30 ft wide and 5 in. thick, but costs varied from $0.39 on Section 8 to $0.80 on Section 15.

Acid. —During preliminary laboratory moisture-density tests the mixtures quickly stiffened, indicating that the acid did not act as a lubricant. The aqueous solution of phosphoric acid, as received on the project, was at 75 percent strength. To obtain more widespread and uniform dispersion, it was diluted to 50 percent and a corrosion inhibitor was added for equipment protection.

Dilute acid was used in three of the test sections with drier subgrade, in a quantity corresponding to 2.8 percent of the dry soil weight. Since subgrade moisture on Section 14 was near optimum, undiluted acid was used in it. This was also supposed to be at the rate of 2.8 percent. The P & H, however, was moved from a lime section where it had been set to cut about 4.5 in. to allow for the bulking effect. Through an oversight, no change was made in the depth of cut, which on acid was supposed to be 5 in. Consequently 75 percent acid was applied to Section 14 at the rate of 4.3 percent.

Compaction. —Densification was first attempted by use of a sheepsfoot tamper but the acid-soil mixture showed an unusual affinity for steel. At times the mixture even picked up in a solid mass and exposed the subgrade. This peculiarity was again demonstrated when a 12-ton steel wheel roller was used and the adhering mixture left a severely potholed, rough subbase surface.

Reduction of the moisture by aeration was most effective in reducing the tendency to adhere. This, and the time element between mixing and compacting was quite critical, since the mixture stiffened in a relatively short time. Therefore, when proper moisture prevented clinging, it was impossible to obtain specified density in the viscid mixture. An initial knockdown pass by a pneumatic roller densified the mixture enough that sheepsfoot tampers penetrated only part of the layer of treated material. A relatively light steel wheel vibratory compactor was used, but densified only the top half or less of the 5-in. depth. The mixture also stuck to it. Since a supercompactor was not available, the choice of roller type was eliminated and all compacting was done with the pneumatic.

If enough moisture was added to bring the mixture to optimum, the clods were wet only on their surface, the finer material sponged up most of the water, and the pneumatic bogged down. In this latest crisis the moisture was reduced and the acid subbase was completed. In three of the four sections, the moisture was from 1.6 to 4.5 percent below optimum. The other section was 2 percent over. In none of the four was specified compaction obtained; the deficiency ranged from 3 to 13 percent.

The consensus was that everything within reason had been tried. Yet, despite all efforts, none of the four acid sections had been built to comply with specifications for pulverization, for moisture, or for density.

Finishing.—Finishing the acid subbase involved trimming to proper cross-section, sprinkling and pneumatic rolling, all within 2 hr after addition of acid. This was not always accomplished within the allotted time.

Sealing and Curing.—An MC-0 prime of 0.3 gal/sq yd was applied to the subbase, and the primed sections were barricaded for 7 days. The prime abraded severely when the sections were opened to traffic. This loosened material was probably the mixture which was trimmed and shifted during finishing operations and which did not bond to the subbase when rerolled.

During the curing period a peculiar condition developed, noticeable to some extent in all acid sections. Bulges occurring somewhat at random were, in some stretches, aligned into narrow, more or less continuous, wandering lines with a striking resemblance to mole runs. Two theories were advanced to explain their formation: (a) that CO_2 gas generated by acid reaction with carbonates had built up enough pressure to heave the primed surface, and (b) that concentrations of clay balls had swelled after absorbing moisture.

Regardless of the undetermined cause of the localized bulging, tests in the section showed a general loss in density, from 87 percent when constructed to 77 percent 8 days later. In the same period, moisture tests showed an increase from 1.6 percent below optimum to 5.5 percent above. Meanwhile, the mixture had changed in appearance from a cohesive, somewhat glutinous material to a granular type which was easily dug with a pocket knife. Density was restored to the original 87 percent by pneumatic rolling, but no improvement in cohesion was noticed. This was the condition of nearly all acid subbase when it was covered with base.

Tests.—Moisture-density tests were made in each of the four sections on typical soil and on acid-soil mixtures sampled immediately after passage of the P & H. In preliminary laboratory tests, addition of acid had increased the weight and decreased optimum moisture. Field tests followed this trend on but two of the four sections.

Tests for particle size, plasticity and shrinkage limit were made on native soil and fresh acid-soil mixtures. Similar tests were made on Shelby tube samples taken when the subbase was 280 days old and impossible to core, but still soft enough to take a push sample. Comparisons revealed that the engineering properties of the soil were improved by addition of phosphoric acid. However, the P. I. at 280 days was considerably higher than when constructed, in both the 2.8 and 4.3 percent areas.

Unconfined compressive strength was determined at various ages on 2- by 4-in. specimens molded on Sections 2, 8 and 15 from the fresh field mix and cured in the humid room at Jefferson City. Results are shown in Table 12.

TABLE 12

COMPRESSIVE STRENGTH OF 2- BY 4-IN. SPECIMENS MOLDED FROM FIELD MIX (2.8 PERCENT ACID)

Station	Test Sect.	$CaCO_3$ Equiv. (%)	Strength (psi)			
			7-Day	14-Day	28-Day	6-Mo
8+00	2	—	0	0	0	11
8+50	2	—	0	2	0	15
11+00	2	—	0	1	8	8
48+00	8	—	13	23	19	44
51+00	8	—	0	2	4	15
52+50	8	—	0	3	10	—
92+50	15	0.41	5	6	8	12
94+50	15	0.96	0	0	2	8
97+00	15	0.73	6	10	8	22

A Shelby tube sample from the subbase (age 280 days) at Station 97+00 had a

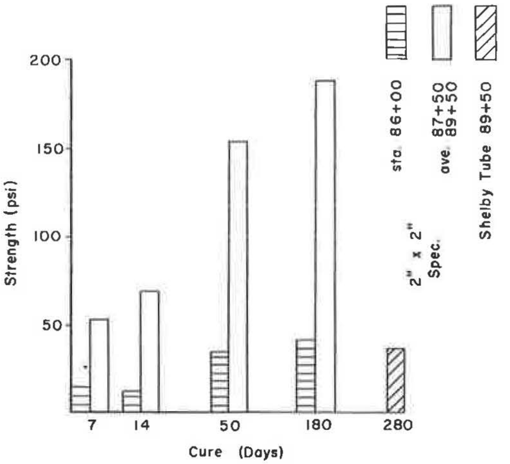


Figure 6. Soaked strength, 4.3 percent acid section.

strength of 4.5 psi. The molded specimens and Shelby tube samples from these sections showed little resistance to moisture, either falling apart during immersion or having very low strengths.

Soaked strengths of 2- by 2-in. specimens molded from Section 14 field mix are shown in Figure 6. Results from Station 86 are shown individually because the calcium carbonate equivalent in this area was 1.66 percent, and the extra 1 percent acid probably did not neutralize all the carbonates. Specimens from this station withstood immersion, but strength has shown only a slight increase with age. The specimen cured 180 days cracked in storage.

Specimens molded from two other stations have shown an increase in strength with age. The following tabulation shows the values in Figure 6 when converted to the average gain in soaked strength per day for each increment of curing age.

Days	Gain in Strength (psi per day)
0-7	7.8
7-14	2.1
14-50	2.4
50-180	0.2

Although the reason is not known, it is conjectured that the better soaked strengths of the specimens molded from this section, when compared to Sections 2, 8, and 15 may be due to the higher acid content or the higher than optimum moisture content.

The soaked CBR of the acid-soil field mix (Section 14) was 74.4, compared to 3.7 for the untreated soil. Swell of the acid-soil mix was negligible, but the untreated soil swelled 3.6 percent.

Lime Subbase

As would be expected, subbase construction procedures and results on the four lime test sections were much the same as reported in the first part of this report. The maximum density was reduced and optimum moisture increased. Bulking was calculated at about 13 percent and the P & H machine set to cut 4.5 in. The average compacted subbase measurement was a little more than 5 in., and the thickness control was generally excellent.

Test results for plasticity, gradation and shrinkage limit made on newly processed mixture showed the same general improvement in the clay soil as reported earlier. Several cores taken from the lime sections at the age of 280 days were similarly tested and, with one exception, showed continuing improvement with age. The average L. L., P. L. and P. I. all increased from that determined during construction.

Unconfined compressive strengths on field-molded moist-cured soaked specimens of various ages indicated that the rate of gain in strength was most rapid during the first 7 days. This coincides with the curing period during which no traffic was allowed to use the sections. Strengths showed an increasing trend from 75± psi at 7 days to more than 400 psi at 6 months. Soaked 280-day cores, constructed and cured under different conditions than in the laboratory, showed almost exactly the same strength as the 28-day molded specimens. The CBR of the mixture was almost 20 times that of the untreated soil, and the swell was reduced almost to zero by addition of lime. The volume of the cores was calculated from measurements and, on the average, showed an increase in density from that at time of construction. The somewhat lower moisture content in the cores may have caused shrinkage and densification.

Rolled Stone Base and Subbase

Four of the 660-ft test sections had a 5-in. rolled stone subbase placed in one lift, and compacted to at least 95 percent. A 4-in. lift of the same base material was placed over two of the sections, while 8 in. in two lifts was constructed on the other two. The only construction difficulties or delays were caused by variations in moisture. When near optimum, pneumatic rolling produced an average density of 99. 5 percent.

Base gradation was similar to that on the other projects, with a number of samples exceeding the specified maximum passing the No. 4. The P. I. at construction averaged 6, the maximum specified. In March 1963, shoulder samples averaged 8 and cores drilled from the base in May averaged 9. The latter figure may have been influenced by the inclusion of drill cuttings in the samples. Tests showed the minus No. 200 content to have been increased 7 percent by use of the drill. In both the March and May investigations, base moisture was below optimum.

Asphaltic Concrete

The two courses of asphaltic concrete were built to comply with standard Missouri specifications. About 6 wk after opening to traffic, permeability tests by two methods indicated low permeability.

Performance

Deflection. —Deflections on the test sections were measured by Benkelman beam in the fall of 1962; the spring, summer and fall of 1963; and twice in the first half of 1964. Since spring readings are most critical in Missouri, remarks will be confined largely to the results obtained in those periods. The first two sets of readings showed that the average deflection is higher in spring than in fall, regardless of design, and the weaker the design the greater is the increase. Neither would be unexpected.

Two 660-ft sections were underdesigned, with 3 in. of asphaltic concrete on a 4-in. rolled stone base on untreated subgrade. From deflection on those sections, it was evident that as the crushed stone thickness was increased to 8 in. on subgrade and to 4 and 8 in. on the 5-in. stone subbase, the percentage reduction in deflection usually increased. This would also be expected, since the stronger structure should deflect less.

Apparent incongruities have been noted, as for example in the Fall 1962 readings. Deflections indicated that the lime subbase was no better than the untreated subgrade, and the phosphoric acid subbase was actually worse than native soil. More recent measurements tend to refute the early readings, leading to the belief that subbase strength is increasing as cementation occurs with greater age. This may be especially applicable to the acid sections, which lacked density and showed an unbound, granulated structure when primed and covered with base. In May 1963, push samples of the acid subbase were quite firm and well bonded together.

The 1964 spring readings indicated that only one 3-4-0 section had an average deflection of more than 0. 04 in., although maximums exceeding that figure were measured on all of the 16 sections. Signs of distress were evident on both 3-4-0 sections, perhaps portending more extensive signs of failure in the near future.

Equivalencies of Stone vs Lime. —Based on deflections alone, it was indicated that 5-in. lime subbase was equivalent to about 1 in. of rolled stone base in the sections of 3-in. asphaltic concrete—8-in. stone—5-in. lime design. But where the stone base thickness was reduced to 4 in., the 5 in. of lime subbase was equivalent to a little more than 2 in. of stone.

Rutting-Roughness. —Rutting has been negligible, and roughometer measurements have produced little reliable information. In November 1962 and in February and May 1963, roughness measurements on all the thicker sections were invariably greater in the southbound lane; there is no known explanation for this phenomenon.

Summary

The Missouri Highway Department's experience during the construction, testing and subsequent investigations of the 16 test sections justifies the following statements.

1. With some exceptions, evidence developed in the few years since construction of these projects is insufficient to warrant drawing hard and fast conclusions or making definite comparisons between different designs.
2. Various bits of information have been accumulated whose significance, if any, is at present obscure. They may, however, be relevant at some future time when all available data are assembled for a final picture of the total performance of these test sections.
3. Specifications for the acid mixture may have been unrealistic. Laboratory tests provided data for writing special provisions for the use of this experimental material. Requirements for soil pulverization and for moisture and density of the mixture could not be fulfilled despite every possible effort.
4. The designs, based on the results of preliminary laboratory research, called for a 5-in. compacted subbase, using 6 percent lime or 2.8 percent phosphoric acid, both figured on the dry weight of the soil. For comparison, a third type of subbase was also included, using 5 in. of crushed stone. The lime content was closely controlled by the lime spreader. The acid content had to be adjusted through a spraybar on the P & H stabilizer. Periodic clogging of a few of the nozzles, neutralization of some acid by unanticipated calcareous material in the soil, and forgetting to change the cutting depth caused some unknown variations in the acid quantity, which probably ranged from about 2 to 4.3 percent instead of the desired quantity. Thus, rather unintentionally, the experimental aspects of the job were somewhat amplified.
5. Both hydrated lime and phosphoric acid reacted with the glacial clay soil to improve its engineering properties, as measured by laboratory tests for plasticity, gradation and shrinkage. Examination of test results, however, reveals that neither 2.8 nor 4.3 percent acid has yet improved the soil as much as did 6 percent lime.
6. Drying and pulverizing the subgrade in the acid sections was a prolonged operation, and the results of all the work were often destroyed by rain. No such difficulties were encountered in the lime sections which quickly dried, permitting work to proceed. Rain may even have accelerated the reaction between soil and lime and expedited the breakdown of clayey clods. Therefore, what was a distinct disadvantage in the acid sections may even have been of some benefit in the lime sections.
7. Specified density was easily obtained in lime subbase with a minimum of rolling but could not be obtained in any of the four acid sections. Requirements for the latter were somewhat higher than for lime—100 percent of T 99 in four layers as compared to 95 percent by the standard method.
8. Moisture was specified to be within 2 percent of optimum on lime and 1 percent on acid subbase during compaction. In general, the lime sections complied, but none of the acid sections were within the limits.
9. Acid sections were supposed to be finished within 2 hr after the addition of acid, and lime sections within 72 hr. The tendency of the acid-soil mixture to adhere to steel compactors eliminated from use all but the pneumatic roller. A moisture content near optimum facilitated compaction but tended to bog down the roller. Lowering the moisture caused the mixture to stiffen, and proper density could not be attained. The 2-hr time limit was frequently exceeded; the 72-hr limit never was.
10. When the utmost effort failed to produce required density in the acid subbase, it was sprinkled, finished by pneumatic rolling, primed and barricaded for 7 days. During the curing period fluffing developed, and the subbase lost density and became loose and granular. The primed mixture bulged erratically but was restored to original density by rerolling.

11. Tests on Shelby tube samples have since indicated the development of a more tightly bound, cohesive material with passage of time. Strength test results are somewhat ambiguous, but little gain has been shown. A marked rebound in plasticity has been noted. Cores could not be drilled from the acid subbase, but 9-mo push samples and 6-mo molded specimens, in general, exhibited little compressive strength. Cylinders molded from material with the highest acid content did show a trend toward greater strength with increase in age. Most specimens from mixtures with lesser acid content disintegrated when submerged in preparation for compressive strength testing.

12. No test section has been subjected to abnormally heavy loads, although current volume is reported equal to that anticipated, at the time of design, for 1980. It is believed that repairs necessary on projects to the north are readily explainable by a peculiar, nonrecurring combination of several factors—pavement age, truck traffic, time of year, and lack of maintenance.

13. Construction of the overlying base and asphaltic concrete courses was normal on all sections. It has not been shown, to date, that base P. I. somewhat in excess of 6 is detrimental to the performance of this flexible pavement.

14. Maximum spring deflections have recently exceeded 0.04 in. on all of the 16 sections, regardless of design. The average, however, has been above 0.04 only on one of the two weakest sections. A full-width transverse crack and a depressed alligator-cracked area presently give evidence of weakness and may be a sign of more extensive distress to be expected. All sections are being closely observed for future developments, and further investigations will be made whenever necessary.

Summation

Since the last portion of this experimental job was completed and opened to traffic less than 2 yr ago (as of July 1964 writing), this is only an interim report to describe the preliminary research, design, construction and current performance of the three projects, including the 16 test sections. Final conclusions as to comparative performance of different materials and designs will be possible only in the future, when answers will undoubtedly replace conjectures about indications.

On these Missouri projects, both lime and phosphoric acid improved the engineering properties of the clay subgrade, reducing plasticity, volume change and percentage of fines, while increasing strength and stability.

When mixed with glacial Shelby clay, lime almost immediately reacts with the soil and improves its characteristics. When compacted, the mixture provides a firm subbase which can effectively replace some thickness of crushed stone base. In areas where stone is scarce and, therefore, expensive, or where satisfactory material must be imported, lime-stabilized subbase can also be more economical than an equal thickness of crushed stone.

Bid prices per ton on these projects were $30 for bulk lime and $125 for 75 percent phosphoric acid. By dry soil weight, 6 percent lime was used, whereas the acid percentage varied from about 2 to more than 4. A total of 79.11 tons of acid was used in four test sections of 660 ft each. On the remaining 16.87 mi, 3041.6 tons of lime were used.

On F-524(2), Gentry Co., the only project on which both lime and phosphoric acid were used, costs to the state were approximately $23,000/mi for acid and $8,900/mi for the lime. Costs to the contractor would probably present a still more unbalanced comparison, in view of the great amount of time and effort invested in fruitless attempts to meet the specifications on acid subbase for pulverization, moisture content and density.

If any future phosphoric acid stabilization should be contemplated in Missouri under specifications similar to those in effect on these sections, it is probable that manipulation costs will be higher than on this job and the cost differential between lime and acid stabilization will be even greater. Based on Missouri's experience and present knowledge, the gap in costs might be somewhat narrowed by development of more efficient pulverizing equipment or a phosphoric acid additive to facilitate pulverization, or by writing less stringent specifications.

Cost data, in combination with other factors, indicate that lime stabilization may be most efficiently used in subbase construction in areas where clay soils respond well to such treatment and satisfactory rock deposits are scarce. Acid might compete only if the cost differential can be considerably reduced and equivalent or greater benefits to the soil can be demonstrated.

At present the definite statement can be made that for more than 2 yr, all pavement composed of a 5-in. lime-soil subbase, 8-in. rolled stone base and 3-in. asphaltic concrete surface has given satisfactory service. This is true also of 15 of the 16 test sections.

Early deflection measurements indicated that, inch for inch, crushed stone was better as subbase material than either the lime-soil or acid-soil mixtures. Recent measurements indicated somewhat of a reverse trend, perhaps reflecting a gain in strength as acid and lime sections age. Future investigations will without doubt provide information to permit accurate and final evaluation of the various designs.

Based on Missouri's experience with acid stabilization, it is believed that, before any such future work, extensive laboratory testing should determine whether such rigorous specifications as were in effect here are necessary or desirable. Additional investigation should be made into the long-time effects and permanence of any improvements in soil properties induced by the acid.

On these projects the preponderance of evidence points to the superiority of lime over phosphoric acid as a stabilizing agent. Lime subbase was easy to construct to specified requirements, dried quickly after the frequent rains and did not delay progress, provided a stable foundation and excellent working platform for building the base, and has shown a consistent gain in strength. None of this can be said about acid sections.

Missouri's experience on these projects leads to the following recommendations which are believed to deserve serious consideration in the design of any future similar subbase and base for flexible pavement.

1. Extend the stabilized subbase through the shoulders instead of building a trenched section with relatively impermeable shoulders.
2. Eliminate the necessity for a rotting period and remixing pass of equipment by requiring only that 100 percent of the lime-soil mixture pass the 1½-in. sieve, thereby greatly expediting progress. Tests on one of these projects where such a specification was in effect led to the belief that the lime's stabilizing effect suffered no impairment by such a procedure.
3. Specify a bituminous prime for lime subbase instead of curing by periodic applications of water.
4. Liberalize plasticity requirements in rolled stone base.
5. Increase rolled stone base density requirements.
6. If soil and traffic conditions demand a base thickness of 8 in. or more, treat at least the top 4-in. lift with cement or bituminous material.
7. Specify asphaltic concrete thickness greater than 3 in.
8. Write and enforce a specification that will make certain that work on asphaltic concrete stops in time for pavement to be opened to traffic and receive normal maintenance during the winter. Associated with this requirement would be one exacting strict control of heavy construction traffic over new pavement.

It is believed that incorporation of some or all of these suggestions into the design will result in a much higher type road. Costs will undoubtedly be higher, but the service life and performance record of the roadway would be upgraded sufficiently to overbalance the additional costs.

REFERENCE

1. Jones, W. G. Lime-Stabilized Test Sections on Route 51, Perry County, Missouri. Highway Research Board Bull. 193, pp. 32-39, 1958.

Split-Tensile Strength of Lime-Stabilized Soils

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The tensile strength characteristics of lime-soil mixtures are of considerable importance in any type of rational pavement design procedure. There is little information at this time concerning the tensile strength properties of lime-soil mixtures since the majority of the reported work has dealt primarily with compressive strength.

This paper presents the principles of the split test (a diametral compression test for determining tensile strength), an evaluation of the test, and the application of the test procedure to lime-soil mixtures. Split-tensile and compressive strength test results are given for 11 soils.

The results show that lime-soil mixtures develop substantial tensile strength and that the split-tensile strength is closely correlated with unconfined compressive strength. Factors such as soil type, lime treatment, and length of curing period influence the magnitude of the split-tensile and compressive strengths, but do not affect the ratio between the strengths. The investigation clearly indicates that the split test has considerable merit as a test procedure for evaluating the tensile strength properties of lime-soil mixtures.

•AN INCREASING emphasis has been placed on the use of stabilized highway materials in recent years. Through the use of stabilizing agents, low-quality materials can be economically upgraded to the extent that they may be effectively utilized in the pavement structure. Stabilizing agents also improve natural materials of medium and high quality, and materials ranging from well-graded crushed stones to highly plastic clays have been successfully stabilized. Although these natural materials are very dissimilar in many respects, they characteristically have low tensile strengths. Stabilization with lime, lime-fly ash, portland cement, etc., imparts a tensile strength to these materials. The compressive strength of the material is also considerably increased. Generally speaking, the compressive strengths are several times greater than the tensile strengths. The stress-strain curves for a typical stabilized material are nearly linear to failure with very little inelastic yielding (so-called brittle behavior). The modulus of elasticity of the stabilized material is normally several times greater than that for the natural material. Figure 1 illustrates stress-strain curves for a typical lime-soil mixture.

Stabilized highway materials are generally incorporated into the pavement structure as base courses, subbases, or subgrades. Burmister (1) and others have shown that in a layered system of elastic materials, where the overlying layers have higher moduli of elasticity than underlying layers, tensile stresses are developed at the interfaces between the layered materials. This layered system analysis is commonly presumed to be applicable to a highway pavement where the stiffer materials are used in the upper layers. Since many stabilized materials are relatively weak in tension, any type of rational design procedure must take into account their tensile strength. Unfortunately, much of the research and development work concerning stabilized

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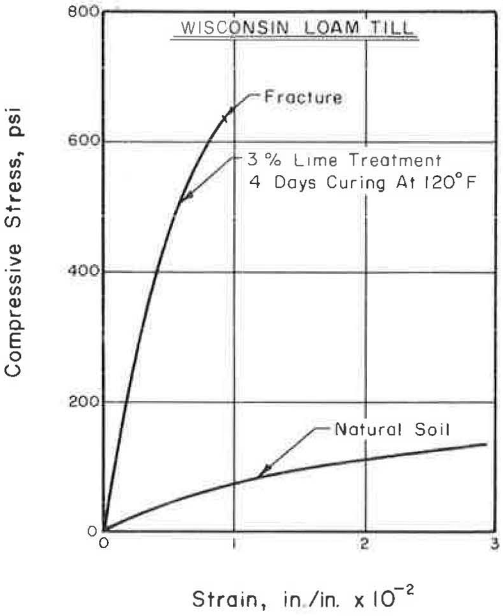

Figure 1. Typical stress-strain curves.

materials has utilized unconfined compression and triaxial testing methods and there is a lack of basic knowledge concerning their tensile strength properties.

If these stabilized materials are to be used in an efficient and economical manner, it is imperative that a satisfactory test method be developed for determining their tensile strength. The investigation described in this report is concerned with preliminary studies of the split-tensile strength of lime-stabilized soils.

SPLIT TEST

The split test was developed independently by Carneiro and Barcellos (2) in Brazil and Akazawa (3) in Japan. The test procedure has been primarily utilized to evaluate the tensile strength of concrete (ASTM Designation: C 496-64T) but it has many merits as a tensile test and could easily be adapted for stabilized highway materials, such as lime-soil mixtures, which exhibit a brittle-type behavior and have relatively low tensile strengths.

The split test is conducted by placing a cylindrical specimen horizontally between two loading surfaces and loading the specimen along two opposite generatrices as shown in Figure 2. For brittle materials weak in tension, the specimen fails in tension along the loaded diameter, A-B, of the cylinder.

Theory of Split Test

The theoretical solution of the split test is based on the theory of elasticity. Frocht's equations (4) for the stresses at a point in terms of rectangular coordinates (Fig. 3) are as follows:

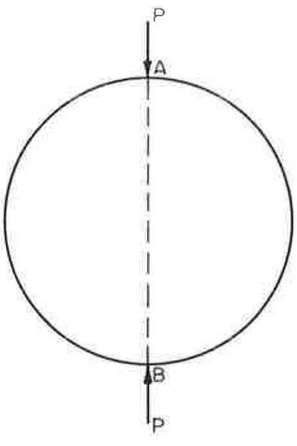

Figure 2. Split test.

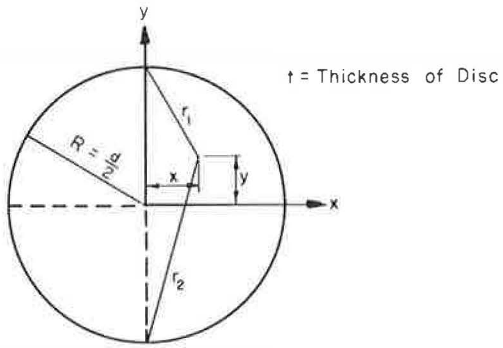

Figure 3. Coordinate system.

$$\sigma_x = \frac{-2P}{\pi t}\left[\frac{(R - Y)\,X^2}{r_1^4} + \frac{(R + Y)\,X^2}{r_2^4} - \frac{1}{d}\right] \tag{1}$$

$$\sigma_y = \frac{-2P}{\pi t}\left[\frac{(R - Y)^3}{r_1^4} + \frac{(R + Y)^3}{r_2^4} - \frac{1}{d}\right] \tag{2}$$

$$\tau_{xy} = \frac{2P}{\pi t}\left[\frac{(R - Y)^2\,X}{r_1^4} - \frac{(R + Y)^2\,X}{r_2^4}\right] \tag{3}$$

where

σ_x, σ_y, τ_{xy} = stress components with respect to rectangular coordinates;
x, y = rectangular coordinates;
P = load applied to specimen;
t = thickness of cylindrical specimen;
d = diameter of cylindrical specimen;
R = radius of cylindrical specimen; and
r_1, r_2 = location coordinates.

For the horizontal diameter of the cylinder, the X-axis, $Y = 0$, $r_1 = r_2 = X^2 + R^2$, and the stress equations simplify to:

$$\sigma_x = \frac{2P}{\pi td}\left[\frac{d^2 - 4X^2}{d^2 + 4X^2}\right]^2 \tag{4}$$

$$\sigma_y = \frac{-2P}{\pi td}\left[\frac{4d^4}{(d^2 + 4X^2)^2} - 1\right] \tag{5}$$

$$\tau_{xy} = 0 \tag{6}$$

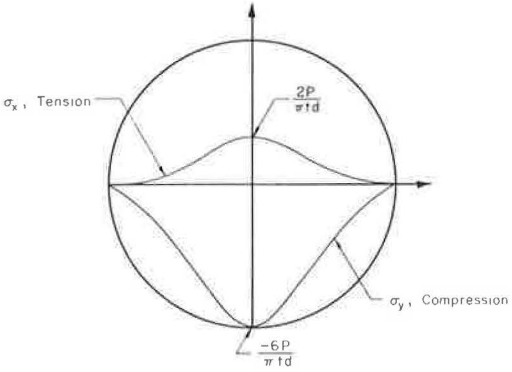

Figure 4. Stress distribution on X-axis.

The vertical stress, σ_y, along the X-axis is always a compressive stress and varies from a maximum at the center to zero at the circumference. At the center, the magnitude of σ_y is $\frac{-6P}{\pi td}$ and the accompanying horizontal stress, σ_x, is a tensile stress equal to $\frac{2P}{\pi td}$. This indicates that the material being tested must have a compressive strength at least three times its tensile strength if it is to fail in tension. The stress distribution along the X-axis is shown in Figure 4.

For a vertical plane through the center of the cylinder along the Y-axis, Frocht's equations for the stresses reduce to:

$$\sigma_x = \frac{2P}{\pi td} \tag{7}$$

$$\sigma_y = \frac{-2P}{\pi t}\left[\frac{2}{d - 2y} + \frac{2}{d + 2y} - \frac{1}{d}\right] \tag{8}$$

$$\tau_{xy} = 0 \tag{9}$$

The horizontal tensile stresses, σ_x, along the vertical plane have a constant value of $\frac{2P}{\pi td}$ and the vertical compressive stresses vary from $\frac{-6P}{\pi td}$ at the center of the disc to ∞ at the end of the loaded diameter. These high compressive stresses at the loading points will cause failure, thus preventing failure in the central portion of the vertical diameter of the specimen due to tensile stresses (5). Photoelastic studies have shown that the point of maximum stress concentration can be moved away from the load point by applying a distributed load through a loading strip. In addition, the distributed load changes the σ_x stresses in the vicinity of the loading strip to compressive stresses, placing the material in the immediate area beneath the loading strips under the influence of compressive stresses (5). Most brittle materials are fairly strong under such a state of stress and, therefore, the specimen fails in tension in the central part of the loaded diameter.

A schematic representation of the test specimen and the loading strips is shown in Figure 5. The utilization of the loading strips somewhat alters the stress distribution in the specimen. According to Wright (6) the horizontal stress distribution σ_x along the vertical diameter is closely approximated by:

$$\sigma_x = \frac{-2P}{\pi td}\left[1 - \frac{d}{2a}\ (\alpha - \sin\alpha)\right] \tag{10}$$

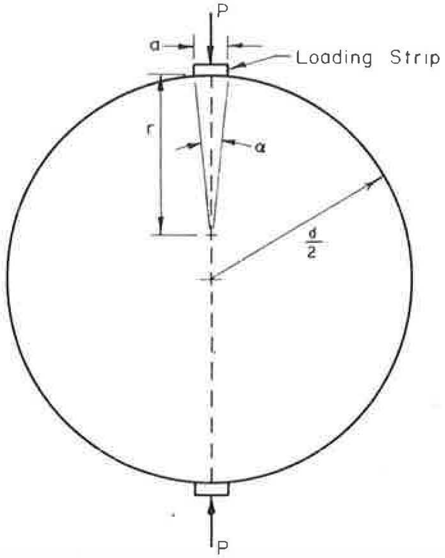

Figure 5. Loading strips.

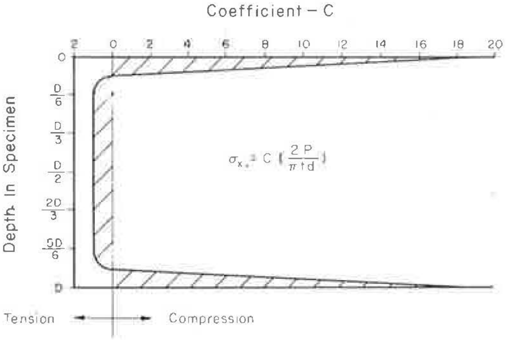

Figure 6. Horizontal stress distribution on Y-axis for loading strip width equal to d/12.

provided the width of the loading strip, a, is less than d/10. The resulting horizontal stress distribution on the vertical diameter is shown in Figure 6. Peltier's (7) work on loading strips indicated that the tensile stresses remain uniform over a reasonable proportion of the diameter if the loading strip width is less than d/5. It thus appears that when loading strips of the widths suggested above are utilized, the tensile stresses over a substantial portion of the loaded diameter have a value of approximately $\frac{2P}{\pi td}$.

Loading strip characteristics and cylinder size are two factors which have received considerable attention. According to Mitchell (5), loading strip characteristics such as width, thickness, and type of material, affect the type of rupture but not the tensile stress at failure. Rudnich et al. (8), in their evaluation of the split test for use with ceramic materials, concluded that of the various types of failure that may occur in the split test, only a shear-type failure is unsatisfactory for determining the split-tensile strength. It is recommended that the loading strip be of a rather pliable material which can conform to any surface irregularities in the specimen.

The effect of cylinder size on the split-tensile strength of concrete was investigated by Carneiro and Barcellos (2), Akazawa (3), and Wright (6). All investigators agreed that the size of cylinder had very little effect on the test results but that larger cylinders gave results with a smaller coefficient of variation.

Evaluation of Split Test

Since the theoretical analysis of the split test is based on the theory of elasticity (4), it is logical that the test would provide a good indication of the tensile strength of materials which behave elastically to failure. Rüsch and Vigerust (9) used the split-tensile test to evaluate the tensile strength of concrete and concluded that the tensile splitting strength is near the true strength, especially for high-strength concrete which is more nearly an elastic material.

Although the tensile stress distribution along the vertical diameter of a test specimen is approximately constant, the complete stress distribution on the diameter is quite complicated. Bawa (10) pointed out that the vertical stress, σ_y, has a large variation along the vertical diameter and, therefore, the stress difference, $\sigma_y - \sigma_x$, is highly variable.

With brittle materials, strain as well as stress may be important in determining the tensile strength of the material. Since there is not an uniaxial state of stress in the split test, Poisson's ratio may have an effect on the indicated tensile strength. Bawa (10) and Ramesh and Chopra (11) emphasized that Poisson's ratio cannot be ignored in a biaxial state of stress such as exists in the split test. If strain is an important factor in determining the tensile strength, higher strengths will be obtained if Poisson's ratio is low and lower tensile strengths will be obtained if Poisson's ratio is high. Bawa (10) observed this strain influence for tests on cement mortar.

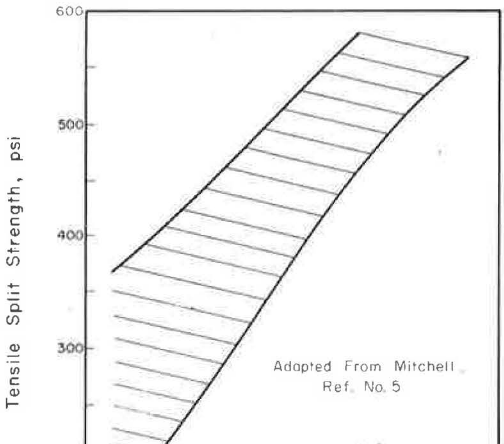

Figure 7. Tensile strength vs compressive strength for concrete cylinders.

A major disadvantage of the split test is that it does not resemble the actual field service conditions of many materials which exhibit a "slab type" of behavior. Many engineers prefer a flexural test for evaluating such materials.

Correlation With Other Test Methods

Mitchell (5) concluded, as a result of his literature review, that the split test gives tensile strengths less than the flexure test and greater than the briquet test with the split test having better reproducibility than the other forms of tension tests. The literature indicates that the split-tensile strength of concrete is approximately 50 to 60 percent of the flexural strength. Many investigators attempted to establish a general relation between split-tensile and unconfined compressive strengths of concrete. Figure 7 shows the general trends of several of these investigations. The hatched area encompasses the range of test results reported.

Summary

The split test appears to be a test method that can be easily adapted for evaluating the tensile strength of stabilized highway materials. Theoretical and experimental data indicate that a relatively uniform tensile stress distribution exists on the major portion of the loaded diameter of the test cylinder. This uniform stress distribution cannot be obtained with any other type of tensile test procedure currently being used to evaluate stabilized highway materials.

INVESTIGATION

The objectives of this investigation were to evaluate the split-tensile strength characteristics of cured lime-soil mixtures and the relation of split-tensile strength to unconfined compressive strength. The factors of lime type and percentage, curing period, and soil type were varied to determine their influence on split-tensile strength characteristics.

Materials

Eleven soils of diverse physical and mineralogical properties and three types of lime were used in the study.

Soils. —Loess-derived, Wisconsinan till-derived, and Illinoian till-derived soils were used in the investigation. The soils were sampled in the field, air dried in the laboratory, pulverized, screened to remove the plus No. 4 material, and stored for subsequent use. Selected physical, engineering and mineralogical properties were determined for the soils. A summary of selected soil properties is presented in Table 1.

Limes. —High-calcium hydrated lime, monohydrated dolomitic lime, and a by-product high-calcium lime were used as stabilizing agents. The limes were produced by conventional lime manufacturing procedures, with the exception of the by-product lime which is obtained from the manufacture of acetylene gas. Properties of the limes are presented in Table 2.

Mixing

Proper quantities of lime and air-dry soil were thoroughly blended in a Lancaster mixer. The amount of water required to bring the lime-soil mixture to optimum

TABLE 1

SELECTED SOIL PROPERTIES

Designation	Type	Source	AASHO Classification	$< 2\mu$ Clay (%)	L. L. (%)	P. I.	Predominant Clay Mineral
1	Accretion gley	Effingham Co.	A-6 (7)	18	33.7	18.4	Montmorillonite
2	Accretion gley-1	Sangamon Co.	A-6 (10)	25	32.5	14.2	Mixed layer
3	Accretion gley-2	Sangamon Co.	A-6 (12)	26	35.9	21.9	Mixed layer
4	Bryce B	Iroquois Co.	A-7-6 (18)	52	53.1	28.8	Illite
5	Cowden B	Montgomery Co.	A-7-6 (19)	34	53.9	31.4	Montmorillonite
6	Cowden C	Montgomery Co.	A-6 (9)	20	32.4	12.6	Montmorillonite
7	Cowden B	Randolph Co.	A-7-6 (19)	38	54.2	32.5	Montmorillonite
8	Illinoian B	Sangamon Co.	A-6 (11)	29	37.2	19.2	Mixed layer
9	Illinoian till	Sangamon Co.	A-6 (6)	14	25.5	11.0	Illite
10	Illinoian till	Effingham Co.	A-6 (6)	17	24.6	11.7	Illite
11	Ottawa A-6	LaSalle Co.	A-6 (8)	25	25.2	10.8	Illite

TABLE 2

PROPERTIES OF LIMES

Lime Designation	Type	%$Ca(OH)_2$	%MgO	%$Mg(OH)_2$	% Passing No. 325 Sieve
A	High-calcium hydrated	96	--	--	95
B	Monohydrated dolomitic	58.8	33.3	1.7	85
C	By-product high-calcium hydrated	96	--	--	76

moisture content was then added and mixing was continued for approximately 2 min. Following mixing, the lime-soil mixture was covered and allowed to stand for 1 hr before specimens were compacted.

Sample Preparation

A series of sixteen 2-in. diameter by 4-in. specimens were prepared for each test condition, i.e., lime type, soil type and curing period. The specimens were molded in three equal layers with each layer receiving a compactive effort of 20 blows of a 4-lb hammer dropping 12 in. Each layer was scarified to provide bond between the adjacent layers. After proper trimming, the specimens were extruded from their molds and cured.

All specimens were compacted at approximately optimum moisture content as determined by a moisture-density test. The moisture-density test was conducted in a manner similar to AASHO Designation: T 99-57, except that 2-in. diameter by 4-in. molds were used and the compactive effort was applied through 20 blows of a 4-lb hammer having a 12-in. drop. This compactive effort produces maximum dry densities and optimum moisture contents similar to those obtained from testing by Method A of AASHO Designation: T 99-57 for moisture-density relations of soils. Optimum moisture contents and maximum dry densities for the various lime-soil mixtures are presented in Table 3.

Curing

After compaction, trimming, and extrusion, the specimens were placed in 1-gal metal cans and the can lids were sealed with Perma-Tex. The sealed cans were then placed in a 120 F curing cabinet for periods ranging from 1 to 75 days.

TABLE 3

COMPACTION PROPERTIES OF LIME-SOIL MIXTURES

Soil	Lime		Max. Dry Dens. (pcf)	Opt. Moisture (%)
	Type	%		
1	B	7	114.3	15.3
2	A	7	106.0	17.0
3	A	5	112.0	15.8
4	A	5	97.3	25.8
5	B	7	95.5	24.5
6	C	3	112.6	15.0
7	B	5	98.2	23.0
8	C	5	108.5	17.8
9	A	3	121.0	13.0
10	C	3	124.3	11.5
11	A	3	119.6	14.3
	A	5	116.4	15.0
	B	3	118.5	13.4
	B	5	117.4	14.6
	C	3	119.5	14.2
	C	5	116.0	15.1

Testing Procedure

At the termination of the curing period, eight alternate specimens (1, 3, 5, etc., or 2, 4, 6, etc.) were tested in unconfined compression and the remaining eight were tested in split tension. A Riehle hydraulic machine with a strain rate of 0.05 in./min was used for all testing. The unconfined specimens were tested in the usual manner. Loading strips 0.25 in. wide and approximately 0.07 in. thick were used with the split-tension specimens. The 0.25-in. wide loading strips gave a width to diameter (a/d) ratio of ⅛. Specimen moisture content at the time of testing was determined and found to be approximately optimum.

Testing Program

The testing program was divided into two phases. Only one soil type was used in Phase I, but 11 soils were used in Phase II.

Phase I.—This part of the program involved only the A-6 subgrade soil, a calcareous Wisconsinan till, from the site of the AASHO Road Test near Ottawa, Ill. Lime type, lime percentage, and curing periods were varied over wide limits to evaluate the influence of such variations on split-tensile strength and the ratio of split-tensile strength to unconfined compressive strength. Lime treatments, curing periods, and test results are presented in Table 4.

Phase II.—Eleven different soils were used in Phase II. The prime objective was to determine the effect of soil type on the ratio of split-tensile to unconfined compressive strengths. The soils were stabilized with the amount of lime required to produce maximum 28-day compressive strengths (73 F curing temperature and sealed container curing). These optimum lime requirements were established by the author in a previous investigation. The type of lime used (A, B, C) and the curing period (7, 15, 20, 30, 50 days at 120 F) were assigned to the soils in a random fashion. Lime treatments, curing periods, and test results are shown in Table 5.

TABLE 4

PHASE I TEST RESULTS
(Ottawa A-6 Soil)

Lime		Curing (days)	q_u (psi)[a]	Split[a] Strength (psi)	Split/q_u
Type	%				
A	3	1	173	20.8	0.12
		5	270	40.3	0.15
		10	434	62.3	0.14
		20	559	72.7	0.13
		30	410	58.0	0.14
		40	557	67.3	0.12
		50	703	90.1	0.13
		75	1033	157.0	0.15
A	5	1	201	28.2	0.14
		5	231	26.0	0.11
		10	527	75.4	0.14
		20	620	69.3	0.11
		40	790	85.4	0.11
		50	1298	184.0	0.14
		75	959	143.0	0.15
B	3	1	199	25.1	0.13
		5	349	47.2	0.14
		11	494	60.1	0.12
		20	615	65.4	0.11
		30	763	96.3	0.13
		40	778	87.3	0.11
		50	764	80.4	0.11
		75	897	107.0	0.12
B	5	1	196	19.4	0.10
		5	362	42.2	0.12
		10	371	46.5	0.13
		15	529	70.7	0.13
		20	823	124.0	0.15
		31	1136	160.0	0.14
		40	1236	183.0	0.15
		75	1474	178.0	0.12
C	3	3	323	35.4	0.11
		7	449	51.6	0.11
		20	757	108.0	0.14
		31	815	115.0	0.14
		53	875	129.0	0.15
		75	1217	157.0	0.13
C	5	3	310	40.9	0.13
		7	497	60.7	0.12
		20	646	96.9	0.15
		30	712	99.6	0.14
		40	857	99.5	0.12
		50	862	107.0	0.12
		75	1582	200.0	0.13

[a] Average of eight specimens.

PRESENTATION AND ANALYSIS OF TEST RESULTS

Phase I

Examination of the test results from Phase I (Table 4) shows that lime-soil mixtures possess substantial tensile strength. The split-tensile strengths range from approximately 20 to 200 psi. Figure 8 illustrates the typical variation in compressive strength, split-tensile strength, and the ratio of split-tensile strength to compressive strength (S_T/q_u) for different curing periods. Longer curing periods generally increased compressive strength and split-tensile strength, regardless of lime type and percentage, but the S_T/q_u ratio varied only from 0.10 to 0.15. It is apparent that split-tensile

TABLE 5

PHASE II TEST RESULTS

Soil	Lime		Curing (days)	q_u (psi)[a]	Split Strength (psi)[a]	Split/q_u
	Type	%				
1	B	7	15	1004	133	0.13
2	A	7	66	739	92	0.13
3	A	5	50	1618	207	0.13
4	A	5	20	640	97	0.15
5	B	7	15	608	85	0.14
6	C	3	30	798	119	0.15
7	B	5	31	408	59	0.15
8	C	5	7	806	103	0.13
9	A	3	20	863	104	0.12
10	C	3	50	1051	154	0.15
11	B	3	40	778	87	0.11

[a] Average of eight specimens.

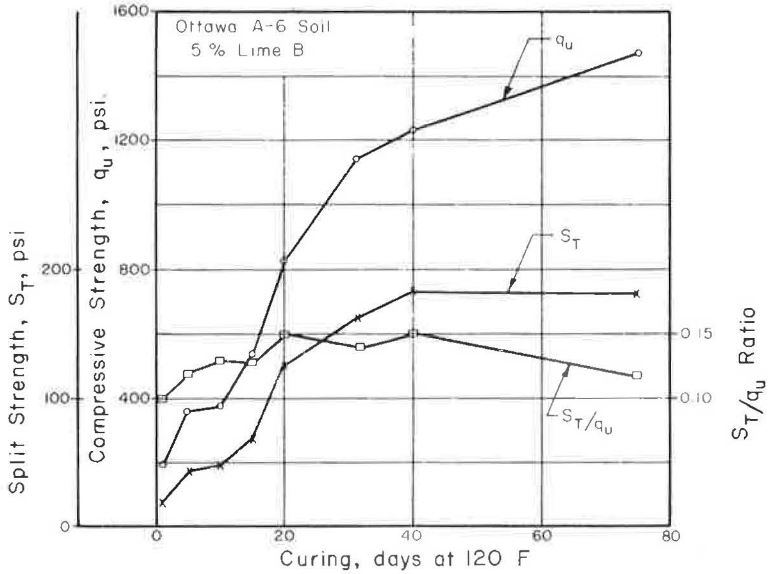

Figure 8. Curing effects.

and compressive strength vary in a similar manner and that the ratio between them exhibits little variation.

In Figure 9, the S_T/q_u ratio for selected lime treatments is plotted as a function of q_u to determine if this ratio is constant over a wide range of compressive strengths. Linear regression analyses (Table 6) show that for every lime treatment the S_T/q_u ratio is not a function of compressive strength (slope of regression line is not significantly different from 0, $\alpha = 0.05$). Since the ratio is not related to compressive strength, the best estimate of the S_T/q_u ratio for a given lime type and percentage is the average for all curing periods.

Analysis of variance test results (Table 7) also indicate that the S_T/q_u ratios for various lime types and percentages are not significantly different ($\alpha = 0.05$).

In summary, the S_T/q_u ratio for this soil (Ottawa A-6) is not influenced by compressive strength (curing period), lime type, or lime percentage. Factors which increase or decrease the compressive strength of lime-soil mixtures influence split-tensile strength in a similar manner.

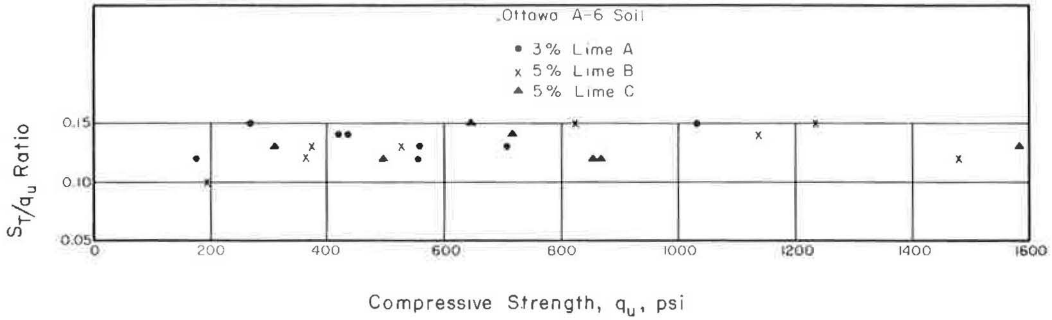

Figure 9. S_T/q_u ratio vs compressive strength.

TABLE 6

LINEAR REGRESSION ANALYSIS
(Ottawa A-6 Soil)

Description: This test was conducted to determine if the S_T/q_u ratio is linearly related to the magnitude of the compressive strength, q_u. Varying compressive strengths were obtained by altering the length of the curing period.

Hypothesis: $\beta = 0$, or the slope of the regression line between S_T/q_u and q_u is not significantly different from 0.

Lime		F Calculated
Type	%	
A	3	0.003
A	5	0.56
B	3	2.99
B	5	2.06
C	3	3.2
C	5	0.03

NOTE: None of the calculated F values were significant ($\alpha = 0.05$); therefore, the hypothesis is accepted in every case.

TABLE 7

ANALYSIS OF VARIANCE TEST RESULTS
(Ottawa A-6 Soil)

Description: This test was conducted to determine if there is a significant difference in the S_T/q_u ratios for different lime types and percentages. The S_T/q_u ratio shown below is the average of all test results for a given lime type and percentage, regardless of curing period.

Hypothesis: S_T/q_u ratios are equal for the various lime types and percentages.

Lime		Avg. S_T/q_u
Type	%	
A	3	0.135
A	5	0.129
B	3	0.121
B	5	0.130
C	3	0.130
C	5	0.130

Calculated F value = 0.05.
F value not significant at $\alpha = 0.05$; therefore, hypothesis is accepted.

Phase II

Since the results of Phase I indicate that lime type and curing period do not influence the S_T/q_u ratio, the random assignment of lime type and curing period for the 11 soils included in Phase II is justified. The results from Phase II of the test program (Table 5) show a wide variation in compressive and split-tensile strengths, depending on soil type, lime percentage and curing period. It is evident, though, that the S_T/q_u ratio remains essentially constant. Analysis of variance test results (Table 8) indicate a significant difference ($\alpha = 0.05$) between the S_T/q_u ratio for the individual soil types. Although significant differences exist between ratios for some of the soils, the range is only from 0.113 to 0.155. For practical purposes, an overall average of approximately 0.13 for the S_T/q_u ratio may be appropriate.

GENERAL DISCUSSION

It is evident that cured lime-soil mixtures possess substantial tensile strength, but, like other typically brittle materials, it is small in comparison to its compressive strength. The average S_T/q_u ratio of 0.13 compares favorably with results reported for concrete (5, 12) but appears to be slightly higher. Though the split-tensile strength is low, it is emphasized that the modulus of rupture (flexural strength) for other materials reported in the literature is from approximately 1.5 to 3.0 times larger than its split-tensile strength (8). With a correlation factor of this magnitude, the tensile strength properties of lime-soil mixtures are comparable with the flexural strengths of other stabilized highway materials with approximately the same compressive strength.

TABLE 8

ANALYSIS OF VARIANCE TEST RESULTS, INFLUENCE OF SOIL TYPE

Description: This test was conducted to determine if there is a significant difference in the S_T/q_u ratios for different soil types. The S_T/q_u ratios shown are the averages of the ratios of the eight pairs of specimens (one pair is a split specimen and an unconfined specimen) for each lime-soil mixture.

Hypothesis: The S_T/q_u ratios are equal for the different soil types.

Soil	Lime		Curing at 120 F (days)	Avg. S_T/q_u
	Type	%		
Bryce B, Iroquois Co.	A	5	20	0.155
Cowden B, Randolph Co.	B	5	31	0.149
Illinoian till, Effingham Co.	C	3	50	0.147
Cowden B, Montgomery Co.	B	7	15	0.143
Accretion gley-2, Sangamon Co.	A	5	50	0.129
Illinoian B, Sangamon Co.	C	5	7	0.128
Illinoian till, Sangamon Co.	A	3	20	0.126
Accretion gley-1, Sangamon Co.	A	7	6	0.126
Cowden C, Montgomery Co.	C	3	30	0.116
Illinoian B, Effingham Co.	B	7	15	0.113
Ottawa A-6, LaSalle Co.	B	3	40	0.113

Calculated F value = 6.8.
F value is significant at $\alpha = 0.05$; therefore, hypothesis is rejected.

Results of Duncan's Multiple Range Test are shown above.

NOTE: Any two means not indicated by the same line are significantly different. Any two means indicated by the same line are not significantly different.

The tensile strength of a lime-soil mixture should greatly contribute to its performance as a highway construction material. Since the lime-stabilized material can withstand tensile stresses, a lime-soil pavement layer should behave differently than a flexible-type pavement. The work of Nichols (13, 14) appears to substantiate the foregoing statement. In his investigations, he attributed the reduced deflections of pavement sections with stabilized subgrades (lime or cement) to the "slab strength afforded by a semi-rigid subgrade." The improved performance noted for many pavement sections with lime-treated subgrades may also be partially attributable to the slab strength which can be developed if the material possesses substantial tensile strength.

CONCLUSIONS

Based on the results of this investigation, the following conclusions were drawn:

1. The split test has considerable merit as a test procedure for evaluating the tensile strength properties of lime-soil mixtures;
2. Lime-soil mixtures possess substantial tensile strength which is influenced by such factors as soil type, lime treatment and curing period;
3. The ratio of split-tensile strength to compressive strength (S_T/q_u) shows no statistically significant variation for a particular soil type;
4. Small but significant differences were observed in the S_T/q_u ratios for the different soil types; and
5. The best available estimate of the S_T/q_u ratio for lime-soil mixtures is an overall average of 0.13.

ACKNOWLEDGMENTS

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REFERENCES

1. Burmister, D.M. The Theory of Stresses and Displacements in Layered Systems and Applications to the Design of Airport Runways. Highway Research Board Proc., Vol. 23, pp. 126-148, 1943.
2. Carniero, F.L. L. B., and Barcellos, A. Concrete Tensile Strength. Int. Assoc. of Testing and Res. Lab. for Mater. and Struct., Bull. No. 13, March 1953.
3. Akazawa, T. Tension Test Method for Concrete. Int. Assoc. of Testing and Res. Lab. for Mater. and Struct. Bull. No. 16, Nov. 1953.
4. Frocht, M.M. Photoelasticity. Vol. 2. New York, John Wiley and Sons, 1957.
5. Mitchell, Neal B. The Indirect Tension Test for Concrete. Mater. Res. and Std., Oct. 1961.
6. Wright, P.J.F. Comments on Indirect Tensile Test on Concrete Cylinders. Mag. of Concr. Res., Vol. 7, No. 20, 1955.
7. Peltier, R. Theoretical Investigation of the Brazilian Test. Union of Testing and Res. Lab. for Mater. and Struct., No. 19, 1954.
8. Rudnick, A., Hunter, A.R., and Holden, F.C. An Analysis of the Diametral-Compression Test. Mater. Res. and Std., Vol. 3, No. 4, April 1963.
9. Rüsch, H., and Vigerust, G. Discussion of Tensile Splitting Test and High Strength Concrete Test Cylinders, by Sven Thaulow. ACI Jour., Part 2, Dec. 1957.
10. Bawa, N.S. The Analysis of Stress Distribution in Indirect Tensile Tests on Concrete Cylinders. Ind. Concr. Jour., Vol. 31, No. 3, April 1957.
11. Ramesh, C.K., and Chopra, S.K. Determination of Tensile Strength of Concrete and Mortar by the Split Test. Ind. Concr. Jour., Vol. 34, No. 9, 1960.
12. Grieb, W.E., and Werner, G. Comparison of the Splitting Tensile Strength of Concrete with Flexural and Compressive Strengths. Public Roads, Vol. 32, No. 5, Dec. 1962.
13. Nichols, F.P. Effects of Compaction and Subgrade Stabilization on Deflections and Performance of Virginia Pavements. Proc. Int. Conf. on Struct. Design of Asphalt Pavements, Ann Arbor, Mich., Aug. 1962.
14. Nichols, F.P., Jr. Deflections as an Indicator of Flexible Pavement Performance. Highway Research Record No. 13, pp. 46-65, 1963.

Discussion

DAVID L. TOWNSEND, Queen's University, Kingston, Ontario, Canada.—The author's application of the split-tensile strength test to the study of lime-stabilized soils represents an interesting and potentially very useful step towards a clearer understanding of the mechanical properties of these soils. There is some justification in considering that the tensile strength may be a better criterion to apply for failure conditions than the usual unconfined compression test, and it is possible that it may have some wider applications with regard to durability of the stabilized soils.

In connection with a study of the durability of lime-stabilized soils under freezing and thawing conditions, T.W. Klym, at the discussor's suggestion, conducted a preliminary study of the suitability of the split-tensile test as a measure of resistance to heaving. The results indicated as much lack of relationship as the unconfined compression test, and further testing was discontinued in favor of developing a better understanding of the mechanism by which lime-stabilized soils resist deterioriation under freezing and thawing conditions. However, in view of the author's results, it is pertinent to augment his reported information with these preliminary results.

The testing methods were as follows. Normal dolomitic hydrated lime was manually mixed with five soils and allowed to cure for 24 hr before being dynamically compacted to 100 percent standard proctor compaction in a 2.0-in. diameter by 3.0-in. long mold and cured in sealed containers at 70 F for various periods. The sample size was dictated by the durability test methods. The split-tensile specimens were tested at a rate of 0.05 in./min between loading strips with an a/d ratio of $^1/_8$; and the unconfined compression tests were conducted at 0.10 in./min in a standard soil testing compression unit. As this was a preliminary study, it was felt that the difference in testing rates would not be significant insofar as indicating if the tensile strength could be related to durability.

The results of the individual tests are given in Table 9. Clay mineral identification was performed by standard X-ray diffraction analysis, and other index tests follow the standard procedures. The actual unconfined compression test values are given, and these have been corrected by a factor of 0.96 to convert to a standard length-to-diameter ratio of 2:1. Although this conversion factor is based on concrete samples, it is felt to be reasonably valid for these weakly cemented soils. The amount of lime for each soil was based on the amount that would normally be used for these soils under highway construction conditions and is approximately 4 percent more than the amount required for fixation as indicated by plasticity tests. Unconfined compression and split-tensile strength tests were also performed immediately after sample preparation, but it is felt that these values are not pertinent to this discussion.

From the very limited results, it may be apparent that:

1. There is considerable variation in the observed values of split-tensile strength for any one curing period and soil-lime combination (in some cases, as high as 20 percent from a mean value); and
2. There is a variation in the ratio of S_T/q_u for the five soils tested which is similar to those reported by the author. However, in two cases, the ratio appears to be larger or smaller than the values reported.

There may be some small variation in the ratio of S_T/q_u when related to the activity of the soil, but there is far from sufficient evidence to establish any trend.

In many engineering feasibility studies, it is customary to use only two or three test samples for the measurement of physical properties. However, from these results, it may appear that there can be significant variation within the split-tensile strength test. If, in the future, designs are to be related to this strength, it would

TABLE 9

Soil No.	Classification	Clay Minerals	% Clay	I_p	Activity	% Lime	14-Days Curing				28-Days Curing				4-Months Curing			
							Act. q_u (psi)	Corr. q_u (psi)	Act. S_T (psi)	Ratio	Act. q_u (psi)	Corr. q_u (psi)	Act. S_T (psi)	Ratio	Act. q_u (psi)	Corr. q_u (psi)	Act. S_T (psi)	Ratio
S-2	CI	Mont.	22	25	1.1+	6	174	167	16.6		228	219	27.4		465	446	41.7	
							172	165	13.9		240	230	18.4		422	405	49.5	
							176	169	19.3		246	236	25.8		462	443	56.2	
								167	16.6	0.10		228	23.9	0.10		431	49.1	0.11
A-1	CH	Mont.	75	66	0.9	7	198	190	21.8		252	219	27.4		328	315	25.1	
							206	198	17.3		250	240	30.6		322	309	23.1	
							190	183	19.2		222	213	22.8		301	289	23.5	
								190	19.4	0.10		232	26.6	0.11		301	23.9	0.08
M-2	CH	Mont.	71	56	0.8	7	260	250	27.1		320	307	28.8		376	361	36.4	
							257	247	27.9		295	283	33.9		369	354	41.9	
							265	254	29.4		296	284	30.7					
								250	28.1	0.11		291	31.1	0.11		358	39.2	0.11
Q-1	CL	-	32	18	0.6	6	110	106	14.6		148	142	15.1		298	286	44.1	
							116	111	17.6		139	134	23.0		305	293	43.5	
							118	113	15.2		142	136	22.1		301	289	53.1	
								110	15.8	0.14		134	20.1	0.15		289	46.9	0.16
O-1	CI	Kaol.-ill.	44	21	0.5	6	127	122	13.6		165	158	18.7		378	363	51.5	
							113	108	13.0		150	144	23.1		370	355	38.6	
							110	106	12.7		142	136	26.1		377	362	48.2	
								112	13.1	0.12		146	22.6	0.15		360	46.1	0.13

appear that the variation within the test itself should be known, so that extrapolations to erroneous values are not made. It would be appreciated if the author could indicate the typical variations which developed within the eight samples of a series used in the various parts of his study.

MARSHALL R. THOMPSON, *Closure.*—Mr. Townsend's comments are appreciated. The values he reports for the ratio of split-tensile strength to compressive strength are of the same general magnitude as those presented in the paper.

In reply to Mr. Townsend's question, the coefficient of variation, $\frac{\sigma}{\bar{x}}$, obtained by using eight samples in a series was approximately 10 percent for both the split-tensile specimens and the unconfined compression specimens.

Mechanisms of Soil-Lime Stabilization

An Interpretive Review

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Literature on the mechanisms of the physicochemical and chemcal reactions responsible for soil-lime stabilization is critically reviewed. The importance of cation exchange, flocculation, and carbonation in the stabilization processes is discounted. The rapid "ameliorating" effects of lime on the properties of plastic soils are documented from the literature, and the distinction between these short-time effects and the long-time cementitious reactions responsible for the development of strength is noted.

The cation crowding hypothesis is rejected as an explanation of the ameliorating effects. New results summarized show that the cation crowding is more accurately described as physical adsorption of calcium hydroxide on the clay surfaces. The ameliorating effects may be due to very rapid cementation of the primary clay particles at points of contact within the flocs formed on the addition of lime. This rapid cementation is attributed to immediate formation of tetracalcium aluminate hydrate by reaction of $Al(OH)_x$ groups at the edges of the clay particles with adsorbed calcium hydroxide on the faces of adjacent planar surfaces. This may be supplemented by somewhat slower formation of tobermorite gel at the same reaction sites.

The nature and properties of the reaction products responsible for slow development of strength in soil-lime systems are discussed. In general, the products formed are usually tobermorite gel and a calcium aluminate hydrate phase distinguished by a 7.6 Å basal spacing that is stable to moderate drying; other compounds formed under special circumstances include tricalcium aluminate hexahydrate and calcium silicate hydrate (I). Phases involving both silica and alumina in distinguishable lattice positions are not usually formed. [illegible]

The scant information in the literature on the chemical mechanisms involved in the lime-clay mineral transformations is reviewed. Available literature would tend to suggest a "through-solution" mechanism dependent on the rate of dissolution of the clay (or of silica and alumina from the clay), since it is implied that the reaction depends on attainment of a high pH and since addition of sodium hydroxide accelerates strength development. On the other hand, evidence that the reaction proceeds from an initial state of physical adsorption of lime on the surface of the clay would suggest a surface chemical reaction not dependent on prior dissolution of material from the clay.

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• SOIL STABILIZATION by the addition of quicklime or hydrated lime is an ancient art, successfully adapted in modern times by perceptive engineers of the Texas Highway Department and now widely practiced. It is not, however, the purpose of this report to discuss either the history or the engineering aspects of soil-lime stabilization as currently practiced, but to attempt to bring together and critically evaluate current knowledge of the mechanism or mechanisms responsible for the stabilization process. Once these are well understood, it should be possible to develop practical stabilization procedures on a more rational basis, so as to take full advantage of this useful and cheap chemical agent.

The responses of soil to treatment with lime are complex and often dramatic. A number of explanations have been proposed to account for these unusual responses, including:

1. Cation exchange, i.e., replacement of the exchangeable sodium, magnesium, or other cations previously held by the soil clay by calcium cations derived from the lime;
2. Flocculation of the clay, and consequent increase in effective grain size;
3. Carbonation, i.e., reaction of the lime with carbon dioxide from the atmosphere to form calcium carbonate, which has been said to exert cementing action; and
4. So-called pozzolanic reactions with soil constituents to generate new minerals of a cementitious nature.

Of these phenomena, the first three may occur but are inadequate explanations, as we shall attempt to show. The latter explanation is at least partially correct, and in this paper a more specific picture of the nature of these reactions is presented.

The factor of cation exchange has been mentioned by many authors, yet familiarity with the cation exchange properties of soils should have eliminated this as a serious explanation for the stabilizing effects of lime on soil. As soil scientists know, many natural soils are largely calcium saturated. For example, recent work shows that the montmorillonitic soils of the southwestern United States (Grumosols in soil science classification systems), are normally two-thirds to three-quarters calcium saturated (1). Despite this predominance of calcium in the exchange complex, these soils exhibit all the classic deficiencies associated with montmorillonite soils when used as subgrades and, thus, require stabilization. Although it may be suggested that complete, rather than predominant, calcium saturation is required for stabilization, it has been demonstrated that even when lime is added in very large excess to dilute clay suspensions, under conditions where cation movement is facilitated, complete exchange for sodium does not take place (2). Exchange in comparatively dry compacted soils is undoubtedly less complete.

The concept that flocculation plays a major part in soil-lime stabilization is often voiced, but careful examination of previously known facts shows that this also is an inadequate explanation. Some soils, including most of the red and yellow soils of the southeastern part of the United States, are flocculated in nature, as can be seen by shaking the soil in water and examining the resulting flocs. Despite this natural flocculation these soils are not stable, and they do respond to lime treatment. Furthermore, it is well known that many chemical agents, including various salts, alcohols, acids, and ketones, induce immediate flocculation when mixed with clays, yet are valueless for stabilization. The fact that flocculation of clay occurs as a consequence of the addition of lime is a well-known phenomenon, but the achievement of flocculation is clearly not the mechanism by which lime stabilizes soils.

The hypothesis that soil-lime stabilization depends on the carbonation of the lime to form calcium carbonate can be dismissed by reference to any of a great number of studies in which reaction with atmospheric carbon dioxide was precluded by sealing the samples and in which the characteristic modification of properties and development of strength associated with lime stabilization were observed. As demonstrated by Eades, Nichols, and Grim (3), carbonation does take place in the field, but the strength gain said to accrue by virtue of cementation of soil grains by calcium carbonate has yet to be conclusively demonstrated. It appears that the additional benefit by long-term reaction of uncarbonated lime with the soil itself would far outweigh any such contribution, and carbonation is probably a deleterious rather than a helpful phenomenon in soil stabilization.

Having rejected these inadequate hypotheses, we may now turn our attention to the more nearly adequate, but still incomplete, explanations of the mechanisms of lime-soil interaction that constitute the subject of this paper. Experience indicates that there are at least two distinct stages of reaction involved: (a) the immediate or rapid processes responsible for the "amelioration" of the water-sensitive properties of untreated clay soil, and (b) the slower, long-term reactions resulting in formation of the final cementitious products that are indicated by the gradual development of strength in compacted soil-lime mixtures. These will be discussed separately.

RAPID AMELIORATION EFFECTS

Atterberg Limits

It has been shown by many workers (4, 5, 6) that addition of lime to plastic soils results in reduction of the plasticity index (P. I.). Usually an immediate increase in the plastic limit (P. L.) is observed on the addition of lime. The amount of this increase varies directly with the amount of lime added, up to some limiting lime content; further increments of lime usually bring little or no additional increase. The point of inflection of the plot of lime added vs P. L. has been dignified by the appellation "lime fixation point" (7). The implications involved in this term will be discussed later.

The effect on the liquid limit (L. L.) is less easy to summarize, since the published data conflict. Some authors report decreases in L. L. on addition of lime (4, 5), but others report that this parameter may increase substantially (8, 9). Still other workers (6, 10, 11) report that both increases and decreases occur, depending on the individual soil being tested.

These discrepancies require some explanation. It should be pointed out that the L. L. of a clay is far more sensitive to the kind of cation present than is the P. L. Calcium-saturated clays have substantially lower L. L. than the same clays saturated with sodium or certain other cations (12). Since some (although not complete) cation exchange occurs on addition of lime, the effect of the lime may vary, depending on the extent to which preexisting cations other than calcium are exchanged. Studies on clays of unknown original cation status are consequently difficult to interpret.

Clare and Cruchley (8) give data showing a drastic increase in L. L. with added lime for a particular clay which was almost completely calcium saturated in its natural state. They also state that these data are typical for the other clays tested as well; the other clays varied in original calcium saturation from 35 percent upward. We, as well as many others, have observed that mixtures of lime and calcium-saturated clay, stored so as to prevent evaporation or carbonation, assume an increasingly dry appearance; in extreme cases, a free-flowing slurry mixture may stiffen so that it will no longer pour or take the shape of the container. This behavior would, of course, be reflected in an increase in L. L. It seems that, in general, the L. L. will increase on the addition of lime in the absence of a strong specific tendency in the opposite direction resulting from cation exchange effects.

When the L. L. increases on the addition of lime, the increase is not usually as great as the accompanying increase in P. L. Thus, the separate effects on L. L. and P. L. usually combine to yield a rather sharp decrease in P. I. Extra increments of lime beyond some fairly small percentage do not usually result in significant further decreases; in fact, sometimes the trend is reversed and incremental increases in P. I. are found. It should be kept in mind that the P. I. is a composite parameter that may not accurately reflect real changes in the system; for example, equal increases in L. L. and P. L. following a given treatment would be reflected as "no change" in the P. I.

Aging of the system generally results in further changes. Sometimes this effect is small. Lund and Ramsey (6), for example, found that very little change occurred in the Atterberg limit values of soil-lime mixtures after the first hour. However, often substantial time-dependent changes ensue. For example, Wolfe and Allen (13) report substantial increases in P. I. for a number of lime-soil mixtures when cured for 2 days, as compared with samples tested immediately after the addition of lime. For more extended curing periods (7 to 28 days) the effect was reversed, and significant decreases in P. I. in most cases to a nonplastic condition, were recorded.

Grain-Size Distribution

Changes in the effective grain size distribution (mechanical analysis) are observed almost immediately following the addition of lime to a clay soil. Data secured by conventional sieve and hydrometer analysis show that a major decrease in the content of clay-size particles occurs within the first hour; subsequent results after periods ranging up to 240 days show only small additional changes (6). The new "grains" produced as a result of the lime treatment are mostly sand sized and, although relatively weakly bonded, are for the most part capable of withstanding a 5-min dispersion period in a mechanical mixer (6, 13).

Moisture-Density Relations

It has been reported by many workers that the density to which a soil can be compacted at a given moisture content is usually reduced significantly on addition of lime, and delay in compaction results in further reduction. The moisture content needed to achieve maximum density for a given compactive effort usually increases, sometimes rather significantly. However, according to results summarized by Herrin and Mitchell (14), lime in excess of a relatively limited amount, on the order of 5 percent by weight of soil, generally produces little additional increase in the optimum moisture requirement.

Volume Change Effects

Lund and Ramsey (6) report a drastic reduction in the volume change. This effect was observed within the first hour after the addition of lime; with the soil tested, about 3 percent lime sufficed to give the maximum effect. A corresponding immediate increase was noted on the shrinkage limit itself. Similar results were reported by Wolfe and Allen (13).

Mitchell and Hooper (15) reported that lime (in this case, dolomitic lime) markedly reduced the swelling of specimens tested under a modest surcharge pressure. The effect was observed after an aging period of 24 hr. A reduction in expansion pressure on addition of lime was observed by Wolfe and Allen for some soils, but not for others (13).

Soil Suction

Clare and Cruchley (8) report significant increases in the amount of moisture held against a given suction following the addition of lime.

Permeability

Although few data are available, the permeability of compacted soil-lime mixtures has been reported to be much less than that of compacted soil alone (13).

These various short-term effects can be regarded as different aspects of the amelioration effect of lime on the properties of clay soils. We will attempt to define exactly what chemical or physico-chemical action is reflected in these sometimes drastic changes.

Considerable evidence seems to point to the conclusion that these rapid responses are not the result of the pozzolanic reactions to which permanent strength gains are attributed. It has been well documented that the development of strength of specimens, particularly after soaking, is a fair index of the amount of cementitious compound formed (16). The experience of many workers, as summarized by Herrin and Mitchell (14), and the results documented by Anday (17) indicate that strength development in soil-lime systems is a comparatively slow process characteristically requiring periods of several weeks to many months at normal temperatures.

As pointed out by Mitchell and Hooper (15), if significant amounts of lime were consumed by permanent cementing reactions at early ages, delay between mixing and compaction should adversely affect the strength of the final product. They found no such effect for delays up to a day, provided all samples were compacted to the same density.

Similarly, if significant permanent cementing were to occur at early ages, subsequent remolding of the specimen should bring about a strong reduction in the ultimate strength developed. Unpublished data by Shepard (18) indicate that remolding after as long as a week of curing following initial compaction has no deleterious effect on the ultimate strength of soil-lime mixtures. In contrast, Shepard found that soil-cement mixtures, in which cementation occurs rapidly on hydration of the cement, did suffer severe reductions in ultimate strength on remolding after a week of curing.

Thus, extensive development of pozzolanic reaction products seems to be ruled out as a mechanism responsible for the rapid amelioration of the properties of plastic soils by lime. What then is the nature of the mechanism responsible for these effects?

Extensive investigations undertaken by workers at Iowa State University (7, 19-23) have led to the previously mentioned concept of the lime fixation point. This was defined as that percentage of lime beyond which additional increments of lime produced no appreciable increase in P. L. It was hypothesized that excess calcium cations, derived from the lime, in some fashion "crowd onto" the clay particles and cause them to become "electrically attracted," the process resulting in flocculation with weak bonds between the flocs. Additional lime, which produces calcium cations in excess of those which could crowd onto the clay, produces no further change in the P. L. Calcium held by the clay in amounts up to the lime fixation point was considered to be immune to further reaction with the clay to form cementitious compounds. The sedimentation velocity of flocs formed by adding lime to a clay suspension reached a maximum near the lime fixation point, indicating that floc size was at a maximum at this point (21).

Ho and Handy (22) cited evidence that calcium was retained by bentonite from lime-bentonite slurries after a very limited washing treatment. The amount of calcium retained increased with increasing percentage of lime in the slurry, and for modest percentages of lime was far more than the normal cation exchange capacity (C. E. C.) of the clay. These writers hypothesized that the calcium in excess of the normal C. E. C. was held at new exchange sites generated at the edges of clay particles by increasing dissociation of acidic -Si(OH)$_x$ groups with increasing pH; hence, the term pH-dependent exchange. However, it was noted (without explanation) that the amount of calcium retained continued to increase steadily with increments of lime far beyond the lime fixation point and no break in the curve was evident.

Ho and Handy also studied several additional features of the lime-bentonite system. Small additions of lime (up to about 2.2 percent) were found to increase the relative viscosity of calcium bentonite slurries; however, aging of the samples continually increased the lime content at which maximum relative viscosity was attained. These authors also made differential thermal analyses (DTA) of dried lime-bentonite slurries, and disclosed that samples to which only a small amount of lime had been added failed to generate the normal endothermic response characteristic of the lime.

In another paper, Ho and Handy (23) reported the results of measurements of the zeta potential (a quantity related to the net negative or positive charge carried by the clay particles in dilute suspension) of lime-treated bentonite aged either for several days ("fresh") or for a year ("aged"). Calcium-saturated bentonite had a zeta potential of about -20 millivolts. Small additions of lime caused a slight increase in this value and larger additions generated a modest decrease for both fresh and aged samples. Sodium-saturated bentonite had a zeta potential of about -40 millivolts; additions of lime to this clay up to about 6 percent caused strong proportional reductions to -26 millivolts for the fresh samples and -22 millivolts for the aged samples, and higher additions of lime caused no further change. Ho and Handy presented a detailed analysis of their results which we consider highly speculative. It is sufficient to note at this point that the large effect found for the sodium bentonite was apparently due to the ordinary process of cation exchange of calcium for sodium, and in accord with the earlier results of Prikryl and Esterka (2) this process was not quite complete.

We were far from satisfied that the interpretations of the early ameliorative effects of lime on clay suggested by the Iowa State University workers could be correct in detail. The apparently unlimited crowding of positively charged cations onto the limited external surface of the montmorillonite would appear to be an unlikely phenomenon; if it were to occur, one would expect that a large decrease and eventual reversal of the negative charge of the clay would take place, due to the limited number of -$Si(OH)_x$ groups on the edges of the clay particles that could dissociate to generate negative charges to balance the cation uptake.

We carried out a series of investigations in an attempt to shed more light on these matters. Details of these investigations will be submitted for publication elsewhere, but some of their results can be summarized here.

Measurements were carried out on Wyoming bentonite which had been saturated with calcium cations by standard laboratory procedures and checked for completeness of saturation. Small quantities of the calcium-saturated clay were shaken with a large amount of saturated calcium hydroxide solution for various periods at room temperature, and then separated by centrifugation. The supernatant solutions were recovered and promptly analyzed for calcium content (using versenate titration) and hydroxyl content (by titration against standard acid). The following results were obtained:

1. A very rapid reduction of the concentration of both calcium and hydroxyl ions in the solution was observed.
2. The proportion of the ions remaining in solution was stoichiometric, indicating that equivalent proportions of hydroxyl and calcium ions (two hydroxyl for each calcium) had been taken up by the clay.
3. About 3 percent of lime (by weight of the clay) was found to be adsorbed within the first 5 min, the minimum time in which the manipulations could be performed. This amount corresponds roughly to the lime fixation point of the Iowa State workers.
4. Further sorption continued at a declining rate as long as the experiments were continued (3 wk). At that time the total amount of lime removed from the solution amounted to about 20 percent of the weight of the clay.
5. Washing the centrifugally separated clay with an amount of distilled water equal to the volume of the saturated lime solution originally used resulted in essentially complete removal of the adsorbed lime, if carried out shortly after the onset of the experiment. However, the ratio of hydroxyl to calcium ions removed was slightly less than stoichiometric.
6. After a sample had been in contact with the lime solution for several days and had adsorbed much additional lime, a single wash as outlined removed only part of the lime; subsequent additional washings removed smaller and smaller amounts. The ratio of hydroxyl to calcium ions removed in successive washings declined from values fairly close to stoichiometric (about 1.7) to about 1.
7. In DTA of dried clay which had sorbed about 4 percent of lime from solution (in about an hour of contact), no lime was detected. A mechanical mixture of dry clay with the same amount of dry lime gives a readily detectable endothermic response at 500°, characteristic of crystalline lime.
8. No X-ray diffraction peaks for crystalline calcium hydroxide were observed for the clay recovered from lime solution as mentioned previously; the corresponding dry mixture produced easily detectable peaks under the same diffraction conditions.
9. Measurements of surface charge of the calcium bentonite were carried out on untreated clay and on clay recovered after immersion in saturated lime solution for an hour. The method used was that which Pike and Hubbard used for a similar determination of the surface charge of hydrating cement particles (24). No significant change in negative charge was found.
10. In other experiments, lime was added to suspensions of calcium-saturated bentonite of restricted water content. At lime contents of 4 percent or less by weight of clay, the electrical conductivity of the suspension was found to decrease to very low values in less than 24 hr. This was regarded as an indication that the lime was being rapidly adsorbed, with the consequent decrease in concentration of the lime electrolyte in the pore solution. Removal of some of the pore solution by high-speed centrifugation and subsequent chemical analysis confirmed this interpretation.

These data indicated that the response to lime in clay-water systems of restricted water content was similar to that occurring in dilute suspensions.

No evidence of heat production or removal was found to accompany the addition of lime to calcium bentonite suspensions of restricted water content, in tests carried out over several hours with a moderately sensitive calorimeter. This is a further indication that the lime sorption process is one of physical adsorption rather than chemical reaction.

These results are interpreted to yield a picture of the initial reactions somewhat different from that previously developed elsewhere.

1. Calcium hydroxide is physically adsorbed from solution at a very rapid rate by calcium-saturated clay, and presumably also by clay saturated with other cations. This adsorption is largely reversible at very early stages, but is soon followed by reaction to produce calcium silicate hydrates. The conclusion that lime is physically adsorbed on clay surfaces was also reached by Prikryl and Esterka (2).
2. The adsorption removes calcium ions and hydroxyl ions from solution concurrently, and does not reflect a crowding of cations only onto new exchange sites generated at high pH levels, as previously postulated.
3. Since both ions are sorbed in equivalent amounts (within the limits of the accuracy of the analysis), no significant change occurs in the net negative charge of the clay particles. This does not preclude small changes in the zeta potential after some period of chemical reaction.
4. The amount of calcium and hydroxyl ions sorbed immediately (about 3 percent by weight of clay) would correspond roughly to sorption of a little more than a monomolecular layer of calcium hydroxide on the external surfaces of the clay. This sorption corresponds roughly to the lime fixation point and suggested that the latter represents approximately a monolayer of lime on the external surfaces of the particular clay concerned.
5. The very large amount of slow sorption beyond this value is thought to reflect several additional processes, but principally the slow reaction of adsorbed lime with the clay surfaces to produce calcium silicate and calcium aluminate hydrates. We think that, at least in wet systems, these products spall from the clay surface, thus liberating fresh clay surface for further adsorption and reaction. Physical adsorption of lime onto the newly formed reaction products is also likely. Finally, a slow, restricted entry of lime into the interlayer spaces of the clay may take place.

The observation that adsorption of lime occurs in clay-lime systems does not in itself provide any particular indication of how the clay properties are so drastically changed within a short time after the addition of lime. The mechanics of this process requires further elucidation.

Strong flocculation is commonly observed when lime is added to clay. In general, according to current colloid-chemical concepts (25), clays flocculate on addition of electrolytes because of the modifying effect of the electrolyte on the extension of the electrical double layer from the surfaces of the clay particles. The electrolyte represses the double layer, thus reducing the electrostatic repulsive forces between clay particles. This results in a net attraction, especially between negatively charged faces and positively charged edges of adjacent particles, and a "card-house" or "double-T" structure develops. The flocculated clay is still sensitive to water and, generally speaking, removal of the electrolyte from the pore solution results in a process whereby the double layer again spreads out, the repulsive forces between particles increase, the flocs weaken and are reduced in size, and eventually the system deflocculates.

In our experiments it was noted that removal of lime from the pore solution results in only low concentrations of electrolyte being present after a few hours; yet the initially formed flocs persist and become, if anything, more pronounced and stable with time. Obviously, the properties of the clay are altered more profoundly by lime than by nonreactive electrolytes such as salts.

One of the authors has demonstrated (26) that lime reacts almost instantaneously with hydrous alumina of high surface area to generate a well-crystallized compound, tetracalcium aluminate hydrate, C_4AH_{13} (in the shorthand notation commonly used for these compounds, $C = CaO$, $A = Al_2O_3$, $H = H_2O$, $S = SiO_2$, etc.). This compound is also probably formed on the hydration of C_3A in portland cement (27). We postulate that in lime-clay systems a similar immediate reaction takes place between the alumina-bearing edges of the clay particles and the lime adsorbed on the clay surfaces. In particular, we postulate that this reaction occurs at the points of contact between the edges of one particle and the faces of adjacent particles in the card-house structure of the flocs. Calcium silicate hydrate (tobermorite gel) probably forms at these points of contact also, but more slowly, perhaps over a period of some hours. Formation of very small quantities of these cementing products at the points of contact is thought to be sufficient to stabilize the flocs and knit the particles together so that plasticity, shrinkage and swelling, and other normal clay-water interactions are distinctly inhibited. Thus, the clay becomes ameliorated but not really stabilized, since the flocs have not been bound together well enough for the mass to retain any significant strength after soaking. However, the individual particles are cemented together well enough within the flocs to resist dispersion, and the flocs may act as single grains in mechanical analysis (6).

To develop a material capable of holding together and resisting applied loads after soaking, compaction is required to obtain a minimum void volume, and time must be allowed to permit the slow continuing chemical reaction to develop additional cementing products sufficient to fill the voids at least partially.

As shown by Jambor (16), the strength developed in lime-pozzolan systems depends to some extent on the kind of cementing agent formed, but also in great degree on the proportion of void space occupied by the cementing agent—the "gel/space ratio" familiar in portland cement technology.

The formation of calcium aluminate and calcium silicate hydrates at particle contacts very early in the reaction process does not lend itself to easy demonstration, due to the very small amounts of these products involved and the nature of the system. The concept is offered as a working hypothesis consistent with the known properties of the system. Further work is under way to provide additional information in an effort to prove or disprove this hypothesis.

PRODUCTS OF LONG-TERM REACTION BETWEEN LIME AND CLAY

The reactions occuring over a long period of time and, in the last analysis, responsible for the stabilization of the soil are of at least as much concern as the ameliorating responses just described. Most soils consist largely of uncombined silica and of silicates of various kinds; alumino-silicates usually predominate in the clay fraction. Consequently, it is not surprising that reaction with lime produces compounds falling largely into two classes: hydrated calcium silicates and hydrated calcium aluminates.

Except when formed under hydrothermal conditions, the calcium silicate hydrates are invariably poorly crystallized and difficult to detect. Three such phases are known in the literature: (a) tobermorite gel, also called C-S-H (gel); (b) C-S-H (I); and (c) C-S-H (II). The gel is a high-calcium phase normally generated in, and responsible for, the strength of portland cement concrete. C-S-H (I) has a distinctly lower calcium content, a somewhat different morphology, and is distinguishable by a very strong exothermic peak at about 850 C on DTA. Although C-S-H (I) may be synthesized in a fairly well-crystallized form in the laboratory, the gel phase is invariably more poorly crystallized and has a maximum of only three peaks on X-ray diffraction. C-S-H (II) is a high-calcium phase whose exact properties are open to some doubt. All of these compounds are presumed to have a layer structure similar but not necessarily identical to that of the well-crystallized mineral tobermorite. The latter may be synthesized readily by hydrothermal means, but its occurrence in soil-lime reaction products produced under normal atmospheric conditions is extremely unlikely.

There are several types of calcium aluminate hydrates. The one commonly formed at normal temperatures is a member of the tetracalcium aluminate hydrate group. This group comprises a number of crystalline modifications and partially dehydrated states, as well as phases of the same basic structure but incorporating carbonate groups in place of some of the hydroxyls. The form generally produced by clay-lime reactions is most similar to a phase with the composition $C_4AH_{12} \cdot CO_2$ and has a basal spacing of 7.6 Å which remains unchanged on drying. In contrast, pure C_4AH_{13} has a basal spacing of 7.9 Å when moist, and the spacing diminishes stepwise to about 7.4 Å or less on drying. It is possible that the soil-lime product may incorporate some silicon in isomorphous substitution for aluminum.

Reactions of lime with clay minerals at slightly elevated temperatures generally give rise to a quite different calcium aluminate hydrate, C_3AH_6. This is a cubic phase with a crystal structure different from that of the tetracalcium aluminate hydrates; it forms preferentially at temperatures above about 30 C and once formed is stable at room temperature. A good recent monograph covering the status of knowledge of both calcium silicate and calcium aluminate hydrates is that edited by Taylor (28).

Goldberg and Klein (29) carried out the first published X-ray study of lime-clay reaction products, but succeeded in detecting only calcium carbonate, which was probably produced during air drying of the sample prior to X-raying. Eades and Grim (30) reacted lime with pure clay minerals at 60 C and found that new minerals formed. Kaolinite so treated yielded peaks for a poorly crystallized calcium silicate hydrate of unspecified type, and a peak at 5.1 Å (and others) which we interpret as being due to C_3AH_6. Eades and Grim could not detect any new crystalline product of lime-montmorillonite reaction, although it was obvious that reaction had occurred. Illite reacted somewhat, but no positive statement was made as to the nature of the reaction product or products.

Hilt and Davidson (31) examined the product of a long-time reaction of lime and montmorillonite at room temperature, and finally identified it as a C_4AH_{13} type material. Its properties were consistent with the previously mentioned product having a 7.6 Å spacing. The published X-ray diffractometer traces indicated weak peaks attributable to poorly crystallized calcium silicate hydrates as well. Glenn and Handy (32) studied the products of reaction between several clay minerals and various forms of lime at room temperature, but their results were not completely interpreted. They found in general that poorly crystallized calcium silicate hydrates and the 7.6 Å calcium aluminate hydrate were found in the kaolinite systems; in addition, an unknown 12.6 Å compound was formed when dolomitic lime was used. Montmorillonite yielded both the 7.6 Å calcium aluminate hydrate and materials whose diffraction peaks were ascribed to a more nearly pure C_4AH_{13} phase.

Glenn and Handy observed no reaction of lime with quartz, and little reaction with muscovite or vermiculite. In contrast, in studies of the products formed in field soil stabilization projects, Eades, Nichols, and Grim (3) found that considerable reaction had occurred in quartz- and mica-bearing soils. When examined under the petrographic microscope, the quartz and mica particles were found to have developed fuzzy outlines, and visible cementing gel was present not only as a coating on the grains but also in cracks within the grains. Examination by X-ray methods disclosed the presence of calcium silicate hydrates as well as of calcium carbonate. The authors did not refer to the possibility that calcium aluminate hydrates might also have formed.

Diamond, White, and Dolch (33) reported that kaolinite and montmorillonite reacted with lime to produce different products under different reaction conditions. At 60 C, both clays produced calcium silicate hydrate classified as C-S-H (I) and the kaolinite produced the cubic C_3AH_6; no crystalline calcium aluminate compound was formed from montmorillonite. At lower temperatures the products from both clays were considered to be tobermorite gel and the 7.6 Å calcium aluminate hydrate. Small peaks at about 9 Å were found in the montmorillonite products under both conditions; but they disappeared on washing, and no explanation for their presence could be offered. These authors found that both the tobermorite gel and the 7.6 Å calcium aluminate hydrate product were formed by reaction of lime with mica, illite, and even pyrophyllite. Reaction with quartz generated tobermorite gel, even at 60 C.

Glenn and Handy (34) recently reported further results on long-term studies of reaction products of lime and montmorillonite at room temperature. In general, the formation of tobermorite gel, C-S-H (I), C_4AH_{13}, and possibly C-S-H (II) were reported. Interestingly enough, both fresh mixtures of clay and lime and mixtures which had been allowed to react at room temperature for several years yielded only well-crystallized tobermorite on hydrothermal treatment. The aluminum present in the product was thought to be incorporated as an isomorphous replacement for silicon within the tobermorite lattice.

It is noteworthy that in all the studies discussed so far, no quaternary compounds (that is, compounds in the lime-silica-alumina-water system) were reported. Apparently, where separate calcium aluminate phases were not developed, the aluminum present was incorporated isomorphously in the calcium silicate phase. In contrast to these results, Benton (35) reported that reactions of lime with calcined kaolinite yielded the quaternary compound gehlenite hydrate, C_2ASx, also called Stratling's compound. This compound was formed in addition to the more usual products, a poorly crystallized calcium silicate hydrate and the 7.6 Å calcium aluminate hydrate.

In a Japanese study (36) the same gehlenite hydrate and the 7.6 Å calcium aluminate hydrate were reported to result from reactions at normal temperatures between lime and Kanto loam soils, which consist mostly of the amorphous clay mineral allophane. At higher temperatures the same soils produced C_3AH_6 and a hydrogarnet quaternary phase instead of these compounds. When gypsum was also included in the treatment of these soils with lime, excellent stabilization results were obtained. In this case, ettringite or so-called cement bacillus, a sulfate-bearing phase formed in hydrating cements, was the major cementing agent formed.

MECHANISM OF DEVELOPMENT OF CEMENTATION PRODUCTS

The early physico-chemical reactions of lime that produce the amelioration effects on soil clays and the nature of the cementing compounds that produce the final cemented product have been discussed, but little has been said about the mechanics of the chemical and structural transformations that generate the final compounds. Very little is known about the details of the reaction processes involved, and most information is speculative in nature. This lack of knowledge is one of the major gaps in our understanding of the soil-lime system.

Eades and Grim (30) have suggested that with kaolinite, "the reaction seems to take place by lime eating into the kaolinite particles around the edges with a new phase forming around a core of kaolinite." Electron micrographs by Diamond, White and Dolch (33) tend to confirm this idea, in part, since the edges of residual kaolinite particles were ragged and irregular, as if they had been attacked chemically. The probability that partial exfoliation had taken place was also raised by the latter authors, who called attention to the fact that since two distinct crystalline hydration products were produced, a simple topotactic solid-state mechanism was not a reasonable explanation.

Recently Sloan (37) reported an electron-microscope study of the effects of treating kaolinite with sodium hydroxide and with lime in dilute suspensions. He confirmed that under such conditions the primary attack on the kaolinite particles took place at the particle edges. With lime he observed what appeared to be nucleation of a reaction product at or near the edges of the kaolinite particles; however, this product did not appear to be one previously reported to occur in lime-clay reactions, and the exact significance of this observation is uncertain.

With illite and montmorillonite, Eades and Grim concluded: "Following the saturation of the interlayer positions with calcium ions the whole clay mineral structure deteriorates without the formation of substantial new crystalline phases." In contrast, Diamond, White and Dolch reported that new crystalline phases (tobermorite gel and the 7.6 Å calcium aluminate hydrate) were formed from these two minerals and, even in advanced stages of the reaction when most of the clay had been decomposed, the clay still remaining retained its crystallinity almost intact. For example, a 2M illite retained all the X-ray peaks characteristic of this polymorph, and montmorillonite retained its characteristic (060) spacing.

Eades, Nichols, and Grim (3) suggested that a pH high enough to dissolve silica is an essential feature of the process of formation of calcium silicate hydrates by lime-clay reactions; this idea was seconded by Diamond, White and Dolch (33) who envisioned solution from the edges of the clay crystals as a possible mode of reaction. The addition of sodium hydroxide to a lime-soil mixture has been found by several workers to accelerate strength development (13, 38). Moh (39) bases his explanation of these benefits in both soil-lime and soil-cement systems partly on the hypothesis of an increased rate of solubilization of potentially reactive silica.

These arguments imply that the reaction path proceeds through solution, with silica liberated from the clay reacting with dissolved lime. In contrast our results seem to suggest that reaction proceeds between adsorbed lime and the surface layers of the clay with which it is in contact. Such a mechanism has been proposed to account for the rapid generation of tobermorite gel by lime sorbed on silica gel surfaces at moderately elevated temperatures (40).

It may be that, as seems to be the case in portland cement hydration, both "adsorbed-state" and "through-solution" reactions take place and both contribute to the development of the final product. However, any ultimate resolution of these rather difficult problems must await further intensive experimental work.

SUMMARY

Among physico-chemical mechanisms commonly suggested as explanations of the stabilizing effects observed in soil-lime stabilization, cation exchange replacement of existing cations by calcium, flocculation, and carbonation have been prominent. Since many soils in need of stabilization are naturally calcium saturated, flocculated, or both, the first two mechanisms can no longer be seriously considered. Since soil-lime systems sealed from contact with carbon dioxide develop the normal indications of stabilized soils, carbonation must also be rejected.

The effects of lime on soils are such that two stages of reaction can be detected: (a) an early stage in which the properties of the plastic soil are greatly ameliorated but little permanent strength is developed, and (b) a subsequent stage marked by the slow development of strength and the accumulation of soil-lime reaction products. Among the effects observed in the first stage are large increases in the P. L., generally leading to a reduction in the P. I.; a sharp reduction in the apparent content of clay size particles as they are bound into flocs stable against the dispersion incident to mechanical analysis; increase in the moisture and the compactive effort required to achieve a given density; and reduction in such parameters as swell pressure, volume change on drying, and permeability. These changes are commonly produced in periods ranging from minutes to a few hours after the addition of lime.

Recent work by various workers has suggested the existence of a lime fixation point, that is, an amount of lime that must be added to a given soil to maximize these effects. This lime was considered to be unavailable for further reaction. The mechanism of the effect was discussed in terms of an apparent crowding of excess calcium cations onto the clay following the addition of lime. This in turn was tentatively attributed to pH-dependent exchange sites originating at the edges of the clay particles.

The results of our recent experimental work contradict these hypotheses. It is shown that the cation crowding effect is, in reality, one of physical adsorption of calcium hydroxide onto the clay surfaces. We postulate that the ameliorative effects are due to almost immediate but limited chemical reaction at the points of contact between the edges and faces of primary clay particles within the flocs formed by the normal electrolyte effect of added lime. This reaction is visualized as the formation of small amounts of tetracalcium aluminate hydrate by reaction of the exposed $Al(OH)_x$ groups at the edges of the clay surfaces with lime sorbed on the faces of adjacent surfaces. This immediate reaction is supplemented by somewhat slower reaction of the silica with lime to generate tobermorite gel.

The nature of the compounds held responsible for the slow development of strength in soil-lime systems has been discussed in some detail. The exact products formed vary somewhat with the kind of clay and the reaction conditions, especially temperature. There are commonly at least two phases produced, a calcium silicate hydrate and a calcium aluminate hydrate. The former is usually tobermorite gel; the latter is a well-crystallized hexagonal compound, which is probably an impure (substituted) tetracalcium aluminate hydrate and is characterized by a 7.6 Å basal spacing independent of drying conditions. At temperatures only slightly above normal room temperature a different calcium aluminate hydrate phase, the cubic tricalcium aluminate hexahydrate, is produced.

It was found that quartz, mica, and other phases considered less reactive than clays may also react under appropriate conditions and give rise to similar cementitious products. Quaternary phases in which silicon and aluminum atoms occur in distinguishable lattice positions are not commonly formed, except when lime reacts with previously calcined clay or with the amorphous clay mineral allophane.

The present lack of detailed knowledge of the mechanisms of the chemical reactions that produce these final cementitious products is discussed. Evidence exists that the reactions are favored by conditions of high pH which would make silica more soluble; addition of sodium hydroxide to lime-clay systems produces significant strength gains at early ages. This implies a mechanism involving reaction of dissolved silica and alumina with calcium ions. On the other hand, evidence that the reaction is preceded by sorption of the calcium hydroxide from solution strongly implies a direct surface reaction with the clay. It may be that both mechanisms are operative.

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REFERENCES

1. Johnson, W. M., Cady, J. G., and James, M. S. Characteristics of Some Brown Grumosols of Arizona. Proc. Soil Science Soc. of Amer., Vol. 26, pp. 389-393, 1962.
2. Prikryl, J., and Esterka, F. Equilibrium Studies of Sorbed Clay Mineral Cations and Electrolytes in Alkaline Solutions. (In Czech). Silikaty, Vol. 4, pp. 135-145, 1960.
3. Eades, J. L., Nichols, F. P., Jr., and Grim, R. E. Formation of New Minerals with Lime Stabilization as Proven by Field Experiments in Virginia. Highway Research Board Bull. 335, pp. 31-39, 1962.
4. Wang, J. W. H., Mateos, M., and Davidson, D. T. Comparative Effects of Hydraulic, Calcitic, and Dolomitic Limes and Cement in Soil Stabilization. Highway Research Record No. 29, pp. 42-54, 1963.
5. Jan, M. A., and Walker, R. D. Effect of Lime, Moisture, and Compaction on a Clay Soil. Highway Research Record No. 29, pp. 1-12, 1963.
6. Lund, O. L., and Ramsey, W. J. Experimental Lime Stabilization in Nebraska. Highway Research Board Bull. 231, pp. 24-59, 1959.
7. Hilt, G. H., and Davidson, D. T. Lime Fixation in Clayey Soils. Highway Research Bull. 262, pp. 20-32, 1960.
8. Clare, K. E., and Cruchley, A. E. Laboratory Experiments in the Stabilization of Clays with Hydrated Lime. Geotechnique, Vol. 7, pp. 97-111, 1957.
9. Zolkov, E. Influence of Chlorides and Hydroxides of Calcium and Sodium on Consistency Limits of a Fat Clay. Highway Research Board Bull. 309, pp. 109-115, 1962.
10. Woods, K. B., and Yoder, E. J. Stabilization with Soil, Lime, or Calcium Chloride as an Admixture. Proc. Conf. on Soil Stabilization, M. I. T., pp. 3-19, 1952.
11. Taylor, W. H., Jr., and Arman, A. Lime Stabilization Using Preconditioned Soils. Highway Research Board Bull. 262, pp. 1-19, 1960.
12. Grim, R. Applied Clay Mineralogy. p. 207. New York, McGraw-Hill, 1962.
13. Wolfe, R. E., and Allen, J. R. Laboratory Evaluation of Lime Treatment of Typical Minnesota Soils. Minnesota Dept. of Highways, Final Rept. of Invest. No. 601, 1964.
14. Herrin, M., and Mitchell, H. Lime-Soil Mixtures. Highway Research Board Bull. 304, pp. 99-138, 1961.

15. Mitchell, J. K., and Hooper, D. R. Influence of Time Between Mixing and Compaction on Properties of a Lime-Stabilized Expansive Clay. Highway Research Board Bull. 304, pp. 14-31, 1961.
16. Jambor, J. Relation Between Phase Composition, Over-all Porosity, and Strength of Hardened Lime-Pozzolana Pastes. Mag. Concr. Res., Vol. 15, pp. 131-142, 1963.
17. Anday, M. C. Curing Lime-Stabilized Soils. Highway Research Record No. 29, pp. 13-26, 1963.
18. Shepard, F. D. Effects of Recompaction on the Strength of a Lime and Cement Stabilized Soil. Unpublished report of study carried out in laboratories of Virginia Council of Highway Investigation and Research, Charlottesville, 1963.
19. Laguros, J. G., Handy, R. L., and Reign, L. L. Effect of Exchangeable Calcium on Montmorillonite Low-Temperature Endotherm and Basal Spacing. Highway Research Board Bull. 349, pp. 51-58, 1962.
20. Pietsch, P. E., and Davidson, D. T. Effects of Lime on Plasticity and Compressive Strength of Representative Iowa Soils. Highway Research Board Bull. 335, pp. 11-30, 1962.
21. Pinto, C. de S., Davidson, D. T., and Laguros, J. G. Effect of Lime on Cement Stabilization of Montmorillonitic Soils. Highway Reaearch Board Bull. 353, pp. 64-83, 1962.
22. Ho, C., and Handy, R. L. Characteristics of Lime Retention by Montmorillonitic Clays. Highway Research Record No. 29, pp. 55-69, 1963.
23. Ho, C., and Handy, R. L. Electrokinetic Properties of Lime-Treated Bentonites. Paper presented at 12th Nat. Conf. on Clays and Clay Minerals, Atlanta, Ga., Oct. 1963.
24. Pike, R. G., and Hubbard, D. Source of the Non-Migratable Ionic Charges Developed by Portland and High-Alumina Cements During Hydration. Highway Research Board Proc., Vol. 37, pp. 256-270, 1958.
25. Van Olphen, H. An Introduction to Clay Colloid Chemistry. New York, Interscience Publishers, 1963. 301 pp.
26. Diamond, S. Rapid Reaction of Lime with Hydrous Alumina. Nature, Vol. 204, p. 183, 1964.
27. Buttler, F. G., Dent Glasser, L. S., and Taylor, H. F. W. Studies on $4CaO \cdot Al_2O_3 \cdot 13H_2O$ and the Related Natural Mineral Hydrocalumite. Journal of Amer. Ceram. Soc., Vol. 42, pp. 121-126, 1959.
28. Taylor, H. F. W., ed. The Chemistry of Cements. New York, Academic Press, 1964.
29. Goldberg, I., and Klein, A. Some Effects of Treating Expansive Clays with Calcium Hydroxide. ASTM Spec. Publ. 142, pp. 53-71, 1952.
30. Eades, J. L., and Grim, R. Reactions of Hydrated Lime with Pure Clay Minerals in Soil Stabilization. Highway Research Board Bull. 262, pp. 51-63, 1960.
31. Hilt, G. H., and Davidson, D. T. Isolation and Investigation of a Lime-Montmorillonite Crystalline Reaction Product. Highway Research Board Bull. 304, pp. 51-64, 1961.
32. Glenn, G. R., and Handy, R. L. Lime-Clay Mineral Reaction Products. Highway Research Record No. 29, pp. 70-82, 1963.
33. Diamond, S., White, J. L., and Dolch, W. L. Transformations of Clay Minerals by Calcium Hydroxide Attack. In Clays and Clay Minerals, Proc. 12th Conf. New York, Pergamon Press, in press.
34. Glenn, G. R., and Handy, R. L. X-Ray Studies of Lime-Bentonite Reaction Products. Paper presented at 66th Ann. Meeting, Amer. Ceram. Soc., Chicago, Ill., 1964.
35. Benton, E. J. Cement-Pozzolan Reactions. Highway Research Board Bull. 239, pp. 56-65, 1960.
36. Ariizumi, A., Maki, T., and Oba, M. On Stabilization of Kanto Loam by Means of Lime Gypsum Admixtures. Ann. Rept. of Roads, Japan Roads Assoc., Tokyo, pp. 22-31, 1962.

37. Sloan, R. L. Early Reaction Determination in Two Hydroxide-Kaolinite Systems by Electron Microscopy and Diffraction. Paper presented at 1st Ann. Meeting, Clay Minerals Soc., Madison, Wis., 1964.
38. Davidson, D. T., Mateos, M., and Barnes, H. F. Improvement of Lime Stabilization of Montmorillonitic Clay Soils with Chemical Additives. Highway Research Board Bull. 262, pp. 33-50, 1960.
39. Moh, Za-Chieh. Soil Stabilization with Cement and Sodium Additives. Jour. Soil Mech. and Found. Div., ASCE, Vol. 88, Pt. SM-6, pp. 81-105, 1962.
40. Greenberg, S. A. The Chemical Reactions of Calcium Hydroxide, Silica, and Water Mixtures at 82°C. Jour. Phys. Chem., Vol. 61, pp. 373-374, 1957.

Discussion

R. L. HANDY, T. DEMIREL, CLARA HO, R. M. NADY, C. G. RUFF, ET AL., Iowa State University, Ames. —We sincerely congratulate the authors for this timely review and skillful appraisal of the mechanisms of soil-lime stabilization. In this discussion we would like to call attention to two statements in the paper.

> The point of inflection of the plot of lime added vs P. L. has been dignified by the appellation "lime fixation point."

This is quite correct. However, for reasons of purification we later substituted "lime retention point" for "lime fixation point" (22). The term "fixation" was a rather unfortunate appellation because analogy with K^+ or NH_4^+ fixation implies an irreversible cation adsorption phenomenon. As pointed out by the authors, this is not so; the $CA(OH)_2$ is readily removable at early ages even though the increase in plastic limit (P. L.) appears to be immediate.

> Ho and Handy presented a detailed analysis of their results which we consider highly speculative.

We agree completely. In addition we would paraphrase this with only the names changed, perhaps with "Diamond and Kinter" for "Handy and Ho." Not that we regard speculation as sinful; on the contrary, speculation is high art, vital for exiting resonant vibrations in others and for decreasing free energy of the authors. With this in mind, let us discuss in detail the speculations of Diamond and Kinter, and also include a few from Iowa State University.

The authors' speculation of physical adsorption of lime is not in disagreement with other hypotheses, since all evidence points to physical rather than chemisorption. However, this adsorption should be explained. If it is an ionic adsorption, it should be consistent with double layer theory, since one cannot adsorb 60 meq or more of negative OH ions on 100 g of clay surface without some kind of incentive. Also, the love for OH^- appears rather special, since other anions (sulfate, chloride, etc.) are rejected, even when in company with calcium. We, therefore, have suggested an ionic adsorption whereby OH^- and Ca^{++} ions repress the double layer (ion "crowding"), a widely accepted trend when an electrolyte is added to a clay suspension.

Then, since the lime adsorption reaction is uniquely dependent on pH of the suspension, it would appear that OH^- ions react at broken bonds or react with H^+ ions of the clay, increasing the negative charge so it can adsorb more calcium in stoichiometric proportion. Actually, the occurrence of a pH-dependent charge was not invented by us especially for this purpose; it is recognized and well established in the literature, colloid chemists preferring to call it "an electrical double layer created by adsorption of potential-determining ions" (25). The amount of charge available is believed to depend on the pH, broken bonds, and the dissociable hydrogen ions available. We may even speculate that with a long enough time and a high enough pH, montmorillonite could even

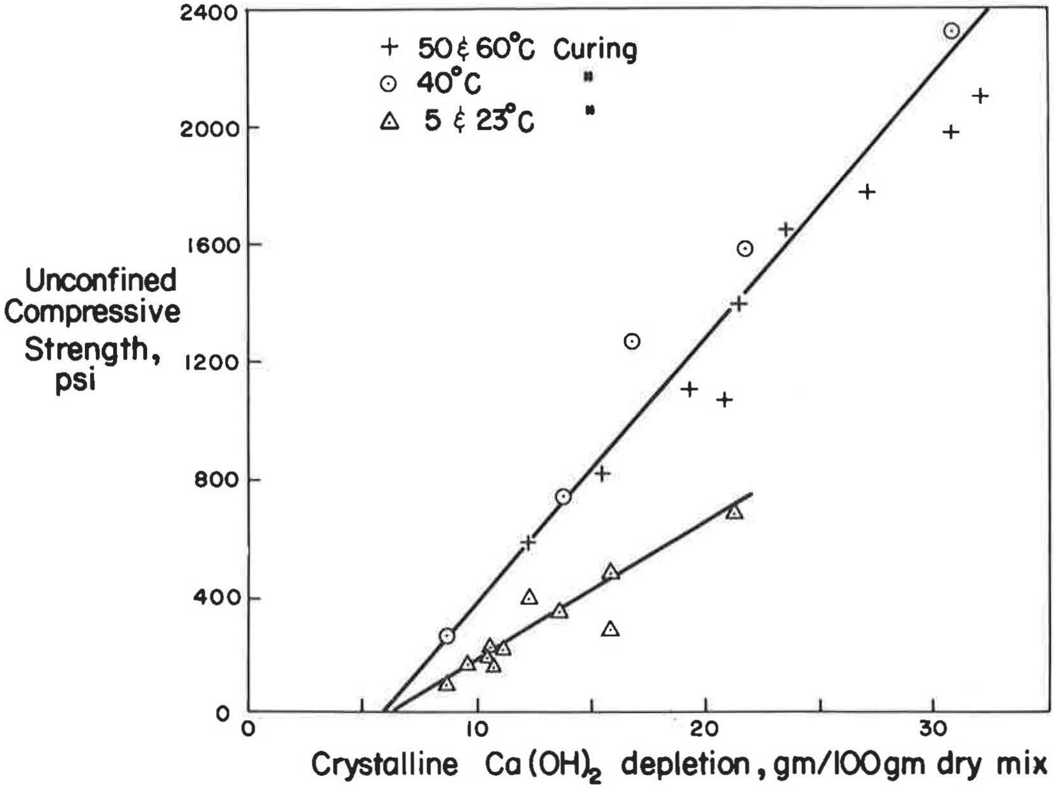

Figure 1. Relation of unconfined strength to $Ca(OH)_2$ reacted in lime-Ca-bentonite-water cylinders cured at various times and temperatures.

dehydrolyze from the octahedral layer to contribute as much or more charge than the permanent charge derived from isomorphous substitution. Unfortunately, this is pure speculation, and soil chemists usually work in the pH range from 3 to 8. Recent work on 60 Wisconsin soil samples averaging 13.3 percent clay gave C.E.C. = 30 + 4.4 X, where X is the pH; the correlation coefficient was 0.979 (3). That is, increasing the pH 5 points (corresponding to an increase in OH ion concentration of 10^5 times) increased the cation exchange capacity 22 meq/100 g. Although applying this relationship to a lime-bentonite mixture involves extrapolation to higher clay content and higher pH range, the prediction is at least in the correct order of magnitude.

Apparently Diamond and Kinter favor molecular rather than ionic adsorption. They also suggest that adsorption results in pozzolanic reaction where the lime coats the clay crystal edges, and cementation of these edges to faces to give a stable floc "card-house" structure, a very attractive and valuable speculation, certain to excite some resonant vibrations. However, we feel that their adsorption mechanism needs additional clarification. From the surface area calculation the authors apparently assume adsorption of a monolayer OH^- and Ca^{++} ion patchwork in the ionic ratio 2 to 1. Perhaps they will entertain some questions: Why aren't other salts adsorbed (except at high pH)? Why did repeated leaching eventually remove more Ca^{++} than $2(OH)^-$? (This is predicted by double layer theory in which OH's react with clay.) What is the shape of the $Ca(OH)_2$ molecule, and how does the molecular adsorption pattern fit the clay mineral structure, both on the silica tetrahedral sites and over the holes? Also, if a $Ca(OH)_2$ is adsorbed as a monolayer, we would hope that X-ray diffraction will eventually indicate some between the layers, since the same forces of adsorption presumably exist there as on the outside.

The final point we wish to make is the "lime retention" point. Regardless of the details of the adsorption mechanism, the lime retention or adsorption point is a valuable

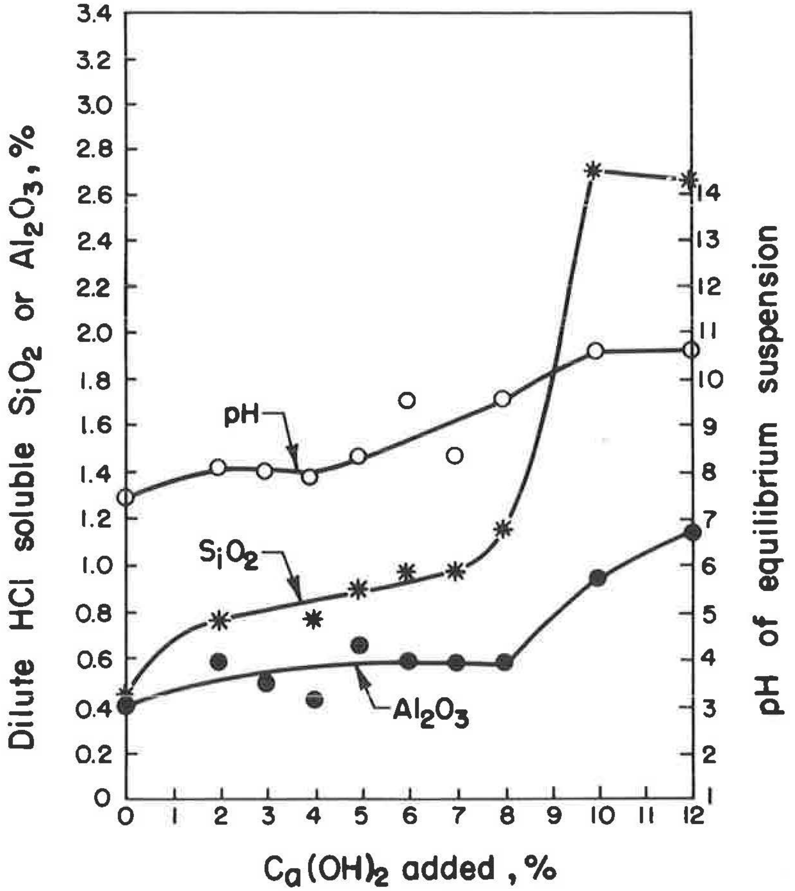

Figure 2. Relations of dilute HCl-soluble SiO_2, Al_2O_3 and equilibrium pH of $Ca(OH)_2$-Na-bentonite suspensions cured at room temperature for more than 2 yr.

concept, particularly useful for engineering purposes. To be sure, the lime retention point slowly changes on curing, probably as a result of localized pozzolanic reactions as suggested by Diamond and Kinter. Of more practical importance is that pozzolanic reactions sufficient for cementation do not occur unless lime is added in excess of the lime retention point, readily estimated for the P. L. test. This is further illustrated by new data presented in Figures 1 and 2.

Figure 1 shows the compressive strength of ½-in. diameter by 1-in. lime-Ca^{++}-bentonite-water cylinders vs loss of crystalline lime from the system as measured by X-ray diffraction. Curing was at various times and temperatures. The initial mixes contained 41.6 percent lime, corresponding to a C/S ratio of 1.0 with all lime and clay reacted. Extrapolation of the strength curves to zero strength suggests that about 6 to 7 percent lime was unavailable for much cementation reaction, the strength being about the same as for the pure clay. This lime percentage is somewhat higher than the lime retention point from plasticity tests.

Figure 2 shows silica and alumina soluble by dilute acid treatment of Na-bentonite-lime suspensions which had been sealed and cured at room temperature for over 2 yr. The soluble silica and alumina are assumed to be mainly from pozzolanic reaction products, although as can be seen, about 0.4 percent of each was leached from untreated clay. The amount of reacted silica is approximately doubled by addition of 2 to 7 percent lime, perhaps in part due to glassy or silicious impurities in the bentonite. The amount of reacted alumina is increased only slightly by addition of 2 to 8 percent lime. However, in the neighborhood of 8 to 9 percent lime, both silica and alumina take a sharp jump, indicative of pozzolanic reaction. It is interesting that in this region pH has stabilized at 10.5, which other evidence suggests is the threshold for pozzolanic reaction (42).

References

41. Helling, Charles S., Chesters, G., and Corey, R. B. Contribution of Organic Matter and Clay to Soil Cation-Exchange Capacity as Affected by pH of the Saturating Solution. Proc. Soil Sci. Soc. Amer., Vol. 28, No. 1, pp. 517-520, 1964.

42. Davidson, L. K., Demirel, T., and Handy, R. L. Soil Pulverization and Lime Migration in Soil-Lime Stabilization. Highway Research Record No. 92, 1965.

SIDNEY DIAMOND and EARL B. KINTER, Closure—We thank Drs. Handy, Demirel, Ho, Nady, Ruff, et al. of the Iowa State University staff for their "resonant vibrations." Differences in interpretation of data always occur in fields of active scientific investigation, and we are not surprised that our interpretations are not universally accepted.

With regard to specific points raised by the discussors, we would offer the following statements:

1. We concur that the "love (of clay surfaces) for OH^- ions appears rather special," not only because other anions are rejected, but also because this love is far less torrid when the OH^- ions are accompanied by cations other than calcium. For example, we have immersed calcium-saturated Wyoming bentonite in solutions of sodium hydroxide of the same concentration as saturated lime solutions (0.04 N). Such sodium hydroxide solutions have a pH distinctly higher than those of saturated limewater, approximately 13.6 as compared to 12.4. In a typical experiment in which ½ g clay was shaken in 100 ml of solution, we found that 48 meq of sodium ions per 100 g of clay were removed from solution by the clay, and 23 meq of calcium ions per 100 g of clay were returned to the solution. Thus, after correcting for cation exchange, an excess of about 25 meq sodium ions per 100 g of clay was adsorbed by the clay, along with approximately the equivalent amount of hydroxyl ions. Thus, in this higher pH system, the clay picked up only about a quarter as many hydroxyls as it does under comparable conditions in saturated lime solutions of lower pH. Obviously, adsorption of OH^- ions is not uniquely dependent on the pH of the system, but increases markedly in the specific joint presence of calcium and hydroxyl ions. This is not consistent with the argument advanced by the discussors that the process is one of formation of an augumented double layer created by adsorption of potential-determining (hydroxyl) ions, coupled with indifferent adsorption of accompanying cations.

Prikryl and Esterka (2) performed the reverse experiment, that is, immersion of sodium-saturated bentonite (and other clays) in calcium hydroxide solutions. They found that the ratio of equivalents of Ca adsorbed from solution to equivalents of Na released to solution (which would be 1 if only ordinary cation exchange were taking place) increased uniformly with increasing initial concentration of calcium hydroxide. Again, these results are not explainable on the hypothesis suggested by the discussors, but were taken by Prikryl and Esterka (and by the present writers) to be indicative of physical adsorption of calcium hydroxide.

2. The Wisconsin soil data cited by the discussors (41) yield a statistical prediction of a 22 meq/100 g pH-dependent gain in cation exchange capacity, from pH 7 to pH 12. This is not really of the same magnitude as the amount of lime removed from solution by montmorillonite. Our data show adsorption of the order of 100 meq/100 g clay (3½ percent lime by weight of clay) in 10 min, and more than 120 meq/100 g of clay (4½ percent lime) in less than an hour. However, the apparent adsorption of sodium hydroxide, cited under point 1 of this closure, is of the same magnitude, and it may well be that adsorption of sodium hydroxide by bentonite is explainable on

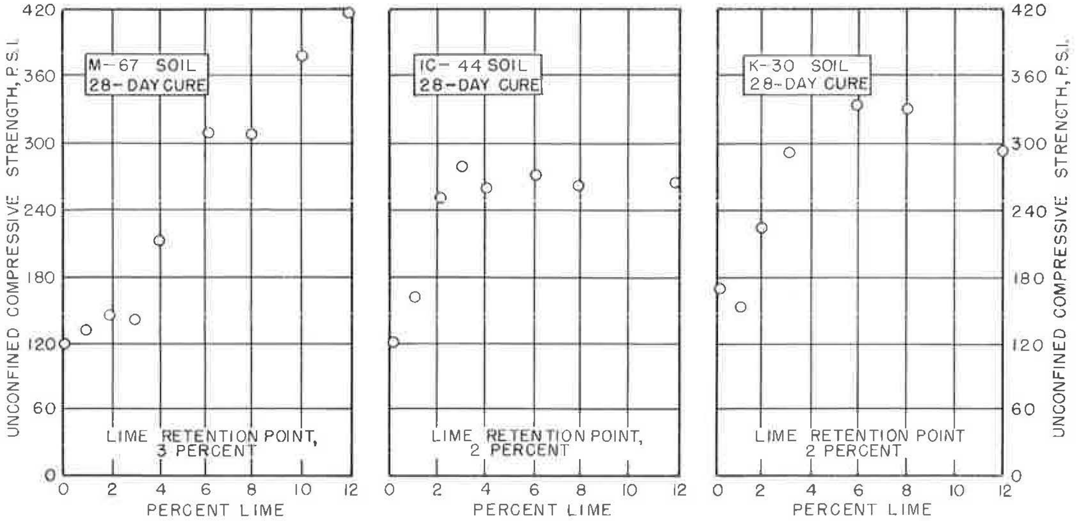

Figure 3. Unconfined compressive strength as function of lime content for a montmorillonite, an illite-chlorite, and a kaolinite soil (7, Figs. 3, 4, 5).

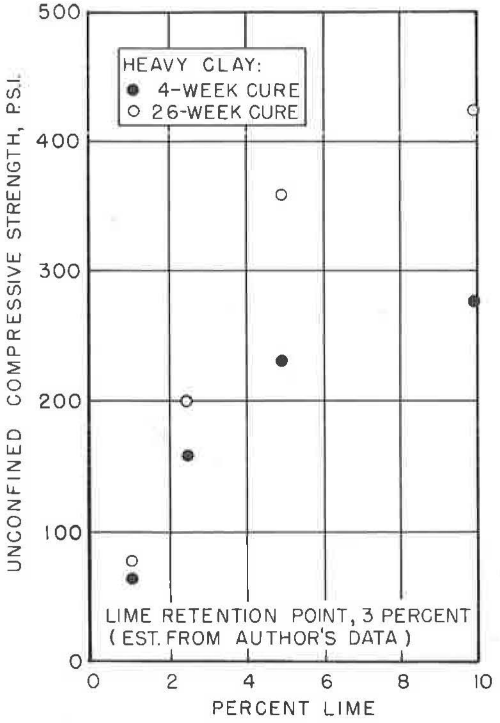

Figure 4. Unconfined compressive strength as function of lime content for heavy clay (London clay) (46, Fig. 1).

the basis of pH-dependent exchange. We do not imply that this is so, only that the possiblity exists.

3. In response to the question of why other salts (we take this to mean "other electrolytes") are not adsorbed, we are forced to plead ignorance. Unfortunately, we are not alone, since very little published research has been done on adsorption from solution, and most of this concerns adsorption of organic compounds. A useful discussion of the available literature was given by Brunauer and Copeland (43).

4. With regard to the question of the balance of 2 OH^- and Ca^{++} ions released into solution on repeated washing of lime-treated clay, we consider that two factors are responsible. First, during the time of processing, extending over periods of one to several hours, some reaction between the adsorbed lime and the clay has undoubtedly occurred to produce small amounts of tobermorite gel and probably C_4AH_{13}-type compounds. These substances release hydroxyl and calcium ions in ratios less than the 2:1 ratio characteristic of lime. The second cause of the discrepancy is the increasing degree of carbonation one must expect on repeated handling of lime-bearing materials in the atmosphere.

5. The shape of the calcium hydroxide molecule (or ion-triplet) is not, of course,

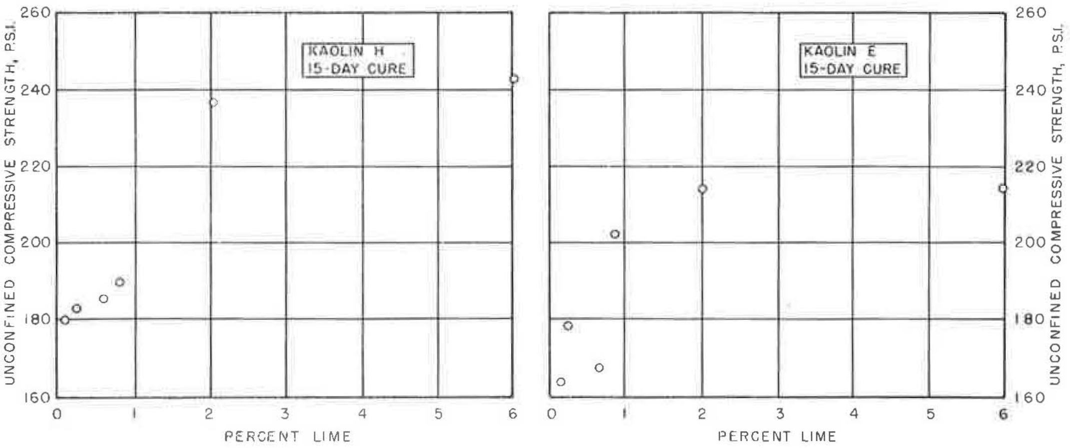

Figure 5. Unconfined compressive strength as function of lime content for two kaolins; lime retention point not available (47, Fig. 7).

amenable to direct determination. Some clue may be obtained from the crystal structure of portlandite, which is hexagonal with a = 3.59 Å and c = 4.91 Å. The structure is built up of layers of octahedra, each of which has a calcium ion at its center and a hydroxyl group at each of its six apices; each hydroxyl group is shared by three octahedra (28).

If this arrangement were to be preserved in calcium hydroxide adsorbed on a clay surface, one would expect the thickness of each layer to be about 4.9 Å and the coverage area per molecule to be that of a 60° rhombus with 3.59 Å sides, that is, 11.2 Å^2. In a study of the chemisorption of lime on silica gel, Greenberg (44) used an area of 25 Å^2 per $Ca(OH)_2$ molecule and found that this gave reasonable results. The present writers have adsorption isotherm data which indicate a coverage area of about 25 Å^2 on montmorillonite and a somewhat higher value on kaolinite. Thus, the indications are that adsorbed calcium hydroxide molecules are not as closely packed on the clay surface as they are in crystalline portlandite.

The potential fit of the portlandite structure, that is, CaO_6 polyhedra, to the hexagonal arrangement of silica tetrahedra that constitutes an idealized representation of most clay mineral surfaces was diagrammed by Taylor and Howison (45), and was shown to be poor, the calcium polyhedra being too large for the hexagonal silica network.

6. Finally, we must address ourselves to the question of the lime retention point. The hypothesis postulates that no significant reaction occurs, and hence, no strength gains accrue, for treatment with lime in amounts less than the lime retention point.

The discussors kindly inclosed two new figures to reinforce this argument. Unfortunately, the present writers have considerable difficulty in appreciating the significance of these figures.

Figure 1 relates strength development to depletion of crystalline lime in systems, all of which contained 41 percent lime by weight of clay. Only very low densities are obtainable in systems with such unusually high lime contents. We suggest that because of these low densities, strength gains obtainable in such systems bear little relationship to those that can be obtained in well-compacted systems of lime contents near the lime retention point, that is, 2 to 4 percent. The lines plotted on this figure intersect the zero strength axis at more than 6 percent lime. Surely the discussors do not mean to imply that a calcium bentonite treated with 6 percent lime and properly compacted and cured will not develop any significant strength.

The relevance of the discussors' Figure 2 to the question at issue is unfortunately not apparent to the present writers.

Data are available from the literature, however, which bear directly on the question of strength gains, and hence reactivity, in soils treated with amounts of lime below the lime retention point. If the concepts of the discussors are correct, a plot of strength vs lime content for properly compacted and cured specimens should show no strength gain below the lime retention point, a distinct inflection at this point, and strength increases from then on as some function of increased lime content. In contradistinction, we suggest that with many soils, small increments of lime less than the lime retention point do in fact add to the strength of such samples as a result of chemical reaction. On this basis, a plot of strength vs lime content should show increases in strength with very small increments of lime starting at or near zero.

Figures 3, 4, and 5 are offered from data in the literature (7, 46, 47). Only the data points are submitted; no trend lines are drawn. Readers of this discussion are invited to draw their own conclusions.

References

43. Brunauer, S., and Copeland, L. E. Surface Tension, Adsorption. In Handbook of Physics, Chapter 7. New York, McGraw-Hill, 1958.
44. Greenberg, S. The Chemisorption of Calcium Hydroxide by Silica. Jour. Phys. Chem., Vol. 60, pp. 325-330, 1956.
45. Taylor, H. F. W., and Howison, J. W. Relationships Between Calcium Silicates and Clay Minerals. Clay Minerals Bull., Vol. 3, pp. 98-111, 1956.
46. Dumbleton, M. J. Lime-Stabilized Soil for Road Construction in Great Britain. Roads and Road Construction, Vol. 40, pp. 321-325, 1962.
47. Ormsby, W. C. Kaolin-Lime-Water Systems: Strength and Rheological Properties. NBS Tech. Rept., 1964.

Soil Pulverization and Lime Migration in Soil-Lime Stabilization

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T. DEMIREL and R. L. HANDY, Iowa State University

•THIS INVESTIGATION was initiated to study the effect on soil-lime stabilization of one important variable, the degree of pulverization before compaction. For soil-lime stabilization it is generally recommended that 60 to 70 percent of all material, exclusive of gravel and stones, should pass the No. 4 sieve, and 100 percent of this material should pass the 1-in. sieve (1). Some specifications have called for as much as 100 percent of all material, exclusive of gravel and stone, to pass the No. 4 sieve (2). Soils with greater than 20 to 30 percent clay and with a plasticity index (P.I.) greater than 20 percent, however, are sometimes difficult or even impossible to pulverize (3). Curing of partially mixed soil and lime before final mixing improves pulverization, but this procedure has been found to be detrimental to the lime-soil reaction due to exposure to air containing carbon dioxide (2).

Field observations by the authors and others (4) of stabilized unmixed soil lumps within cured soil-lime and soil-cement indicated that free lime may migrate into and stabilize these lumps. The goal of the present study was to evaluate the effects and effectiveness of such lime migration, with the eventual possibility of a relaxation of soil-lime pulverization requirements.

DIFFUSION PHENOMENA

Diffusion in solids is similar to that in liquids except for certain complications due to mechanical defects and grain boundary interfaces through which diffusion can occur. Therefore, diffusion in a solid may take place either homogeneously through the crystal lattice, or as grain boundary or interface diffusion process. Diffusion of these two types will occur at very different rates (5).

Because of the interactions between clay and exchangeable ions and between clay and water, the diffusion process in clay-water systems is very complex; these two interactions could affect each other. Different diffusion mechanisms with different diffusion coefficients and activation energies can be operating in a clay-water system at the same time (6). Diffusing cations have been found to move by at least two different mechanisms Cation migration in an ovendry soil is evidence for diffusion by movement along the exchange sites on a clay particle surface and by jumping from the exchange sites on one particle to exchange sites on an adjacent particle. Diffusion of cations occurs at a faster rate through the pores of a clay-water system.

The diffusion of anions through a bentonite-water system was found to take place exclusively within the pores or "channels" (6). In accordance with this conclusion, anion diffusion did not occur in an ovendry bentonite. Significantly, in a clay-water system containing free water, anions will diffuse faster than cations if the concentration gradient is small, and cations and anions can diffuse separately. Barber has concluded that calcium ion movement in soil is due primarily to water movement but, if a concentration gradient exists, diffusion is possible (7).

That the diffusion of lime in soil is possible and even probable has been shown as early as 1939 and perhaps even earlier. The movements of ions in gels and soils have been divided into four groups according to mechanism (8): (a) free diffusion of ion pairs in the intermicellar pores and channels, (b) adsorption of ions by colloidal particles

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which themselves diffuse as a result of Brownian movement or with the aid of some transporting agent, (c) exchange of ions between surfaces and intermicellar liquids, and (d) surface migration and contact exchange, independent of the nature of the intermicellar liquid.

By the use of radioactive tracer techniques the apparent self-diffusion coefficients of calcium ions added as a chloride were found by Lai and Mortland (6) to be 2.36 ± 0.07, 1.95 ± 0.06, and 1.75 ± 0.05 (10^6 sq cm/sec) for bentonite-water systems containing 17.9, 23.9 and 28.0 percent by weight of bentonite. Gast (9) determined the self-diffusion coefficients of calcium ions through a bentonite-water system containing 3 percent bentonite by weight to be 0.607 ± 0.045 and 2.14 ± 0.25 (10^6 sq cm/sec) for calcium ions added as hydroxide and as chloride, respectively. In both investigations Ca-saturated bentonite was used so that cation exchange would not interfere with diffusion.

In interpreting diffusion coefficient determination data both authors (6, 9) attached considerable significance to viscosity of water adjacent to clay particles. Lai and Mortland refer to Low's work (10) in their interpretation, and Gast refers to Kemper's work (11). Both Low and Kemper measured viscosity of water in a clay-water system indirectly, but the influence of all possible variables was not investigated. Definite conclusions about viscosity of water in a clay-water system on the basis of these two investigations would require considerable speculation. Although viscosity undoubtedly has an important influence on the diffusion of ions in a clay-water system, speculation about the manner and magnitude of influence is impractical without further investigation.

Investigators thus far have succeeded only in determining the apparent diffusion coefficients of cations in a soil-water system. It has been generally concluded that cations in these systems have at least two different diffusion mechanisms and, therefore, two different diffusion coefficients. Determination of these two coefficients has not yet been achieved.

Anions may have a different diffusion coefficient than cations (6). Electrolyte concentrations high enough to cause chemical reaction and cation exchange would affect diffusion through soil-water systems. Consideration of all these factors would require a very theoretical and detailed expression for the rate of diffusion of lime through soil. The validity of applying one of the solutions given for the diffusion equation by Barrer (12) is questionable. Until more is known of all the factors influencing ion migration through a clay-water system a more general approach, as presented below, is justifiable.

Lime Diffusion, a Boundary Process

The development of a reaction product layer at the boundary of lime and soil is the result of a series of processes which may be classified as boundary processes of solid-state reactions. These boundary processes may include (a) the transfer of lime into the soil, (b) a chemical reaction between the two, (c) formation of nuclei and growth of the reaction product, and (d) further diffusion of lime into the soil through the reaction product layer. The first three of these processes are generally referred to as phase boundary processes, and the last as a diffusion process (13). The reaction rate may then be expressed as

$$\frac{dn}{dt} = c \frac{\Delta\mu}{W_R + W_D} \qquad (1)$$

where W_R is the reaction resistance defined as the reciprocal of the velocity of all phase boundary processes, W_D is the diffusion resistance, and $\Delta\mu$ is the free energy change involved in the reaction.

When only one of the reactants is able to move, two different rate-determining conditions may control the reaction rate. One of the possible rate-determining conditions exists when the diffusion resistance is much larger than the reaction resistance ($W_D >> W_R$), and diffusion is occurring at a much slower rate than is the boundary reaction. In this case the reaction rate can be written:

$$\frac{dn}{dt} = c\frac{\Delta\mu}{W_D} = c\,\frac{\Delta\mu}{1/D\int_{\text{Phase Boundary II}}^{\text{Phase Boundary I}} dl/A} \tag{2}$$

where D is the diffusion coefficient defined by Frick's law, l is the thickness of diffusion layer, and A is the cross-sectional area of diffusion layer. Assuming a constant cross-section and constant D, this equation when integrated gives the following expression for the rate of growth of the product layer:

$$l^2 = k't \tag{3}$$

where k$'$ is a constant, and t is time. The other controlling condition exists if the reaction resistance is much larger than the diffusion resistance ($W_R \gg W_D$) and the reaction is taking place at a much slower rate than is diffusion. If this is true, a general expression for the reaction rate cannot be formulated due to the complex nature of the phase boundary processes. Expressions must be formulated for each particular reaction involved.

These two conditions are extreme cases and often both the diffusion rate and the phase boundary reaction rate must be considered in the overall reaction rate (13). If the total free energy change is separated into parts due to diffusion and to phase boundary processes, the general expression for the reaction rate can be resolved into:

$$bl + l^2 = kt \tag{4}$$

Eqs. 3 and 4 relate the thickness of the reaction product layer with time. The application and solution of an appropriate expression of this type to a soil-lime system is of significant importance to an investigation of the effect of pulverization on soil-lime stabilization. The applicability of Eq. 3 has been verified for the reaction between calcium carbonate and mullite (14).

METHODS OF INVESTIGATION

Materials

The soil used in this investigation was a Sharpsburg series plastic loess from Clarke County, Iowa. A sample was taken from the B horizon at a depth between 12 and 46 in. The physical and chemical properties of this soil are given in Table 1.

Commercially available calcitic hydrated lime (Kemikal, U.S. Gypsum Co.) was used.

Soil Preparation

Soil samples were brought into the laboratory in two large metal containers. One of the containers had been sealed in the field with plastic wrapping and cellophane tape so the field moisture content of this soil could be maintained. The other can was merely covered with a metal lid. The sealed container was placed in a humidity room where the relative humidity is maintained at 95 ± 5 percent, and the contents of the other can were spread and allowed to air dry for several days.

TABLE 1

SOIL PROPERTIES

Property	Value
Textural composition, AASHO (% by wt.)	
Sand, 2.0 to 0.074 mm	3.1
Silt, 0.074 to 0.005 mm	53.6
Clay (5 μ), <0.005 mm	43.3
Clay (2 μ), <0.002 mm	35.0
AASHO classification	A-7-6(12)
Textural classification	Clay
L.L. (%)	41.2
P.L. (%)	21.0
P.I.	20.2
Predominant clay mineral	Ca-montmorillonite
pH	7.0

The soil sample which had been stored at its field moisture content in the humidity room was pulverized by hand to pass a 1-in. sieve and be retained on a No. 4 sieve. All material from this sample passing the No.

4 sieve was discarded. The material passing the 1-in. sieve and retained on the No. 4 sieve (lump sample) was again placed in a metal container, sealed with plastic wrap and cellophane tape, and replaced in the humidity room until used. Lumps were kept at the field moisture content of about 16 percent to minimize density drops during compaction of specimens. The size distribution by weight of the soil lumps was as follows:

Percent passing 1-in. sieve, 100.0;
Percent passing ¾-in. sieve, 55.6;
Percent passing ½-in. sieve, 35.8;
Percent passing ⅜-in. sieve, 24.9; and
Percent passing No. 4 sieve, 0.0.

The air-dry soil was then crushed in a jaw crusher to pass a No. 4 sieve and stored in a metal container until used. Representative samples for mechanical analysis and for determination of Atterberg limits were obtained from this air-dried sample.

Moisture-Density Relationship

The percentage of lime used was chosen arbitrarily. Since in Iowa 6 percent lime is a frequently used amount for soil-lime stabilization, this amount, based on the oven-dry weight of the soil, was added to a representative sample of air-dried soil (100 percent pulverized) for the density-moisture relationship study. The ingredients were placed in a metal mixing bowl and given a thorough hand mixing. Then the soil-lime mixture was mixed with a Hobart kitchen mixer, Model C-100, at low speed for 1 min. Further stirring by hand insured mixing of material in the bottom of the bowl. The mixing bowl was again placed on the mixer and water was added from a graduated cylinder while mixing proceeded for 1 min. Scraping of the sides of the bowl and an additional 1 min on the kitchen mixer completed the mixing process. A damp cloth was placed over the bowl to prevent evaporation during molding.

Specimens 2 in. in diameter by 2 ± 0.05 in. high were molded with a compaction apparatus developed at the Iowa Engineering Experiment Station (15) to give approximate standard AASHO T 99-57 density. With the exception of dry-mixing, the mixing and molding procedure was repeated for increasing moisture contents of the soil-lime mixture to obtain the density-moisture relationship.

The optimum moisture content for maximum AASHO density of 100 percent pulverized plastic loess soil containing 6 percent of hydrated calcitic lime was determined to be 23.2 percent. Although the unconfined compressive strength specimens were prepared with soil samples containing various percentages of lumps using the Rainhart Co. compaction apparatus, the optimum moisture content was, for convenience, determined with 100 percent pulverized soil using the Iowa State compaction apparatus. The difference between the density obtained at 23.2 percent moisture for the two methods was about 1 pcf, an insignificant amount.

Unconfined Compressive Strength Specimens

The primary purpose of the unconfined compressive strength study was to secure comparative data which would indicate the effect of pulverization on soil-lime stabilization. For this purpose unconfined compressive strength specimens containing 100, 80, 60, 40 and 20 percent lumps were prepared.

The necessary amounts of soil passing the No. 4 sieve, corrected for hygroscopic moisture, and the necessary amount of soil lumps passing the 1-in. sieve and retained on the No. 4 sieve, corrected for their moisture content, were weighed to the nearest gram, placed in a metal mixing bowl, and given a short-period preliminary hand mixing with a large metal spoon. The desired amount of lime was then weighed to the nearest gram and added to the mixing bowl. Further hand mixing was initiated to insure a relatively intimate mixture of soil and lime before water was added to bring the moisture content of the mixture to the optimum moisture content determined for 100 percent pulverized soil. Then the mixture was hand mixed with a spoon for 6½ min. Following this, the bowl was covered with a damp cloth to prevent evaporation, and moisture samples were taken.

Proctor specimens 4 in. in diameter and 4.6 in. high were molded using a Rainhart Co. compaction apparatus, Model 662. The motor-driven rammer of this apparatus is calibrated to deliver the same compactive effort as the standard AASHO T 99-57 rammer. After being molded and extruded, the specimen was weighed, marked, and sealed in plastic wrap with cellophane tape to limit carbonation of the lime by carbon dioxide from the air and to limit moisture loss. After each group of three specimens were molded, moisture samples were taken; the wrapped samples were placed in the curing room in which the relative humidity was maintained at 95 ± 5 percent and the temperature at 70 ± 5 F. The specimens were cured for periods from 7 to 270 days.

At the end of the selected curing periods, the specimens were removed from the curing room and tested to failure using a Universal hydraulic testing machine. The rate of strain applied to the specimens was held constant at about 0.1 in./min. The ultimate load in pounds was recorded for each specimen; the load was read to the nearest 5 lb for specimens tested with the low range of the machine (up to 6,000 lb) and to the nearest 20 lb for samples tested with the medium range (up to 24,000 lb).

Pressed fiber pads, trimmed to approximately the same diameter as the samples, were used on the top and bottom of the samples during load testing to insure uniform load application.

Lime Migration Study Specimens

Soil and water to bring the moisture content to optimum were mixed using the same soil, equipment, and procedure used for mixing the materials for determination of the moisture-density relationship, except that no lime was used and, therefore, dry mixing was unnecessary.

Molding of specimens was accomplished using the Iowa State compaction apparatus mentioned previously. The procedure and equipment used were the same as those used for the moisture-density relationship study except that a 6-in. high and 2-in. diameter Plexiglas mold was used instead of the regular metal one. After being molded, the specimens were positioned in the Plexiglas molds so that one face was flush with the end of the mold. The specimens were then weighed and their heights were measured.

A lime slurry containing 7 parts water to 5 parts lime, by weight, was poured to a depth of $1\frac{1}{2}$ in. on the top of the specimens contained in the Plexiglas molds. These molds were then sealed at both ends with plastic wrap and cellophane tape to prevent moisture loss and carbonation from carbon dioxide in the air and were placed in an upright position in the curing room. The samples remained in the curing room at 95 ± 5 percent RH and a temperature of 70 ± 5 F until testing.

At specified intervals of time, two of the specimens were removed from the curing room for testing. The plastic wrap was removed from the ends of each mold, a sample was taken of the lime slurry, and the remainder of the slurry was removed from the surface of the sample and discarded. After measurement of the sample height, the sample was extruded in eight small increments and sliced even with the edge of the mold, using a wire cheese cutter. All of the slices were $\frac{1}{4} \pm 1\frac{1}{32}$ in. thick with the exception of the first slice, which was taken from the end opposite the one placed in contact with the lime slurry and was usually less than $\frac{1}{4}$ in. thick due to shrinkage and variation from 2 in. in the molded height of the specimen. The slices and lime slurry samples were placed in small airtight jars to prevent carbonation.

Partial drying of the slices and lime slurry specimens was accomplished by placing the uncapped jars containing the samples into a vacuum desiccator containing calcium chloride for moisture absorption and Askarite (NaOH and asbestos) for prevention of carbonation. The desiccator was then evacuated and sealed, and the samples were vacuum dried for 36 to 48 hr.

After being dried, the samples were removed from the desiccator, ground with a mortar and pestle, and replaced in capped, airtight jars. All procedures were carried out with the knowledge that more than a minimum amount of exposure to air could cause undesirable carbonation of the samples.

Two-gram portions of each of the slices and lime slurry samples along with a sample of untreated soil were weighed into 50-ml Erlenmeyer flasks, and water was added to the flasks to bring the volume of the soil-water and soil-lime mixtures to 45 ± 5 ml.

After each flask was sealed with a cork stopper, the mixtures were mechanically agitated for $\frac{1}{2}$ hr. The suspensions were then allowed to settle for about 4 hr before the supernatant liquid was poured from the flasks into smaller beakers, and the pH of this supernatant was measured. The pH measurements were taken with a Beckman pH meter. To minimize interference of suspended soil particles with the pH measurement, the beaker was gently agitated by hand while the measurement was being taken.

A 3-g portion of each partially dried and ground slice sample was weighed into a 60-ml Buchner funnel equipped with a coarse fritted glass filter. The samples in the funnels were covered with filter paper and were then leached with successive 100-ml quantities of 1 N potassium chloride solution until less than 1 meg of cation per 100 g of ovendry samples was being leached from the sample by 100 ml of the solution. After being leached with potassium chloride solution, the samples were leached with 0.1 N hydrochloric acid in two successive 100-ml quantities.

The amounts of cation removed from the soil by leaching with the two solutions were determined by titrating to the nearest 0.05 ml with standard EDTA in the presence of pH 10.5 ammonium hydroxide-ammonium chloride buffer. Erichromeschwartz I was used as the indicator. In the case of the hydrochloric acid leachate, adjustment of the pH to a level between 6 and 8 with sodium hydroxide was necessary before addition of the indicator and buffer.

X-Ray Diffraction

X-ray diffraction studies were performed on representative samples of the original soil and on samples taken from unconfined compressive strength specimens and from the lime migration study specimens. The samples were either air dried or dried in a desiccator containing calcium chloride. Samples were pulverized with a mortar and pestle and mounted in a brass ring or on a glass slide. All lump samples taken from the unconfined compressive strength specimens were carefully scraped to insure removal of all adherent matrix material.

A General Electric Co. Model XRD-5 diffraction unit with copper Kα radiation was used. The diffractometer settings used for all studies were a 1° or 3° beam slit, medium resolution soller slits, 0.2° detector slit, a 2°/min scan rate and a 3-sec time constant.

STRENGTH STUDY

Unconfined Compressive Strength

The results of the unconfined compressive strength tests (Figs. 1 and 2, Table 2) show that the presence of soil lumps in a compacted soil-lime mixture decreases the strength of the mixture. After 14 days of humid curing the average difference between the strength of the specimens containing 0 percent soil lumps and those containing 80 percent soil lumps was 26 psi, or about 20 percent of the strength with no lumps. The strength difference between these two extremes of percentage of soil lumps after 90 days of curing was about 35 psi, or about 13 percent.

The strength differences of the intermediate specimens containing 20, 40 and 60 percent of soil lumps was relatively small at all curing periods. The strength differences after 14-days curing between the specimens containing 20 percent soil lumps and those containing 40 and 60 percent soil lumps were only 7 and 9 psi, respectively. These differences after 90 days of curing were 7 and 13 psi. Figure 1 indicates that at a curing time of about 150 days, the strengths yielded by these three mixtures were about equal and only about 20 psi less than those of specimens containing no soil lumps. All four groups of specimens showed strengths above 300 psi.

The rate of strength gain was relatively constant for all specimens tested at curing times up to about 150 days; after this the rate slowed. This change was slower for specimens with no lumps, indicating that the strength differences increase after 200 days until strength equilibrium is reached.

Soil lumps were clearly visible in the specimens tested after curing periods up to 190 days. The lumps were darker in color than the matrix of the specimens. The color difference between the soil lumps and the matrix surrounding the lumps became less pronounced after 190 days.

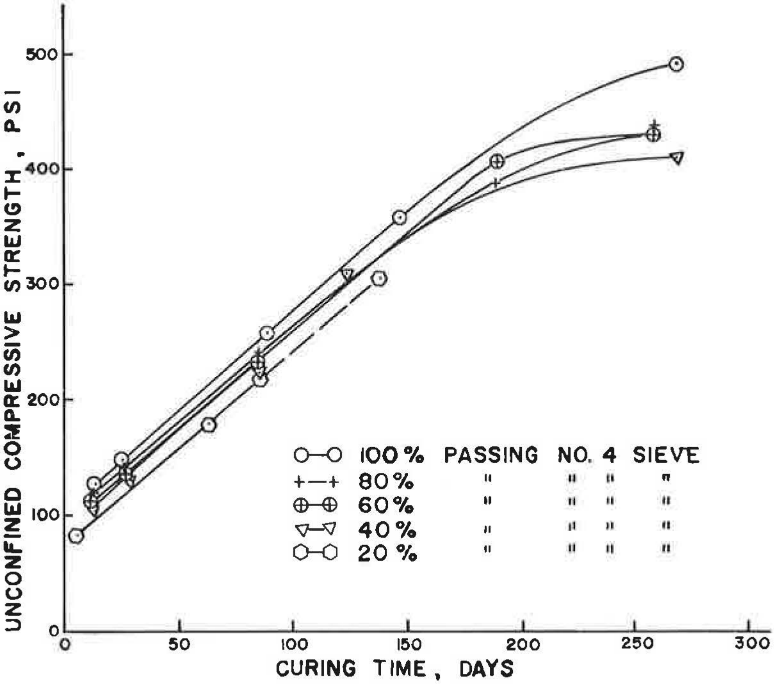

Figure 1. Effect of curing on unconfined compressive strength of specimens containing various percentages of soil lumps.

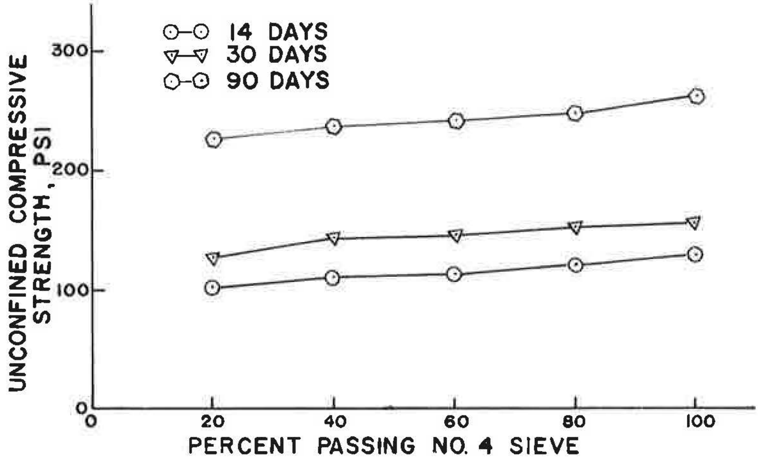

Figure 2. Effect of lump percentage on unconfined compressive strength of specimens cured for various lengths of time.

The lumps were quite easily separated from the matrix of specimens cured from 30 to 190 days. Oddly enough, the lumps appeared to be even stronger than the matrix surrounding them, and it was possible to crumble away the surrounding matrix and leave the lumps intact. Although some of the larger lumps appeared to have soft centers after 30 days of curing, even the largest lumps appeared to be uniform in hardness after 90 days of curing.

TABLE 2

UNCONFINED COMPRESSIVE STRENGTHS OF SPECIMENS CONTAINING VARIOUS PERCENTAGES OF SOIL LUMPS

Percent Passing No. 4 Sieve	Avg. Moisture Content (%)	Avg. Dry Density (pcf)[a]	Curing Period (days)	Avg. Unconfined Compressive Strength (psi)[a]
100	23.2	98.4	14	128
	23.2	98.1	30	156
	23.3	98.1	90	260
	23.1	97.7	147	359
	23.1	98.1	270	494
80	23.1	97.5	14	119
	23.2	96.6	30	151
	23.4	97.1	90	247
	23.1	97.4	190	389
	22.9	97.4	260	432
60	23.1	97.2	14	112
	23.1	97.0	30	145
	22.9	97.3	90	240
	23.0	98.0	190	410
	23.1	97.5	260	431
40	23.0	97.8	14	110
	23.2	98.1	30	143
	23.4	97.6	90	234
	23.3	97.9	128	318
	23.1	97.3	270	411
20	23.0	97.6	7	86
	23.4	97.2	14	102
	23.6	97.3	30	126
	22.9	97.0	65	178
	23.1	97.2	90	225

[a] Results average from three specimens tested.

After 190 days of curing, the strength of the lumps and the matrix of the specimens became more uniform; crumbling away the matrix surrounding the lumps was difficult. The matrix strength and the bonding between the lumps and the matrix had increased.

Lumps removed from specimens containing 60 percent lumps at the time of compaction and cured for 90 days were immersed in water for 48 hr. Although the natural soil slaked quite rapidly, these lumps did not slake, and no loss of strength was apparent.

X-Ray Diffraction

The results of X-ray diffraction studies are summarized in Table 3. A lump sample from the specimen containing 60 percent lumps and cured for 190 days was pulverized and dispersed in water with an ultrasonic dispersion apparatus for 10 min. X-ray diffraction of a glass slide prepared with the dispersed liquid showed a strong calcium carbonate peak at a d-spacing of 3.03 Å. X-ray diffraction of an undispersed lump sample from the same specimens showed only a very weak calcium carbonate peak, suggesting carbonation during testing. Another glass slide was prepared from the dispersed lump sample, and precautions were taken to prevent exposure of the slide

TABLE 3

X-RAY DATA SUMMARY

Sample	Treatment	Reduced Peak Spacings (Å)	New Peak Spacings (Å)	Remarks
190-days cured specimens containing 60% lumps: lumps	Pulverized and dispersed in distilled water, then deposited on glass slide.		3.03, strong	Calcium carbonate.
	Pulverized and dispersed in CO_2-free distilled water, then deposited on glass slide with precautions to prevent exposure to CO_2.		3.03, strong	Calcium carbonate.
	Pulverized and dispersed in benzene, then deposited on glass slide.		3.03, very weak	Trace amount of calcium carbonate.
14-days cured specimens containing 80% lumps: Matrix	6-days additional storage in airtight containers.	15 (montmorillonite)	8.1, 7.6, 3.9 4.9, 3.9, 2.6	Calcium aluminate hydrate (16). Calcium hydroxide.
Lumps		15 (montmorillonite)	8.1, 7.6, 3.9, 2.66	Calcium aluminate hydrate, no calcium hydroxide.
270-days cured specimens containing 60% lumps: matrix and lumps	None		8.1, 7.6, 3.9, very weak. Halo between 2.88 and 3.13	Calcium aluminate hydrate. Calcium silicate hydrate (?)
128-days cured specimens containing 60% lumps: matrix and lumps	Autoclaved at 126 C for 7 days	15 (montmorillonite)	7.96, very strong	Calcium aluminate hydrate.
128-days cured specimens containing 60% lumps: matrix and lumps	Autoclaved at 126 C for 14 days	15 (montmorillonite)	6.4, 4.7, 2.8, 2.7, 2.15 Halo between 2.88 and 3.13	Afwillite (17). Calcium silicate hydrate (?)

to carbon dioxide in the air. In spite of these precautions X-ray diffraction of this slide also showed a strong calcium carbonate peak. When sodium oxalate was added to the supernatant of the lump dispersion, a calcium oxalate precipitate was formed. This formation of the calcium oxalate precipitate was evidence that dissociated calcium ions were in the supernatant and indicated that precautions taken to prevent carbonation of the glass slide X-ray diffraction sample were unsuccessful.

Another lump sample, taken from the specimen containing 60 percent lumps and cured 190 days, was dispersed in benzene for 15 min with the ultrasonic dispersion apparatus. X-ray diffraction of a glass slide prepared from the lump dispersion produced a very weak calcium carbonate peak. Addition of water to the slide and exposure to air containing carbon dioxide increased the intensity of the calcium carbonate.

X-ray diffraction patterns from the natural soil and from matrix and lump samples were examined in an effort to detect the presence of pozzolanic reaction products and to detect differences between lump and matrix samples. A general trend observed in lump and matrix samples was the decrease in montmorillonite peak.

Calcium aluminate was identified on diffraction patterns from samples taken from unconfined compressive strength specimens which had contained 80 percent of soil as lumps and has been cured 14 days. The samples had been stored an additional 6 days in airtight jars before they were X-rayed, bringing the actual curing time to 20 days. New peaks were observed at 8.1, 7.6, 4.9, 3.9 and 2.6 Å in the diffraction pattern from the matrix sample. Glenn (16) has identified the 8.1, 7.6 and 3.9 Å peaks as calcium aluminate hydrate. The 4.9, 3.9 and 2.6 Å peaks were from calcium hydroxide in the matrix material. Diffraction patterns from the lump sample failed to produce any calcium hydroxide peaks, but peaks at 8.1, 7.6, 3.9 and 2.86 Å showed the presence of calcium aluminate hydrate in the samples.

X-rayed samples from specimens containing less than 80 percent of soil as lumps failed to produce strong pozzolanic reaction product peaks in their diffraction patterns. This was true of both matrix and lump samples. After 270 days of curing, the X-ray diffraction patterns of lump and matrix samples taken from specimens containing 60 percent of soil as lumps showed only very weak peaks at the d-spacing given for calcium aluminate hydrates. The most definite evidence of a pozzolanic reaction was the presence of a diffraction halo between 3.13 and 2.88 Å. This halo appeared on the diffraction patterns of both the lump and matrix samples, and is an indication of calcium silicate hydrate.

In an attempt to crystallize any pozzolanic reaction products present in a specimen containing 60 percent soil as lumps and cured 128 days, both lump and matrix samples of these materials were placed in an autoclave maintained at 126 C for 7 days. Diffraction patterns from both materials yielded one strong new peak at 7.96 Å, identified as calcium aluminate hydrate. When the two samples were autoclaved for 14 days this peak disappeared, and most of the strong intensity peaks listed by Glenn and Handy (17) for afwillite, a calcium silicate hydrate, appeared. A halo between 2.88 and 3.13 Å on the two diffraction patterns suggested the presence of other phases of calcium silicate hydrate.

Discussion

Three stabilization mechanisms may be suggested for a compacted soil-lime-water mixture containing some of the soil as lumps, with the percentage of lime in the matrix in excess of the lime retention point (18, 19) and therefore available for a pozzolanic reaction. The lumps may be surrounded by a waterproof skeletal matrix of soil and lime-soil reaction products. In this case, the lumps add almost no strength to the compacted mixture but are prevented from undergoing moisture and volume changes which might disrupt the matrix. The matrix development becomes the strength-determining factor of the compacted mixture.

The other two possible mechanisms involve the assumption that lime will migrate into the lumps of the compacted soil-water-lime mixture. First, the amount of lime migrating into the lumps may only be sufficient to cause a thin reaction product crust on the surface of the lumps and flocculation of soil within the crust. In this case the lumps will have considerable strength, depending on the crust thickness. Second, it may be that enough lime will migrate into the lumps to cause a pozzolanic reaction throughout the lumps and not just in a thin crust at the lump surface. The strongest mixture would result from this mechanism.

Strength results, X-ray diffraction studies, and slaking resistance of lumps removed from tested unconfined compressive strength specimens demonstrated not only that lime movement takes place in a soil-lime-water mixture, but that lime movement occurs in sufficient quantities to cause a pozzolanic reaction in soil lumps within the mixture.

Since the rate of strength gain was nearly the same for the five different soil-lime-water mixtures tested after curing periods up to 150 days, it is logical to assume that the early strength differences among the compacted mixtures were due to differences in a reaction occurring during the early stages of curing. The first phase of a soil-lime reaction is flocculation, but before flocculation can occur within soil lumps, lime movement is necessary. Initial flocculation of lumps is confined primarily to the surfaces, but initial flocculation of the matrix soil is quite extensive because of closer contact with lime. A larger percentage of rapidly flocculated soil in the specimens containing fewer lumps produces an almost immediate strength difference, and larger percentages of lumps will produce lower initial strength.

Increasing the percentage of soil lumps increases the lime content of the matrix material. This increase in lime content will produce more soil-lime contacts and increase initial flocculation of the matrix and lump surfaces. The increase of initial flocculation in the matrix and on the lump surfaces will partially compensate for the unreacted soil within the lumps. The degree of compensation will depend on the gradation or distribution of soil lumps and lime-matrix material.

Similarity in the strengths of specimens containing 20, 40 and 60 percent lumps showed that the amounts of initial flocculation in these specimens were nearly equal. The decreased strength of the sample containing 80 percent lumps indicated a decrease in initial flocculation because of an inferior distribution of soil lump and lime-matrix material.

The relatively equal rates of strength gain for all specimens tested after curing periods of up to 150 days were due to a constant rate of lime-soil reaction rate.

Increased differences between the unconfined compressive strengths of specimens cured over 150 days were due to differences in the distribution of pozzolanic reaction products. Since 6 percent hydrated calcium lime was mixed with the portion of soil passing the No. 4 sieve during the preparation of unconfined compressive strength specimens, the amount of lime in the matrix material at molding varied from 7.5 to 30 percent as the amount of soil as lumps in the specimens varied from 20 percent to 80 percent. Specimens containing a smaller percentage of soil lumps had a better distribution of reaction products and, thus, a higher ultimate strength. A pozzolanic reaction within the soil lumps depended on lime movement, and the quantity of lime movement depended on the distance from a lime source. Since the distance from a lime source in the matrix soil was much less than in the lump soil, more pozzolanic reaction occurred in the matrix soil. The difference in the amount of pozzolanic reaction occurring in the lumps and the matrix increased with the percentage of lumps.

Uniformity of pozzolanic reaction products in a soil-lime-water system containing some soil as lumps would be impossible unless sufficient lime for a reaction with all the clay minerals and other reactants in the system was available. Even then, attainment of uniformity would require a long time.

In verification of this, the percentage of lime (30 percent) present in the matrix material of specimens containing 80 percent of soil lumps before molding was responsible for the formation of calcium aluminate hydrate in a concentration large enough to be detected on X-ray diffraction charts after only 20 days of curing. Increased lime in proximity with soil lump surfaces also resulted in the formation of an X-ray-detectable concentration of calcium aluminate hydrate within the lumps of these samples. X-ray diffraction charts from samples of strength specimens containing a smaller percentage of lime in the matrix material failed to show the presence of calcium aluminate hydrate until after 270 days of curing.

Although calcium silicate hydrate was evidenced only by a halo on X-ray diffraction charts for specimens cured at room temperature, this lime-soil reaction product was undoubtedly formed at the same time as calcium aluminate hydrate. Poor crystallinity prevented a positive identification of this reaction product from X-ray charts.

RATE STUDY

The purpose of the lime migration study was to determine the rate and extent of the penetration of lime into the soil; this was accomplished by pH and calcium ion content determinations described previously.

pH Changes

The results of pH measurements are given in Table 4. To plot the distribution of hydroxyl ions with distance, the pH scale should first be changed to a logarithmic or extended linear scale, since the average distance of fairly thick slices from the lime source does not correspond to the average pH value. Such a transformation, however, magnifies the hydroxyl ion concentration scale out of proportion to the distance scale and makes the distribution curve uncertain in the critical range. To obtain a satisfactory distribution curve, thinner slice samples should have been taken; this, however, was practically impossible.

Table 4 does show a significant change in the hydroxyl ion concentration within soil specimens as the distance away from the lime source increased. The departure from the pH of the natural soil becomes even more evident as the time allowed for diffusion to take place is increased.

The pH appeared to have reached equilibrium in the first ¼ in. of soil away from the lime source at some time between 28 and 42 days. This phenomenon may have been due to the buffering action of clay present in the liquid poured from the top of soil water mixture prepared for pH determination. The apparent pH equilibrium may also have been due to movement of hydroxyl ions out of the first ¼ in. of soil as fast as they were moving in. This would indicate equilibrium of the flocculation reaction and possibly even the pozzolanic reaction in the first ¼ in. of soil.

Calcium Ion Migration

Calcium ion content determinations from slices of the diffusion study samples taken at increasing distances from the calcitic lime source are presented in Figure 3 and Table 5. Results of this determination were plotted in Figure 3 as a bar graph. Lines imposed on the bar graph balance the areas above and below the bars plotted for each slice taken after different times allowed for diffusion. For example, on Figure 3, area a is equal to area b. Theoretically, these lines gave a reasonable indication of the dis-

TABLE 4

RESULTS OF pH MEASUREMENTS AT INCREASING DISTANCES AWAY FROM SOURCE OF LIME[a]

Distance from Lime (in.)	pH						
	0 Days	3 Days	7 Days	14 Days	28 Days	42 Days	63 Days
0-¼	7.00	10.80	11.00	11.15	11.25	11.45	11.45
¼-½	7.00	7.30	7.55	8.60	9.75	10.45	10.85
½-¾	7.00	7.15	7.25	7.30	7.55	7.75	8.80
¾-1	7.00	7.00	7.10	7.15	7.45	7.55	8.00
1-1¼	7.00	7.00	7.20	7.15	7.30	7.50	7.85
1¼-1½	7.00	7.05	7.20	7.10	7.30	7.45	7.85
1½-1¾	7.00	7.00	7.10	7.10	7.30	7.45	7.80

[a] Results average from two slice samples.

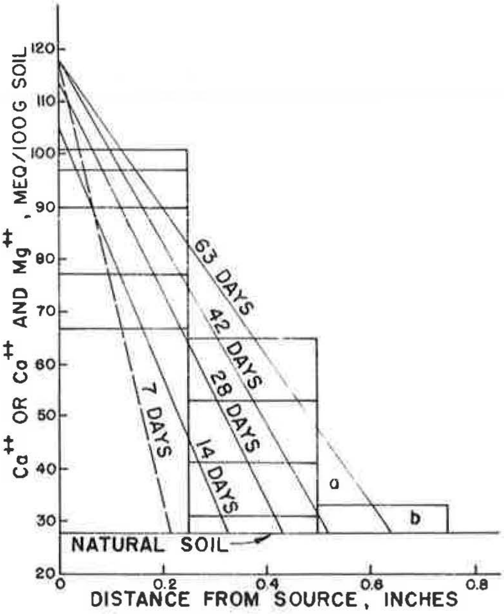

Figure 3. Ca^{++} or Ca^{++} and Mg^{++} ions present in $\frac{1}{4}$-in. slices taken at increasing distances from hydrated calcitic lime source.

tribution of calcium ions within the diffusion samples. The intersection between these imposed lines and the natural soil line was the maximum penetration of a significant amount of lime.

The calcium penetration distance, determined by the intersection of the lines imposed on the bar graph in Figure 3 and the natural soil base line, was plotted vs the square root of time in days (Fig. 4). A straight line through the origin fitted the points quite well and verified the validity of Eq. 3 for the rate of a diffusion-controlled solid-state reaction. The slope of the straight line in Figure 4 was equal to the constant k′. The rate at which lime penetrated this particular soil-water system is defined by the expression

$$l = 0.081\, t^{1/2} \qquad (5)$$

where l equals the lime penetration distance in inches and t equals the time in days.

The application of this rate equation to many other soil-lime water systems is probably valid providing the constant is adjusted. Differences in clay content, clay minerals, density, adsorbed cations, and

TABLE 5

Ca^{++} PENETRATION

Distance from Lime (in.)	Leaching Solution	Leached Cations (meq/100 g dry soil)[a]					
		3 Days	7 Days	14 Days	28 Days	42 Days	63 Days
0-¼	KCl	43.73	48.30	56.53	64.41	72.69	60.25
	HCl	18.86	18.74	21.20	25.78	24.97	41.23
	Total	62.59	67.04	77.73	90.19	97.66	101.48
¼-½	KCl	23.36	23.13	25.62	29.80	34.42	40.17
	HCl	5.15	5.70	5.50	11.70	19.01	25.13
	Total	28.51	28.83	31.12	41.50	53.43	65.30
½-¾	KCl	23.51	23.48	23.21	22.85	23.22	28.87
	HCl	5.02	5.70	3.99	5.20	3.44	4.47
	Total	28.53	29.18	27.20	28.05	26.66	33.34
¾-1	KCl	22.95	24.47	23.48	23.02	22.73	23.17
	HCl	5.01	4.66	4.70	5.02	3.59	4.12
	Total	27.96	29.13	28.18	28.04	26.32	27.29
1-1¼	KCl	23.42	23.64	23.76	23.28	23.33	23.02
	HCl	5.21	4.83	4.52	4.32	3.60	4.12
	Total	28.63	28.47	28.28	27.60	26.93	27.14
1¼-1½	KCl	23.41	24.16	24.94	23.26	23.24	23.02
	HCl	4.83	5.01	4.68	4.86	3.93	3.95
	Total	28.24	29.17	29.62	28.12	27.17	26.97
1½-1¾	KCl	23.80	23.83	24.16	23.54	23.35	23.29
	HCl	4.83	4.49	5.00	4.33	4.09	4.31
	Total	28.63	28.32	29.16	27.87	27.44	27.60

[a] Average results from two slice samples; results for natural soil: KCl, 24.06; HCl, 3.96; total, 28.02.

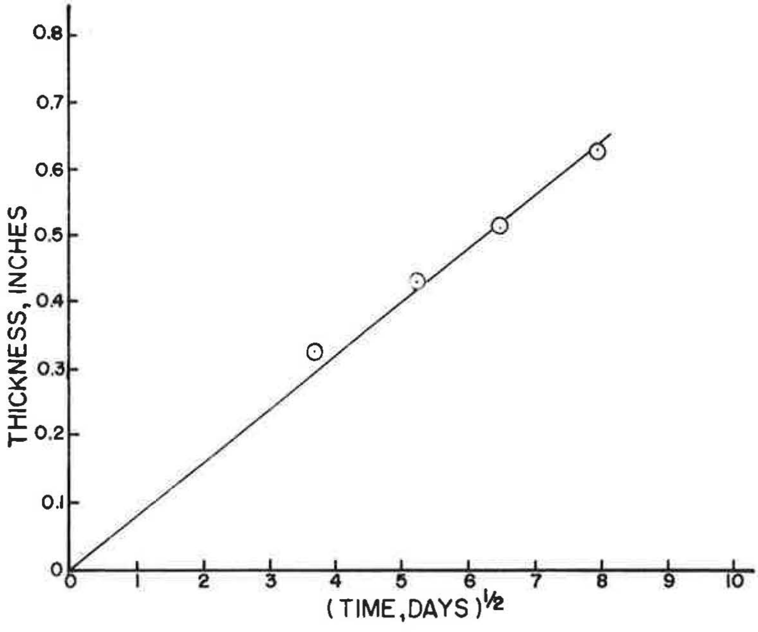

Figure 4. Thickness of lime penetration layer in lime migration study specimens vs square root of time.

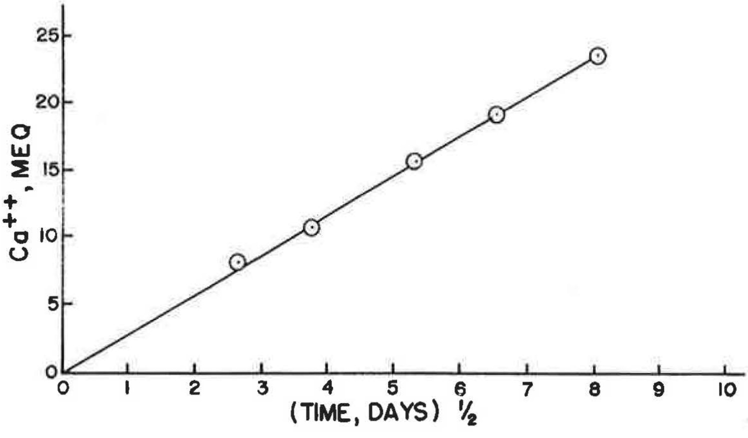

Figure 5. Total amount of calcium moving into 2-in. diameter lime migration study specimens vs square root of time.

temperature will affect the diffusion coefficient involved in the soil-lime reaction rate and will change the value of the constant.

The expression for the rate of lime penetration into a lime-soil water system will apply as long as there is solid lime available for the lime-soil reaction. Only after all lime enters into solution will there be a deviation from this relationship. The amount of lime penetration and pozzolanic reaction occurring after all lime enters into solution, however, will be insignificant, because lime solubility in water is quite low.

Agreement between the rate of lime migration into the specimens used in this investigation and an accepted solid-state reaction rate equation proves that water movement into the specimens was negligible. A small hydraulic head resulting from placement of the lime slurry on top of the compacted soil specimens and a slightly less than 100 percent saturation condition present in the specimens apparently caused no significant water movement.

Figure 5 shows the relation between the total amount of calcium ions which diffused into the migration study specimens with time. The amount of calcium was calculated from the areas under the bars in Figure 3 and the density and dimensions of the specimens. A linear relationship between the total amount of lime diffusing and the square root of time is shown by this plot.

The diffusion coefficient for the particular soil-water system investigated could not be determined from data taken in this investigation. The constant in the rate equation formulated is a function of the change in free energy of the reaction, the diffusion coefficient, the cross-sectional area of the diffusion layer, and a constant. A diffusion coefficient determination would require an investigation of the temperature dependence of the equation.

Plastic Limit Changes

P.L. tests were performed on slice samples from specimens into which migration of lime had occurred for periods of 28, 42, and 63 days. The results shown in Figure 6 are evidence of lime movement in an amount sufficient to increase the P.L. of soil contained in the migration study samples. Although large slice samples made the actual P.L. distribution impossible to determine and the 28-day results are erratic, the P.L. results are generally consistent with the calcium ion determination data. At 63 days there was a significant increase in the P.L. of the soil located at ½ to ¾ in. away from the lime source. The lime penetration distance calculated from calcium ion determination data demonstrated that the lime had diffused approximately 0.64 in. in 63 days.

Pozzolanic Reaction Products

X-ray diffraction patterns of ¼-in slice samples taken from the portions of the lime migration specimens immediately adjacent to the lime sources showed the presence of

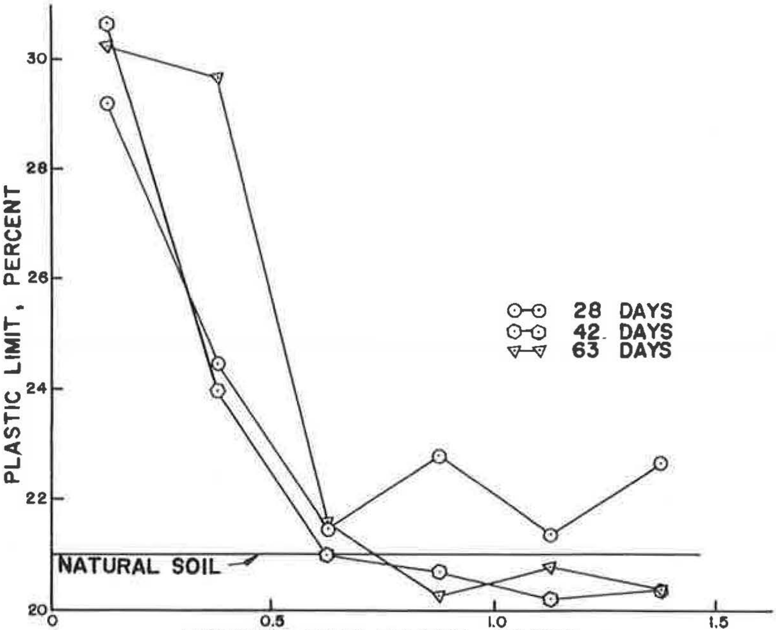

Figure 6. Plastic limit of slice samples taken from lime migration study specimens after various times vs distance from lime source.

calcium aluminate hydrate. All calcium aluminate hydrate peaks at d-spacing of 8.1, 7.6 and 3.9 Å were present on the diffraction charts of two samples studied. One of the samples was from 14-day diffusion specimens and the other was from 63-day specimens.

A ¼-in. slice sample from the 63-day diffusion specimens taken ¼ to ½ in. away also gave weak X-ray diffraction peaks at d-spacing of 8.1 and 7.6 Å. A sample from this pattern of the other diffusion specimens failed to produce these peaks. The actual extent of the pozzolanic reaction product layer in the 63-day specimens was impossible to determine from the X-ray diffraction study, but it is obvious that the layer boundary was somewhere between ¼ and ½ in. away from the lime source.

X-ray diffraction findings and the results of the calcium ion determinations in Table 5 indicate that when the secondary leaching solution of hydrochloric acid removed an amount of calcium ions over 5 to 6 meg/100 g of dry soil from the diffusion study samples, a pozzolanic reaction had taken place. Although a small amount of reaction products was probably dissolved and removed by leaching with potassium chloride, increased calcium ions in the hydrochloric acid leachate is the most logical indication of a pozzolanic reaction. Results of this investigation show that a pozzolanic reaction can be detected by changes in the calcium ion concentration in a secondary hydrochloric acid leachate from a lime-soil mixture which has been previously leached with a potassium chloride solution to remove all dissociated calcium ions.

Increased calcium ions were detected in the hydrochloric acid leachate from the slice samples taken ¼ to ½ in. away from the lime source of the 28-day lime migration specimens. The average pH of these slice samples was 9.75; the average pH of the slice samples nearest the lime source was 11.25. This indicated that a minimum pH of approximately 10.5 was necessary for a pozzolanic reaction, which agrees with previous work by Ho and Handy (19).

CONCLUSIONS

1. Although soil lumps lower the strength of a compacted soil-hydrated calcitic lime-water mixture, the effect diminishes with time as the lumps are stabilized as a result of lime movement in the system. The degree of lump stabilization depends on time and the size of the lumps.
2. The rate of hydrated calcitic lime penetration by diffusion into the particular soil-water system investigated is given by Eq. 5. This expression relating lime penetration and time will hold as long as solid lime is available to the soil-water system. Although an adjustment may be needed in the constant, this expression should be applicable to any similar soil-water-lime systems.
3. Water movement may assist the movement of lime within a soil-water system but is essential only because it provides a medium for lime diffusion.
4. Lime diffusion can occur in sufficient amounts to cause both flocculation and pozzolanic reactions in the soil-water system. The amount of reaction depends on time and the availability of lime. A minimum pH of approximately 10.5 is necessary for pozzolanic reaction.
5. Calcium aluminate hydrates crystallize in a montmorillonitic soil-lime-water system after less than 20 days of curing at room temperature. Calcium silicate hydrates probably are formed at the same time as the calcium aluminate hydrates, but poor crystallinity prevents absolute detection by X-ray diffraction after such a short curing time.
6. Providing no carbonate is present, pozzolanic reactions may be detected from a change in the calcium ion content of the hydrochloric acid leachate from soil-lime-water previously leached free of dissociable calcium ions with a potassium chloride solution.
7. Some soils posing a pulverization problem may be more effectively and economically stabilized with lime if pulverization requirements are relaxed. Rather than specifying a minimum percent passing any given sieve, a specification might indicate a maximum lump size, which could relate to time allowable for complete stabilization.

ACKNOWLEDGMENTS

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REFERENCES

1. Lime Stabilization Construction Manual. ARBA Tech. Bull. No. 243, 1934.
2. Hydrated Lime Stabilizes a Clay Fill in Florida. Eng. News-Record, Vol. 172, No. 20, pp. 40-42, 1964.
3. Pulverization Characteristics of Soils. Soil Cement News, No. 35, p. 4, 1950.
4. Investigations of Quicklime as a Stabilizing Material. U. S. Dept. of Commerce, Tech. Rept. No. 3-455, March 1962.
5. Glasstone, S., Laidler, K. J., and Eyring, H. The Theory of Rate Processes. New York, McGraw-Hill, 1941.
6. Lai, T. M., and Mortland, M. M. Self-diffusion of Exchangeable Cations in Bentonite. Proc. 9th Nat. Conf. on Clays and Clay Minerals, Vol. 9, pp. 229-247, 1960.
7. Barber, S. A. A Diffusion and Mass-Flow Concept of Soil Nutrient Availability. Soil Sci., Vol. 93, pp. 39-49, 1962.
8. Jenny, H., and Overstreet, R. Surface Migration of Ions and Contact Exchange. Jour. Phys. Chem., Vol. 43, pp. 1185-1196, 1939.
9. Gast, R. G. An Interpretation of Self-diffusion Measurements of Cations in Clay-Water Systems. Jour. Colloid Sci., Vol. 17, pp. 492-500, 1962.
10. Low, P. F. Viscosity of Water in Clay Systems. Proc. 8th Nat. Conf. on Clays and Clay Minerals, Vol. 8, pp. 170-181, 1959.
11. Porter, L. K., Kemper, W. D., Jackson, R. D., and Stewart, B. A. Chloride Diffusion in Soils as Influenced by Moisture Content. Soil Sci. Soc. of Amer. Proc., Vol. 24, pp. 460-472, 1960.
12. Barrer, R. M. Diffusion in and Through Solids. New York, Macmillan, 1941.
13. Cohn, G. Reactions in the Solid State. Chem. Rev., Vol. 42, pp. 427-429, 1948.
14. Pole, R. G., and Taylor, N. W. Kinetics of Solid-Phase Reactions of Certain Carbonates with Mullite, Silica, and Alumina. Jour. Amer. Ceram. Soc., Vol. 18, pp. 325-337, 1935.
15. O'Flaherty, C. A., Edgar, C. E., and Davidson, D. T. Iowa State Compaction Apparatus for Measurement of Small Soil Samples. Highway Research Record No. 22, pp. 48-63, 1963.
16. Glenn, G. R. X-Ray Studies of Lime-Bentonite Reaction Products. Ph.D. thesis. Library, Iowa State Univ., 1963.
17. Glenn, G. R., and Handy, R. L. Lime-Clay Mineral Reaction Products. Highway Research Record No. 29, pp. 70-82, 1963.
18. Hilt, G. H., and Davidson, D. T. Lime Fixation in Clayey Soils. Highway Research Board Bull. 262, pp. 20-32, 1960.
19. Ho, C., and Handy, R. L. Characteristics of Lime Retention by Montmorillonitic Clays. Highway Research Record No. 29, pp. 55-70, 1963.

Discussion

ROBERT M. NADY, Consulting Engineer and Associate Professor, Iowa State University.—The following deals with an application of coarse soil pulverization for soil-lime stabilization. Early in 1964, a problem arose during the design of the pavement for the Mount Pleasant, Iowa, airport. Preliminary design estimates indicated that a soil-lime subbase would be the logical selection. The existing technology of lime stabilization in

Iowa seemed to be largely based on the techniques employed for portland cement stabilization. Prevalent specifications required that 60 to 80 percent of the pulverized material pass the No. 4 sieve, exclusive of any gravel-sized particles present in the mass.

The problem centered around soil pulverization. The soils in the Mount Pleasant region are montmorillonitic with clay-sized fractions in the 25 to 35 percent range. Pulverizing such plastic soils finer than about 1-in. lumps is expensive and time consuming, especially if the soil moisture is high. Therefore, the use of lime as a stabilizer depended on modification of the usual requirements for soil pulverization.

Design

In February 1964, I was retained by a consulting engineering firm to participate in the design of the airport pavement. Several soil samples secured from the site on Feb. 28, 1964, were tested in the laboratory. Though extensive sampling was done, two samples of materials were from sources which would lie in the subbase or upper subgrade of the pavement structure. Properties of these soil materials are given in Table 6.

Based on these subgrade soil properties, a design wheel load of 10,000 lb, and the availability of construction materials in the vicinity of the site, the pavement cross-section was selected as follows:

2 in. asphaltic concrete surface,
4 in. asphaltic concrete base,
6 in. subbase, and
Prepared subgrade.

Lime Tests, Laboratory

Standard subbase construction makes use of the existing soils improved by the addition of granular material. It was felt that a greater degree of improvement than normally realized from granular treatment would be desirable for this project. Experience with granular-stabilized montmorillonitic soils showed that shrinkage and frost susceptibility were modified but not eliminated.

The benefits derived from lime treatment of heavy Iowa soils has been established on several highway projects in the state. The improvement in quality of subgrade soils by the use of lime for both airports and highways has been well documented in other states. Because of the nature of the construction materials in the vicinity of the airport site, it was assumed that lime stabilization of the subgrade

TABLE 6

PROPERTIES OF SUBBASE SOILS

Property	Sample	
	1	2
Color	Dark gray	Dark yellow brown
Depth (ft)	0-2	2-4½
FAA classification	E-10	E-10
% sand, No. 10-270	3.5	5.0
% silt, No. 270-5μ	64.4	67.0
% clay, < 5μ	32.1	28.0
Atterberg limits		
L. L. (%)	67.0	60.1
P. I. (%)	39.8	34.9
CBR, 4-day soaked, 0.1-in. pene.	2.8	3.3
Expansion (%)	4.7	3.3
Moisture increase (% dry wt.)	5.1	3.0

TABLE 7

IMPROVEMENT OF LIME-TREATED SOIL NO. 1

Property	Soil Values	
	Untreated[a]	Treated[b]
CBR, 4-day soaked, 0.1 in.	2.8	45.4
Expansion (%)	4.7	1.9
Moisture increase (% dry wt.)	5.1	3.0

[a] Standard pulverization.

[b] 3 percent lime, 1-in. lumps, approximately 20 percent passing No. 4 sieve.

soil, rather than granular improvement or total replacement, would provide an economical subbase having the desired properties. A detailed cost estimate proved this assumption to be correct, and soil-lime was selected for further study.

As noted earlier, pulverization of heavy clay soils has proved to be a difficult requirement. Current practice on state highway projects requires that from 60 to 80 percent of the processed mixture, exclusive of gravel-sized material, pass the No. 4 sieve. Contractor experience shows that this requirement is sometimes impossible to achieve, even with lime pretreatment of the soil, and it is time consuming and expensive. Therefore, lime appeared to be the logical stabilizing agent. But for construction purposes, a different concept of the degree of pulverization of the mass was necessary.

About this time, preliminary data from the work of Davidson, Demirel and Handy became available. The data indicated good results with lime-soil stabilization with the soil pulverized to a much lesser degree than required by current practice. The migratory effect of the lime reaction was under study, and the early results were positive. The trade literature also contained articles dealing with drill-lime stabilization of soft subbases under existing pavements. Although all the information pointed to favorable results with soil-lime in which a large proportion of soil lumps remained undivided, a laboratory test of the soil from this project site was conducted.

Since soil No. 1 was the weaker of the potential subbase soils (Table 6), it was selected for lime treatment. The supply of the sample was limited, however, and only one specimen could be prepared. Since the CBR values of the untreated soil had already been determined, this method of evaluation was also used for the lime-treated test. Other investigators working with similar soils found that about 3 percent lime gave significant improvement, so this lime content was selected.

The soil was prepared by breaking or cutting it into lumps of 1 in. or less in size. Although no sieve analysis was performed on the material, about 20 percent would probably have passed a No. 4 sieve. Three percent calcitic hydrated lime was added to the prepared soil, and moisture was added to bring the mixture to approximately optimum moisture content. The mixture was lightly turned over with a large spoon in a mixing bowl for a few seconds. Mechanical mixing was ruled out to avoid degradation of the soil lumps. The loose mixture was allowed to stand for 48 hr at 95 percent R.H., simulating a 48-hr field cure if lime pretreatment were used. The mixture was then compacted in a CBR mold using the standard equipment, methods, and sequences for the CBR test. Following the 4-day soaking period, the CBR test was run. The results given in Table 7 show a most gratifying degree of improvement. The test data, together with other available information, formed the basis from which the Project Special Provisions were drawn.

Project Special Provisions

The soil-lime subbase specifications were drawn along the usual lines for the sections covering the description of the work, the materials of construction, and the construction equipment. The section on construction methods reflects the work of Davidson, Demirel, and Handy, as well as the tests results given in Table 7. It is quoted, in part, from the Project Specifications as follows:

> Mixed In-Place. Soil-lime subbase shall be constructed from subgrade soil to the width and thickness shown on the plans. The surface of the subgrade, prior to subbase construction, shall comply with . . . of this section.
>
> Either bagged or bulk lime may be used except that bagged quicklime shall not be permitted. If bulk lime is used, approved spreading equipment shall be provided. If bagged lime is used, the manufacturer's bag weights shall be accepted. Bags shall be placed in a uniform and regular pattern providing the specified quantity of treatment. Spread lime that has been displaced prior to mixing shall be restored. Lime shall not be spread when the layer to be processed or the subgrade is frozen or when the air temperature is less than 40° F.

> The surface of the subgrade shall be scarified to a depth necessary to provide the quantity of material, which when combined with the specified quantity of lime, will produce a compacted subbase of the thickness designated on the plans.
>
> The scarified soil shall be processed with approved graders, discs, harrows, or rotary mixers. Either of the following construction sequences may be directed by the engineer, depending on the moisture content of the subgrade soil at the time of construction.
>
> Sequence 1. For the condition of moderate to high moisture content of the subgrade soil, the scarified soil shall be processed into chunks 3 inches or less in diameter. Upon the surface of the soil, one-half of the specified amount of lime shall be spread. The lime shall be cut in with mixing equipment. The surface of the layer shall then be lightly rolled with a pneumatic roller to effect a surface seal and to prevent the entrance of surface water. The layer shall be left undisturbed for a period of not less than 24 hours and not to exceed 72 hours.
>
> After the passage of the required curing period, the remaining one-half of the lime shall be spread. Mixing shall continue until the lumps of soil mixture are reduced in size to $1\frac{1}{2}$ inches diameter, or less. If required, water shall be added during mixing to obtain and maintain optimum moisture content, plus or minus 2 percent. Following the mixing, compaction and finishing . . . shall proceed. Final compaction shall be completed with 48 hours of final mixing.
>
> Sequence 2. For the condition of low to moderate moisture content in the subgrade, the scarified soil shall be processed into lumps $1\frac{1}{2}$ inches or less in diameter. The total amount of lime as specified shall be spread on the prepared material. Mixing of the soil and lime shall proceed until a uniform mixture is obtained. If required, water shall be added during mixing to obtain and maintain optimum moisture content, plus or minus 2 percent. Following the mixing, compaction and finishing . . . shall proceed. Final compaction shall be completed within 48 hours of final mixing. . . .

Construction of Project Subbase

Subbase processing started on Oct. 12, 1964, and was substantially completed by Oct. 20, 1964. The contractor had no difficulty in meeting the requirements of the specifications. Since the weather was dry and mild throughout the period, Sequence 2 was used. Within the specification limits and the capabilities of the equipment on the job, the contractor requested approval of, and was permitted to try, different orders of mixing and adding lime and water as well as different lime spreading techniques. Although the contractor selected the optimum order for his particular equipment spread, there appears to be little difference between the various construction segments, insofar as the quality of the completed subbase is concerned. Some construction processes are shown in Figures 7 to 15.

Field Testing.—The project was inspected continuously during construction. On seqments where the full depth was processed at one time, a minimum of three passes of the rotary mixer was required to effect complete mixing and the reduction of lumps to $1\frac{1}{2}$-in. or less diameter. Although it was not a specification requirement, testing for the amount passing the No. 4 sieve gave a result of 36 percent for one test and a slightly higher value for an additional test. For the whole project, the fraction passing the No. 4 sieve was probably in the range of 30 to 40 percent.

The Soil Research Laboratory, Iowa Engineering Experiment Station, Iowa State University, conducted plate bearing tests at selected locations on the prepared subgrade and on the completed subbase. The field data was reduced to load/deflection information in appropriate units and is presented in that form in Table 8. It will be noted that the lime treatment has materially increased the bearing capacity of the soil. A given load causes much less deflection of the plate on treated soil than on untreated soil.

Figure 7. Unloading and spotting bagged lime.

Figure 8. Opening bags of lime.

Figure 9. Spreading bagged lime with motor grader.

Figure 10. Spreading lime with drill spreader.

Figure 11. Spreading bulk lime with a long sweep discharge elbow attached to bulk transport; only a slight amount of dusting and air-borne lime.

Figure 12. Completed spread from one load of bulk lime directly down runway edge (note uniformity of spread).

Figure 13. Mixing soil-lime with rotary mixer.

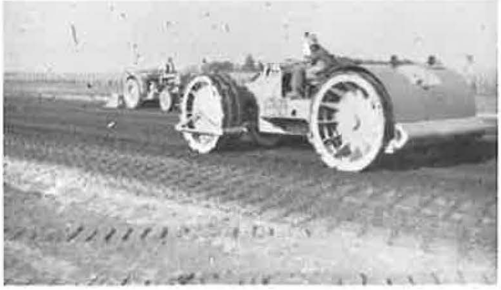

Figure 14. Compacting soil-lime with segmented roll static compactor.

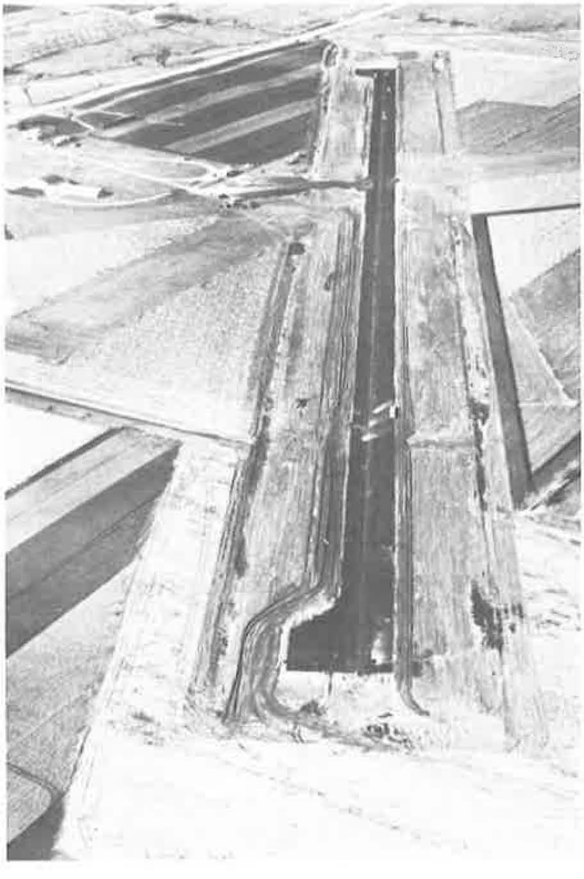

Figure 15. Airstrip with subbase, MC-0 cure, and asphaltic concrete base.

The values of deflection of 0.05 and 0.10 in. were selected for reporting in Table 8, although plate bearing tests were made taking deflection readings up to 0.60 in. in one case, with the maximum readings generally in the range of 0.20 to 0.40 in. Corresponding unit loads ranged generally from 65 to 80 psi at maximum deflection.

Anticipated Performance.—To relate the plate bearing test to performance under design load, data for various aircraft were accumulated. Since no design contact pres-

TABLE 8

RESULTS OF PLATE BEARING TESTS, MOUNT PLEASANT, IOWA, AIRPORT RUNWAY[a]

Location	Applied Load (psi)			
	At 0.05-In. Deflection		At 0.10-In. Deflection	
	Subgrade	Subbase[b]	Subgrade	Subbase[b]
Sta. 6+00, centerline	17.7	—	30.5	—
Sta. 10+00, centerline	17.5	26.0	27.4	43.4
Sta. 15+00, centerline	21.8	29.2	38.2	53.7
Sta. 20+00, right 5 ft	19.5	34.0	38.8	60.4
Sta. 25+00, left 10 ft	26.0	27.4	45.9	52.7
Sta. 30+00, centerline	—	26.0	—	45.0
Average	20.5	28.5	36.2	51.0

[a] Using 12-in. diameter plate.
[b] Of 6-in. soil-lime.

TABLE 9

LOAD AND PRESSURE DATA FOR AIRCRAFT

Aircraft	Fully Loaded Gross Wheel Load (lb)	Tire Contact Area (sq in.)	Contact Pressure (psi)	Pressure at 6-In. Depth (psi)	Pressure at 12-In. Depth (psi)
Aero Commander	3,000	153	19.6	5.7	2.6
Beech Twin Bonanza	3,000	153	19.6	5.7	2.6
Beech 18	4,650	171	27.2	8.2	3.9
Cessna 407 (jet)	3,725	171	21.8	6.6	3.1
Cessna 310	1,930	117	16.5	4.2	1.9
Fornaire Ercoupe	600	90	6.7	1.5	0.6
Mooney	1,300	108	12.0	3.0	1.3
Piper Apache	1,600	126	12.7	3.3	1.5
Navion	1,160	108	11.0	2.7	1.2
Luscombe Silvaire	600	90	6.7	1.5	0.6
DC-3	11,800	238	49.6	17.7	8.8
Convair 340	21,306	304	70.1	27.0	14.2
DC-6	46,300	468	99.0	44.5	25.2

sure criteria are used, the design wheel load of 10,000 lb is approached and exceeded by the aircraft wheel load data. Although not every craft of this size range is represented, the information includes typical light private, commercial, charter, and training planes of types most likely to use the runway. Also, the listing includes multiengine transports in commercial usage in this area (Table 9) for comparative purposes.

Data of Tables 8 and 9 show that under the field conditions at the time of the plate bearing tests, the pavement structure could successfully carry a fully loaded DC-3 and Convair 340 without exceeding the limiting deflection of 0.05 in. A fully loaded DC-6 would cause deflections greater than 0.05 in. but less than 0.10 in. The writer has observed overloaded flexible pavements deflecting as much as $^3/_8$ to $^1/_2$ in. without detectable damage. These large deflections and excessive loads were not repetitive in nature, however, but could be compared to emergency use of the airport by heavy craft.

Table 9 data were developed for a circular tire contact area and a 45° cone angle of stress distribution downward from the surface. An elliptical or oval-shaped contact area, more nearly like an aircraft tire print, would give a greater area of stress spread at the subbase and subgrade depths in the structure. Further, the 45° spread angle enjoys considerable support when applied to layers of granular material. But little is known regarding distribution of stress through layers of bituminous base or lime-stabilized soil. One study currently in progress shows that stresses transmitted through a 6-in. layer of bituminous concrete are about half the level of stresses transmitted through a 6-in. layer of compacted crushed stone. The stresses were distributed over a correspondingly larger area under the bituminous concrete.

Considering the shape of the contact and stress distribution areas, a safety factor between 3 and 4 seems to be operating. However, the plate bearing tests were made during mid-October in 1964 at a time when the soil moisture was low and the subgrade was stable. After the spring thaw, the subgrade may be soft. Based on the earlier CBR tests (Table 6) the subgrade soil will have a subgrade reaction (k) value of about 100 pci when saturated. At a k value of 100 pci, a unit load of 5 psi on the subgrade would produce a deflection of 0.05 in. The subgrade stress load produced by the DC-3 of 8.8 psi thus exceeds the limiting load (5 psi) by about 75 percent. This causes a reduction in the safety factor from the range of 3 to 4 to a range of 1.7 to 2.3. Since the DC-3 wheel load is about 10 percent greater than the design wheel load, a modest safety factor exists under poor subgrade conditions for the design wheel load of 10,000 lb.

Conclusions

Based on the data contained here, site inspection during construction, and all other available data and observations, the following conclusions are drawn:

1. The research data developed and reported here for coarse lump soil pulverization in soil-lime mixtures was verified in a full-scale airport construction project.
2. Pulverization requirements of the project specifications were readily attainable in the field with typical construction equipment and standard methods of processing.
3. The Federal Aviation Agency procedure for thickness design of flexible airport pavements embodies a modest but realistic factor of safety.
4. Plastic soils, which are generally subject to improvement by lime treatment but are difficult to pulverize to a high degree, can be successfully treated with lime if current pulverization requirements are relaxed.
5. Laboratory design testing can be limited to a determination of the susceptibility of a soil to improvement by lime treatment. If a soil reacts favorably, the presence of lumps up to $1\frac{1}{2}$ in. in diameter, and probably greater, will cause but little difference in this determination.
6. Project operational sequence, within the limits of the specifications presented herein, produces no detectable difference in the completed layer. Capabilities of a given equipment spread on a given project can thus be optimized.
7. A new approach to lime treatment of heavy soils is opened. As the use of this technique increases, further economies will be realized.

Acknowledgments

This author is indebted to many whose knowledge and data are summarized in this report or who otherwise assisted in its preparation and presentation. This opportunity is used to thank them all, in particular the City Council of Mount Pleasant, Iowa; Charles Hanna, Chairman of the Mount Pleasant Airport Committee; Donald E. French, general project consultant; William Perry; Ames Engineering and Testing Service and Neil A. Carpenter; Iowa Engineering Experiment Station, the authors of the research paper, Prof. J. M. Hoover, and their associates; the Federal Aviation Agency, Lincoln, Neb., office, and Dean Tatum and Robert Smith; and the Norris Construction Co. and Dick V. McGowen.

L. K. DAVIDSON, T. DEMIREL, and R. L. HANDY, Closure.—The only comment we would make is to repeat a remark overheard in the audience at the time of Professor Nady's presentation: "It takes guts." Additional reflection on the matter forces us to agree. Engineering, particularly when it employs unconventional methods, does "take guts," and we in teaching and research tend to overlook the "guts" involved in getting the technical paper parade into practice. When it is done, it is most gratifying.