

HIGHWAY RESEARCH RECORD

Number 139

Behavior Characteristics
of
Lime-Soil Mixtures

5 Reports

Subject Classification

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Foreword

Three of the papers in this RECORD deal with practical methods of selecting the amount of lime to be used in lime-soil stabilization. Thompson, McDowell, and Eades and Grim all present methods which can be put into use immediately. Design engineers and soils engineers have an excellent opportunity to compare these three methods and select the one which is most suitable for application within their own areas.

Thompson presents the results of a study which will be of high interest to design engineers. He provides a means for evaluating strength characteristics which allows a decision as to whether or not the proposed layer of the stabilized mixture will give the structural strength necessary for its position in the pavement system. The importance of resistance to flexural stresses is pointed out. This consideration may play the most important part in the rational design and use of lime-soil mixtures. Shear strengths of the mixtures were found to be generally sufficient to prevent shear failure so that flexural stress will generally control.

McDowell makes a major contribution, interlacing the benefits of field experience with laboratory research to evolve a chart which can be used to obtain a recommended amount of lime to use in stabilization of subgrades and bases. The chart is not applicable to materials having less than 10 percent passing the No. 40 sieve or to material with a P. I. of less than 3. Research findings give very strong indications that the lime should be relatively pure (90 percent or more Ca or Mg hydroxides) and relatively fine (85 percent or more passing No. 200 sieve). The rehealing properties of some lime-soil mixtures are also pointed out.

Eades and Grim present a "quick test" for determining the amount of lime required to stabilize the soil. This test could be very useful in the field for construction control and in dealing with variations in soils and the attendant changes in lime requirements. Construction control could be greatly improved, resulting in a more economical and efficient use of lime as a stabilizing material.

Townsend and Klym make an important contribution in their study of the effects of frost action on the durability of lime-soil mixtures. They point out the importance of the cementing action by the complex silicate being strong enough to resist the disruptive pressure caused by water being frozen in the larger pore spaces. A utilization of these findings leads toward rational field construction controls which could be instituted immediately in order to obtain for the mixture a better opportunity to gain the necessary strength to resist the disruptive pressure.

Ruff and Ho also add to the basic knowledge of the reactions taking place in lime-soil mixtures. This new knowledge, in opening the way to a better understanding of the reactions, may logically lead to developments that could result in greatly improved strength and durability. This greater understanding of the time-temperature-strength relation will lead to improved specifications for quality control of the construction process.

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Shear Strength and Elastic Properties of Lime-Soil Mixtures

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•THE SHEAR strength properties of a material are essential for the rational analysis and design of a flexible pavement structure. McCleod (1), Hewitt (2), and others have used shear strength parameters for evaluating the load-carrying capacity of flexible pavements. Many flexible pavement design procedures are based on the shear strength properties of the pavement materials. The Texas and Kansas triaxial design techniques and procedures are among the most renowned. McDowell (3) has shown excellent correlation between the Texas triaxial procedures and the performance of the flexible test sections at the AASHO Road Test.

If lime-soil mixtures are to be effectively used as paving materials, it is necessary to develop information concerning their shear strength properties. This information is required for determining the structural behavior of the material in addition to its usefulness in triaxial design procedures.

INVESTIGATION

The objectives of this study were (a) to investigate the shear strength and elastic properties of typical lime-soil mixtures, (b) to evaluate the influence of soil type and curing time on these properties, and (c) to determine the relationships, if any, between shear strength properties and unconfined compression test results.

Materials

Four typical Illinois soils were used in the testing program. Previous laboratory experience with these soils indicated that they reacted very satisfactorily with lime. These soils display a wide range of such properties as clay content, mineralogy, and plasticity. Selected soil properties, the lime treatments used, and the percent of dry weight of soil, are given in Table 1.

A commercially produced, high-calcium hydrated lime was used throughout the study. The lime contained 96 percent Ca(OH)_2 , and 95 percent passed the No. 325 sieve.

Specimen Preparation

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

Compaction.—All specimens were compacted at approximately optimum moisture content as determined by a moisture-density test. This test was conducted in a manner similar to AASHO T99-57 except that 2-in. diameter by 4-in. high 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 Method A of AASHO T99-57 test for moisture-density relations of soils. Optimum moisture contents and maximum dry densities for the various soils and lime-soil mixtures are given in Table 2.

Paper sponsored by Committee on Lime and Lime-Fly Ash Stabilization and presented at the 45th Annual Meeting.

TABLE 1
SELECTED SOIL PROPERTIES

Soil	AASHO Class.	<2 μ Clay (%)	L. L. (%)	P. I. (%)	Carbonates (%)	Predom. Clay Mineral	Lime Treat. (%)
Bryce B	A-7-6 (18)	52	53.1	28.8	NC ^a	Illite	5
Illinoian till	A-6 (6)	14	25.5	11.0	18.6	Illite	3
Fayette C	A-6 (8)	21	31.9	10.1	20.0	Montmorillonite	5
Wisconsinan loam till	A-4 (8)	18	24.5	7.8	13.8	Illite	3

^aNoncalcareous.

TABLE 2
COMPACTION PROPERTIES

Soil	Lime (%)	Max. Dry Density (pcf)	Opt. Moist. (%)
Bryce B	0	101.9	20.5
	5	97.3	25.8
Illinoian till	0	125.3	11.3
	3	121.0	13.0
Fayette C	0	110.5	18.1
	5	108.6	18.3
Wisconsinan loam till	0	122.4	12.0
	3	120.0	12.0

A series of sixteen 2-in. diameter by 4-in. high specimens was prepared for each test condition, i. e., lime percentage, 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.

Curing.—After compaction, trimming, and extrusion, the specimens were placed in 1-gal metal cans, and the can lids sealed with Perma-Tex. The lime-soil specimens were then placed in a 120 F curing cabinet for periods of 1, 2, 4, or 6 days. These curing conditions produce strengths that are comparable to those developed under field curing conditions (4).

The natural soil specimens were allowed to cure for 7 days at ambient room temperature to allow for thixotropic effects.

Testing Procedures

At the termination of the prescribed curing period, the specimens were allowed to cool to room temperature and then tested in triaxial compression (unconsolidated-undrained test).

The 16 specimens were divided into four equal-sized groups and one specimen from each group was selected at random to be tested at a given air-confining pressure (σ_3) of 0, 5, 15, or 35 psi. The samples were loaded at a constant rate of deformation of 0.05/in./min., and periodic readings of applied load and total deformation of the specimen were recorded until maximum load was achieved.

Samples taken from representative specimens following testing indicated that the specimens were at approximately the same moisture content as at the time of preparation.

Determination of C and ϕ

The cohesion intercept, C, and angle of shearing resistance, ϕ , were determined statistically by a process used by Herrin (5). The procedure is described below.

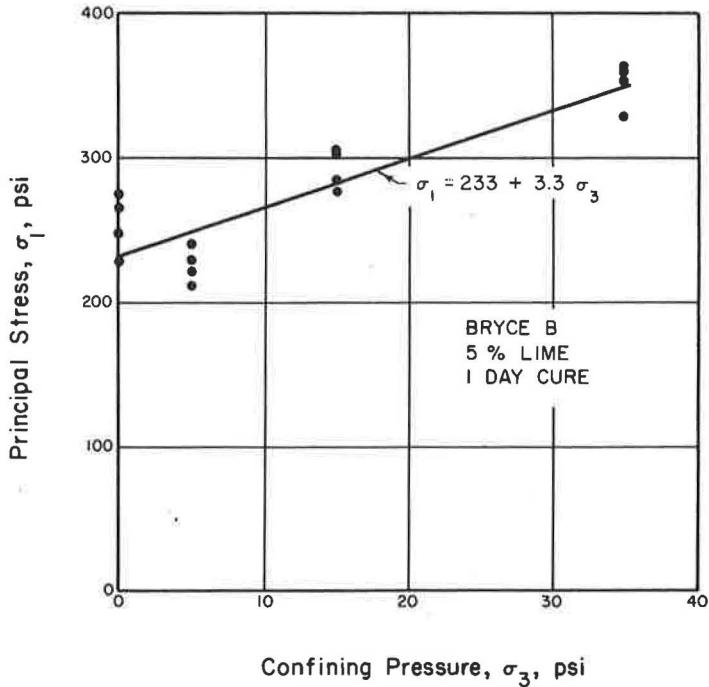


Figure 1. Influence of confining pressure on maximum principal stress.

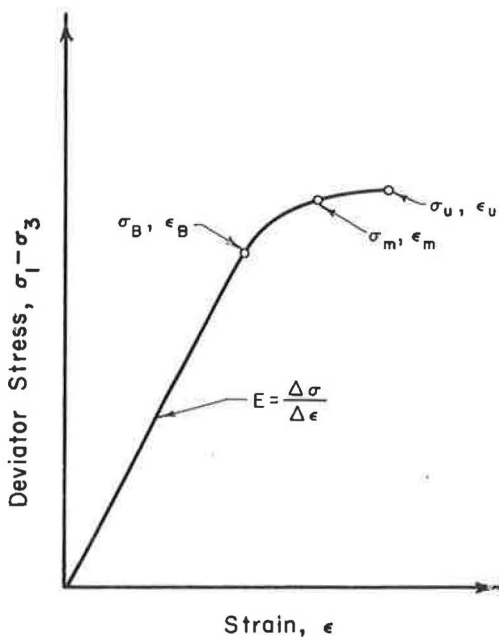


Figure 2. Composite stress-strain curve.

1. A plot of σ_1 , maximum normal stress applied, vs σ_3 , confining pressure, was prepared (Fig. 1).

2. The best least-squares regression equation for the data was determined. The equation (Fig. 1) was of the form $\sigma_1 = a + b \sigma_3$.

3. C and ϕ were calculated from the following relationships:

$$C = \frac{a}{2\sqrt{b}} \quad \sin \phi = \frac{b-1}{b+1}$$

Evaluating Elasticity Properties

A composite or average stress-strain relationship was established for each series of four lime-soil specimens tested at a given confining pressure, σ_3 . The composite relation (Fig. 2) was developed using the technique described in the appendix.

ANALYSIS OF TEST RESULTS

Shear Strength Properties

Pertinent shear strength properties for the natural soils and the lime-soil mixtures included in the investigation are

TABLE 3
SHEAR STRENGTH DATA

Soil	Lime (%)	Shear Strength														
		7 Days at 73 F			1 Day at 120 F			2 Days at 120 F			4 Days at 120 F			6 Days at 120 F		
		C (psi)	ϕ	E (ksi) ^a	C (psi)	ϕ	E (ksi) ^a	C (psi)	ϕ	E (ksi) ^a	C (psi)	ϕ	E (ksi) ^a	C (psi)	ϕ	E (ksi) ^a
Bryce B	0	39	11.5	5.70	64	32.5	29.0	158	13.4	54.0	178	32.7	81.3	159	21.2	73.0
Illinoian till	5	60	13.0	8.60	94	34.2	28.4	113	35.5	63.5	129	32.1	56.7	250	27.0	87.5
Fayette C	3	22	17.6	6.50	108	14.5	52.5	93	37.0	63.0	149	43.7	115.0	386	14.2	160.0
Wisconsinan loam till	0	48	7.5	5.25	105	25.0	48.0	139	13.0	58.3	184	27.4	77.8	164	36.8	91.9
	3															

^aE computed at 15 psi confining pressure.

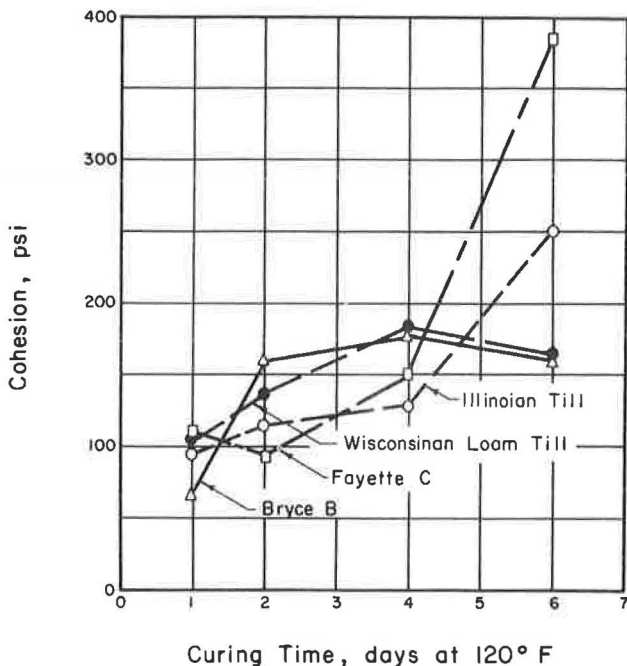


Figure 3. Influence of curing time on cohesion of lime-soil mixtures.

TABLE 4
COHESION (C)-UNCONFINED COMPRESSIVE STRENGTH (q_u)
LINEAR REGRESSION ANALYSIS SUMMARY (Analysis of Variance^a)

Source of Variation	Degrees of Freedom	Sum of Squares	Variance	F
Total	15	87,145		
Regression	1	77,089	77,089.0	107.3 ^b
Residual	14	10,056	718.3	

^aRegression equation: $C \text{ (psi)} = 9.3 + 0.292 q_u \text{ (psi)}$.
^bSignificant at $\alpha = 0.01$.

given in Table 3. The lime treatment greatly improved the shear strength of the soils.

Substantial cohesion increases were obtained with every soil, the magnitude of the increase depending on the soil type and length of curing period. For the 1-day curing periods, the cohesion values of the lime-soil mixtures were 1.5 to 5 times larger than the cohesion of the untreated soils. Longer curing periods generally

produced further increases in cohesion (Fig. 3). Cohesion values for the 6-day curing period varied from 159 to 386 psi.

Statistical analysis of the unconfined compressive strength and cohesion values for the lime-soil mixtures indicated a highly significant linear regression relation (Table 4). The regression equation, $C \text{ (psi)} = 9.3 + 0.292 q_u \text{ (psi)}$ is shown in Figure 4.

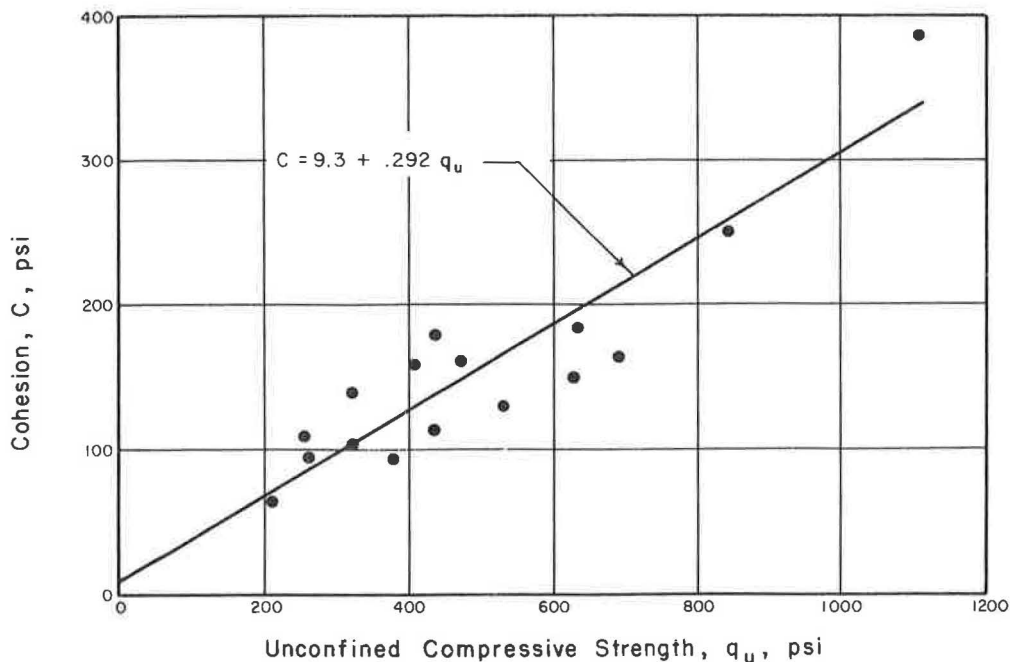


Figure 4. Cohesion vs unconfined compressive strength of lime-soil mixtures.

In every case except two, the cured lime-soil mixtures displayed a larger angle of shearing resistance, ϕ , than the untreated soil. The majority of the ϕ values for the lime-soil mixtures ranged from 25 to 35 deg. The data do not indicate any apparent trend between ϕ and curing time or soil type.

It is very difficult to evaluate ϕ effectively for materials such as lime-soil mixtures, which are characterized by very high cohesion. In this study, the cohesion was so large that small specimen-to-specimen variations in cohesion may have negated the effect of the low confining pressures, 0 to 35 psi, used. More realistic determinations of ϕ would require extremely high confining pressures for eliminating the influence of cohesion variations. Considering the very high cohesion of lime-soil mixtures and the low confining pressures normally assumed to exist in a flexible pavement structure, extremely elaborate high-pressure triaxial tests to obtain a more precise determination of ϕ may not be justified.

Elasticity Properties

A summary of pertinent stress-strain properties (E , ϵ_B , ϵ_u , σ_B , σ_u) for the natural soils and the lime-soil mixtures is given in Table 5. Figure 5 shows typical stress-strain curves.

The modulus of elasticity, E , of the cured lime-soil mixtures ranged from 28,000 to 160,000 psi, and the range was from 5,200 to 8,600 psi for the natural soils. Longer curing periods for the lime-soil mixtures generally produced an increase in E (Fig. 6). Increased confining pressure had little effect on E , the value remaining essentially constant for all magnitudes of confining pressures used.

The increase in E with longer curing periods is similar to the strength vs curing-time relation for lime-soil mixtures. Figure 7 shows the relation between E determined for a confining pressure of 15 psi and the unconfined compressive strength of the lime-soil mixtures. Linear regression analysis of the data indicated a highly significant regression between the variables (Table 6). The regression equation, E (ksi) = 9.98 + 0.1235 q_u (psi) is shown in Figure 7.

TABLE 5
SUMMARY OF STRESS-STRAIN PROPERTIES

Soil	Lime (%)	Curing (days at 120 F)	Confining Pressure (psi)	E (ksi)	ϵ_B (%)	ϵ_u (%)	σ_B (psi)	σ_u (psi)	α_B/σ_u
Bryce B	0	- ^a	0	4.9	1.24	3.04	61	93	—
			5	4.8	1.73	3.58	83	104	—
			15	5.7	1.30	3.41	74	108	—
			35	5.0	1.92	3.12	96	110	—
	5	1	0	27.5	—	1.00	—	210	—
			5	31.1	0.65	1.08	203	254	0.80
			15	29.0	0.73	1.34	211	276	0.76
			35	30.1	0.76	1.59	229	318	0.72
	5	2	0	62.8	0.50	0.89	313	410	0.76
			5	57.6	0.47	1.03	273	403	0.68
			15	54.0	0.53	1.02	288	401	0.72
			35	59.6	0.53	0.96	315	428	0.74
	5	4	0	72.5	0.41	0.86	300	441	0.68
			5	64.6	0.52	0.90	333	440	0.76
			15	81.3	0.41	0.95	335	491	0.68
			35	71.3	0.50	1.16	354	520	0.68
	5	6	0	93.6	0.37	0.68	348	473	0.74
			5	80.1	0.41	0.79	328	461	0.71
			15	73.0	0.46	0.93	335	470	0.71
			35	67.1	0.50	1.21	335	506	0.66
	Wisconsinan loam till	0	- ^a	0	6.4	1.48	2.93	95	133
5				2.3	2.67	4.00	61	78	—
15				5.2	1.33	3.06	70	111	—
35				4.4	2.43	3.44	108	125	—
3		1	0	47.5	0.55	0.96	262	322	0.81
			5	57.1	0.53	0.83	304	365	0.83
			15	48.0	0.57	1.03	274	376	0.73
			35	57.1	0.48	0.88	271	363	0.75
3		2	0	58.1	0.41	0.71	238	322	0.74
			5	55.4	0.54	0.91	299	379	0.79
			15	58.3	0.38	0.94	221	365	0.61
			35	59.0	0.39	0.96	228	364	0.63
3		4	0	86.1	0.59	0.90	508	637	0.80
			5	83.4	0.55	0.92	455	629	0.72
			15	77.8	0.63	1.06	492	644	0.76
			35	75.7	0.69	1.18	521	684	0.76
3		6	0	98.3	0.55	0.88	542	692	0.79
			5	91.9	0.48	0.82	533	637	0.84
			15	91.9	0.55	1.04	506	710	0.71
			35	95.0	0.59	1.15	556	759	0.73
Fayette C		0	- ^a	0	4.3	1.16	1.92	50	63
	5			3.6	1.39	2.04	50	59	—
	15			6.4	0.92	1.60	60	77	—
	35			6.8	1.02	1.82	69	90	—
	5	1	0	41.6	0.44	0.73	185	258	0.72
			5	48.9	0.40	0.96	198	320	0.62
			15	52.5	0.36	0.77	189	290	0.65
			35	54.4	0.37	0.81	199	300	0.66
	5	2	0	60.0	0.50	0.78	303	378	0.80
			5	67.4	0.42	0.83	282	406	0.70
			15	63.0	0.46	0.95	292	422	0.69
			35	78.9	0.36	1.00	283	465	0.61
	5	4	0	155.0	0.34	0.50	523	630	0.83
			5	113.3	0.44	0.84	493	750	0.66
			15	115.0	0.43	0.86	492	770	0.64
			35	124.7	0.47	0.91	582	830	0.70
	5	6	0	187.7	0.41	0.75	770	1110	0.69
			5	153.0	0.58	0.92	685	1180	0.75
			15	160.0	0.56	0.92	935	1210	0.77
			35	205.0	0.40	0.74	832	1162	0.72
	Illinoian till	0	- ^a	0	10.8	1.05	2.15	113	148
5				8.1	1.12	2.51	91	132	—
15				8.6	1.16	2.25	100	134	—
35				10.9	0.92	2.58	100	152	—
3		1	0	29.4	0.71	1.13	210	265	0.79
			5	35.6	0.55	1.11	196	279	0.70
			15	28.4	0.70	1.47	200	308	0.65
			35	38.3	0.63	1.36	243	352	0.69
3		2	0	—	—	—	—	435	—
			5	63.1	0.52	0.97	328	463	0.71
			15	63.5	0.51	1.00	325	460	0.71
			35	71.6	0.53	1.03	379	541	0.70
3		4	0	55.0	0.71	1.24	392	532	0.74
			5	54.7	0.64	1.18	348	502	0.69
			15	56.7	0.59	1.22	332	517	0.64
			35	55.9	0.75	1.24	420	540	0.78
3		6	0	104.1	0.63	1.01	658	845	0.78
			5	91.4	0.70	1.07	640	787	0.81
			15	87.5	0.63	1.13	550	798	0.69
			35	89.2	0.80	1.25	713	908	0.78

^aCured 7 days at room temperature.

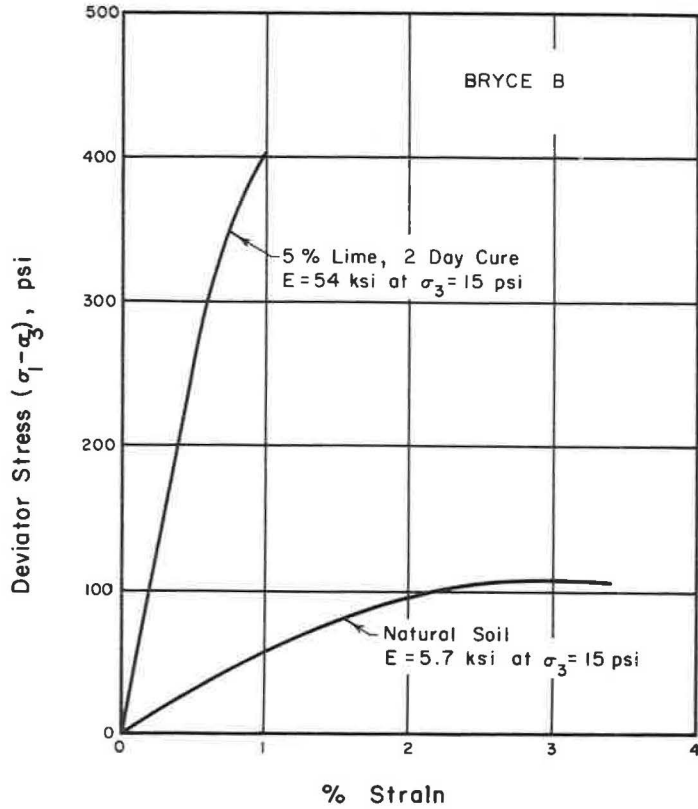


Figure 5. Typical stress-strain curves for natural and lime-treated soil.

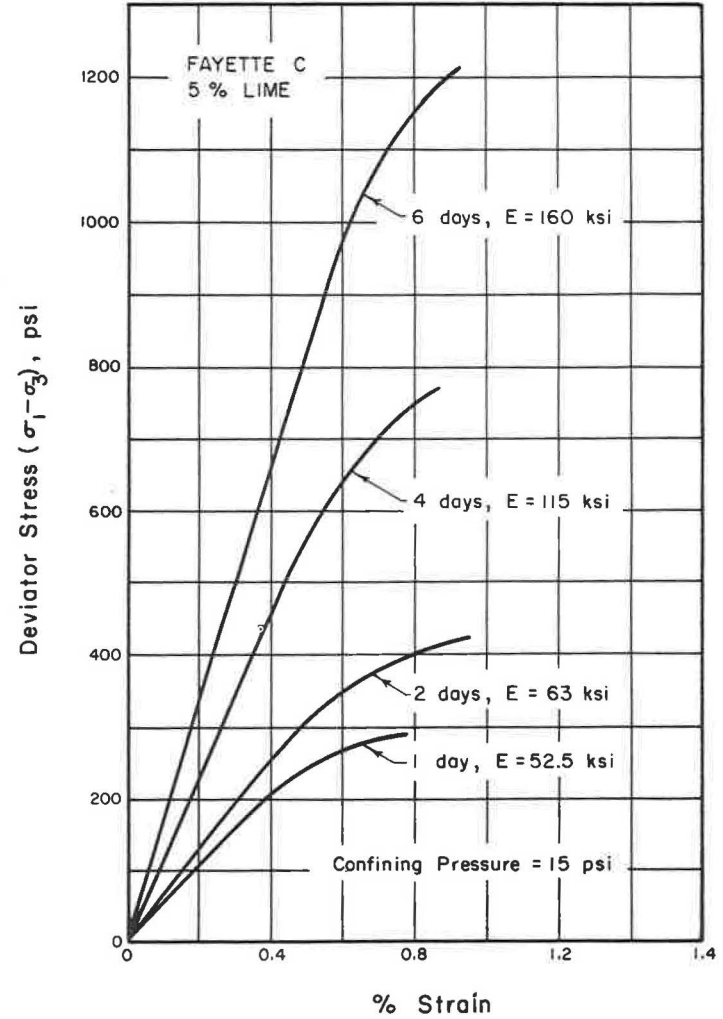


Figure 6. Influence of curing time on stress-strain properties of lime-soil mixture.

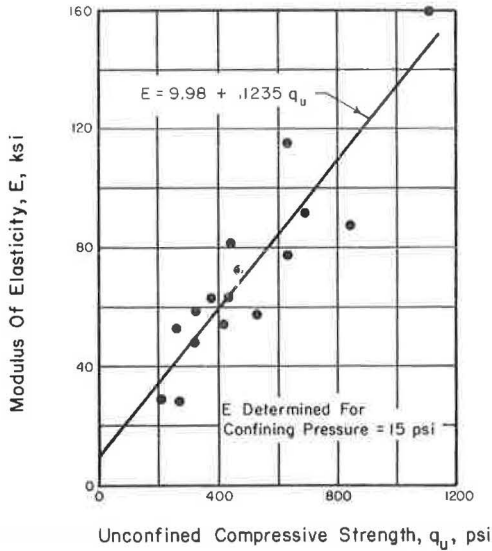


Figure 7. Modulus of elasticity vs unconfined compressive strength of lime-soil mixtures.

TABLE 6
MODULUS OF ELASTICITY (E) - UNCONFINED COMPRESSIVE STRENGTH (q_u) LINEAR REGRESSION ANALYSIS SUMMARY (Analysis of Variance^a)

Source of Variation	Degrees of Freedom	Sum of Squares	Variance	F
Total	15	15,951		
Regression	1	13,252	13,252	68.7 ^b
Residual	14	2,699	192.8	

^aRegression equation: $E \text{ (ksi)} = 9.98 + 0.1235 q_u \text{ (psi)}$.

^bSignificant at $\alpha = 0.01$.

The failure strains, ϵ_u , for the lime-soil mixtures were decreased in relation to those exhibited by the natural soils (Fig. 5). Typical failure strains for the lime-soil mixtures were approximately 1 percent, compared to 2 or 3 percent for the untreated soils. Completely randomized design statistical analyses (Table 7) of the lime-soil mixture failure strains at different confining pressures indicated that for one-half of the mixes an increase in confining pressure produced

larger failure strains. For the remainder of the mixtures there was no statistically significant influence of confining pressure on ϵ_u .

To evaluate the influence of soil type and curing period on failure strains, the average failure strains at a confining pressure of 15 psi for the various lime-soil mixtures and curing periods were statistically analyzed in a randomized complete block design. The analysis indicated that there was no statistically significant difference among the failure strains, regardless of soil type or curing period (Table 8). The average failure strains, ϵ_u , for all of the lime-soil mixtures analyzed was 1.02 percent.

TABLE 7
INFLUENCE OF CONFINING PRESSURE (σ_3) ON FAILURE STRAINS (ϵ_u)
(Statistical Summary of Completely Randomized Designs^a)

Soil	Lime (%)	Curing (days at 120 F)	Treatment ^b Variance	Variance	F
Bryce B	5	1	28.35	2.0	14.2 ^c
		2	2.0	2.5	0.8
		4	6.33	1.42	4.46 ^c
		6	19.33	0.58	33.2 ^c
Illinoian till	3	1	7.33	4.1	1.79
		2	0.5	1.62	0.31
		4	0.0	4.1	0.0
		6	2.0	4.9	0.41
Fayette C	5	1	3.67	0.82	4.5 ^c
		2	4.0	1.8	2.22
		4	10.67	0.91	11.7 ^c
		6	3.67	1.36	2.7
Wisconsinan loam till	3	1	2.0	5.73	0.35
		2	6.33	0.33	19.0 ^c
		4	6.33	1.18	5.36 ^c
		6	8.0	1.09	7.34 ^c

^aIn all analyses degrees of freedom were 3 for treatment and 12 for error.

^bTreatment corresponds to various confining pressures.

^cSignificant at $\alpha = 0.05$ level.

TABLE 8
 INFLUENCE OF SOIL TYPE AND CURING PERIOD
 ON AVERAGE FAILURE STRAINS OF LIME-SOIL MIXTURES
 AT σ_3 OF 15 psi

(a) Statistical Summary of Randomized Complete Block Design				
Curing Period (days at 120 F)	Lime-Soil Mixture			
	Bryce B	Illinoian Till	Fayette C	Wisconsinan Loam Till
1	13.5 ^a	11.2	7.7	10.5
2	10.3	10.1	9.8	9.5
4	9.6	12.3	8.6	10.5
6	9.3	11.3	9.3	10.4

(b) Analysis of Variance				
Source of Variation	Degrees of Freedom	Sum of Squares	Variance	F ^a
Total	15	29.1		
Curing period	3	1.4	0.47	0.27
Lime-soil mixture	3	12.3	4.1	2.40
Error	9	15.4	1.71	

^aAverage strain values are $\times 10^{-3}$ in./in.

^bF values not significant at $\alpha = 0.05$.

TABLE 9
 INFLUENCE OF CONFINING PRESSURE (σ_3) ON ϵ_B (Statistical Summary
 of Completely Randomized Designs^a)

Soil	Lime (%)	Curing (days at 120 F)	Treatment ^b Variance	Error Variance	F
Bryce B	5	1	0.43	1.30	0.33
		2	0.67	1.17	0.57
		4	1.33	0.34	3.88 ^c
		6	1.17	0.33	3.6 ^c
Illinoian till	3	1	1.0	5.5	0.18
		2	0.0	0.73	0.0
		4	5.67	8.8	0.64
		6	3.0	1.8	1.67
Fayette C	5	1	1.07	1.63	0.66
		2	1.37	0.79	1.73
		4	1.13	0.42	2.69
		6	4.07	0.65	6.3 ^c
Wisconsinan loam till	3	1	0.9	1.9	0.47
		2	1.93	0.72	2.68
		4	1.37	0.85	1.61
		6	0.1	0.43	0.23

^aIn all analyses degrees of freedom were 3 for treatment and 12 for error.

^bTreatment corresponds to confining pressure.

^cSignificant at $\alpha = 0.05$

TABLE 10
 INFLUENCE OF SOIL TYPE AND CURING PERIOD
 ON AVERAGE ϵ_B

(a) Statistical Summary of Randomized Complete Block Design				
Curing Period (days at 120 F)	Lime-Soil Mixture			
	Bryce B	Illinoian Till	Fayette C	Wisconsinan Loam Till
1	7.2 ^a	6.9	4.1	5.7
2	5.2	5.2	4.4	4.3
4	4.6	6.4	4.2	6.2
6	4.4	7.0	5.0	5.7

(b) Analysis of Variance				
Source of Variation	Degrees of Freedom	Sum of Squares	Variance	F ^b
Total	15	41.9		
Curing	3	3.0	1.0	0.29
Lime-soil mixture	3	7.7	2.56	0.74
Error	9	31.2	3.47	

^aAverage strain values are $\times 10^{-3}$ in./in.

^bF values not significant at $\alpha = 0.05$.

TABLE 11
 INFLUENCE OF SOIL TYPE AND CURING PERIOD
 ON AVERAGE σ_B/σ_U RATIO

(a) Statistical Summary of Randomized Complete Block Design				
Curing Period (days at 120 F)	Lime-Soil Mixture			
	Bryce B	Illinoian Till	Fayette C	Wisconsinan Loam Till
1	0.76 ^a	0.66	0.71	0.78
2	0.72	0.70	0.71	0.69
3	0.70	0.71	0.71	0.76
6	0.71	0.73	0.77	0.77

(b) Analysis of Variance				
Source of Variation	Degrees of Freedom	Sum of Squares	Variance	F ^b
Total	15	2.13		
Curing	3	0.0	0.0	0.0
Lime-soil mixture	3	0.84	0.28	1.96
Error	9	1.29	0.143	

^aAverage σ_B/σ_U ratio based on all specimens.

^bF values not significant at $\alpha = 0.05$.

The strains at which the stress-strain curves departed from a linear relation, ϵ_B , were analyzed to determine if confining pressure, soil type, or curing period influenced the value. Completely randomized design statistical analyses of the data indicated that for 13 of the 16 lime-soil mixtures there was no significant difference for ϵ_B at different confining pressures (Table 9). Thus, ϵ_B for a given lime-soil mixture and curing period could be averaged to provide a representative ϵ_B . Statistical analysis of the average ϵ_B values indicated that there was no significant difference among the various lime-soil mixtures (Table 10). The overall average of ϵ_B , regardless of soil type, confining pressure, or curing period, was 0.54 percent.

The data suggest that σ_B , the deviator stress corresponding to the strain ϵ_B , varies in relation to σ_u , the ultimate deviator stress. Close analysis of the data indicates that for a given lime-soil mixture and curing period the σ_B/σ_u for all confining pressures was therefore taken to be representative of a lime-soil mixture cured for a particular period. Table 5 gives the average σ_B/σ_u values for the various mixtures, curing periods, and confining pressures. Statistical analysis of the data indicates no significant difference among the σ_B/σ_u ratios, regardless of soil type and curing period (Table 11). The overall average, 0.72, is therefore the best estimate of σ_B/σ_u for these lime-soil mixtures.

DISCUSSION

Shear Strength

The major effect of lime on the shear strength properties of a reactive fine-grained soil is to produce a substantial increase in cohesion, with some minor increase in ϕ . At the low confining pressures normally considered to exist in a flexible pavement structure, the cohesion increase is of the greatest significance.

McCleod (1) has demonstrated the value of cohesion to the ultimate bearing capacity of a flexible pavement structure. He stated, "The important contribution that the existence of cohesion, C , in any layer, surface, base, or subbase makes to the ultimate strength of a flexible pavement is one of the major conclusions indicated. . . ." Using McCleod's ultimate strength approach, it can be shown that if the lime-soil mixtures studied in this investigation were used in pavement components of normal thicknesses, the ultimate strength would be much larger than required for normal wheel loading and traffic conditions.

Theoretical analyses of shear stresses in pavement systems have been conducted by many investigators. In some instances, the maximum shearing stress is taken as one-half the difference of the principal stresses (6), but Mehta and Veletsos (7) evaluated the vertical shear stress component at various points in a layered system. Although the maximum shear stress in a pavement is dependent on the properties and characteristics of the particular pavement system and wheel loading condition, the maximum theoretical shear stress is substantially less than the tire contact pressure if the pavement is of normal thickness, greater than 6 to 8 in.

Assuming a simple pavement system similar to that in Figure 8, and using analytical procedures recommended by McDowell (6) and Mehta and Veletsos (7), the maximum shearing stresses in such a system are less than approximately 50 psi. Shear stresses of this magnitude would not cause shearing failure in any of the materials investigated in this study.

Modulus of Elasticity

The large E increase produced by lime treatment of a fine-grained soil has important implications regarding pavement system behavior. If a subgrade soil is stabilized, the modular ratio, $E_{\text{lime-soil}}/E_{\text{subgrade}}$, will be much larger than unity. Inasmuch as the modulus of elasticity of the lime-soil mixture increases with curing time, the effective modular ratio for the pavement system also varies. For the materials in this investigation the modular ratios for different mixtures ranged from approximately 3 to 25. Theoretical analyses have shown that in such a layered pavement system, substantial flexural stresses are developed in the layer containing the lime-soil mixture.

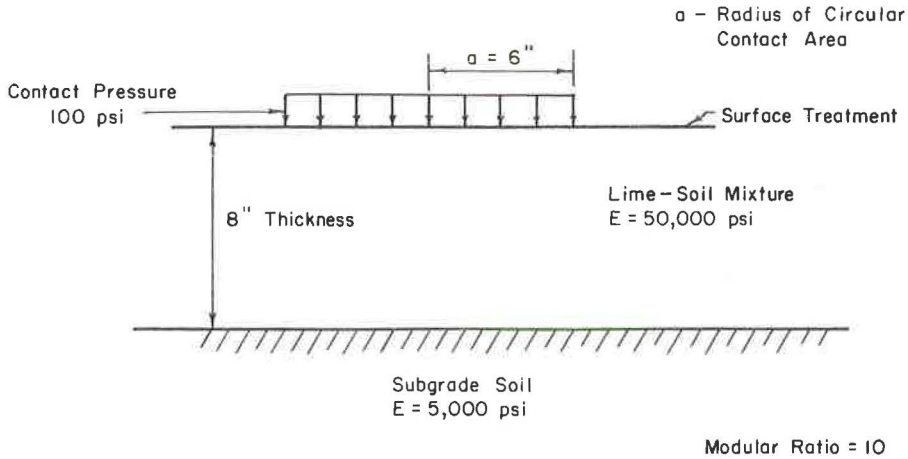


Figure 8. Typical lime-soil pavement section.

Thompson's work (8) indicated that lime-soil mixtures possess tensile-strength properties capable of resisting the flexural stresses developed. The behavior of the lime-soil layer over the untreated subgrade is therefore dissimilar from a typical flexible pavement due to the slab action which can be developed by the lime-soil mixture. The slab action of lime-soil mixtures has been considered by McDowell (3) in the Texas triaxial design procedure, and reductions in pavement layer thicknesses are permitted if the material has a substantial tensile strength as measured by the cohesiometer test.

Stress-Strain Properties

The typical stress-strain curves (Figs. 5 and 6) show that lime-soil mixtures under compressive states of stress at low confining pressures are fairly brittle materials and display a limited amount of inelastic yielding.

A significant finding of the study is that the ultimate strain, ϵ_u , at 15 psi confining pressure, is not significantly different for the various mixtures included in the study, although a large range of ultimate strengths, σ_u , were obtained. The finding suggests that the stress-strain properties of lime-soil mixtures are primarily determined by the hydrated calcium silicate and calcium aluminate cementing agents produced by lime treatment. Because the same basic cementing agents are present in all of the lime-soil mixtures, when the critical limiting strain for the cementing agents is achieved, failure occurs. Similarly, increased moduli of elasticity and strengths for the various mixtures and curing periods are due to the formation of larger quantities of the cementing agents and more cemented contact points in the material. The 1 percent ultimate strain, ϵ_u , may be an important design consideration because it suggests a limiting strain failure criteria. This behavior can be explained partially by the close positive correlation between E and the strength of the lime-soil mixtures.

As statistical analysis of the stress-strain data indicated that ϵ_u at 15 psi confining pressure, ϵ_B , and σ_B/σ_u were not significantly different for the various lime-soil mixtures, pertinent stress-strain characteristics can be defined for a wide strength range. Average values for these parameters as determined from this study, are

$$\begin{aligned}\epsilon_u \text{ at 15 psi confining pressure} &= 1.02 \text{ percent;} \\ \epsilon_B &= 0.57 \text{ percent;} \text{ and} \\ \sigma_B/\sigma_u &= 0.72\end{aligned}$$

General

The foregoing discussion indicates that in the rational design of a lime-soil mixture pavement layer, the shear strength of the mixture is probably not critical. The high

modular ratios and mixture tensile-strength properties result in the generation of substantial flexural stresses in the lime-soil layer. These flexural stresses are probably the controlling factor in any rational design considerations if subgrade stresses are within an allowable range. The foregoing comments are based on the assumption that the lime-soil mixture is durable and retains its strength and integrity throughout its service life.

SUMMARY AND CONCLUSIONS

The shear strength and stress-strain properties of four soils and four lime-soil mixtures were evaluated. Curing periods for the lime-soil mixtures varied from 1 to 6 days at 120 F. Unconsolidated-undrained type triaxial tests with total deformation measurements were conducted on the cured samples. The test results suggest the following conclusions:

1. Lime treatment substantially increases the shear strength of lime-reactive soils. This improvement is primarily due to a large increase in cohesion with small increases in the angle of shearing resistance, ϕ .
2. The moduli of elasticity, E , of the lime-soil mixtures were much larger than the E of the untreated soils. Modular ratios, $E_{\text{lime-soil}}/E_{\text{soil}}$, of 3 to 25 were obtained for the materials studied.
3. Increased curing periods normally produced lime-soil mixtures with higher shear strengths and moduli of elasticity.
4. The cohesion, C , and modulus of elasticity, E , of the cured lime-soil mixtures investigated can be predicted based on unconfined compression test results.

$$C \text{ (psi)} = 9.3 + 0.292 q_u \text{ (psi)}$$

$$E \text{ (ksi)} = 9.98 + 0.1235 q_u \text{ (psi)}$$

5. Ultimate failure strains for the lime-soil mixtures were decreased in relation to the untreated soils. For the mixtures studied, the failure strains at 15 psi confining pressure were not significantly different for the various lime-soil mixtures and curing periods. The average ultimate strain was approximately 1 percent.
6. The strains at which the lime-soil stress-strain curves departed from a linear relation, ϵ_B , were not significantly different for the mixtures studied. The average value for ϵ_B was 0.54 percent.
7. The stress, σ_B , corresponding to ϵ_B was equal to approximately 0.72 σ_u for the different lime-soil mixtures, irrespective of soil type and curing period.
8. In typical flexible pavement structures the shear strengths of the mixtures studied were sufficient to prevent shear failure of the lime-stabilized layer. Flexural stresses in the stabilized layer will probably be the controlling factor in a rational design procedure for lime-soil mixtures of a quality comparable to those included in this investigation.

ACKNOWLEDGMENTS

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REFERENCES

1. McCleod, Norman A. An Ultimate Strength Approach to Flexible Pavement Design. Proc. Assoc. of Asphalt Paving Technologists, Vol. 23, 1954.
2. Hewitt, William L. Analysis of Stress in Flexible Pavements and Development of a Structural Design Procedure. Highway Research Board Bull. 269, 1960.
3. McDowell, Chester. Road Test Findings Utilized in Analysis of Texas Triaxial Method of Pavement Design. Highway Research Board Spec. Rept. 73, pp. 314-349, 1962.

4. Anday, M. C. Curing Lime-Stabilized Soils. Highway Research Record 29, pp. 13-24, 1963.
5. Herrin, Moreland. Effects of Aggregate Shape on the Stability of Bituminous Mixes. PhD thesis, Purdue Univ., 1954.
6. McDowell, Chester. Wheel-Load-Stress Computations Related to Flexible Pavement Design. Highway Research Board Bull. 114, pp. 1-16, 1955.
7. Mehta, M. R., and Veletsos, A. S. Stresses and Displacements in Layered Systems. Civ. Eng. Studies, Structural Res. Series No. 178, Univ. of Illinois, 1959.
8. Thompson, Marshall R. Split-Tensile Strength of Lime Stabilized Soils. Highway Research Record 92, pp. 69-80, 1965.

Appendix

METHOD FOR DETERMINING COMPOSITE STRESS-STRAIN RELATION

1. Graphs of deviator stress, $\sigma_1 - \sigma_3$, vs strain were prepared for each specimen.
2. An initial tangent modulus of elasticity (E_1, E_2, E_n , etc.) was evaluated for each specimen from the straight-line portion of its stress-strain curve. An average modulus of elasticity, E , was determined by $E = E_1 + \dots + E_n/n$.
3. The deviator stress at which the stress-strain curve departed from a linear relation was designated $\sigma_{B1}, \sigma_{B2}, \sigma_{Bn}$, etc. An average stress corresponding to this point was determined by $\sigma_B = \sigma_{B1} + \dots + \sigma_{Bn}/n$. A strain value for σ_B was evaluated by $\epsilon_B = \sigma_B/E$.
4. The maximum deviator stresses for the four stress-strain curves $\sigma_{u1}, \sigma_{u2}, \sigma_{un}$, etc., were averaged to provide an average ultimate strength, σ_u .
5. An average ultimate strain value, ϵ_u , corresponding to σ_u was evaluated by averaging, for the four specimens, the magnitude of the strains that occurred between σ_{Bn} and σ_{un} and adding this average strain increment to ϵ_B .
6. An intermediate deviator stress value (σ_m) for the strain, ϵ_m , corresponding to the point midway between ϵ_B and ϵ_u was determined by averaging, for the four specimens, the stresses occurring at the strain halfway between the strains associated with σ_{Bn} and σ_{un} .
7. Utilizing the values computed above, $E, \sigma_B, \epsilon_B, \sigma_u, \epsilon_u, \sigma_m$, and ϵ_m , the composite stress-strain curve was plotted (Fig. 2).

Evaluation of Soil-Lime Stabilization Mixtures

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As the use of lime for stabilization of both fine-grained and granular soil materials continues to increase each year, so do the requests for lime content recommendations. This information is not readily available. Members of our profession customarily turn to whatever means of testing that is available to seek answers. This report comments on ways and means of analyzing soil-lime mixtures from the following standpoints:

1. Strength and P. I. tests on cores taken from old roads;
2. The cracking problem for lime-stabilized base courses;
3. Deflection tests;
4. Relation of strength to percent of lime with emphasis on effect of curing time;
5. The rehealing properties of some mixtures; and
6. The effect of moisture, lime content, purity of lime, and fineness of lime on early loss of cohesion in raw soil.

The report indicates that core strengths are two and one-half to three times as great as would be obtained under normal room temperature curing periods. The report also concludes that for equal percentages of the four types of limes studied, the finer and purer the lime the greater the loss in the cohesion of the raw gumbo soil tested. This loss of cohesion is believed to be one of the key factors influencing efficient and economical mixing in the field. On the basis of testing road cores, both short- and long-time cured laboratory specimens and early effects on cohesion of soil, a chart for recommended lime contents is presented. The chart includes the plasticity index, and the percent minus No. 40 mesh, and is based on the use of a fairly pure fine-hydrated lime.

•SEEKING information on the evaluation of soil stabilization mixtures, we obtained undisturbed cores from old roads of known behavior (Figs. 1 and 2). This is not new, but perhaps our attempts to relate our preliminary laboratory strengths to those obtained from cores may be new to some. The cores taken from a 10½-yr old lime-treated clay project located between Houston and Beaumont are approximately three times stronger than would have been predicted from ordinary preliminary laboratory tests (Fig. 3). During trimming, excess portions were retained for soil constants. The numbers opposite plotting points represent the plasticity index. There was no relation between strength and P. I., and there was no noticeable difference in strength for the two percentages of quick lime used. The cores containing 8 percent treatment of lime gained less in P. I. with age than the mixtures containing 4½ percent (Fig. 4). Although the data are not included in this report, the average soil binder content (although variable) produced by drying and slaking was 17 percent less for the 8 percent mixtures. Figure 5 shows results of tests on raw gravel and cores from lime-treated gravel (Fig. 2). It appears that maximum strengths for this mixture were reached somewhere

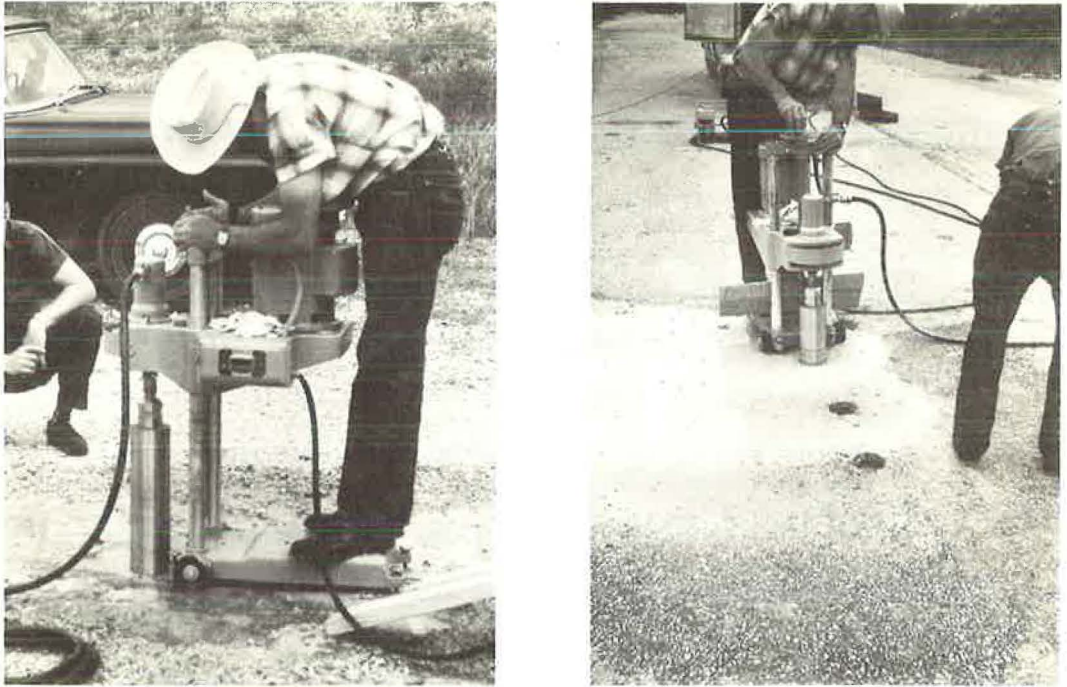


Figure 1. Removal of undisturbed cores from old roads of known behavior.

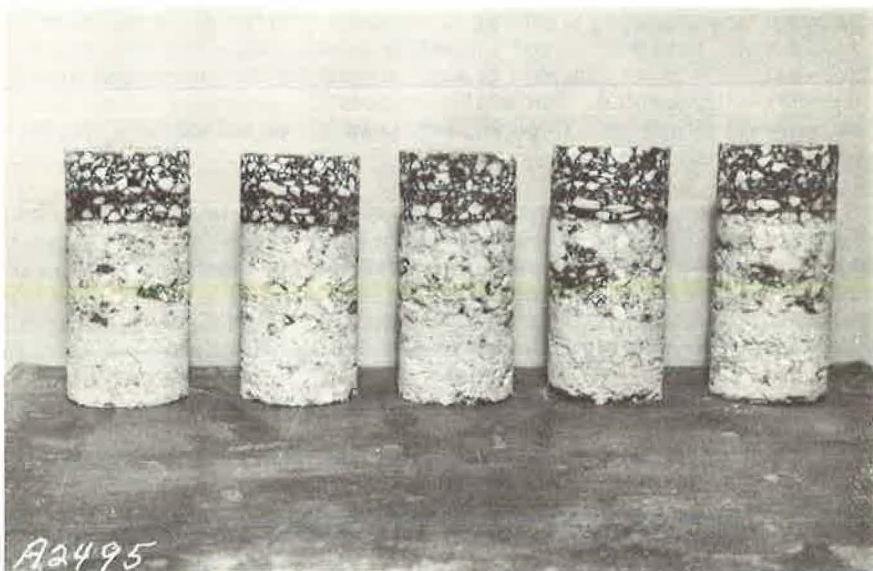


Figure 2. Cores from old roads of known behavior.

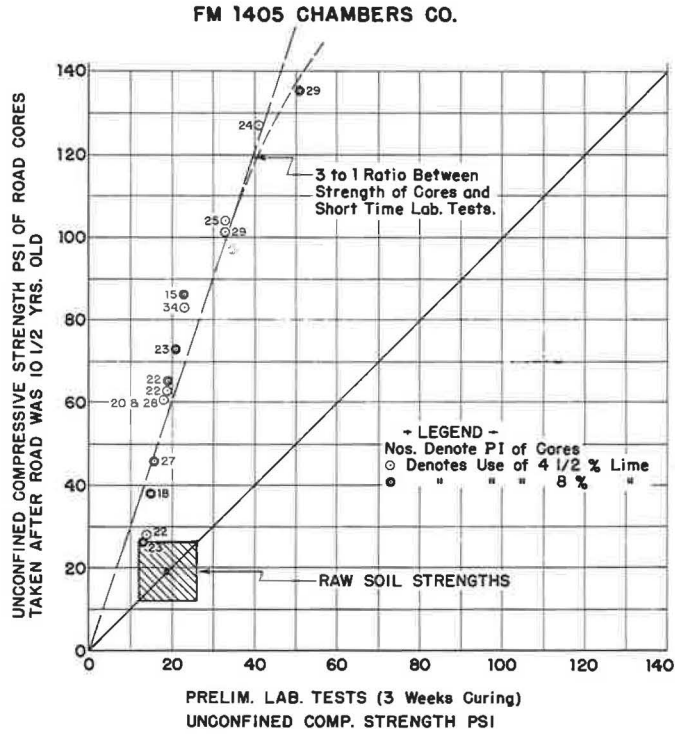


Figure 3. Relation of compressive strengths of clay-lime field cores to preliminary laboratory (short-time) results.

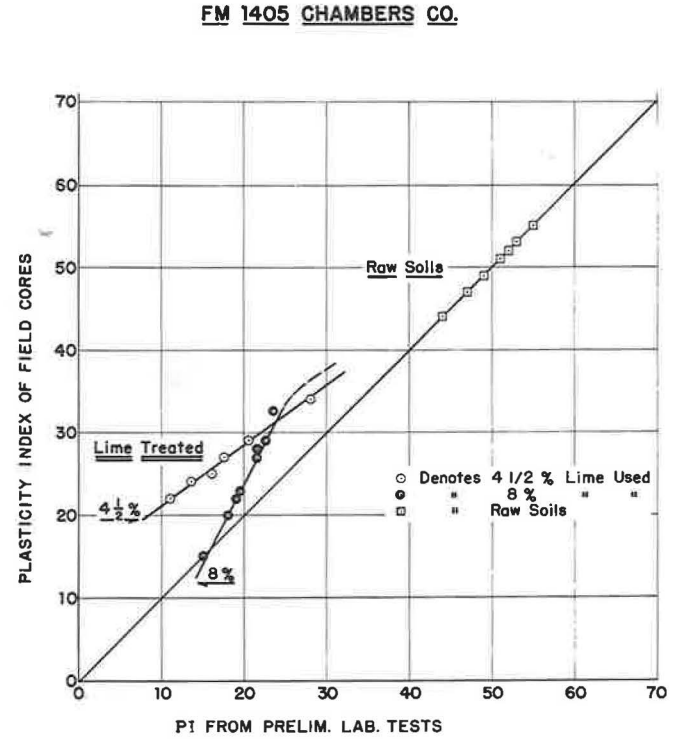


Figure 4. Relation of P.I. of clay-lime field cores to those obtained from short-time tests.

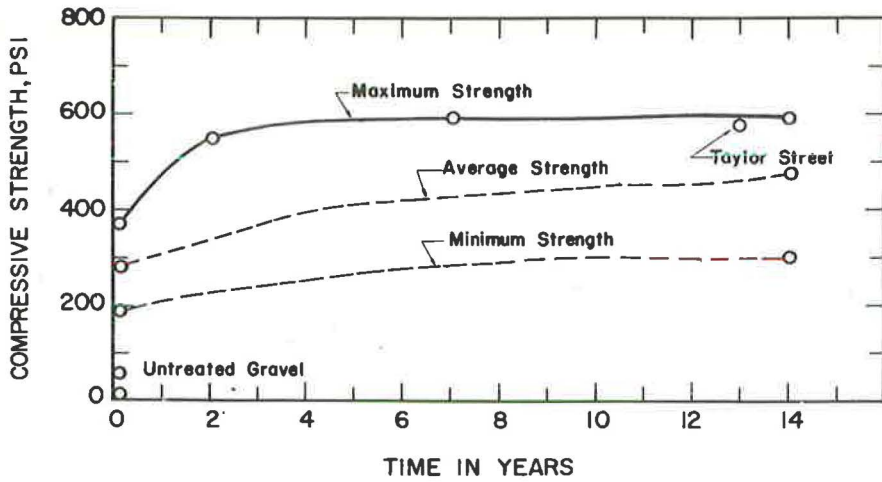


Figure 5. Unconfined compressive strengths.

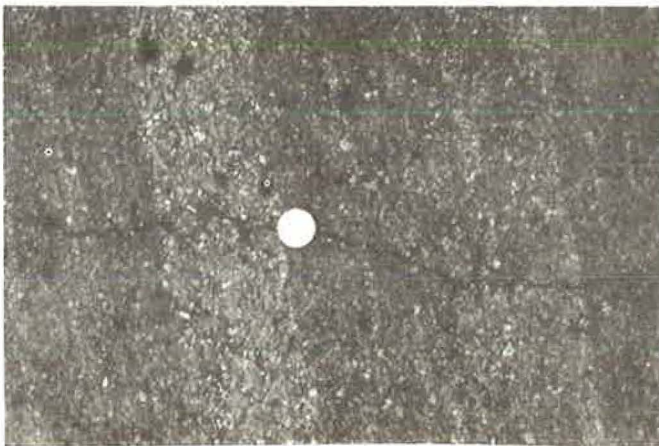


Figure 6. Shrinkage cracking in streets of Llano.



Figure 7. Benkelman beam deflection test.

between 1 and 24 mo. Other long-time tests, not included in this report, indicate soils containing much greater amounts of lime may still gain in strength after 10 yr (1). It is possible that the strength results in Figure 3 for the 8 percent lime might have gained

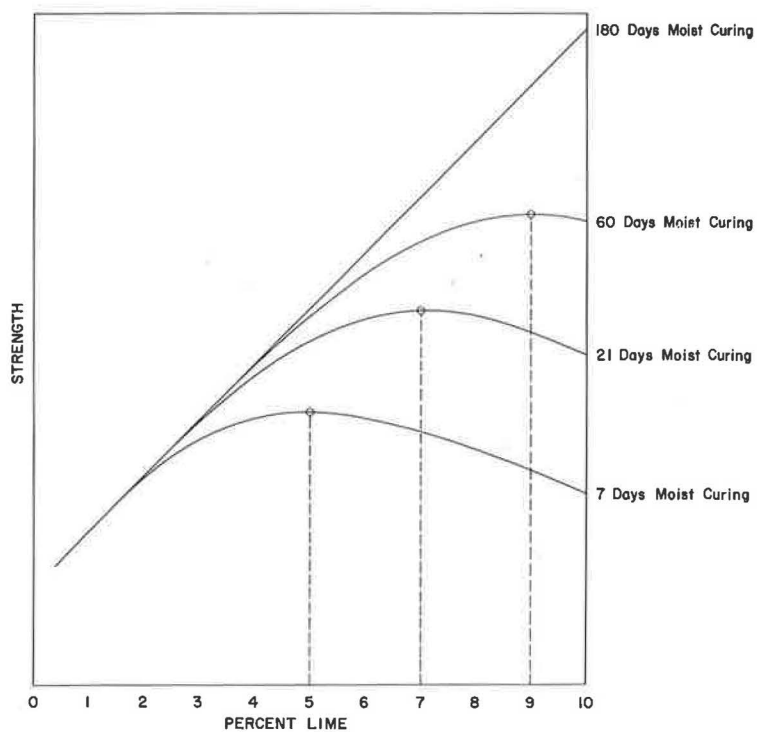


Figure 8. Optimum percent lime.

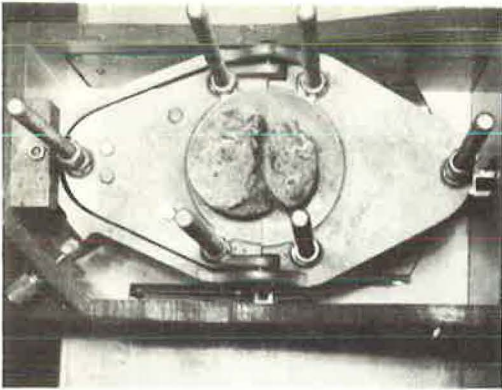


Figure 9. Cohesiometer specimen after breaking first time.

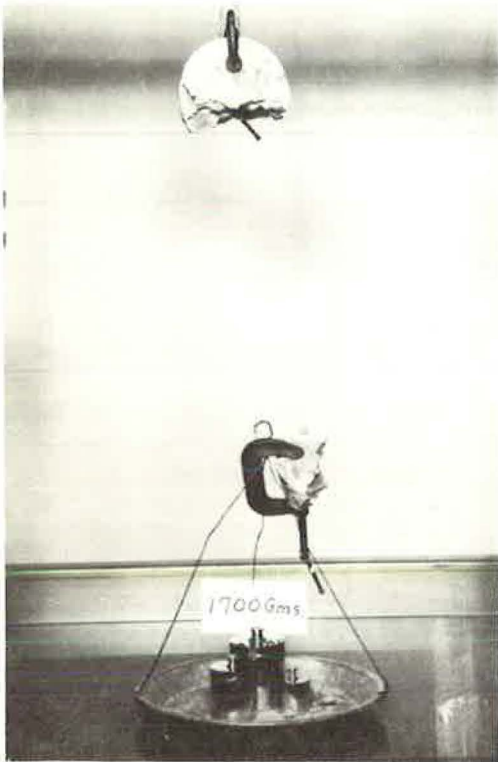


Figure 11. Same specimen as in Figures 9 and 10 breaks along old shear plane when 1,700-gm load is applied.



Figure 10. Rehealed cohesiometer specimen after 18 mo curing in moist room; specimen supported 1,500-gm load.

a great deal with time, but the road was abandoned for the sake of a new location shortly after the cores were taken.

Another method of analysis used for evaluation of treated base courses concerns cracking such as is shown in Figure 6. This problem is complicated by the fact that cracking can be caused by many factors other than shrinkage and other characteristics of the material itself; these causes include overloading when mixture is critically tender, volume change of subgrade, and inadequate depth design. We have nothing new to offer on this subject except to say that we are working on it largely from the standpoint of materials analysis.

Considerable progress in the use of deflection tests (2) to bring out increased beam strength with age has been accomplished by use of the Benkelman beam (Fig. 7).

Despite accomplishments in testing of cores, road cracking, or road deflections, we still have to tie mix design for all future work to some preliminary laboratory test

which can be accomplished in a reasonably short time. Unfortunately, when confronted with an admixture design problem, we probably obtain results for only one of the curves given in Figure 8. Seldom do engineers realize that they have obtained only one of the elusive optimum lime contents that time of curing could change.

Another interesting aspect of evaluating lime stabilization mixtures involves autogenous healing. A core cut from a 12-yr old gravel stabilized with 3 percent lime was tested in a cohesiometer (Fig. 9), then placed in a moist room for approximately 18 mo, during which rehealing took place. A load of between 1,500 and 1,700 gm was required to pull the specimen apart again (Figs. 10 and 11).

MIXABILITY

In the stabilization of highly plastic clays with lime, it is believed that fairly good mixing is essential to successful results, and that efficient mixing is affected by the purity and fineness of the lime perhaps as much as by weather, moisture content, and equipment. It is further believed that for clay soils the cohesion of the soil is its greatest deterrent to pulverization. Figure 12 shows how the cohesiometer was used to evaluate cohesion of a soil having a P.I. of 42. The upper curve shows the effect of moisture content of the raw soil. In this case, 2- by 6-in. diameter specimens were compacted with 50 blows of a 10-lb rammer dropped 18 in. Specimens were held overnight without moisture content change so that moisture films could adjust before testing. A peak occurred in the curve at about optimum moisture for heavy compaction. In the case of soil-lime mixtures, the overnight delay before testing was deleted because we wanted to know the short-time condition of the mix, as in the case of passing it through a mixer. Total time used in molding and testing soil-lime mixtures was approximately 20 min. One percent lime reduced cohesion of the raw soil materially, but as the lime content was increased to 2, 3, 5, 6½ and 8 percent, cohesion was decreased further, perhaps with not much distinction between mixes containing 5, 6½ and 8 percent. These data from the peaks of these curves, expressed as percentages, are shown as the lower curve in Figure 13; data obtained in the same manner for three other limes varying in purity and fineness are shown in the lower right hand corner. To compare the relative effects of the four limes tested, horizontal lines on pertinent percent cohesiometer values were drawn and percentages of lime selected which produced equal effects on cohesion of the raw soil (for 50 percent cohesion, see Fig. 13). Percentages in Figure 14 were obtained in this manner. The data indicate that purity and fineness have an important influence on the lime contents required to produce equal cohesive properties in the soil tested. No procedure for this particular cohesiometer test is presented, inasmuch as a choice of details of the procedure is being studied. Soil materials of low cohesive characteristics probably would not be affected similarly, but most lime sold to treat soils is used for treating clayey soil materials where there is a mixing problem. Pure lime is enhanced for stabilization of clays as its degree of fineness is increased. The benefits of finely ground limes are most noticeable during pulverization and mixing. It is commonly assumed that fairly good mixing is an important quality in road-building mixtures. Also important is the contractors' ability to achieve such a mixture rapidly, in the most economical manner. It is believed that the finer the lime used, the more clay contracts and reactions there will be to ameliorate the soil in a reduced period of time.

SELECTION OF LIME CONTENT

Because there are so many variables in construction, we doubt that there is a strictly representative strength test method available for determining lime content for soil stabilization purposes which can be performed within a reasonable length of time. It is believed that Figure 15 offers the best method available for selecting lime contents for stabilization of soils; however, some form of strength test to check the lime's reaction with soils is necessary. We use a minimum unconfined compression value of 50 psi for subgrade or subbase and 100 psi for base materials which must carry a surfacing. In the preparation of Figure 15 the following factors were taken into consideration:

1. construction delays during mixing due to change of weather;
2. purity and fineness of lime;
3. imperfect mixing and compacting;
4. imperfect sealing and curing;

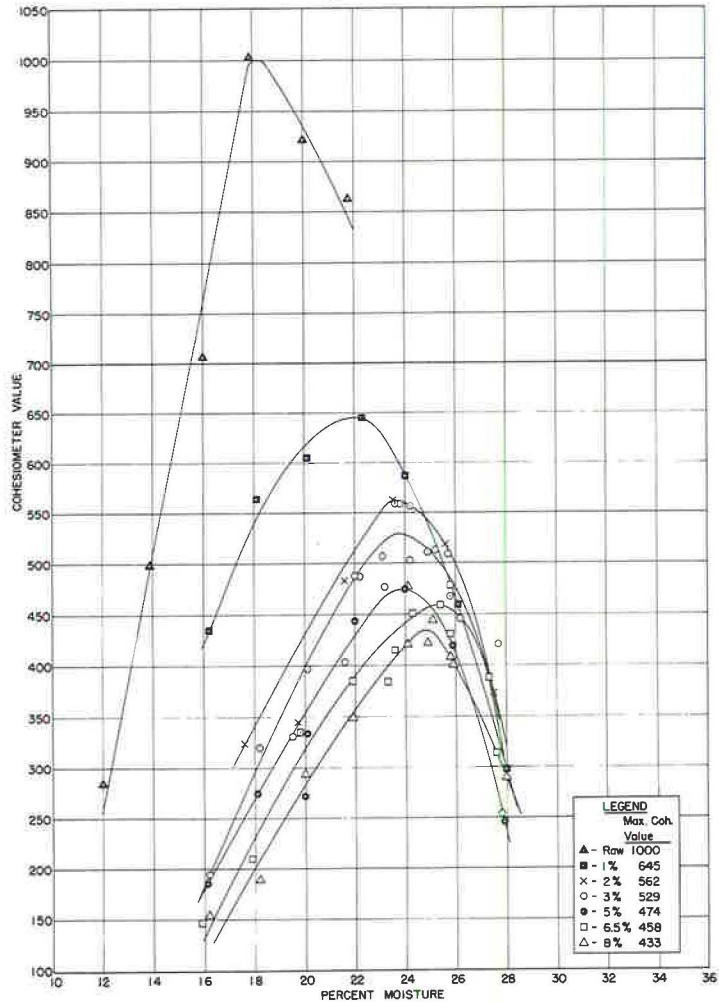


Figure 12. Relation of cohesimeter value to percent moisture for black gumbo manor clay soil containing various percentages of a fine pure lime.

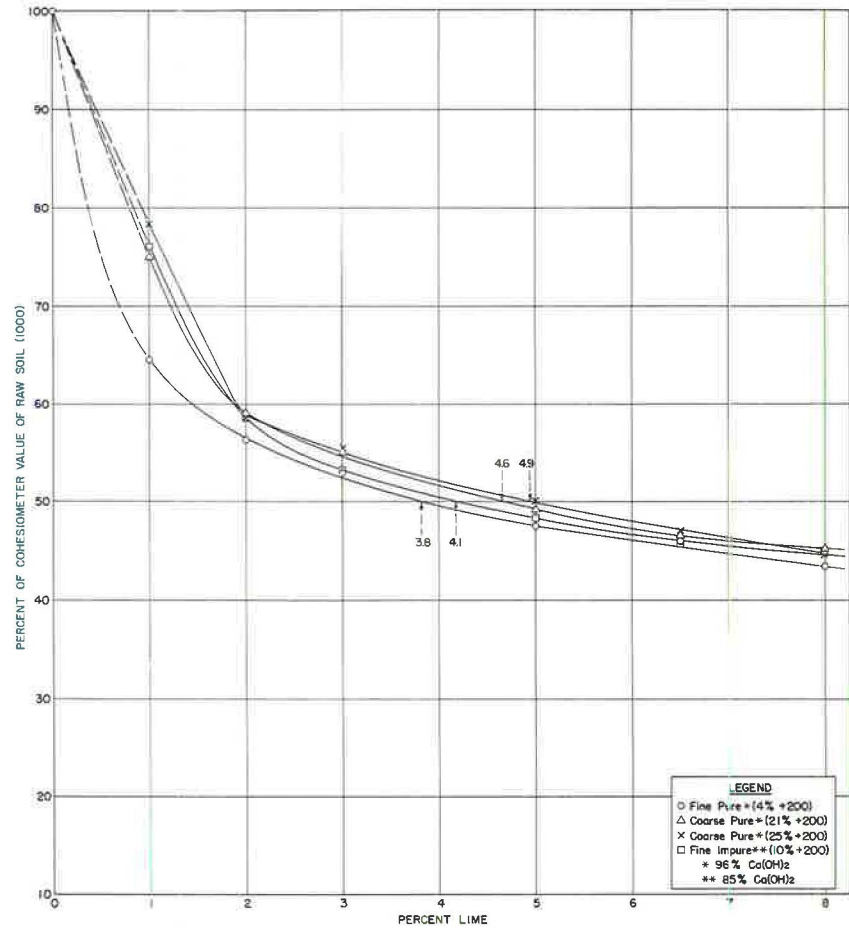


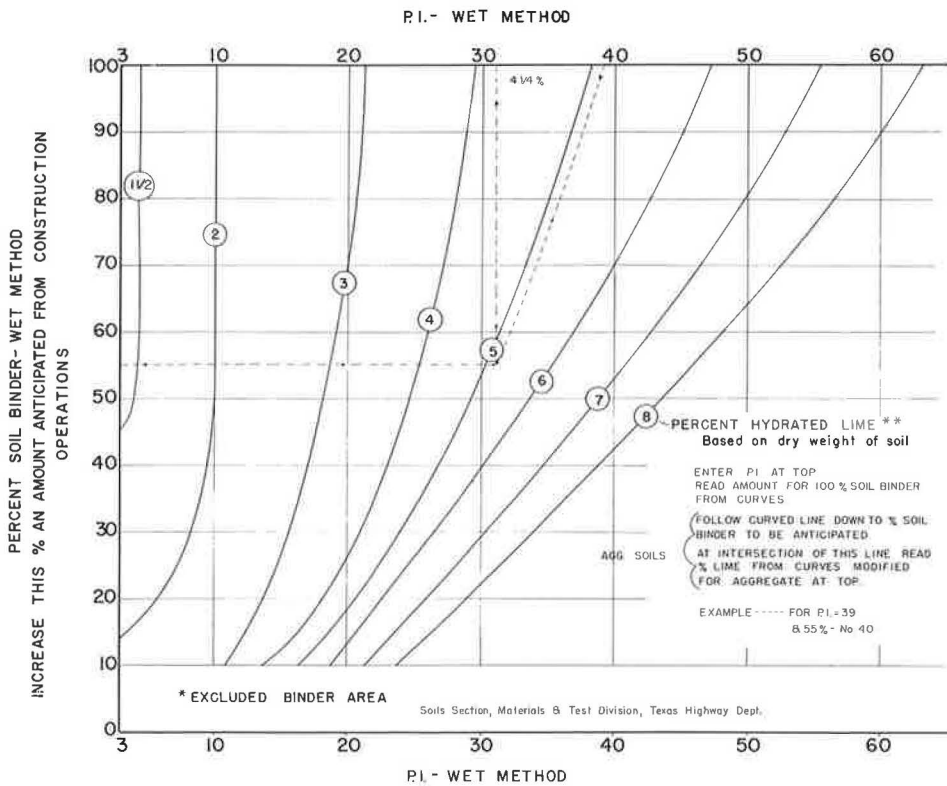
Figure 13. Effect of various types of limes on optimum cohesimeter values of manor clay.

Cohesimeter Values in % of Raw Soil	Percent Lime Req'd. to Produce Equiv. Cohesimeter Values			
	Fine Pure* 4% + 200	Coarse Pure+ (A) 25%+200	Coarse Pure* (B) 21%+200	Fine Impure** 10% + 200

100	0.0	0.0	0.0	0.0
75	0.5	1.2	1.0	1.1
70	0.7	1.4	1.2	1.3
65	1.0	1.6	1.5	1.5
60	1.4	1.9	1.9	1.9
55	2.3	3.0	2.9	2.5
50	3.8	4.9	4.6	4.1
45	6.7	7.8	8.2	7.4

* 96% Ca(OH)₂
** 85% Ca(OH)₂

Figure 14. Percentages of various types of limes required to produce equivalent cohesimeter values.



* Exclude use of chart for materials with less than 10% - No. 40 and cohesionless materials (P.I. less than 3)

** Percent of relatively pure lime usually 90% or more of Ca and/or Mg hydroxides and 85% or more of which pass the No. 200 sieve. Percentages shown are for stabilizing subgrades and base courses where lasting effects are desired. Satisfactory temporary results are sometimes obtained by the use of as little as 1/2 of above percentages. Reference to cementing strength is implied when such terms as "Lasting Effects" and "Temporary Results" are used

Figure 15. Recommended amounts of lime for stabilization of subgrades and bases.

5. strengths from long-time curing tests;
6. permanence of effects desired from treatment; and
7. plasticity of soil and durability of portion retained on No. 40 sieve.

The percentages shown (Fig. 15) are for long-time cementing effects in which the layer involved becomes a part of the layer system of the pavement. As little as one-half of these amounts may be used if only temporary working table effects are desired. In this case, where the layer is treated with a small amount of lime, it should not be counted as part of the pavement system. The use of such low percentages in the treatment of some "gumbo" soils has made it very difficult to meet the specification requirement for pulverization (60 percent passing the No. 4 sieve). The chart is applicable to all soils except for a few which have a P.I. below 3 and contain less than 10 percent passing the No. 40 sieve. Ordinarily these types of soils are not treated with lime but occasionally this might be necessary. In such cases, a special investigation should be made encompassing a great deal more than mere use of the chart.

CONCLUSIONS

1. A study of cores from old roads can be helpful in the evaluation of soil-lime stabilization mixtures; the data indicate that (a) strengths of cores from properly constructed old roads can generally be expected to be from two and one-half to three times as great as those obtained under normal room-temperature curing periods; (b) the P.I. of the soil-lime mixtures has no relation to strength; and (c) 8 percent lime had less tendency for P.I. to increase with age than 4½ percent mixtures. (Portions slaked for soil binder showed that the 8 percent mixtures produced less soil binder than the 4½ percent mixtures.)
2. A study of cracking, rehealing and load deflections of existing jobs can also contribute to evaluation of lime stabilization mixtures.
3. Short-time strength tests probably will not identify optimum lime contents for most construction, but it is essential that they be made to check against use of soils which are nonreactive when treated with lime. Long-time tests would do a better job of identifying optimum lime contents, but are impractical from the standpoint of time and might not suggest the use of enough lime due to the ideal conditions under which they are run.
4. The use of pure fine limes decreases the cohesion of heavy clay soils more effectively than coarse pure or fine impure limes; therefore, difficulty of mixing should be decreased when the purest and finest limes available are used. Each agency will have to determine practical limits for purity and fineness. In our area it is believed that limes should be at least 90 percent hydrated lime and contain a minimum of 85 percent passing the No. 200 sieve when wet washed.
5. A chart based on the interrelationship of percent of relatively pure fine lime, plasticity index, and percent soil binder is presented and recommended for use. This chart is believed to meet most of the rigorous requirements of the state of the art at this time.

ACKNOWLEDGMENTS

The author is indebted to many who have contributed, encouraged, or assisted in the development of soil-lime stabilization in Texas. A few of these people or organizations have been mentioned; however, the work of the members of the Soil Section and other members of the Materials and Tests Division of the Texas Highway Department, under the guidance of A. W. Eatman, has been a major factor in making this report possible.

REFERENCES

1. McDowell, Chester. Development of Lime Stabilization for Improvement of Load Carrying Capacities of Pavements. Texas Highways Spec. Issue, May 1955.
2. Lund, O. L., and Ramsey, W. J. Experimental Lime Stabilization in Nebraska. Highway Research Board Bull. 231, pp. 24-57, 1959.

Durability of Lime-Stabilized Soils

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The results of a laboratory investigation of the durability of 10 lime-stabilized clayey soils from various parts of Canada are discussed. A fixed quantity of lime was added to each soil, based on the quantities recommended for highway subgrade applications. Durability was assessed by means of cyclic freezing and thawing, and changes in durability were determined by measuring the changes in length of the laterally confined specimens. Standard compression tests were performed on all samples following the cyclic freezing, and, in addition, changes in permeability, soil suction, degree of saturation, and tensile strength were observed on selected samples.

The changes in durability of the lime-stabilized soils are discussed in relation to a tentative theory based on the behavior of portland-cement mortars under similar conditions. The overall agreement with the modified theory was satisfactory, but further detailed testing is needed to establish some of the specific limiting values.

•DURING the past decade, there has been a considerable amount of field and laboratory research into the behavior of lime-stabilized soils. This research has tended to concentrate on three general areas: (a) fundamental mineralogical research into the lime-clay mineral reaction; (b) laboratory research into the variation in mechanical properties due to variation in lime percentage; and (c) field and laboratory research into the optimum construction procedures for various applications of lime-stabilized soils. The results have indicated that lime stabilization, when used intelligently, may have very definite economic advantages in certain clay subgrades. The introduction of this construction process into Canada has been hindered by the lack of substantial research into, or of a general hypothesis related to, the possibly adverse effects of frost action, although some initial work has been completed in the United States (2).

In an attempt to develop a fuller understanding of the behavior of lime-stabilized soils under frost conditions, the Canadian Lime Institute has supported a modest laboratory program at Queen's University for three years. The purpose of this program was twofold: (a) to obtain data for representative Canadian soils under freezing conditions, and (b) to examine critically the results of the tests with regard to explaining and analyzing the durability phenomena.

The absence of any general hypothesis to explain the frost-durability relationship for stabilized soils meant that an "ad hoc" approach had to be used initially; i.e., a substantial amount of somewhat empirical data had to be established before a consistent pattern could be predicted. In turn, this approach necessitated that certain arbitrary decisions concerning testing procedures had to be taken to avoid unnecessary examination of minutiae. As far as possible, the tests should be similar to those used elsewhere, but exact replication was not intended. As a result of this program, a general hypothesis was developed and is presented in this paper in conjunction with the test results. The hypothesis actually evolved in stages during the examination of preliminary

testing, but for convenience in discussion, it is presented here in advance of a discussion of the test results.

In general, the results of the study show that when lime is added to clay soils, heaving due to frost action can be reduced below tolerable limits. Silty soils are improved with the addition of lime, but frost damage may still be severe (based on laboratory results). The criteria for frost susceptibility of stabilized soils appear to differ substantially from those for unstabilized soils.

GENERAL HYPOTHESIS

Recent work (3, 5) has shown that the lime-clay reaction is a calcium-silicate reaction, and this is also discussed by Ingles (9) who suggests that the same complex silicates are formed in the lime-soil mixture as those in portland cement mortar. If this is the case, lime-stabilized soil systems will exhibit the same trends in mechanical properties as those for cement mortars: a distinct gain in strength with time, a strength dependency on the compactness of the specimen and on the type of cementing agent used, and an increased durability with increased entrained-air content. These trends are well documented in the literature. It also suggests that theories of frost behavior, as developed for uncemented soils, will not be strictly applicable to the problem of frost behavior for stabilized soils.

Theories of cement-mortar durability suggest that due to the calcium-silicate reaction, a substantial increase develops in the percentage of total voids as fine pores, and that the water within these pores does not freeze within the normal range of highway pavement temperatures. In this connection, Powers (18) states for cement mortars:

The granules, called gel particles, are exceedingly small, and interstitial spaces among them are correspondingly small. These spaces called gel pores are in fact so small that water cannot freeze in them at any temperature within the range of interest. . . . As hydration of cement proceeds, alkalis become more and more concentrated in the chemically free water. This fact, as well as dimensional factors already mentioned, influences the amount of ice that can exist in the paste at a given temperature. As freezing proceeds, pure ice separates from the solution. Hence freezing increases alkali concentration and lowers the melting point of the ice. . . .

Hardened cement paste in concrete normally has insufficient escape boundaries. Introduction of additional air voids by using air-entraining agents is a means of providing escape boundaries sufficiently close together. . . .

Typically, the gel-pore diameter in hydrated cements is in the order of 20 to 40 Å for 60 percent of the pore volume, and preliminary tests on frost-susceptible clays suggest pore diameters in the order of 5,000 Å. At high water-cement ratios, the permeability of hardened cement paste is in the order of 1.2×10^{-10} cm/sec, whereas silty clays may have permeabilities approaching 10^{-8} cm/sec.

In a generalized form, a modified cement-mortar hypothesis which may be applicable to the durability of lime-stabilized soils under freezing and thawing conditions may be summarized as follows:

1. The addition of lime to the soil creates a flocculation (or aggregation) which alters the normal pore size distribution in two respects: a number of substantially larger pores are created adjacent to the flocs; and within the flocs the pores are exceptionally small. Manifestation of this effect is seen by comparing grain-size distribution curves before and after lime treatment (19).
2. Within the finer pores, as the calcium-silicate reaction progresses, there is an increase in the alkalinity of the pore water due to the presence of sodium and potassium hydroxides, which depresses the freezing point below that normally encountered in highway conditions. Clare and Cruchley (1) have shown that pH values in the order of 12.5 can be expected for lime-stabilized soils.

3. The larger pores between the flocs enable the fluid to flow more readily through the soil, and are manifest by an increased permeability. (There is a paucity of experimental data related to the permeability of stabilized soil in the literature, and one of the auxiliary purposes of this investigation was to obtain some preliminary values.)

4. The complex calcium-silicate reaction develops the usual cementing agents which bind the pore walls together. In this connection, Diamond and Kinter (3) state:

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 tetra-calcium aluminate hydrate. . . .

5. When the temperature is lowered below the depressed freezing point in the larger pores, ice formation begins with the usual volume expansion as the crystal is formed and suction develops. The volume expansion places the remaining fluid under an hydraulic gradient, which can be dissipated through the increased permeability of the soil. The soil pore tends to act as a rigid container due to the cementing action which is developed, and immediate expansion does not take place. Eventually, the available pore space is filled with ice, excess water is dispersed to other parts of the system, and heaving is stopped as the ice-water interface is destroyed.

6. If the cementing action is not sufficiently developed (i. e., in terms of adequate curing time or strength) the hydraulic pressures generated due to crystal growth may exceed the tensile strength of the pore structure. Due to a rearrangement in pore geometry and the resulting loss of confinement of the pore structure, the induced suction is increased. This leads to ice segregation, further volume expansion, and deterioration.

7. The interrelationship between permeability and strength is modified by the degree of saturation of the sample and the freezing temperatures encountered. If an appreciable number of unfilled pores exists near the freezing front and the amount of water which may be transformed into ice is small, it will be easier for the hydraulic pressures to be dissipated. As a rough first approximation, Powers and Brownyard (17) have suggested that for a temperature of -10 C, approximately 75 percent of the soil water will form ice, with the remainder staying in a liquid phase. Walker and Karabulut (22) have suggested similarities between the soil-lime system and that for cement mortars, but the details are not given and the system is presumed to be similar to a closed system.

Accepting this general pattern, the hypothesis suggests that two separate soil properties may be used in the assessment of the durability of the stabilized specimen:

1. The strength of the cemented soil skeleton must be sufficiently developed so that it can resist the expansive forces from the excess hydraulic pressures. The tensile strength will be the most direct measurement, but the compressive strength may also be used in view of the interrelationship between these two tests.

2. The permeability of the soil must be sufficiently improved so that excess hydraulic pressure may be dissipated during the freezing period. The improvement in permeability will also be manifested by a reduced suction potential for the same moisture content. This property will be interrelated to the initial degree of saturation and curing conditions.

As a combined result of adequate strength and permeability, the hypothesis suggests that satisfactory stabilized soils are dimensionally stable. If the permeability and suction, or the degree of cementation and strength gain is inadequate, expansion occurs in the soil pores and heaving results.

Penner (15) has suggested that the application of an external pressure of 19.7 psi would be adequate to prevent frost heaving for a freezing point depression of 0.01 C. In subsequent experimental work (16), he has shown that for potters flint, in a closed

system, the application of approximately 1 kg/cm^2 (14.2 psi) of an equivalent overburden pressure was sufficient to reduce heaving. Hoekstra (8) measured the heaving pressure developed in frost-susceptible soils. For the marginal to non-frost-susceptible soils, a heaving pressure of 20 psi was developed, with greater values recorded for the more susceptible soils. Thompson (21) has shown that tensile strengths ranging from 20 to 200 psi may be anticipated for lime-stabilized soils.

Although the mechanism for heaving in a stabilized soil may be apparently different from that for unstabilized soils, the development of an internal tensile strength may not be too different from the application of an external pressure to the freezing process.

It was necessary to establish an overall survey of the problem before any detailed examination of an individual component began. For example, the permeability criterion would obviously be related to the laboratory freezing rate selected and the method of sample preparation, and a more rapid and severe freeze would require a higher permeability. The compressive strength parameters had been used in other investigations, and the length-change test has been used for studies of soil cement (Packard and Chapman, 14). For these reasons, the laboratory work concentrated on the strength-dimensional stability values with auxiliary tests performed for saturation, moisture content distribution, suction, and permeability.

TESTING PROGRAM

Although the number of durability investigations for lime-stabilized soils is limited, the work of Preus (19) in Minnesota, and of Davidson and his associates in Iowa has shown that the following trends may be anticipated: (a) increasing lime percentage, under comparable curing conditions, reduces lineal expansion; (b) a minimum curing time is necessary before the stabilized soil is capable of withstanding freezing and thawing conditions; and (c) the individual behavior of a stabilized soil could not conveniently be related to a single property such as plasticity.

To relate studies of Canadian soils with previous research, it was decided to perform tests essentially similar to those used in earlier studies. It was also decided to obtain a broad range of soil types for the study, and to use a consistent lime percentage, rather than using only one or two soil types with variations in the lime content and curing procedures. The apparent advantages of this procedure were (a) a representative range of mineralogical and particle size distribution for Canadian soils would be obtained, and

TABLE 1
PHYSICAL PROPERTIES OF 14 SOIL SAMPLES INITIALLY SELECTED
FOR LIME STABILIZATION-FROST INVESTIGATION

Sample No.	Location	L.L.	P.L.	P.I.	Sand (%)	Silt (%)	Clay (%)	Unified Class. Symbol	Activity ^a	Predom. Clay Mineral ^b
A1 ^c	90 mi NW of Edmonton	90	22	68	25	22.5	75	CH	0.91	M, I, Ch
A2	Drumheller	107	25	82	0	12	88	CH	0.93	
A3 ^d	40 mi NW of Edmonton	34	12	22	45	28	27	CL	0.82	
S1 ^d	15 mi N of Regina	32	12	20	39	40	21	CL	0.96	
S2 ^c	40 mi S of Regina	39	13	25	47	31	22	CI	1.14	M, I-K
S3	Regina	81	22	59	5	28	67	CH	0.88	
M1	60 mi NW of Wpg.	62	17	45	15	30	55	CH	0.82	
M2 ^c	20 mi N of Wpg.	79	23	56	5	24	71	CH	0.79	M, I, K-Ch
M3 ^d	Winnipeg	106	26	80	0.5	7	92.5	CH	0.86	
O1 ^c	London	38	17	21	10	46	44	CI	0.48	K-I, Ch
O2	Haileybury	52	20	32	17	41	42	CH	0.76	
O3 ^d	Welland	70	24	46	5	23	72	CH	0.64	
Q1 ^c	Quebec City	36	18	18	2	66	32	CL	0.56	
Q2 ^d	Montreal	52	21	31	18	46	36	CH	0.85	

^aActivity = P.I./clay fraction.

^bM = montmorillonite, I = illite, Ch = chlorite, K = kaolinite.

^c5 soils selected for preliminary investigation, 1962-1963 (stage I).

^d5 soils selected for verification analysis, 1963-1964 (stage II).

(b) secondary mineralogical effects which may develop between a particular soil mineral and lime type might not introduce a distorted test pattern when overall results were compared to the hypothesis.

Fourteen clayey soil samples were collected from widely distributed points after examination of geological and pedological maps, and after consultation with various members of provincial highway departments. The results of the standard identification tests and predominant clay minerals are given in Table 1. Subsequently ten samples were selected for detailed testing.

The lime percentage for each soil was determined by the method suggested by Hilt and Davidson (7) by which the "lime-fixation capacity" is obtained for each soil; i.e., the lime percentage above which changes in the plastic limit are minimal (Fig. 1). A reasonable correlation could be established between the fixation capacity and the percentage of clay sizes (Fig. 2). The design percentage was then obtained by adding an additional amount (+4%) above the fixation value for each of the soils.

Current construction practice allows the lime to be mixed and blended with the soil for a period of at least 24 hr before compaction, and this practice was followed for each sample at the design lime percentage. In addition, compaction is usually specified at 100 percent standard density at optimum moisture content conditions. Although the working hypothesis suggests that initial degree of saturation and dry unit weight (or strength) are important variables, it was decided to test the samples at the usual specified conditions. Subsequent studies could be used to establish what field limits can be tolerated. The results of the standard compaction tests for the untreated and treated soils, along with the design percentages of lime, are given in Table 2.

The specific freeze-thaw method selected consisted of several modifications to the British freeze-thaw method (Fig. 3). The test simulates the "open-ended" freezing

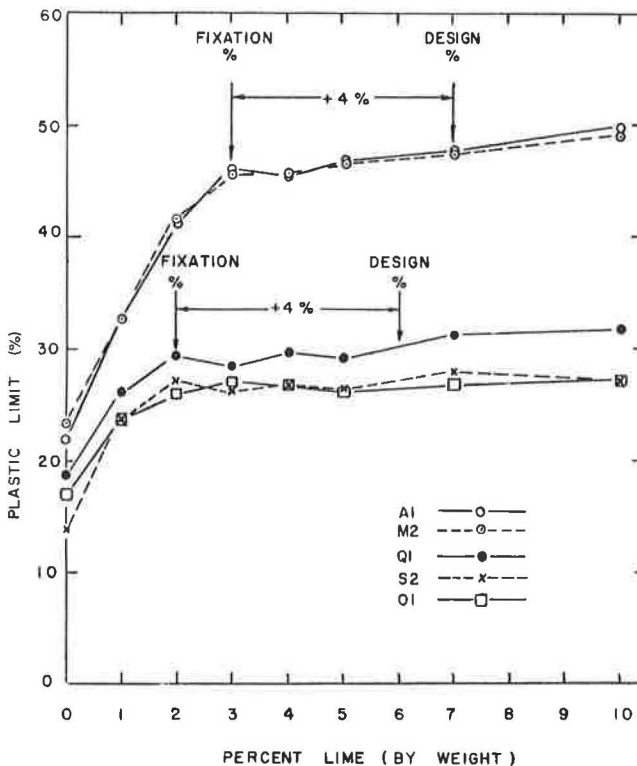


Figure 1. Relationship between plastic limit and percent lime illustrating method of selecting design percentage for stage I.

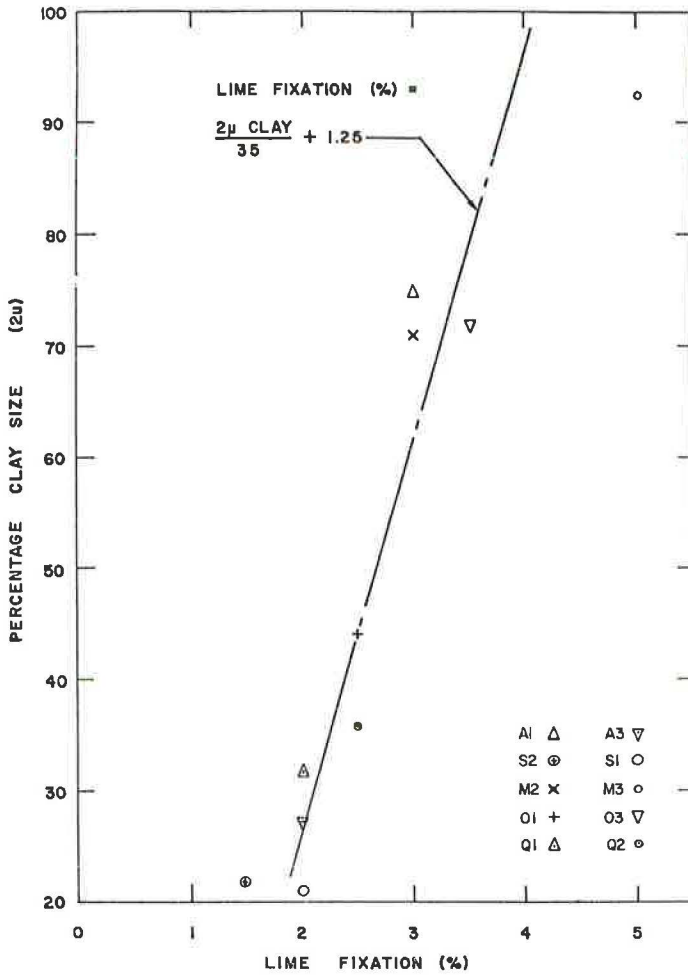


Figure 2. Relationship between fixation lime percentage and clay-size content.

TABLE 2
STANDARD COMPACTION TEST RESULTS AND
DESIGN PERCENTAGES OF LIME

Sample No.	Percent Lime		Untreated Soil		Treated Soil	
	Fixation	Design	Std. Density (pcf)	Opt. Moist. (%)	Std.	Opt.
A1	3	7	91.5	23.0	83.0	30.0
A3	2	6	114.8	13.5	107.8	16.0
S1	2	6	124.0	12.0	110.0	15.0
S2	1.5	6	103.0	18.0	99.5	17.0
M2	3	7	89.5	25.0	84.0	35.0
M3	5	7	88.0	28.0	80.4	28.0
O1	2.5	6	109.5	17.0	101.0	22.5
O3	3.5	7	94.6	18.0	87.3	26.0
Q1	2	6	108.4	17.0	98.7	24.5
Q2	2.5	7	101.5	23.0	91.8	20.0

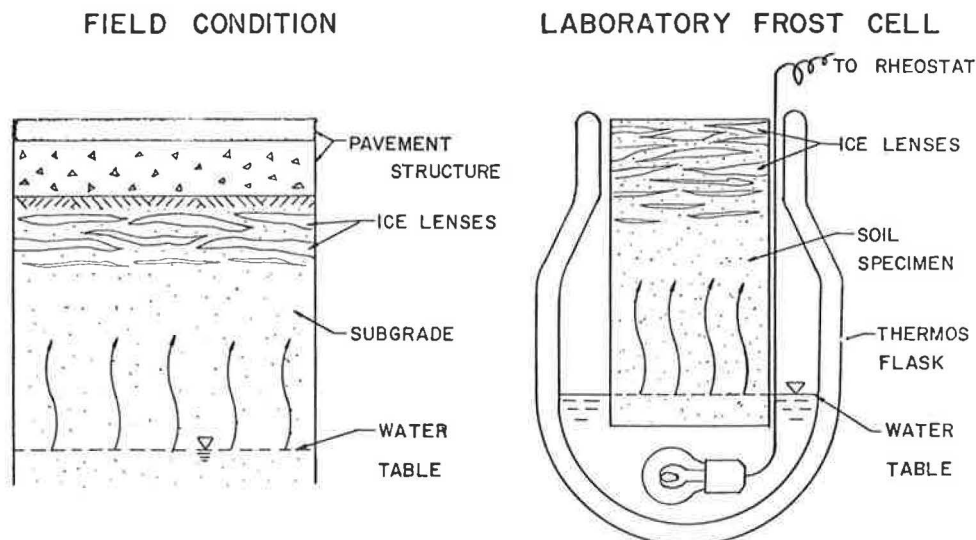


Figure 3. Design of laboratory frost cell to simulate field frost condition.

conditions that occur in the field, and has been correlated by Kalankamary and Davidson (11) to freezing conditions in Iowa. A cursory study of the general climatic differences between Iowa and central Canada showed that Canada has a slightly lower mean annual temperature so that some minor modifications were adopted. The main features of the freeze-thaw tests were

1. Freezing temperature was maintained at 12 F;
2. The number of freeze-thaw cycles was kept at 7;
3. The sample was subjected to 16 hr of freezing, followed by 8 hr of thawing at 72 F; and
4. An open-ended system was used, allowing additional water to reach the 3- × 2-in. diameter samples at all times.

To insure that the sample was subjected to substantial frost penetration, thermocouple measurements were taken during the initial series, and the foregoing cycle was selected so that the frost penetration was at least 2 in.

After the soil specimens had been compacted to 100 percent standard density and moisture by dynamic and static means, they were sealed in Saran Wrap, and cured at room temperature for varying periods of time. Selected samples were then subjected to freeze-thaw testing, or used as control samples. Moisture content distribution was measured in 1/2-in. thick layers for treated and untreated soil specimens after various periods of curing before or after the freeze-thaw cycling. In addition, length changes were measured during the tests by means of dial gages mounted on top of the specimens. Following the freezing cycles, selected samples were subjected to unconfined compression tests, tension tests, permeability tests, suction tests, and analyzed for moisture content.

EXPERIMENTAL RESULTS

Inasmuch as the samples were to be subjected to a variety of tests after the freeze-thaw cycling, an inordinate number of samples would be necessary if a statistical evaluation was required for each lime percentage and curing time for each soil. It was felt that the variety of soil types tested would present an overall pattern for this initial stage, and subsequent statistical details could be assessed by later testing.

The assessment of durability for a stabilized soil is purely subjective, especially when results are restricted to laboratory work, and a variety of criteria may be selected.

For the purposes of this research, a non-frost-susceptible stabilized soil was considered to be a sample which exhibited less than 0.02-in. heave (1.0% heave in the freezing zone) after 7 cycles of freezing and thawing under the prescribed conditions. This value was selected for the following reasons:

1. Dimensional stability of the soil is of more importance than residual strength, because in most cases in Canada lime stabilization is utilized for subgrade construction where stresses are a minimum.
2. Lineal change of freeze-thaw specimens of soil cement correlates with other assessments of durability. For a different method of freeze-thaw testing, Packard and Chapman (14) used 0.1 percent lineal change as a durability measure.

Strength Relationships

The results of the unconfined compression test determination for the ten samples are given in Table 3, and typical experimental results are shown in Figures 4, 5 and 6 for three of the soils tested. (The samples were numbered as follows: the first letter represented the particular province, this number being the sample number within that province, and the second number represented the percentage of lime added. Thus an A1-7 soil represents the first sample from Alberta with 7 percent lime added). Examination of the test results suggests that heavy and silty clays (CH) should develop a minimum strength of 200 psi before being subjected to frost action, and that clayey silts (CL) should develop a minimum strength of 300 psi.

Theoretically, the measurement of the developed tensile strength appears to be a more logical method of assessing the behavior of the cemented soil when subjected to frost action because the expansive pressures must overcome the tensile strength of the cell walls if heaving is to take place. Metcalf and Frydman (13) have shown that the tensile strength, as measured by the Brazilian or indirect tension test, is between one-twelfth and one-tenth of the unconfined compression strength for stabilized soils; Thompson has suggested a value approximately one-eighth the unconfined strength.

These previous investigations imply that the unconfined compression strength could be an adequate measure of the tensile strength; but for comparison purposes, indirect tensile tests were performed on samples of the first five soils tested. These results are also given in Table 3, and may be summarized as follows: for heavy and silty clays (CH), the minimum tensile strength should be between 20 and 30 psi; for silty clays (CL), the minimum tensile strength must exceed 45 psi.

Recently, Wissa et al. (23) have shown that the internal friction of a lime-stabilized silt remains essentially constant as the curing time is increased. A second trend showed that the cohesion intercept increases on a logarithmic scale as curing time increases, in a manner similar to that reported by Dumbleton (4) for the unconfined compression strength. Assuming that the Griffith crack theory is applicable to these brittle lime-stabilized clays, this second trend suggests that the tensile strength will also increase logarithmically with curing time. The increase in strength with time for the limited tensile and compressive strengths is shown in Figure 7 and confirms these trends. The implications of the trend suggest that lime-stabilized soils will become increasingly durable to freezing and thawing conditions as the curing proceeds, and this is also confirmed by the numerical values of lineal expansion recorded in this series. The Griffith crack theory (c.f. Jaeger, 10) also suggests that for a range of values of ϕ' from 30 to 37 deg, the tensile strength will vary from one-seventh to one-eighth of the unconfined compressive strength. A constant value is not to be expected because the ratio varies with the angle of internal friction, but this range of the ratio is generally consistent with these test results.

Permeability and Suction

The results of the unsaturated permeability tests on the first five samples are shown in Figure 8, and the method of test is given in the Appendix. In general, there was a substantial increase in the permeability for the non-frost-susceptible heavy clay soils, presumably due to the flocculation developed. The silty clay soils showed erratic or

TABLE 3
UNCONFINED COMPRESSION STRENGTH, TENSILE STRENGTH AND
LENGTH CHANGE FOR LIME-STABILIZED SOIL SAMPLES^a

Sample No.	q _u (psi)			s _t (psi)			Heave (in.)		
	14 Days Curing	28 Days Curing	4 Mo. Curing	14 Days Curing	28 Days Curing	4 Mo. Curing	14 Days Curing	28 Days Curing	4 Mo. Curing
A1-7	190	232	301	19	27	24	0.015	0.007	0.002
A3-6	130	180	456	-	-	-	0.088	0.005	-
S1-6	292	384	550	-	-	-	0.004	0.004	-
S2-6	167	228	431	17	24	49	0.258	0.005	0.005
M2-7	250	291	358	28	31	39	0.003	0.005	0.005
M3-7	130	156	112	-	-	-	0.020	0.048	-
O1-6	112	146	360	13	23	46	0.463	0.191	0.008
O3-7	191	206	307	-	-	-	0.474	0.259	-
Q1-6	110	134	289	16	20	47	0.522	0.114	0.080
Q2-7	229	268	408	-	-	-	0.004	0.004	-

^aq_u = average unconfined compression strength from test samples, corrected by a factor of 0.96 to convert to standard length vs diameter ratio;

s_t = tensile strength as determined by the indirect tensile test.

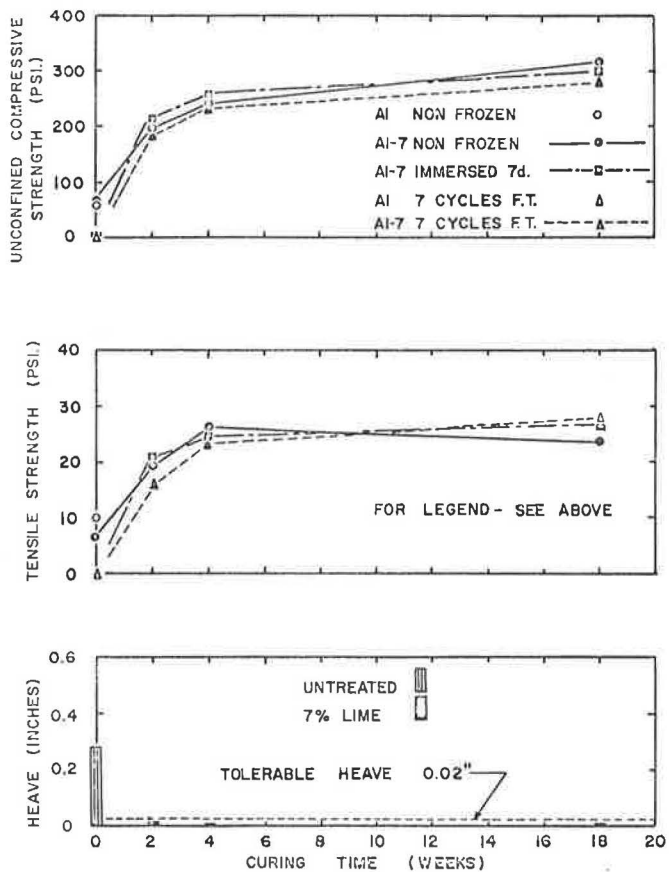


Figure 4. Relationship between unconfined compressive strength, tensile strength, and heave against curing time for soil A1.

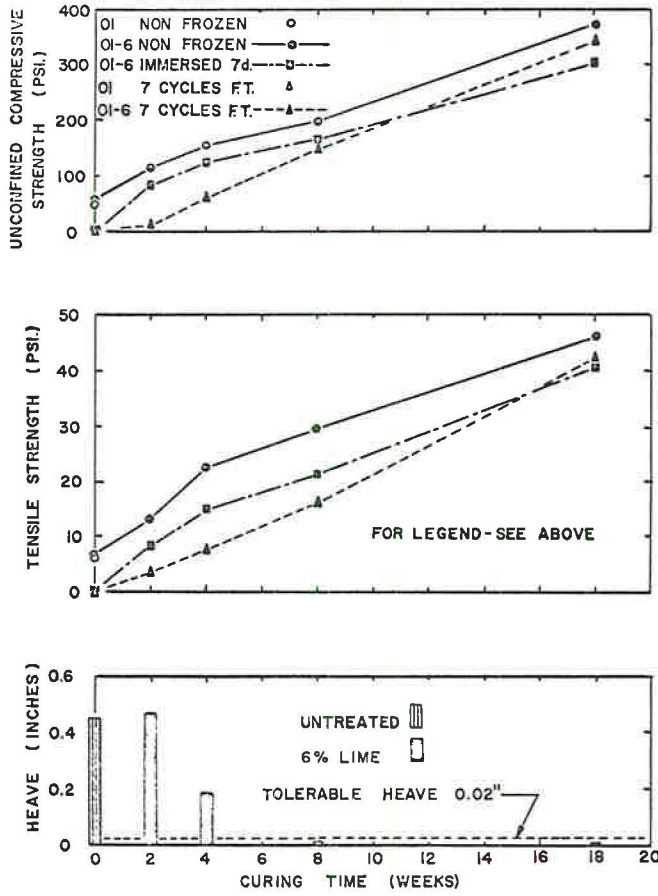


Figure 5. Relationship between unconfined compressive strength, tensile strength, and heave against curing time for soil O1.

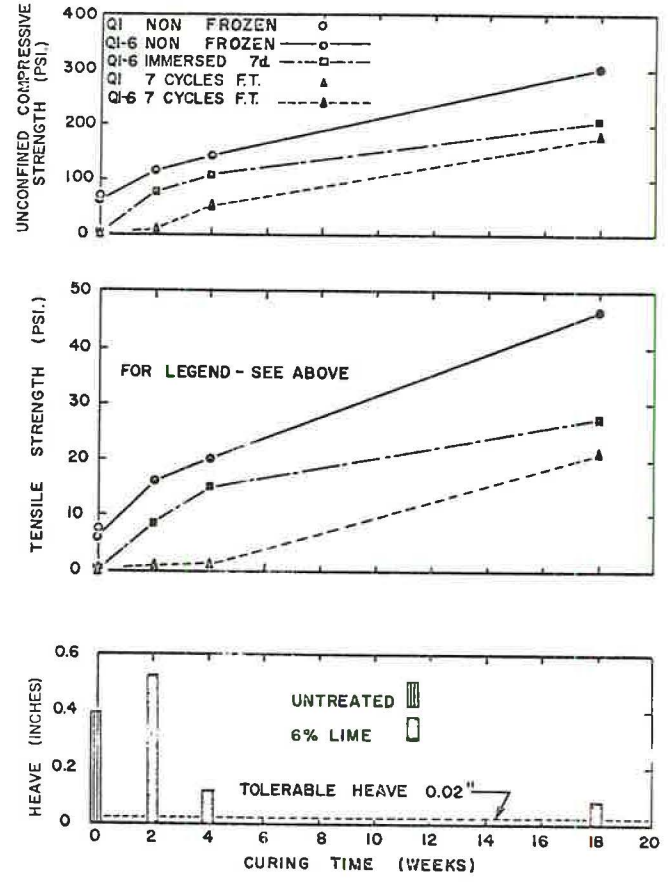


Figure 6. Relationship between unconfined compressive strength, tensile strength, and heave against curing time for soil Q1.

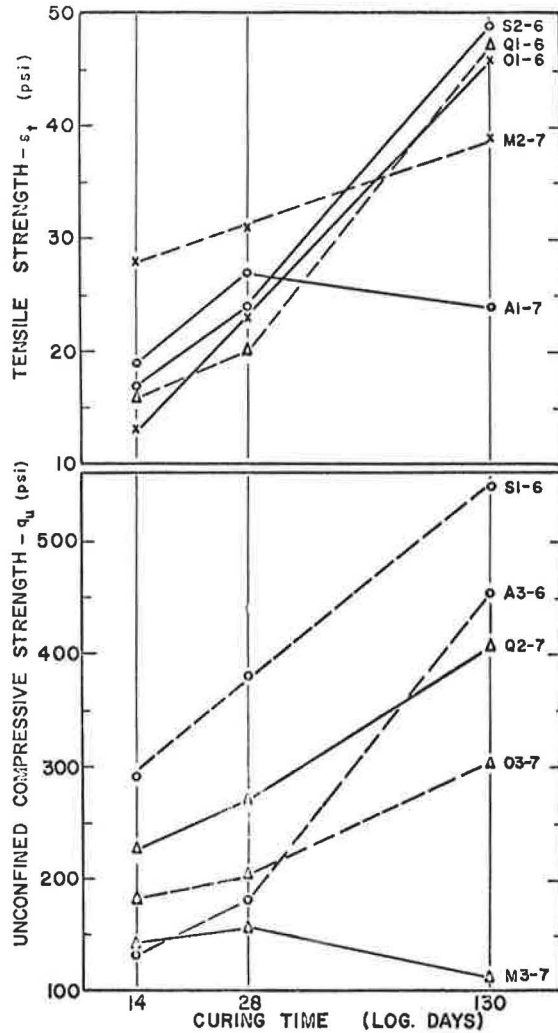


Figure 7. Relationships between unconfined compressive strength, tensile strength, and curing time.

no changes in permeability and were frost-susceptible under these conditions. The difference in void ratios between untreated and treated soils (0.10) may account for a ten-fold change in relative permeabilities. The important point is that permeability changes with time for the treated samples such as S2-6 and A1-7 while at a constant void ratio. These samples developed an adequate resistance to the test freezing and thawing.

Powers (18) claimed that deterioration by frost in hardened cement pastes in concrete occurs mainly due to low permeability. It is suggested that the use of air-entraining agents has proven to be successful in providing adequate escape channels to dissipate the hydraulic pressures associated with the growing ice crystals. The pattern in the lime-stabilized soils appears to be similar.

Recently, Fossberg (6) has suggested that the application of lime to clay soils develops a lowering of the permeability; typically, a reduction from 10^{-6} to 10^{-8} in./min after 28 days curing and at a void ratio of 1.5. However, these samples were molded immediately after the addition of lime so that there was little opportunity for flocculation to develop.

Laguros (12) has reported that the permeability of a soil increased from 8.6×10^{-6} cm/sec to 6.8×10^{-5} cm/sec, although exact details of the "rotting" time are not given.

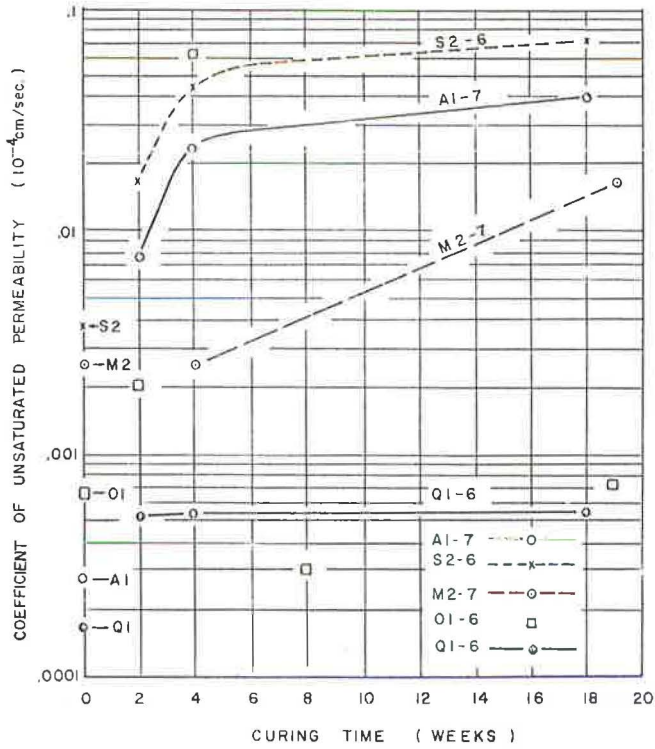


Figure 8. Relationship between coefficient of unsaturated permeability for untreated and lime-treated soils at various curing periods.

Ranganatham (20) computed the value of permeability for lime-stabilized soils from the results of consolidation tests. These samples were allowed to rot for 1 wk (and hence develop flocculation) before testing. In these cases, the permeability increased by at least 10 times, and this trend is consistent with those reported here. It is suggested that when permeability tests are conducted for lime-stabilized soils the samples should be allowed to cure in an uncompacted state similar to the conditions which develop in the field.

The results of the drying curve suction tests are shown in Figure 9, and indicate that the suction potential of the soil is decreased under the foregoing methods of sample preparation. This trend is consistent with the permeability results, as an increase in the size and number of the larger pores is indicated.

The results of Clare and Cruchley (1) and Fossberg (6) show that the suction is increased due to the addition of lime to the soil. However, in both cases, the samples were not allowed to develop any substantial flocculation so that the results are not directly comparable.

Degree of Saturation

Both the suction and permeability characteristics are affected by the initial degree of saturation of the sample. For samples with low initial saturation after molding and curing, the air voids will provide reservoirs into which the excess hydraulic pressures may dissipate, thus reducing the heaving pressures created during ice formation. The relationship between frost-susceptible and non-frost-susceptible soils and the degree of saturation before and after freezing is shown in Figure 10, and it is suggested that

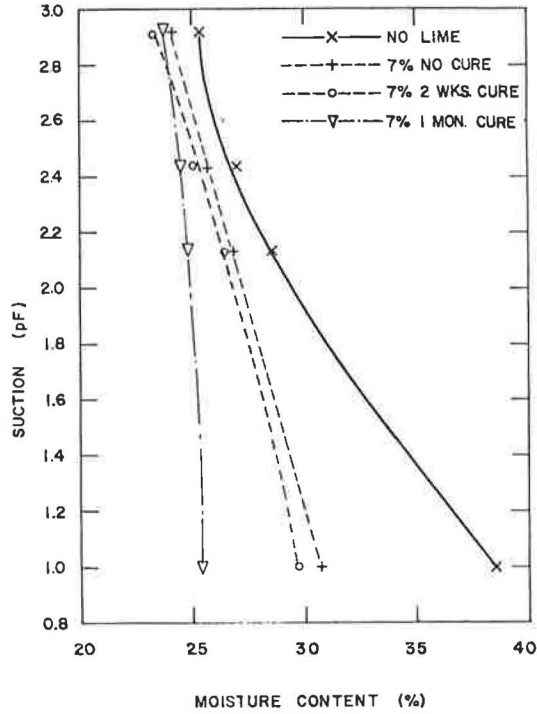


Figure 9. Suction moisture content relationships for specimens of soil Q2 with and without lime addition.

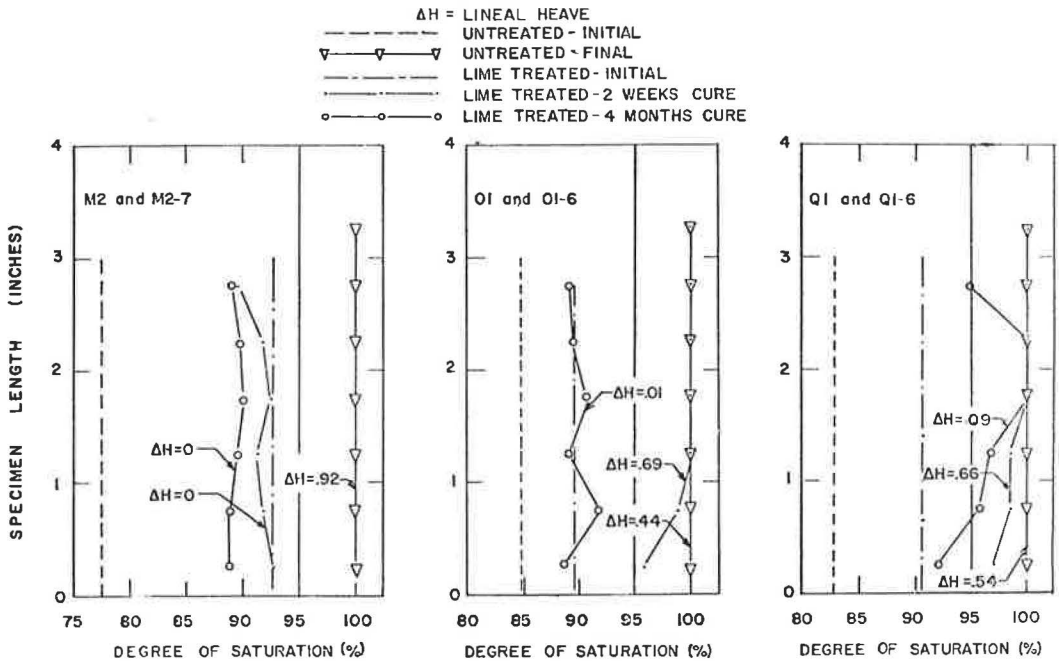


Figure 10. Degree of saturation profiles for untreated and lime-treated specimens before and after frost action.

when lime-stabilized soils possess an initial degree of saturation less than 95 percent, providing that adequate strength is developed, these soils will be non-frost-susceptible.

These results were obtained by cutting the freeze-thaw specimens into 0.5-in. layers and measuring the moisture content before and after the full cycle was completed. All samples were frost-susceptible in the untreated state. Samples M2-7 and O1-6 were non-frost-susceptible after 2 wk and 4 mo cure, respectively, whereas sample O1-6 still heaved after 4 mo. The absence of an altered moisture profile in the durable soil-lime specimens suggests that no further water is being drawn into the system.

The saturation data should be considered as tentative at this stage, because the volumetric expansion and moisture distribution will also depend on the freezing temperatures selected and the type of sample preparation.

DISCUSSION

There are several unanswered questions of substance related to the development of this hypothesis. For example, what is the exact nature of the pore-size redistribution? What influence has the pore-water alkalinity and cyclic-freezing temperature on the freezing point depression and suction?

The behavior of the soil-lime system appears to parallel that of the cement-mortar system insofar as the development of a durable soil under freezing conditions. An alteration of the soil void system is developed which indicates an increase in the number of large voids due to flocculation, and a corresponding increase in the number of small voids within the cement-gel (and within which presumably the water does not freeze). This void size distribution, when coupled with the developed strength, reduces the frost susceptibility of the natural soils.

In the absence of field data, the results of this preliminary laboratory study do not permit the establishment of limiting design criteria for use in routine highway construction. However, several important concepts have been indicated which should be useful for further laboratory research, and several design criteria are indicated which can be used for the preliminary field tests.

The experimental criterion for dimensional stability, a 1 percent length change within the frozen zone of the sample, is presumed to be adequate for subgrade conditions. This dimensional stability is developed for heavy clay samples when a compressive strength on the order of 200 psi is developed, or a tensile strength of 30 psi. These values are applicable for samples compacted to 100 percent standard density and moisture. The durable samples had unsaturated permeabilities larger than 5×10^{-8} cm/sec, but the interrelationship between strength, permeability, and freezing conditions needs to be further investigated before these values may be used for all soils.

The determination of the lime-fixation point is one of the primary steps in arriving at a suitable lime percentage. Generally, if the amount of lime used is below the percentage needed for fixation, the soil will merely be modified. This means that its plastic behavior will be altered, but that no substantial strength development will take place. If the design lime percentage exceeds the fixation point by 4 percent, the treated soils would be resistant to frost deterioration, providing that the soil activity exceeds 0.75, and the silt content does not exceed 50 percent. The modifying effect of lime on the durability of underdesigned stabilized soils, and the possibility of utilizing fly ash to promote the development of the complex calcium-silicate cements need further investigation.

The primary purpose of the research was to establish a general hypothesis for the durability of typical lime-stabilized Canadian soils. The arguments developed are consistent with trends developed elsewhere and the results obtained. It is suggested that there is a rational explanation for the arbitrary requirements for non-frost-susceptible stabilized soils, such as requiring a minimum unconfined compressive strength after cyclic freezing. The results also suggest that the reason for the differing frost behavior of various soil types lies in the amount of complex silicates which may be developed, and which in turn will influence the tensile strength and amount of flocculation.

REFERENCES

1. Clare, K. E., and Cruchley, A. E. Laboratory Experiments in the Stabilization of Clays with Hydrated Lime. *Geotechnique*, Vol. 7, June 1957.
2. Davidson, D. T., Mateos, M., and Barnes, H. F. Improvement of Lime Stabilization of Montmorillonitic Clay Soils with Chemical Additives. *HRB Bull.* 262, pp. 33-50, 1960.
3. Diamond, S., and Kinter, E. B. Mechanisms of Soil-Lime Stabilization: An Interpretive Review. *Highway Research Record* 92, pp. 83-103, 1965.
4. Dumbleton, M. J. Investigation to Assess the Potentialities of Lime for Soil Stabilization in the United Kingdom. R.R.L. Tech. Paper 64, Her Majesty's Stationery Office, London, 1962.
5. Eades, J. L., Nichols, F. P., and Grim, R. E. Formation of New Minerals with Lime Stabilization as Proven Field Experiments in Virginia. *HRB Bull.* 335, pp. 31-39, 1962.
6. Fossberg, P. E. Some Fundamental Engineering Properties of a Lime-Stabilized Clay. *Proc. 6th I.C.S.M.F.E.*, Montreal, Vol. 1, p. 221, Univ. of Toronto Press, 1965.
7. Hilt, G. H., and Davidson, D. T. Lime Fixation in Clayey Soils. *HRB Bull.* 262, pp. 20-32, 1960.
8. Hoekstra, P., Chamberlain, E., and Frate, T. Frost-Heaving Pressures. *Highway Research Record* 101, pp. 28-38, 1965.
9. Ingles, O. G. Bonding Forces in Soils. *Proc. Australian Road Res. Board*, Vol. 1, Pt. 2, 1962.
10. Jaeger, J. C. *Elasticity, Fracture, and Flow*. Wiley, 1962.
11. George, K. P., and Davidson, D. T. Development of a Freeze-Thaw Test for the Design of Soil-Cement. *Highway Research Record* 36, pp. 77-96, 1963.
12. Laguros, J. G. Lime-Stabilized Soil Properties and the Beam Action Hypothesis. *Highway Research Record* 92, pp. 12-20, 1965.
13. Metcalf, J. B., and Frydman, S. A Preliminary Study of the Tensile Stresses in Stabilized Soil Pavements. *Proc. Australian Road Res. Board*, Vol. 1, Pt. 2, 1962.
14. Packard, R. G., and Chapman, G. A. Developments in Durability Testing of Soil-Cement Mixtures. *Highway Research Record* 36, pp. 97-122, 1963.
15. Penner, E. Pressures Developed in a Porous Granular System as a Result of Ice Segregation. *Highway Research Board Spec. Rept.* 40, pp. 191-199, 1958.
16. Penner, E. The Mechanism of Frost Heaving in Soils. *HRB Bull.* 225, pp. 1-21, 1959.
17. Powers, T. C., and Brownyard, T. L. Physical Properties of Hardened Cement Pastes. *ACI Jour.*, p. 990, 1947.
18. Powers, T. C. Basic Considerations Pertaining to the Freezing and Thawing Tests. *Proc. ASTM*, Vol. 55, pp. 1132-1155, 1955.
19. Preus, C. K. Laboratory Evaluation of Lime Treatment of Typical Minnesota Soils. State of Minnesota, Dept. of Hwys., Investigation No. 64, 1964.
20. Ranganatham, B. V. Soil Structure and Consolidation Characteristics of Black Cotton Clay. *Geotechnique*, Vol. 11, p. 331, 1961.
21. Thompson, M. R. The Split Tensile Strength of Lime Stabilized Soil. *Highway Research Record* 92, pp. 69-83, 1965.
22. Walker, R. D., and Karabulut, C. Effect of Freezing and Thawing on Unconfined Compressive Strength of Lime-Stabilized Soils. *Highway Research Record* 92, pp. 1-9, 1965.
23. Wissa, A. E., Ladda, C. C., and Lambe, T. W. Effective Stress Strength Parameters of Stabilized Soils. *Proc. 6th I.C.S.M.F.E.*, Montreal, Vol. 1, p. 412, Univ. of Toronto Press, 1965.

Appendix

TEST METHOD FOR UNSATURATED COEFFICIENT OF PERMEABILITY

The coefficient of unsaturated permeability was determined for both untreated and lime-treated soils at varying curing periods before and after frost action. The apparatus operated on the falling head principle and the essential components and assembly methods are shown in Figure 11.

The test specimen, coated with a silicone lubricant to prevent lime-grease reaction, was placed on the screen inside the lucite tube (Fig. 11). The bottom of the tube containing the specimen on the screen was then immersed in about $\frac{1}{4}$ in. water so that the water level was even with the bottom of the specimen. Preparatory investigations using only wax around the entire specimen proved unsuccessful because wax shrinkage allowed water to escape along the sides of the specimen. A combination of a $\frac{3}{4}$ -in. wax seal at the bottom of the specimen and a grease seal around the remaining portion of the specimen proved effective. Melted wax was then poured through a thin-stemmed funnel until it reached the level of the grease fittings located in the lower portion of the apparatus. When the wax was thoroughly set, automotive grease was forced through the four grease fittings uniformly until the specimen was completely encased.

The top of the apparatus was then assembled and de-aired water was poured into the burette until the apparatus was completely filled. Air bubbles were removed through

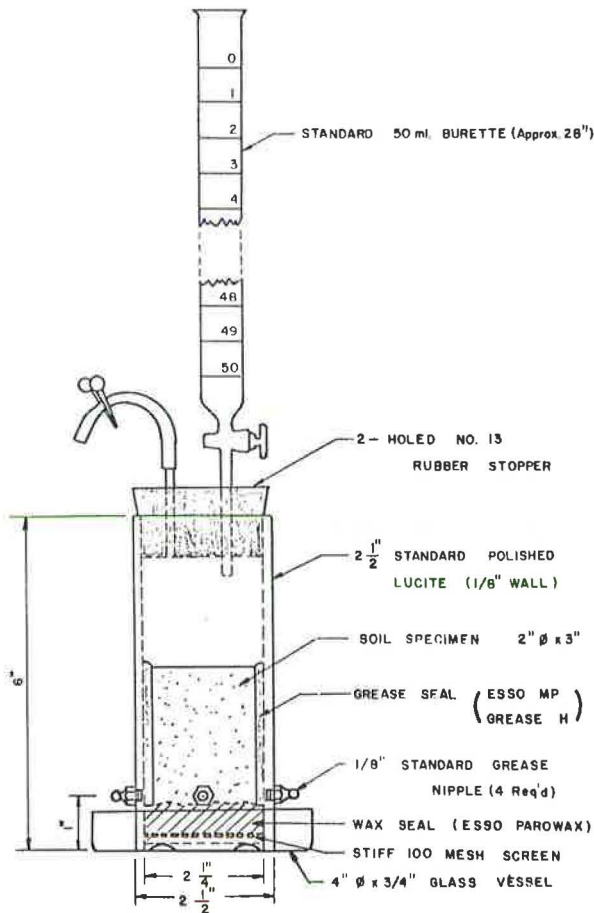


Figure 11. Apparatus used to measure unsaturated coefficient of permeability.

the vent at the top of the rubber stopper. No attempt was made to remove air from the specimens because under field conditions the degree of saturation would be less than 100 percent. De-aired water was used as a standard in all tests. Before testing commenced, the apparatus was kept in operation until steady flow conditions prevailed, generally requiring the apparatus to operate overnight. Typical test values measured ranged from 3.8×10^{-4} cm/sec to 1.7×10^{-8} cm/sec.

Time-Temperature Strength-Reaction Product Relationships in Lime-Bentonite-Water Mixtures

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The interrelation of curing time, curing temperature, strength, and reactions in lime-bentonite-water mixtures was examined. Samples were molded at constant density and moisture content and then cured for periods of from 1 to 56 days at constant temperatures that ranged from 5 C to 60 C. After the appropriate curing time the samples were tested for unconfined compressive strength. The broken samples were then analyzed by X-ray diffractometer and spectrophotometer to determine the identity of the reaction products present after each curing period.

It was found that the strength gain of lime-clay mixtures cured at different temperatures is due to different phases of the complex reaction, $\text{lime} + \text{clay} \rightarrow \text{CSH}(\text{gel}) \rightarrow \text{CSH}(\text{II}) \rightarrow \text{CSH}(\text{I}) \rightarrow \text{tobermorite}$. The farther the reaction proceeds, the higher the strength. There was also evidence of lattice substitutions in the structure of the calcium silicate hydrates at curing temperatures of 50 C and higher. No consistent relationship between time, temperature, strength, and the S/A ration of reaction products existed, but in order to achieve high strengths the apparent C/S ration had to be less than two.

The curing temperature had an effect on the strength developed by a given amount of reacted silica in the cured lime-clay mixture, but at a given curing temperature the cured sample that had the largest amount of reacted silica gave the highest strength.

Evidence was found to indicate that during the clay reaction some calcium is indeed adsorbed onto the clay structure rather than entering into a pozzolanic reaction.

Finally, it was determined that it is possible to determine the amount of silica and alumina in lime-clay reaction products by spectrophotometric analysis with sufficient accuracy for comparison purposes. The spectrophotometric analysis techniques used during the investigation were simple and were not time consuming.

•LIME and portland cement have been tested extensively to determine their efficiency in the stabilization of various soils. Portland cement pastes have also been thoroughly investigated to determine the mechanisms by which cementation occurs. Less investigation has been made of the soil-lime reaction, but what has been done indicates that the reaction products are similar to those formed during the hydration of portland cement. However, most of the work that has been done in determining the mechanism of the soil-lime reaction has been concerned with the effect of time on the strength of the soil-lime mixture, and the effect of time and/or temperature on the reaction products formed. Little attention has been given to the interrelation of time, temperature, and strength of lime-stabilized soil.

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The objective of this investigation is to study this interrelation of curing time, curing temperature, strength, and reactions in lime-clay mixtures. It will be noted that previous time-temperature-strength studies were done before lime-clay reaction products were known or identifiable, so it seemed appropriate to draw together these two paths of research, using one to explain or show the significance of the other.

REVIEW OF LITERATURE

Lime-Clay Reaction

The production of hydrates by a lime-clay reaction has been termed a "pozzolanic" reaction. A pozzolan is a siliceous or siliceous aluminous material which has little or no cementitious value, but will react with calcium hydroxide in the presence of moisture to form cementitious compounds (1, 23). A pozzolan can be either a natural or artificial material.

The beneficial effect of lime on the workability and strength of clayey soils has long been recognized. However, it is only in the past few decades that the mechanisms by which this improved strength and workability are obtained have gradually become understood.

It now appears that the lime-clay reaction takes place in two steps. Davidson and Handy (6) suggest three basic reactions: (a) the calcium ions cause a reduction in the plasticity of the clay; (b) the lime may be carbonated by the carbon dioxide in the air resulting in the formation of a weak cement; and (c) a cementitious reaction takes place between the lime and clay mineral. According to Hilt and Davidson (14) before any pozzolanic reaction takes place, the lime content of the lime-clay mixture must exceed the amount needed to modify the clay.

Studies made on the products of lime-clay systems have shown that calcium silicate and calcium aluminate hydrates are formed in various forms at temperatures ranging from room temperature up (9, 10, 13, 26, 44).

Several investigators have studied the strength characteristics of the calcium silicate hydrates and have characterized all the products as cementitious (19, 24, 28, 34, 40). Furthermore, the formation of calcium aluminate and calcium silicate hydrates during the hydration of portland cement is believed to be responsible for the strength and cementing ability of portland cement concrete (3, 5, 22), although the aluminates play a less significant role (3). The calcium silicate hydrate known as tobermorite gel, a poorly crystalline colloidal substance, has been called the "heart of concrete" (4). Jambor (18) found that the kind and microstructure of calcium silicate hydrate as well as the amount formed has an effect on the strength developed by hardened lime-pozzolana pastes.

Reaction Products

The major reaction products now recognized as forming during the lime-clay reaction are tobermorite, CSH(I)¹, CSH(II), CSH(gel), calcium aluminate hydrates, and hydrogarnet (9, 13, 18, 44).

The first four of these compounds are members of what is known as the tobermorite group of calcium silicate hydrates (36). The structure of tobermorite is similar to that of some clay minerals (25, 39). All members of the tobermorite group, however, do not have precisely the same structure. These calcium silicate hydrates, in addition to the structural similarities, also have the small particle size, large surface area, and a number of other properties analogous to clay minerals (7, 37).

Time, Temperature, Strength Interrelationship

As long ago as 1886 it was recognized that there was some relationship between curing temperature, curing time, and strength of portland cement and lime-sand cement (42). Since that time other investigators have studied the interrelation of

¹It is customary in cement chemistry to denote the following compounds by short symbols: CaO = C; SiO₂ = S; Al₂O₃ = A; Fe₂O₃ = F; MgO = M; H₂O = H. Thus, nCaO · pSiO₂ · qH₂O is represented by CSH.

strength, time, and temperature of concrete (3, 29, 30, 31, 35). Three principal approaches to the problem have been made. Bergström (2) suggested a "maturity" rule such that any given concrete would attain the same strength if $A(T - \theta)$ were constant. In this expression, A = time of curing, T = curing temperature (C), and θ = the temperature at which no increase in strength occurs.

Plowman (30) modified this to the form

$$\text{UCS} = \text{Constant} + \log A(T - \theta)$$

Rastrup (31), after studying the hydration process, based his maturity rule on the form

$$\text{UCS} = f(T_a)$$

where

$$T_a = \int_0^A 2^{(T - \theta)/10} dA$$

T_a is the maturity at curing temperature T , after a curing time A .

The effect of curing temperature on the strength of soil-lime mixtures has also been studied (8, 27). It was found in these studies that the effect of increased curing temperature was to increase the strength of the soil-lime mixtures at the same age. Metcalf (27) also showed that although none of the maturity rules above hold exactly for soil-lime mixtures, his results most nearly follow the rule proposed by Rastrup.

Metcalf (27) assumed that the reaction between lime or cement and clay could be represented by the Arrhenius equation

$$k = B e^{-E/RT}$$

where

- k = the reaction rate,
- E = the activation energy,
- R = the gas constant,
- T = the temperature (in degrees Kelvin), and
- B = constant.

He then plotted his results on the basis

$$\log(\text{UCS}) = B' - B''/T$$

where B' and B'' are constants. He found that the results of cement-stabilized soils plot a constant slope over the range 0-65 C, indicating that the hardening action in that temperature range is essentially similar and independent of the type of soil.

For lime-stabilized clays, however, Metcalf found that not only was the slope of the curves different for the different clays, but that there was an abrupt change of slope in the vicinity of 45 C. He made no comment concerning the change of slope at 45 C, but, he did conclude that the lime-clay reaction responsible for the strength of the lime-clay mixture is not the same for all clay minerals.

PROCEDURES

Materials

The clay used in this investigation was a natural Ca-saturated "Panther Creek southern bentonite" from White Spring, Mississippi. The lime was a powdered analytical reagent grade calcium hydroxide. Distilled water was used in the preparation of all specimens.

Molding

In order to reduce the variables to a minimum, only one ratio of lime to clay was examined. A C/S ratio of approximately 1.0 was chosen for the study because Wang

(44) had established that this C/S ratio produced a variety of reaction products over the curing times and temperatures to be studied. The clay used in the investigation was chosen because some information concerning its reaction with lime had already been developed by Wang.

Proper amounts of lime, clay, and water were hand mixed until a uniform mixture was obtained; then the amount of material required to attain the maximum density was weighed out and placed in the mold. The specimens were molded into 1-in. high by $\frac{1}{2}$ -in. diameter cylinders and to constant density at optimum moisture content for maximum density.

The use of 1-in. high by $\frac{1}{2}$ -in. diameter strength-test specimens results in considerable savings of time and materials. The results obtained reflect the cohesive strength of the lime-clay system, and are sufficiently valid for comparative studies. The molding apparatus has been described in detail by Roderick (32).

Curing

After being molded, the samples were placed on a rack inside an airtight hard plastic Lustr-o-wave bowl in which distilled water had been so placed that the water was not allowed to come in contact with the specimens. The bowls were further sealed at the top with cellophane tape to prevent carbonation and the loss of moisture, before being placed in the appropriate curing chambers. Electric ovens were used as chambers for the 40 C, 50 C, and 60 C curing. The 23 C curing was performed in a temperature-controlled humidity room. The 5 C curing was done in a small laboratory refrigerator.

After completion of specified curing, the specimens were tested for strength by a proving ring type of compression apparatus, accurate to ± 25 psi. The average of the five specimens was reported as the unconfined compressive strength.

Following the strength testing, all the broken samples of the same curing time and temperature were placed in a desiccator and vacuum dried over a mixture of CaCl_2 and ascarite for a period of at least 48 hours. The dried samples were ground by hand to pass the No. 200 sieve and were stored for X-ray and chemical analyses.

X-Ray Diffraction

A General Electric XRD-5 diffractometer and nickel-filtered copper $K\alpha$ radiation were used for examining the presence of crystalline reaction products and for the determination of the unreacted calcium hydroxide in the cured mixtures. The powder samples were pressed into disc-shaped brass rings with a 1,000-psi presser to avoid effects of preferred orientation and to give good reproducibility (33). To further avoid the effects of preferred orientation, the discs were continually rotated while exposed in the X-ray beam (12).

Determination of Calcium Hydroxide Content

The quantitative determination of the calcium hydroxide content of the cured mixture was made using the internal standard method outlined by Klug and Alexander (21).

Quartz powder was used as the internal standard. The standard series of mixtures used in preparation of the calibration curve was made up from finely ground quartz powder, calcium hydroxide, calcium carbonate, and the clay. The mixtures were placed in 4-gm glass vials with three small pieces of rubber and mixed in a Spex Model 8000 vibratory mixer/mill for five minutes to assure thorough mixing. Five samples of each mixture were then examined by X-ray analysis, and the intensities of the $d = 2.62 \text{ \AA}$ (calcium hydroxide) and $d = 2.45 \text{ \AA}$ (quartz) peaks were compared. Because of the linearity of the curves and the reproducibility of the data, it was not necessary to use additional mixtures.

In the determination of the intensities, allowances were made for the background intensity. Figure 1 shows a composite chart of the X-ray traces of quartz, calcium hydroxide, montmorillonite, and tobermorite for copper $K\alpha$ radiation from 32° to $40^\circ 2\theta$; the background under the calcium hydroxide peak at approximately 34.1° was assumed to be equal to the background at around 39° . Similarly, the background under

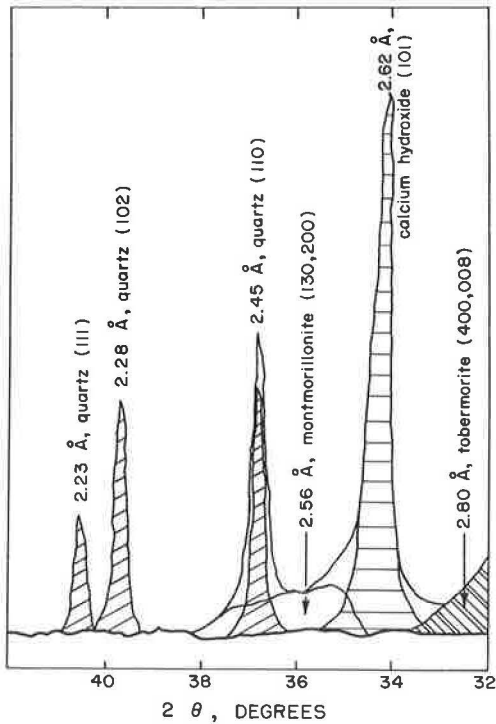


Figure 1. Composite chart of X-ray traces of quartz, calcium hydroxide, montmorillonite and tobermorite for copper $K\alpha$ radiation from 32° to $40^\circ 2\theta$.

tensity at 38.2° , and the distance from the calcium hydroxide peak to this line was considered the peak intensity of the calcium hydroxide. The heights were measured rather than areas, because observations showed that there was no appreciable difference in line broadening.

The quantitative analysis for calcium-hydroxide in the cured mixtures was carried out under the same conditions as the preparation of the calibration curves. In this case, 0.5 gm of ground quartz was mixed with 2.5 gm of the ground cured sample, and the calcium hydroxide content was found by comparison of the ratios of the intensities of the lines indicated above with those of the standard curve. The hygroscopic moisture content of the sample was determined, and the calcium hydroxide content was reported as grams per 100 gm oven-day mixture.

Determination of Silica and Alumina Content

The amounts of silicates and aluminates formed in the reaction mixtures may be determined by chemical means because these reaction products are soluble in dilute HCl, whereas the solubility of the clay mineral is negligible under certain specified conditions (22).

Preliminary Tests for Optimum Conditions for Extraction

Time of Extraction.—The procedure is designed to extract the maximum amount of Ca-silicates and aluminates formed with the minimum dissolution of clay minerals. Preliminary tests had shown that pure tobermorite synthesized from quartz and lime at 175°C in a saturated steam autoclave can be dissolved completely in a sufficient amount of 0.1 N HCl within 30 min by continuous shaking. It also had been shown that clay or soil samples are not subjected to any extensive breakdown under this condition.

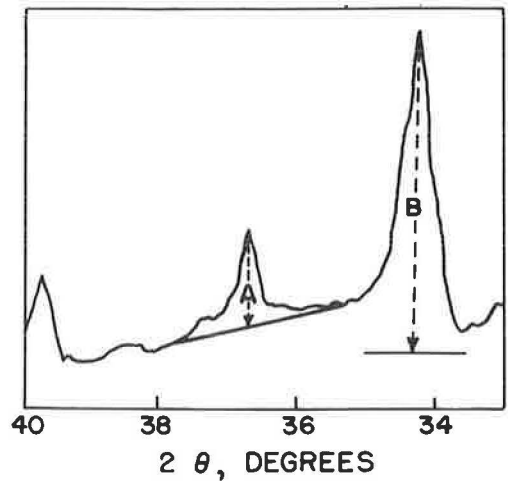


Figure 2. Example of calculation of the intensities of the quartz and calcium hydroxide peaks.

the quartz peak at 36.8° was taken as a straight line connecting the background at 38.2° and 35.9° . These observations were used in the calculation of the intensities of the quartz and calcium hydroxide peaks as shown in Figure 2. A line was drawn connecting the intensity at 38.2° to the intensity at 35.9° , and the distance from the quartz peak to this line was considered the peak intensity of the quartz.

Another line was drawn parallel to the in-

Proper Concentration of HCl.—The reaction products must be dissolved in HCl of such strength as to give a final supernatant pH of between 1 and 2. The pH is critical for two reasons. First, the rate of polymerization of silicic acid dissolved is a minimum in this pH range (17). Second, the aluminum ion is stable at this pH range, but slowly polymerizes and precipitates as $\text{Al}(\text{OH})_3$ at a pH greater than 4.5. In addition, the strength of the HCl must be such that all the calcium silicates and aluminates are dissolved and at the same time a minimum amount of clay is dissolved. The optimum strength of the HCl used for extraction varies depending on the clay and the lime content of the lime-clay mixtures. To determine the optimum acid strength for the lime-clay mixture used to the present investigation, a series of tests was made by shaking 0.5 gm of the sample in 50 ml HCl for 30 min.

From the results of these tests, it was decided that an acid concentration of 0.20 N and a 30-min shaking time best met the criteria.

Sample Extraction.—A suspension of 0.5 gm sample in 50 ml of 0.2 N HCl was placed in a 125-ml Erlenmeyer flask and was shaken for 30 min on a vibratory shaker operated at approximately 400 rpm. The residue was then immediately washed into a 50-ml centrifuge tube and centrifuged at 16,000 rpm for 10 min to obtain a clear supernatant, which was then diluted to 100 ml with distilled water in a 100-ml volumetric flask.

Alumina Determination.—Modifying the procedure for determination of aluminum outlined by Vogel (42), a quantity of extract containing between 0.01 and 0.10 mg of aluminum in a 25-ml volumetric flask was treated with 5 ml of a buffer solution, made up of 77 gm NH_4AC and 57 ml concentrated HAC per liter to give a pH of 4.5. One ml of 0.2 percent freshly prepared aluminon reagent was added, and the mixture made up to 25 ml with distilled water and allowed to stand for 30 min before the color intensity was measured at 520 μ on a Beckman Model B spectrophotometer. A calibration curve was prepared in exactly the same manner by using a standard solution made up of $\text{AlK}(\text{SO}_4)_2 \cdot 12 \text{H}_2\text{O}$ to contain 0.01 mg/ml of aluminum. The aluminum content in each sample was determined from the calibration curve and was expressed in grams of alumina per 100 gm of oven-dry mixture.

Silica Determination.—Following the procedure outlined by Govett (11), an aliquot of extract containing between 0.2 mg and 0.7 mg SiO_2 was added to a 25-ml volumetric flask acidified with 5 ml of 1 N H_2SO_4 and further treated with 5 ml of 0.3 M (with respect to MoO_4^{2-}) ammonium molybdate. The sample was then made up to 25-ml volume with distilled water. The color intensity was measured at 400 μ on a Beckman Model B spectrophotometer not sooner than 2 min nor later than 10 min after the addition of ammonium molybdate. The standard silica solution was prepared by dissolving sodium metasilicate ($\text{Na}_2\text{SiO}_3 \cdot 9 \text{H}_2\text{O}$) in distilled water acidified with H_2SO_4 to give the final pH of about 1.5. Aliquots of the standard solution containing 0.1 to 1.0 mg SiO_2 per 25 ml were used to prepare the standard curve in exactly the same manner as above. The silica content was determined from the standard curve and was reported in grams per 100 gm oven-dry mixture.

RESULTS AND DISCUSSION

Strength vs Time

No significant loss of moisture was found in any of the samples cured at 5 C, 23 C, and 40 C up to 56 days; 50 C up to 28 days; and 60 C up to 14 days. Samples cured at 50 C for 56 days and at 60 C for 28 and 56 days seemed excessively dry, and the containers were completely dry. This lack of excess water is believed to have critical effects on the strengths obtained and the formation of reaction products; therefore, data on the samples cured at those temperatures for those times were ignored.

Figure 3 shows the average and extreme unconfined compressive strengths as a function of curing time for the various curing temperatures. Except for the 3-day curve at 5 C, there is an increase of strength with time at all temperatures with the greater rate of increase at the higher temperatures. This was as expected. The drop in strength for 3-day curing at 5 C appears to be due to experimental error.

Figure 4 shows the unconfined compressive strength as a function of $\log T_a$; with $T_a = 2(T + 11.7)/10$ A, where T = curing temperature in degrees centigrade and A =

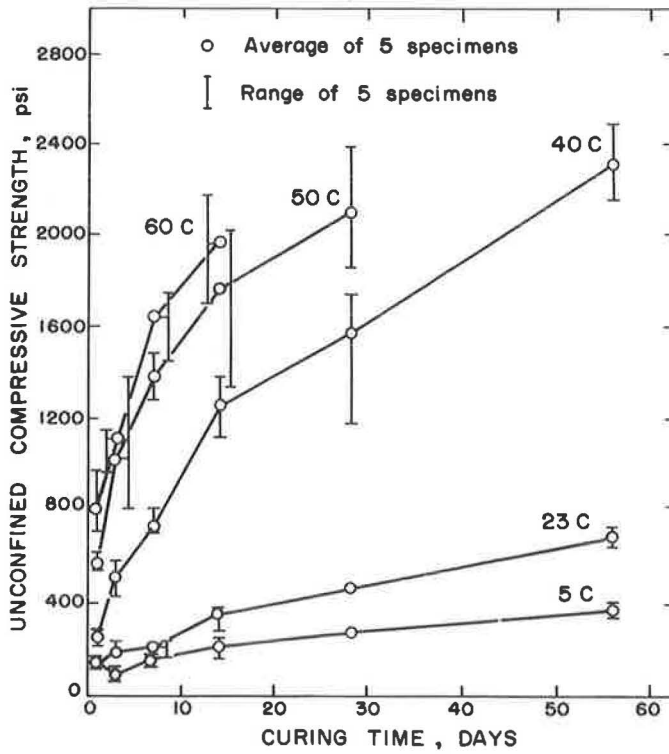


Figure 3. Relation between unconfined compressive strength and curing time for lime-clay mixtures cured at various temperatures.

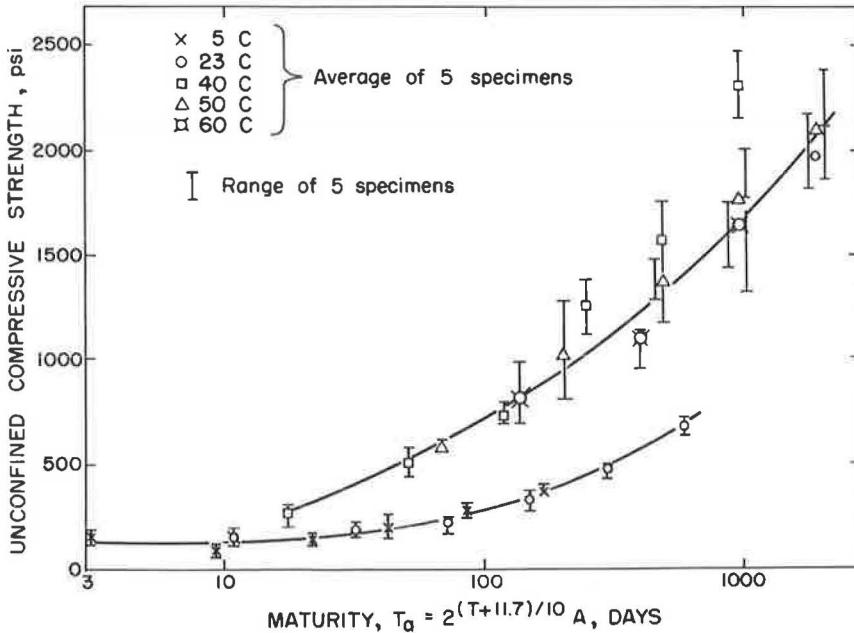


Figure 4. Relation between unconfined compressive strength and "maturity."

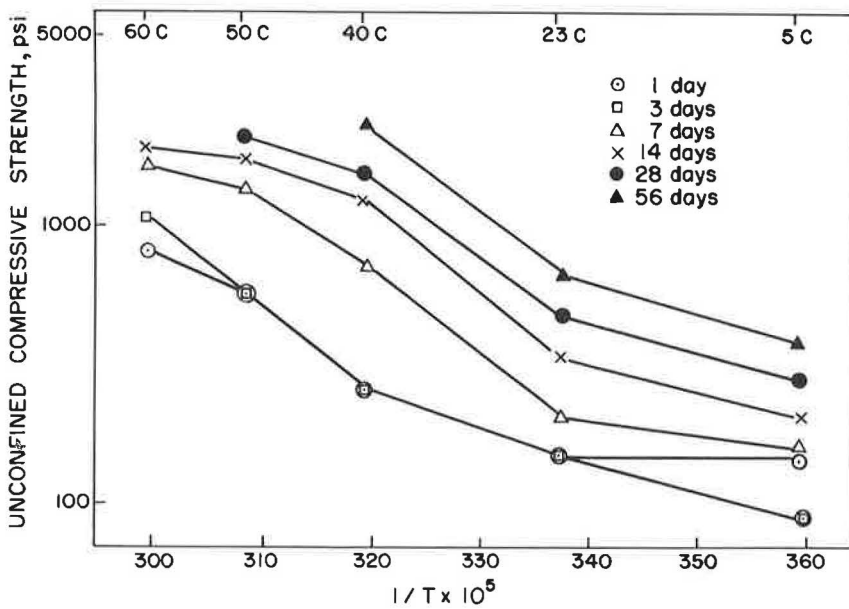


Figure 5. Relation between unconfined compressive strength and temperature after various curing times.

curing time in days. This function of time and temperature was chosen because Metcalf (27) showed that of all the "maturity" laws, this one gave the closest fit to the results of lime-clay stabilization. T_a used in this expression is merely a simplification of Rastrup's (31) maturity rule,

$$T_a = \int_0^A 2^{(T - \theta)/10} dA$$

when the temperature at which the reaction ceases is considered to be -11.7 C and the curing takes place at a constant temperature during the entire curing time. The temperature -11.7 C was chosen because Plowman (30) determined this temperature to be the datum temperature for concrete maturity calculations. Saul (35) had earlier chosen -10 C for the datum on the basis of curve fitting. Plowman made his modification on the basis of direct measurements. A plot of the unconfined compressive strengths of the present investigation was made using -10 C as the datum temperature, and it was found that there was a great deal more scattering of points than when the -11.7 C datum temperature was used. However, even with the increased scatter, the -10 C datum temperature plot had all the characteristics of Figure 4.

Figure 4 shows a grouping of points along two different lines. The lower line is formed by the unconfined compressive strengths of samples cured at 5 C and 23 C; the upper line by the unconfined compressive strengths of the samples cured at 40 C, 50 C, and 60 C. Plotting the average unconfined compressive strength as $\log(\text{UCS}) = B' - B''/T$ in the fashion of Metcalf (27) shows a similar trend (Fig. 5).

Taking equal slopes to indicate that the strength is due to a similar reaction, it appears that three distinct reactions can be identified. The first is produced for 7-, 14-, 28-, and 56-day curing in the range 5 - 23 C, 3-day curing in the range 5 - 40 C, and 1-day curing in the range 23 - 40 C. A second is produced for 1-day curing in the range 40 - 50 C, 3-day curing in the range 40 - 60 C, 7-day curing in the range 23 - 50 C, and 14-, 28-, and 56-day curing in the range 23 - 40 C. The third is produced for 1- and 7-day curing in the range 50 - 60 C, 14-day curing in the range 40 - 60 C, and 56-day curing in the range 40 - 50 C.

One explanation is that a different reaction product is formed at different curing temperatures and that the cutoff temperature lies between 23 C and 40 C. With the same materials used in this investigation and $C/S = 0.892$, Wang (44) found that the mixtures produced CSH (gel), CSH (I), hydrogarnet, and C_4AH_{13} when cured at 40 C for periods of 7 to 180 days, the products formed being dependent on the curing time. The same mixture composition cured at 23 C for periods of from 28 to 180 days produced only CSH (gel) and C_4AH_{13} . Glenn (9) also found, when investigating a similar bentonite-lime mixture with $C/S = 0.69$, that mixtures cured at room temperature for long periods produced C_4AH_{13} , CSH (I), CSH (gel), and possibly CSH (II). Mixtures of the same composition cured for short periods at higher temperatures yielded CSH (gel), CSH (I), and C_4AH_{13} at 40 C; and CSH (I), and possibly CSH (II) or aluminum substituted tobermorite at 80 C.

A sequential reaction is suggested by Taylor (37) for lime-quartz pastes. He states that although many details of the reaction are obscure and others depend on the conditions of the investigation, the general picture seems to be clear: reactions on the quartz surface initially give a lime-rich substance similar to CSH (II). When the overall C/S ratio is low, this reaction proceeds until all the lime is depleted; the CSH (II) then reacts with more quartz giving CSH (I), the C/S ratio of which eventually drops to 0.8. If the temperature and time are sufficient, the CSH (I) then recrystallizes to tobermorite. If the overall C/S ratio is below 0.8, the CSH (I) or tobermorite also reacts slowly with the unused quartz to give gyrolite.

If the lime-clay reaction is similar to the lime-quartz reaction, and on the basis of reaction products it appears that it is, then a complex reaction is taking place. It also appears that this complex reaction is a consecutive reaction of the form $A + B \xrightarrow{k} C \xrightarrow{l} D$, in which k is the rate constant for the first step and l is the rate constant for the second step (20). With $l \geq k$ no C will be formed, and if $l \ll k$ a negligible amount of D will be formed. Since k and l are temperature dependent, at some temperature where k is slightly higher than l , both C and D will be present. As time progresses, the amount of D will build up, whereas the amount of C will increase more slowly. Now if D contributes more to the strength of the mixture than C, at some time when the amount of D in the mixture is great enough, the mixture will behave as though all of its strength was coming from D. At temperatures around room temperature and below, only the first reaction would be taking place. At temperatures of 40 C and higher the second reaction will be taking place at relatively the same rate as the first, so that essentially there is very little if any of the first product formed.

In general, this consecutive reaction theory may be substantiated by data from Wang (44), who showed that CSH (gel) formed at short curing times and low temperatures, and converted to another CSH phase with prolonged curing.

Figure 3 also confirms that although the strength gain of the lime-clay mixtures cured at different temperatures appears to be due to different phases of the complex reaction, it may be possible to utilize accelerated curing methods to approximate the strengths of soil-lime mixtures cured at lower temperatures for longer times. However, care must be exercised in the use of accelerated curing tests. Each soil that is to be subjected to accelerated curing should be investigated thoroughly to assure that disproportionate strengths are not achieved by accelerated curing.

Reaction Products

The discussion of the reaction products formed in the mixtures is made on the basis of X-ray diffraction curves and data obtained from the spectrophotometric analysis of dilute HCl extract. Figure 6 shows the acid-soluble alumina for samples cured at various temperatures for various times, corrected for the 0.13 gm alumina dissolved from the natural clay. Figure 7 shows the acid-soluble silica for samples cured at the various temperatures for various times, corrected for the 0.06 gm silica dissolved from the natural clay.

One-Day Curing.—The X-ray diffraction curves for 1-day curing confirm that reaction products in the tobermorite hydrate group are formed at all temperatures, indicated by peaks at 3.07 and 1.82 Å. All curing temperatures also produce a slight

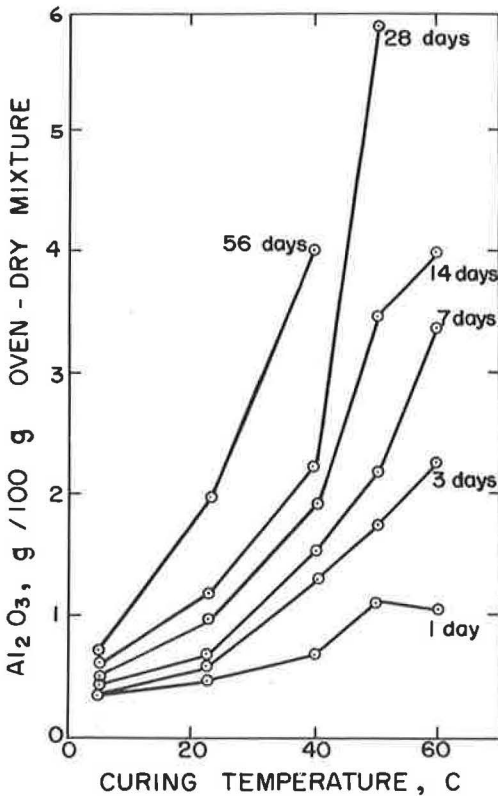


Figure 6. Acid-soluble alumina in sample after curing at various temperatures for various times.

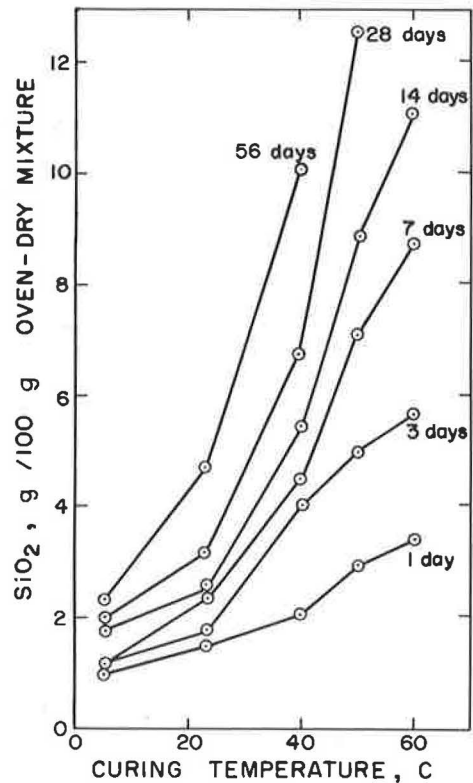


Figure 7. Acid-soluble silica in sample after curing at various temperatures for various times.

amount of C_4AH_n , indicated by peaks in the regions of 7.5, 4.1, 3.99, and 2.88 Å. However, the mixture cured at 23 C gives the most definite peak in the region of 7.5 Å. The mixture cured at 60 C also has broad weak peaks at 3.03 and 1.97 Å. The additional peak at 3.03 Å probably indicates that there are two phases of the tobermorite group present in the cured mixture, i. e., CSH (II) and CSH (gel). The 1.97 Å peak corresponds to a peak found by Diamond (7) in iron or magnesium substituted tobermorite; since no data are available on lattice substitution in CSH (II) or CSH (gel), this may indicate that some substitution is taking place in one or both of these.

At 1-day curing there is little difference in the amount of acid-soluble silica and alumina found after curing at 5 C and that found after curing at 60 C (Figs. 6, 7).

Three-Day Curing.—The X-ray diffraction curves for 3-day curing are essentially the same as those for 1-day curing, except that mixtures cured at all temperatures have weak peaks in the region of 10.5–9.8 Å, indicating the presence of CSH (II). The diagnostic 7.5 Å peak is missing from all curves, but the other C_4AH_n peaks, although weak, are still present. The 1.97 Å peak does not appear in any of the curves, but there is a broad hump in the region of 3.03 Å on all curves. The rate of increase of acid-soluble alumina and silica with respect to temperature is slightly greater in samples at this curing time than it was in these in 1-day curing (Figs. 6, 7).

Seven-Day Curing.—The X-ray diffraction curves of 7-day curing are the same as those for 3-day curing; however, the 7.5 Å peak of C_4AH_n is quite evident at 50 C and 60 C curing temperature, but not at the lower temperatures. At these same curing temperatures the 1.98 Å peak reappears indicating possible iron or magnesium substitution in the calcium silicate hydrate. The rate of increase in acid-soluble alumina with respect to temperature is approximately constant between 23 C and 50 C but

TABLE 1
SUMMARY OF CRYSTALLINE PRODUCTS OBSERVED FROM X-RAY ANALYSIS

Curing Temp. °C	Curing Time					
	1 Day	3 Days	7 Days	14 Days	28 Days	56 Days
5	A?, G	A?, G, II	A?, G, II	A?, G, II	A?, G, II	A, G, II
23	A, G,	A?, G, II	A?, G, II	A?, G, II	A?, G, II	A, G, II
40	A?, G	A?, G, II	A?, G, II	A?, G, II	A?, G, II	A, G, II
50	A?, G	A?, G, II	A, Ga, IIa	A, Ga, IIa	A, Ga, Ia, IIa	N. D.
60	A?, Ga, IIa	A, G, II	A, Ga, IIa	A, Ga, Ia, IIa	N. D.	N. D.

Notes: A— C_4AH_n ; G—CSH (gel); I—CSH (I); II—CSH (II); ?—probable; a—probable lattice substitution; and N. D.—not determined.

increases between 50 and 60 C (Fig. 6). The rate of increase in acid-soluble silica is constant from 5 C to 40 C but increases from 40 C to 50 C and then decreases slightly between 50 C and 60 C (Fig. 7). The latter may be another indication of lattice substitution.

Fourteen-Day Curing.—The X-ray diffraction curves for 14-day curing are the same as those for 7-day curing. A noticeable feature, however, is the change in shape of the curves for 50 C and 60 C curing temperatures at 3.07 Å. At shorter curing times this peak was relatively sharp, but now it has become very broad. At 60 C curing temperature, additional peaks at 2.74 and 2.34 Å occur. The 2.74 Å peak is between a 2.78 Å peak reported by Diamond (7) for CSH (I) and a 2.7 Å peak also reported by him for aluminum-substituted tobermorite. The 2.23 Å peak was not accounted for.

Twenty-Eight-Day Curing.—The diffraction curve for 50 C curing is the same as that of 60 C curing for 14 days, but the curves for other curing temperatures do not exhibit any change from the 14-day curves. There is a great increase in the rate of increase of acid-soluble silica and alumina between 40 C and 50 C (Figs. 6, 7).

Fifty-Six-Day Curing.—The X-ray diffraction curves for 56-day curing are the same as those for 28-day curing except that very distinct peaks appear on all curves in the region of 7.5 Å, indicating the definite formation of crystalline C_4AH_n .

Summary of X-Ray Data.—No evidence of formation of hydrogarnet, ordinarily shown by peaks in the region of 2.68, 3.00, and 1.61 Å, was found in any of the X-ray diffraction curves.

A summary of the crystalline products diagnosed from X-ray diffraction is shown in Table 1. C_4AH_n is reported as present only where there are peaks in the region of 7.5 Å; at other times it is reported as probable, on the basis of other peaks.

At 5 C, 23 C, and 40 C curing temperatures C_4AH_n , CSH (gel) and CSH (II) appear to be the stable phases. The same products appear at 50 C and 60 C curing temperatures, with probable lattice substitutions and CSH (I) appearing after several days of curing. X-ray diffraction curves gave supporting evidence (Table 1) to the theory advanced earlier, that the lime-clay reaction is a consecutive reaction



as evidenced by the trend indicated in the 50 C and 60 C curing. At any given time there may be more than one phase of the reaction present, depending on the curing time and temperature.

C/S Ratio of Reaction Products.—Table 2 gives the apparent C/S ratio of the reaction products at the different curing times and temperature, based on the results of the X-ray diffraction determination of calcium hydroxide in the cured mixture, and the spectrophotometric determination of acid-soluble silica and alumina. These C/S ratios were computed after making these assumptions:

TABLE 2

C/S RATIO OF DISSOLVED REACTION PRODUCTS AFTER ASSUMING ALL ACID-SOLUBLE ALUMINA COMES FROM C_4AH_n PRODUCTS AND ALL CALCIUM HYDROXIDE THAT DISAPPEARS IS UTILIZED IN CALCIUM SILICATE AND CALCIUM ALUMINATE PRODUCTS

Curing Temp. °C	Curing Time					
	1 Day	3 Days	7 Days	14 Days	28 Days	56 Days
5	6.0	6.2	6.6	4.7	5.8	3.8
23	3.4	3.8	2.6	3.6	3.2	2.7
40	2.6	1.9	1.7	1.7	1.8	1.5
50	2.7	1.7	1.7	1.6	0.9	—
60	2.9	1.8	1.2	1.3	—	—

TABLE 3

S/A RATIO OF ACID-SOLUBLE REACTION PRODUCTS AT VARIOUS CURING TIMES AND TEMPERATURES

Curing Temp. °C	Curing Time						
	1 Day	3 Days	7 Days	14 Days	28 Days	56 Days	Avg.
5	5.0	4.0	4.5	5.4	5.3	5.9	5.02
23	5.0	4.7	5.6	4.4	4.6	4.1	4.73
40	4.9	5.1	4.9	4.7	5.0	4.3	4.82
50	4.4	4.8	5.5	4.3	3.6	—	4.52
60	5.5	4.2	4.4	4.7	—	—	4.70
Avg.	4.96	4.56	4.98	4.70	4.62	4.77	4.76

1. All the calcium hydroxide that disappears during curing is assumed to enter into the pozzolanic reaction.

2. All acid-soluble alumina is assumed to come from C_4AH_n reaction products.

The first assumption may be incorrect if as Hilt and Davidson (14) have shown, some of the lime is utilized in the modification of clay and does not enter into the pozzolanic reaction. Ho and Handy (15, 16) have further shown that this lime does not show up on DTA curves, suggesting that the lime is adsorbed onto the clay structure. Since the exact amount of lime utilized in this manner could not be determined, the assumption was made to make a rough comparison rather than an exact determination of the reaction products' C/S ratio.

The second assumption may also be in error because of the indication of lattice substitution in the reaction products. However, since the amount and type of substitutions that do occur cannot be accurately determined, the assumption was considered valid enough for comparison purposes. The assumption of formation of only C_4AH_n products seems valid on the basis of X-ray analysis and the fact that in this temperature range the tetracalcium aluminate hydrates are most likely to be formed (41).

Except after one day of curing, the C/S ratios of the reaction products of the mixtures cured at 40 C, 50 C, and 60 C are all in the range 0.9 to 1.9 (Table 2). Ratios of the mixtures cured one day, or those cured longer at 5 C and 23 C are much higher, 2.6 to 6.6. The reported C/S ratio of the tobermorite group of calcium silicate hydrates is in the range of 0.8 to 2.0 (38).

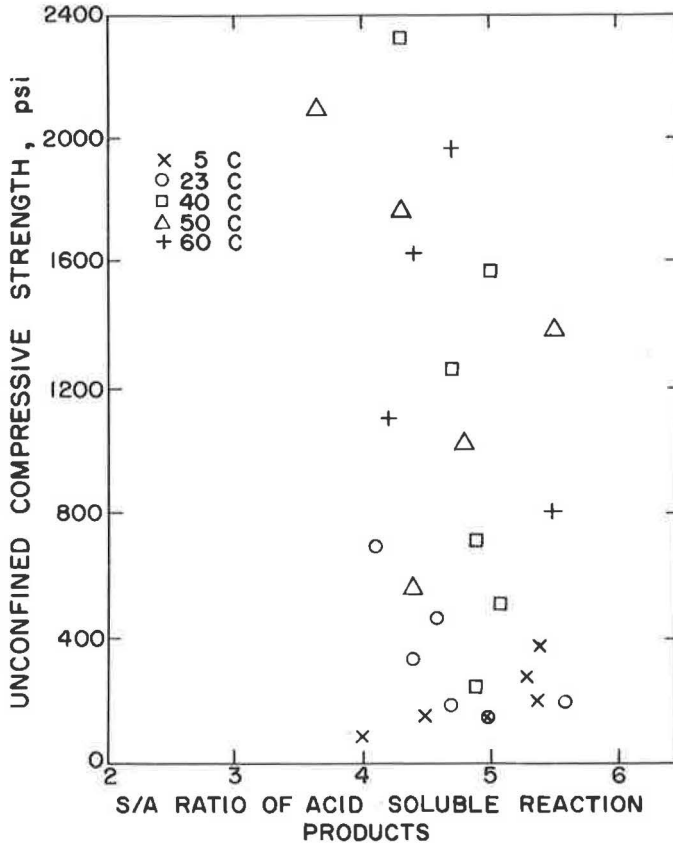


Figure 8. Relation of unconfined compressive strength to the S/A ratio of acid-soluble reaction products.

The validity of the first assumption is indeed questionable at low curing temperatures and short curing times, as the apparent C/S ratio is much higher than is believed possible for the tobermorite group (Table 2). It is so much higher that it seems reasonable to take this as additional evidence that an appreciable amount of calcium is adsorbed onto the clay structure as suggested by Ho and Handy (15, 16). At higher temperatures and longer curing times this effect is reduced because of the dissolution of the clay and the accompanying release of the adsorbed calcium. Otherwise the apparent C/S ratios of the reaction products closely correspond to those expected from the X-ray analysis, i. e., of the order of 0.8 to 2.0.

S/A Ratio of Reaction Products.—Table 3 gives the ratio of acid-soluble silica to acid-soluble alumina at the different curing times and temperatures. Plots of these data against time, temperature, and unconfined compressive strength showed no consistent relationship except that all values of S/A are between 3.6 and 5.9 (Fig. 8). The average is 4.76, or about identical with the S/A ratio of the clay, which is 4.75. The close agreement of the S/A ratio of the dissolved reaction products to the S/A ratio of the clay also indicates that the chemical determination of the acid-soluble silica and acid-soluble alumina is valid.

Relation of Strength to Reaction Products

Jambor (18) has shown that the compressive strength of a mixture is affected by the volume, kind, and microstructure of the cementitious hydration products developed.

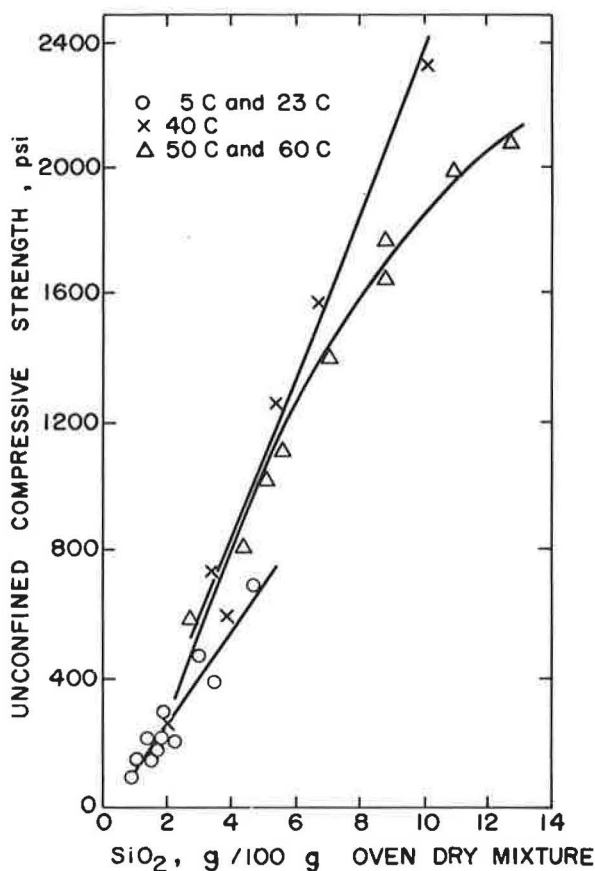


Figure 9. Relation between unconfined compressive strength and calcium hydroxide in cured samples; before curing, the samples all contained 41.6 gm $\text{Ca}(\text{OH})_2/100$ gm oven-dry mixture, giving a C/S ratio of about 1.0.

With this in mind let us examine the relationships, if any, of the various factors that have been determined in the investigation to the unconfined compressive strength of the cured mixtures. As can be seen from Figure 8, the S/A ratio of reaction products has little or no effect on the strength of the cured mixture.

Figure 9 shows the unconfined compressive strength of the mixture as a function of the calcium hydroxide present in the cured samples. It is evident that the strength of the mixture is not a function of the calcium hydroxide alone, but the temperature of curing has an effect on the strength of the mixture developed by the reaction of a given amount of calcium hydroxide. In the samples cured at 5 C and 23 C, the difference in temperature has little effect on the strength developed by the amount of disappearing calcium hydroxide. The same feature is noted in the samples cured at 40 C, 50 C and 60 C. However, there is a noticeable difference in the strength developed by the disappearance of a given amount of calcium hydroxide between samples cured at 23 C and 40 C.

Figure 10 shows the unconfined compressive strength as a function of acid-soluble silica. It appears again that temperature plays a role in determining the strength developed by a given amount of silica. The samples cured at 5 C and 23 C follow one curve, the samples cured at 50 C and 60 C follow another, and the samples cured at 40 C follow still another.

Figure 11 shows the unconfined compressive strength as a function of acid-soluble alumina, and displays the same characteristics as Figure 10. It can only be concluded

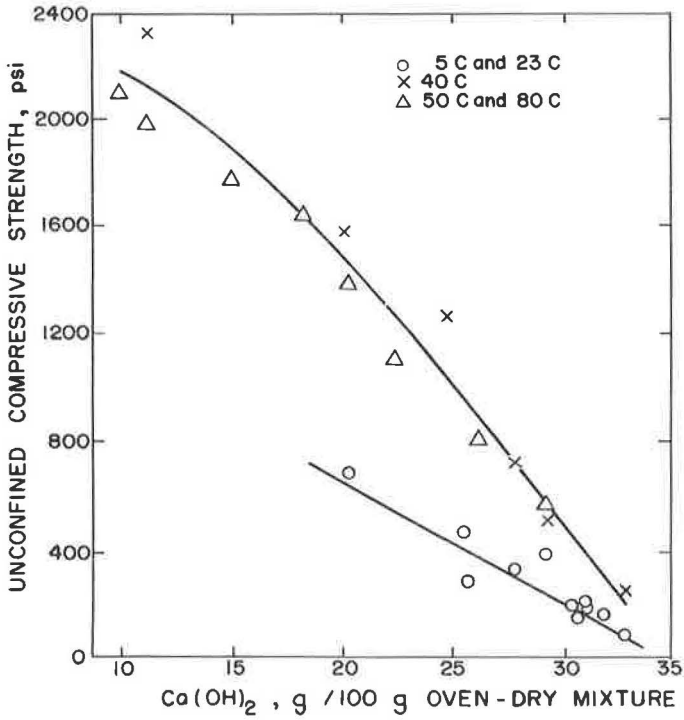


Figure 10. Relation between unconfined compressive strength and acid-soluble silica in cured samples.

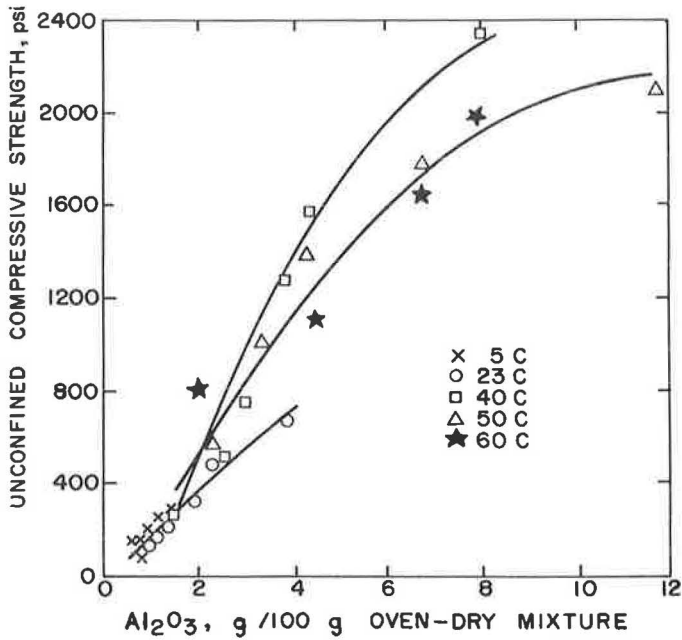


Figure 11. Relation between unconfined compressive strength and acid-soluble alumina in cured samples.

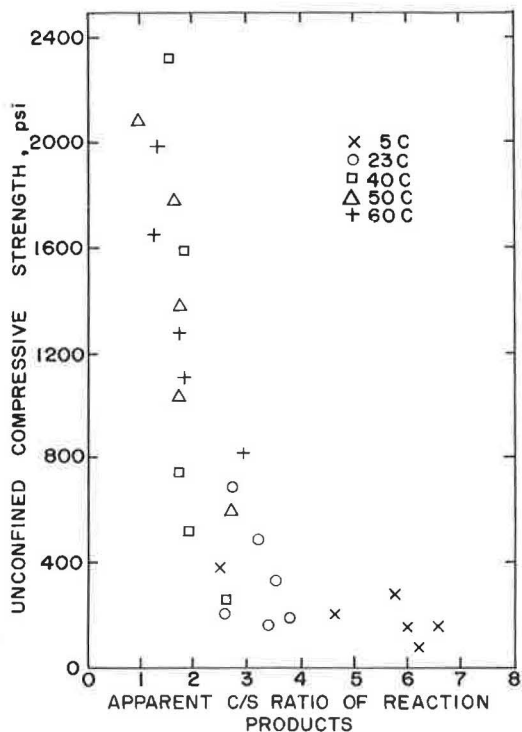


Figure 12. Relation between unconfined compressive strength and apparent C/S ratio of reaction products in cured samples.

that at a given curing temperature the cured sample that has the largest amount of acid-soluble silica or alumina, or the smallest amount of unreacted calcium hydroxide, will have the highest unconfined compressive strength. There is a different phase of the reaction responsible for strength development at 50 C and 60 C curing temperatures than at the 5 C and 23 C curing temperatures, but the more efficient strength development in terms of acid-soluble silica and alumina with the intermediate 40 C curing temperature is not explained.

The average unconfined compressive strength as a function of apparent C/S ratio is shown in Figure 12. Although there is considerable scatter it is evident that for the production of high strengths it is necessary to have an apparent C/S ratio of less than two. These low C/S ratios are produced at curing temperatures of 40 C and higher (Table 2). This probably accounts for the two distinct groupings of points in Figure 1.

It also appears that the farther the consecutive reaction proceeds the lower the C/S ratio becomes and the higher is the unconfined compressive strength (Tables 1 and 2, Fig. 3).

To summarize, the compressive strength of the mixture is affected by the crystalline structure as well as the amount of reaction products formed in the lime-clay reaction. This was evident from the more efficient strength development in terms of acid-soluble silica and alumina, and disappearing calcium hydroxide at the higher curing temperatures. This increased strength development was accompanied by a change in the number of crystalline reaction products detected by X-ray analysis. It appears that the S/A ratio of the reaction products has no apparent effect on strength-developing potential of the reaction products, but the C/S ratio does.

CONCLUSIONS

1. The strength gain of lime-clay mixtures cured at different temperatures is due to different phases of a complex reaction.
2. This complex reaction is lime + clay \rightarrow CSH (gel) \rightarrow CSH (II) \rightarrow CSH (I) \rightarrow tobermorite. The farther the reaction proceeds, the higher the strength.
3. At curing temperatures of 50 C and higher, lattice substitutions take place in the structure of the calcium silicate hydrates.
4. There is no consistent relationship between time, temperature, strength, and the S/A ratio of the reaction products.
5. The curing temperature has an effect on the strength developed by a given amount of silica in a cured lime-clay mixture.
6. At a given curing temperature, the cured sample that has the largest amount of acid-soluble silica or alumina will give the highest strength.
7. To achieve high strengths the apparent C/S ratio must be less than two.
8. Abnormally high apparent C/S ratios at low curing temperatures suggest excess calcium adsorption onto the clay structure rather than immediate entry into a pozzolanic reaction.

9. The total S/A ratio of the lime-clay reaction products equals that of the clay mineral.

10. Alumina in the reaction products occurs mainly in C_4AH_n at all curing temperatures; at curing temperatures of 50 C or higher, some alumina may substitute isomorphously in the calcium silicate hydrate structures.

11. It is possible to determine the amount of silica and alumina in lime-clay reaction products by spectrophotometric analysis with sufficient accuracy for comparison purposes.

ACKNOWLEDGMENTS

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REFERENCES

1. Definitions of Terms Relating to Hydraulic Cement. ASTM Book of Standards, Part 9, p. 224, Philadelphia, 1964.
2. Bergström, S. G. Curing Temperature, Age, and Strength of Concrete. *Mag. of Conc. Res.*, Vol. 5, pp. 61-70, 1953.
3. Bogue, R. H. *The Chemistry of Portland Cement*. 2nd ed. Reinhold, New York, 1955.
4. Brunauer, S. Tobermorite Gel: The Heart of Concrete. *Am. Scientist*, Vol. 50, pp. 210-229, 1962.
5. Czernin, W. *Cement Chemistry and Physics for Civil Engineers*. Chemical Publishing Co., New York, 1962.
6. Davidson, D. T., and Handy, R. L. Soil Stabilization. In Woods, K. B., ed. *Highway Engineering Handbook*. McGraw-Hill, New York, 1960.
7. Diamond, S. Tobermorite and Tobermorite-Like Calcium Silicate Hydrates: Their Properties and Relationships to Clay Minerals. Final Report on Project No. C-36-471. Joint Highway Research Project, Purdue University, Lafayette, Indiana, 1963.
8. Dumbleton, M. J., and Ross, N. F. The Effect of Temperature on the Gain in Strength of Soil Stabilized with Hydrate Lime and with Portland Cement. (Unpubl.). Great Britain Department of Scientific and Industrial Research, Road Research Laboratory, Research Note RN/3655/MJD. NFR, 1960.
9. Glenn, G. R. X-Ray Studies of Lime-Bentonite Reaction Products. Unpubl. PhD thesis, Library, Iowa State University of Science and Technology, Ames, 1963.
10. Glenn, G. R., and Handy, R. L. Lime Clay Mineral Reaction Products. *Highway Research Record* 29, pp. 70-83, 1963.
11. Govett, G. J. S. Critical Factors in the Colorimetric Determination of Silica. *Anal. Chim. Acta*, Vol. 25, pp. 69-80, 1961.
12. Handy, R. L. Fast-Scan X-Ray Diffraction Determination of Calcite: Dolomite Ratio. (To be published in ASTM Proc.)
13. Hilt, G. H., and Davidson, D. T. Isolation and Investigation of Lime-Montmorillonite-Crystalline Reaction Product. *HRB Bull.* 304, pp. 51-65, 1961.
14. Hilt, G. H., and Davidson, D. T. Lime Fixation in Clayey Soils. *HRB Bull.* 262, pp. 20-32, 1960.
15. Ho, C., and Handy, R. L. Characteristics of Lime Retention. *Highway Research Record* 29, pp. 55-69, 1963.
16. Ho, C., and Handy, R. L. The Effect of Lime on Electrokinetic Properties of Bentonite. *Nat. Conf. on Clays and Clay Min. Proc.*, Vol. 19, pp. 267-280, 1964.

17. Iler, R. K. *The Colloid Chemistry of Silica and Silicates*. Cornell Univ. Press, Ithaca, N. Y., 1955.
18. Jambor, J. *Relation Between Phase Composition, Overall Porosity and Strength of Hardened Lime-Pozzolano Pastes*. *Mag. of Conc. Res.*, Vol. 15, pp. 131-142, 1963.
19. Kalousek, G. L. *Studies on the Cementitious Phases of Autoclaved Concrete Products Made of Different Raw Materials*. *Amer. Conc. Inst. Jour.*, Vol. 25, pp. 365-378, 1964.
20. Kittsley, S. L. *Physical Chemistry*. Barnes and Noble, New York, 1955.
21. Klug, H. P., and Alexander, L. E. *X-Ray Diffraction Procedures for Polycrystalline and Amorphous Materials*. John Wiley and Sons, New York, 1954.
22. Lea, F. M. *The Chemistry of Cement and Concrete*. Revised edition of Lea and Desch. St. Martin's Press, New York, 1956.
23. Lea, F. M. *The Chemistry of Pozzolanas*. *Symp. on Chem. of Cement, Stockholm, 1938, Proc.*, Vol. 1, pp. 460-490, 1939.
24. Locher, F. W. *Hydraulic Properties and Hydration of Glasses of the System CaO-Al₂O₃-SiO₂*. *Fourth Internat. Symp. on Chem. of Cement, Washington, 1960. Proc.*, Vol. 1, pp. 267-276, 1962.
25. Magaw, H. D., and Kelsey, C. H. *Crystal Structure of Tobermorite*. *Nature*, Vol. 177, pp. 390-391, 1956.
26. McCaleb, S. B. *Hydrothermal Products Formed from Montmorillonite Clay Systems*. *Nat. Conf. on Clays and Clay Min. Proc.*, Vol. 9, pp. 276-294, 1962.
27. Metcalf, J. B. *The Effect of High Curing Temperatures on the Unconfined Compressive Strength of a Heavy Clay Stabilized with Lime and with Cement*. *Australia-New Zealand Conf. on Soil Mech. and Foundation Eng. Proc.*, Vol. 4, pp. 126-130, 1964.
28. Midgley, H. G., and Chopra, S. K. *Hydrothermal Reactions Between Lime and Aggregate Fines*. *Mag. of Conc. Res.*, Vol. 12, pp. 73-81, 1960.
29. Nurse, R. W. *Steam Curing of Concrete*. *Mag. of Conc. Res.*, Vol. 1, pp. 79-88, 1949.
30. Plowman, J. M. *Maturity and Strength of Concrete*. *Mag. of Conc. Res.*, Vol. 8, pp. 13-22, 1956.
31. Rastrup, E. *Heat of Hydration in Concrete*. *Mag. of Conc. Res.*, Vol. 6, pp. 79-92, 1954.
32. Roderick, G. L. *Use of Polystyrene for Soil Stabilization*. Unpubl. MS thesis, Library, Iowa State University of Science and Technology, Ames, 1963.
33. Rosauer, E. A., and Handy, R. L. *Crystallite-Size Determination of MgO by X-Ray Diffraction Line Broadening*. *Iowa Academy of Science Proc.*, Vol. 68, pp. 357-371, 1961.
34. Sanders, L. D., and Smothers, W. J. *Effect of Tobermorite on the Mechanical Strength of Autoclaved Portland Cement-Silica Mixtures*. *Amer. Conc. Inst. Jour.*, Vol. 29, pp. 127-139, 1957.
35. Saul, A. G. A. *Principles Underlying the Steam Curing of Concrete at Atmospheric Pressure*. *Mag. of Conc. Res.*, Vol. 2, pp. 127-140, 1951.
36. Taylor, H. F. W. *The Calcium Silicate Hydrates*. In Taylor, H. F. W., ed. *The Chemistry of Cements*. Vol. 1, pp. 167-232. Academic Press, New York, 1964.
37. Taylor, H. F. W. *The Chemistry of Cement Hydration*. In Burke, J. E., ed. *Progress in Ceramic Science*, Vol. 1, pp. 89-145. Pergamon Press, New York, 1961.
38. Taylor, H. F. W. *Hydrothermal Reactions in the System CaO-SiO₂-H₂O and the Steam Curing of Cement and Cement-Silica Products*. *Fourth Internat. Symp. on Chem. of Cement, Washington, 1960. Proc.*, Vol. 1, pp. 167-190, 1962.
39. Taylor, H. F. W., and Howison, J. W. *Relationships Between Calcium Silicates and Clay Minerals*. *Clay Min. Bull.*, Vol. 3, pp. 98-111, 1956.
40. Taylor, H. F. W., and Moorehead, D. R. *Lightweight Calcium Silicate Hydrate: Some Mix and Strength Characteristics*. *Mag. of Conc. Res.*, Vol. 13, pp. 145-149, 1960.

41. Turriziani, R. The Calcium Aluminate Hydrates and Related Compounds. In Taylor, H. F. W., ed. The Chemistry of Cements, Vol. 1, pp. 233-286. Academic Press, New York, 1964.
42. Unwin, W. C. On the Rate of Hardening of Cement and Cement Mortars. Inst. of Civil Eng. Proc., Vol. 84, pp. 399-411, 1886.
43. Vogel, A. I. Quantitative Inorganic Analysis, Theory and Practice. 2nd ed., Longmans, Green and Co., Ltd, London, 1951.
44. Wang, J. W. H. Role of Magnesium Oxide in Soil-Lime Stabilization. HRB Spec. Rept. 90, p. 475, 1966.

A Quick Test to Determine Lime Requirements For Lime Stabilization

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The use of hydrated lime, $\text{Ca}(\text{OH})_2$, for modifying, upgrading, and stabilizing soils is increasing greatly. This means highway laboratories have had their work loads increased, and in many instances, more than doubled for a particular job. Before the advent of the use of lime, the laboratory was finished with testing when a soil was classified as unsuitable. Now the same soil is tested and retested to find the percentage of lime required to bring the soil within specifications. In most cases the percentage is determined by compressive strength tests, Atterberg limits tests, or both.

The reaction of lime and soil can be described as a series of chemical reactions. The results of these reactions are expressed as a change in the plasticity, swell, shrinkage or compressive strength of the soil. Therefore, a quick or simple test is needed to show the amount of lime required to react chemically with a soil to bring about these physical changes to an optimum degree.

Laboratory tests, involving mineralogical, physical and chemical characteristics of untreated and lime-treated soils have proven that pH tests can be used to determine the optimum lime requirements of a soil.

•THE USE of hydrated lime, $\text{Ca}(\text{OH})_2$, for modifying, upgrading, and stabilizing soils is increasing greatly. Lime stabilization, having started as an aid in maintenance work, now covers all highway construction. This includes Interstate highways, farm-to-market roads, shoulders and parking lots as well as non-highway uses such as airport runways, building foundations, and railroad subgrades.

The occasional job calling for lime stabilization does not impose a hardship on testing laboratories. If it is a matter of upgrading a soil by decreasing the percentage of fines or reducing the plasticity index of a soil, then samples are cured with varying percentages of lime and sieve analysis or Atterberg limits tests are repeated. However, finding the optimum amount of lime for new construction jobs where the soil classification changes frequently, or for a number of jobs at the same time, greatly increases the work load. A simple and quick test to determine the lime requirements for a soil is a major goal of the soils testing engineer.

The reaction of lime and soil is a chemical reaction or rather a series of chemical reactions. The reactions are basically the reaction of calcium and the organic and inorganic compounds making up the soil. Maclean and Sherwood (7) have reported on the calcium absorption of the hydroxy-quinones, which are found in soils. Besides the absorption of calcium on the exchange sites of the clays, soils may also contain such components as soluble silica which has not been leached from the soil, alumina which is left behind in the weathering profile, sulfates from the decomposition of pyrites, and phosphates if cultivated fields are being drained toward the highway. Therefore, to

determine the amount of lime needed to stabilize a soil, it is necessary to know how much lime will be needed to satisfy all of the reactions. The aforementioned reactions, except for the absorption of calcium on the exchange sites, are instantaneous upon contact with calcium. Results of research at the University of Illinois with radioactive cesium and pure clay minerals proved, for the most part, that ion exchange is complete in one hour. Therefore, it is only necessary to know the amount of lime consumed by any soil in one hour at room temperature. However, since lime is so very alkaline (pH 12.4 at 25 C), the silicates, especially the clays (3), will be attacked, freeing silica and alumina with which the calcium reacts to form calcium silicates—as long as a high alkaline condition is maintained. The lime requirements for this continued reaction could be determined under varying conditions of time and temperature.

One of the objectives of lime stabilization research at the University of Illinois has been to devise a quick and simple test for determining the lime requirements for soil stabilization. Since most of the soils laboratories in the United States have been concerned with proving the merits of lime stabilization, little time has been devoted to test methods. In 1960, Hilt and Davidson (5) reported on research using plastic limits to determine what they termed "lime fixation." This test is satisfactory for showing the lime requirements; however, it is time consuming and an experienced man is needed to roll the silty soils after lime is added. A chemical analysis, such as ASTM C-25-58 (rapid sugar test), on a reacted soil after any given period could be used, but it is also time consuming and only results to the nearest percent are needed.

As stated earlier, lime is a strong base with a very low coefficient of solubility in water, giving a pH of 12.4. Therefore, it was felt that a pH test on soil-lime mixtures, after an hour or any given period of time, could be used to determine the optimum amount of lime for the reactions. Such a test could be performed with unskilled workers.

DESCRIPTION OF MATERIALS

Soils

Approximately one hundred soils have been tested under the conditions of this investigation, and since all of the soils followed the same pattern only the results of four soils are reported. The choice of these soils was based on the type of clay minerals and the Bureau of Public Roads soil classification. The four soils are as follows:

1. A glacial till from Champaign County, Ill., which classifies as an A-4(6). The clay minerals are predominantly illite with a smaller amount of chlorite.
2. Glacial till from Ottawa, Ill., which classifies as an A-6(9). The clay minerals are also illite and chlorite, but chlorite is predominant with mixed layering.
3. Porter's Creek clay, from Kemper County, Miss., which classifies as an A-7-6(20). The clay minerals are illite and montmorillonite.
4. Pierre shale from South Dakota, which also classifies as an A-7-6(20), and the clay mineral is montmorillonite.

Lime

A commercially produced Ca(OH)_2 , hydrated lime, meeting ASTM specification C 207-49, with 98.4 to 99.0 percent passing a No. 325 sieve, was used throughout the testing program.

ANALYTICAL PROCEDURE

Preparation of Materials

The soils were air dried to the point where they could be disaggregated with as little manipulation as possible to pass a No. 10 sieve. Since these soils were predominantly clays, a portion of the minus No. 10 was selected, using a sample splitter, and allowed to completely air dry before further disaggregation to pass the No. 40 sieve for liquid and plastic limit tests. The bulk samples were stored in closed containers so that a uniform moisture content could be maintained.

TABLE 1
PROPERTIES OF SOIL

Property	Glacial Till A-4(6)	Glacial Till A-6(9)	Porter Creek Clay A-7-6(20)	Pierre Shale A-7-6(20)
Textural composition:				
Passing No. 10 sieve	100.0	100.0	100.0	100.0
Passing No. 40 sieve	82.7	93.5	91.7	95.7
Silt (74-5 μ)	33.3	48.1	27.9	7.6
Clay (5 μ)	39.6	31.0	55.0	84.5
Clay (2 μ)	22.8	25.2	37.4	67.0
Physical properties:				
Liquid limit	20.0	28.6	78.8	161.2
Plastic limit	14.5	16.5	31.5	34.5
Plasticity index	5.5	12.1	47.3	126.7
Predominant clay mineral	Illite- chlorite	Illite- chlorite	Illite- montmorillonite	Montmorill- onite

Treatment of Soil

Samples of the minus No. 40 fraction equaling 20 gm of oven-dry soil were weighed to the nearest 0.1 gm and placed in two series of 150-ml plastic bottles with screw-top lids. Varying percentages of lime weighed to the nearest 0.01 gm were added to the two series of bottles. To the lime and soil, 100 ml of CO₂-free distilled water was added, and the three components were mixed by shaking the bottle until all ingredients were thoroughly mixed. One series of bottles was treated at room temperature, the other at 140 F. Curing times varied from one hour to 3 yr for both temperatures. The bottles were shaken for 30 sec every 10 min for the first hour.

Just prior to a pH determination, about 30 ml of the slurry was transferred to a 50-ml plastic beaker for measurement. Immediately after taking the reading, the slurry was returned to the plastic bottle; pH measurements were made at the end of 30 min, 1, 2, 4, 6, 12, 24, 48, 72, and 96 hr. The pH of the slurries of A-4(6) soil was determined at the end of 15 and 30 days. The A-6(9) soil was checked up to 15 days. The Pierre shale was checked every day for 30 days and then at the end of each month for 6 months, and finally once a year for 3 years.

Optimum moisture and standard Proctor density measurements were made for each untreated soil, and for each of the lime contents which had been used for the pH determinations. For the A-4(6) soil the lime contents were 2, 4, 6, and 8 percent; for the A-6(9) soil, 1, 3, 5, 7, and 9 percent; for the Porter Creek clay, 1, 3, 5, 7, and 9 percent; for the Pierre shale, 5, 10, 13, and 15 percent.

At each lime content and curing condition, enough soil for four 2- by 4-in. specimens was blended in a dry state to obtain a uniform mixture. Distilled water was then added to the dry mixture to bring it to its optimum moisture content. The material was then compacted in 2- by 4-in. molds to standard Proctor density. The specimens extruded from the molds were wrapped in damp paper towels and sealed in polyethylene bags to prevent moisture loss and carbonation, and then were stored either at room temperature or in a humidity cabinet at 140 F. A temperature of 140 F was used because Eades, Grim and Nichols (4) showed the correlation between 72 hr at 140 F laboratory curing and field strength after one year.

In the case of the Pierre shale the same procedure as above was followed, except CBR specimens were prepared. The oven-cured specimens were sealed with sheets of rubber and caps made from plate steel. CBR tests were used so that the relationship of swell to the optimum lime content, as determined by pH, could be shown along with strength increases.

EXPERIMENTAL METHODS

Unconfined Compression Test

The soil test specimens were tested unconfined, using a model AP-170 Stability Testing Machine. Loads were indicated on a 2,000-lb or 10,000-lb proving ring, depending on the strength of the sample, which has a dial reading to 0.0001-in. deflection. Strain was applied to the test specimens at a constant rate of 0.05 in./min.

California Bearing Ratio Test

CBR was used to test the Pierre shale. The same equipment that was used for unconfined compression test was used for the CBR test. A 10,000-lb proving ring was used with a 3-sq in. piston. All CBR specimens, regardless of cure, were soaked 4 days before testing. The swell was measured just prior to the test.

Liquid and Plastic Limits

ASTM Methods D-423-61T and D-424-59 were followed, except that the lime-treated samples were taken from the unconfined compression test specimens. The soil from the treated test specimens was broken down into pieces passing a No. 4 sieve and allowed to air dry. Then ASTM Method D-421-58, "Dry Preparation of Soil Samples for Grain-Size Analysis and Determination of Soil Constants," was followed for preparing the treated sample.

The plastic limit of each of the lime-treated samples was determined as the average moisture content of the three threads rolled.

Determination of pH Value

Jackson (6) states that the most valid pH measurement of soil is made in the field moist condition, and as soil suspensions are diluted, the pH rises. In spite of his recommendations, a ratio of soil to water of 1 to 5 was used for several reasons. The pH of the lime solution completely changes the soil moisture and absorbed ion relationship, and the problem becomes one of determining the amount of $\text{Ca}(\text{OH})_2$ needed to maintain a saturated system for a given period. If the ratio is 1 to 2, the soil sticks to the electrodes, which must then be cleaned between each test. Furthermore, when the samples are stored at elevated temperature and some evaporation takes place, the bottles do not have to be watched as closely to make sure the reaction is continuing.

After the unconfined compression tests, pH determinations were made on the treated slurries and on slurries composed of the treated test specimens. The soil from the test specimens was mixed with CO_2 -free distilled water in the same proportion as the soil, lime and water to form the treated slurries. A Coleman Companion pH Meter, model 31, with a Hyalk glass electrode was used with a pH range of 14.

Tests of Solutions With $\text{Ca}(\text{OH})_2$

Since the pH of $\text{Ca}(\text{OH})_2$ at 25 C is 12.40, the electrode and meter should be standardized with a 12.00 pH buffer. When the electrode is unused for a few days, it should be soaked in the buffer before the meter is standardized. The electrode should be washed with CO_2 -free distilled water between each reading to keep the electrode from becoming coated in CaCO_3 due to the carbonation of the lime.

Occasionally it is necessary to soak the electrode in 0.1 N HCl for a few seconds to remove the CaCO_3 . The electrode should be thoroughly rinsed with distilled water to remove the acid. Sometimes, after this treatment, the needle will drift, in which case dip the electrode in a solution of Beckman's "Descote." Descote is an organo-silicon product which removes sorbed water from silicate surfaces, leaving a water-repellent coating.

TABLE 2
ONE-HOUR pH RESULTS VS PLASTICITY INDEX

Percent Lime	Glacial Till A-4(6)		Glacial Till A-6(9)		Porter Creek Clay		Pierre Shale	
	pH	PI	pH	PI	pH	PI	pH	PI
1	12.4	N. P. ^a	11.85	8.3	10.20	49.6	—	—
2	12.4	N. P.	—	—	—	—	—	—
3	—	—	12.3	N. P.	11.85	28.5	—	—
5	—	—	12.4	N. P.	12.20	19.2	11.90	56.7
7	—	—	12.4	N. P.	12.30	N. P.	12.20	28.2
9	—	—	—	—	—	—	12.3	N. P.

^aNonplastic

RESULTS

One-Hour pH Results as Related to Plasticity Index

The plasticity index of all soils tested was decreased with each increase in the percentage of lime until the material became nonplastic (5). For the Champaign glacial till, the pH was 12.40 at the one-hour reading and the soil was nonplastic (Table 2). Since the plasticity index (Table 1) was only 5.5 in the untreated state, this is not surprising.

Ottawa glacial till had a PI of 12.1 in the untreated state and a pH of 12.30 with 3 percent lime, and tested nonplastic (Table 2). Again, this is a marginal soil, and one might expect that 3 percent would reduce the PI. However, the pH test for 1 percent lime was 11.85 and the soil still had a PI of 8.3.

The Porter Creek clay sample was quite plastic in the natural state with a PI of 47.3 (Table 2). At the end of the one-hour pH test it was learned that 7 percent lime was required to hold the pH at 12.3; however, 7 percent held the pH at 12.30 for 48 hr, showing there was still quite a bit of lime in the system. Probably 6 percent lime would be enough to stabilize the soil. The plastic limit test showed that samples treated with 7 percent lime were nonplastic. For this particular soil the pH would not go above 12.30, even with 10 percent lime. This phenomenon occurs with soils which are holding univalent ions such as Na^+ in the exchange position, and as the ions are replaced by the calcium, the electrode becomes sensitive to these ions along with the H^+ ion.

The Pierre shale, which contains only the montmorillonite mineral in the clay fraction, has a PI of 126.7. Seven percent lime held the pH at 12.20 for almost an hour, but it only reduced the PI to 28.2 (Table 2). Nine percent lime held the pH at 12.30 for 24 hr, and the soil tested nonplastic. Here again the pH could not be pushed above 12.30 with additional lime.

Each of the 4 soils became nonplastic when they were combined with the lowest percentage of lime, which resulted in free lime in the system after one hour. The lime percentage was based on the lowest percent of lime required to hold the pH of the lime soil slurry at 12.30 to 12.40 for one hour at room temperature. This is to say that when the Calcium (Ca^{++}) absorbing chemical components of the soil (including the exchange capacity of the soil) were satisfied, the physical characteristics of the soil were altered.

Compressive Strength Test as Related to pH

The compressive strength results for the glacial till A-4(6) are given in Table 3. As the curing period increased, so did the compressive strength. However, the optimum strength for a particular lime content did not occur just as the pH dropped below 12.40 or 12.30, but at about a pH of 11.0. Specimens treated with 8 percent lime and

TABLE 3
 COMPRESSIVE STRENGTH VS pH FOR GLACIAL TILL
 A-4(6) CURED AT 140 F

Time (days)	2% Lime		4% Lime		6% Lime		8% Lime	
	psi	pH	psi	pH	psi	pH	psi	pH
1	78	12.30	60	12.40	—	12.40	—	12.40
2	110	12.25	88	12.40	73	12.40	68	12.40
3	133	12.00	118	12.40	101	12.40	82	12.40
4	131	11.80	133	12.30	123	12.40	109	12.40
15	233	11.20	560	11.50	536	11.80	459	12.05
30	240	10.90	800	11.10	764	11.35	748	11.50

Note: Glacial till + 2 percent lime at room temperature for 144 hours gives a pH of 12.40.

TABLE 4
 COMPRESSIVE STRENGTH VS pH FOR GLACIAL TILL A-6(9)

Time (hr)	1% Lime		3% Lime		5% Lime		7% Lime		9% Lime	
	psi	pH	psi	pH	psi	pH	psi	pH	psi	pH
(a) Room Temperature										
1	41	11.85	42	12.30	45	12.40	45	12.40	43	12.40
6	39	11.85	42	12.30	43	12.35	43	12.35	47	12.40
12	38	11.70	41	12.25	41	12.30	41	12.35	46	12.35
24	43	11.70	45	12.25	46	12.30	45	12.30	40	12.35
(b) Moist Cured at 140 F										
12	43	11.45	72	12.15	82	12.30	82	12.30	83	12.35
24	45	11.30	79	12.05	93	12.20	91	12.25	105	12.30
48	44	11.20	188	11.90	210	12.10	194	12.20	195	12.25
72	54	11.05	211	11.70	301	11.80	258	11.85	255	12.10
96	—	—	—	—	306	11.70	332	11.70	282	11.85
120	—	—	—	—	312	11.60	408	11.60	350	11.70
192	—	—	—	—	343	11.45	415	11.45	417	11.50
264	—	—	—	—	384	11.35	454	11.35	489	11.35

cured for 4 days at 140 F still had a pH of 12.40, showing that the system still contained $\text{Ca}(\text{OH})_2$.

The unconfined compressive strength for specimens with 8 percent lime and cured for four days was only 109 psi, while specimens with 2 percent lime cured for four days had an unconfined compressive strength of 131 psi. As curing continued and the lime was consumed, the pH dropped; in the case of the 2 percent lime-treated specimen, the pH at 15 days was 11.20 and the unconfined compressive strength was 233 psi. At the end of 30 days, the samples treated with 8 percent lime had a pH of 11.50 and the unconfined compressive strength had increased to 748 psi.

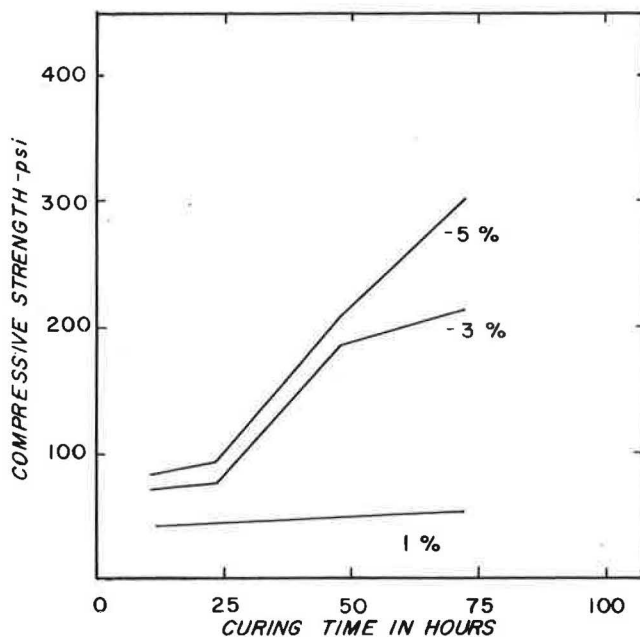


Figure 1. Lime-treated glacial till moist cured at 140 F.

Table 4 shows that there is little or no change in the strength for the first 24 hr, if the A-6(9) specimens are cured at ambient temperature. This can be explained by the fact that lime kills the plasticity of the soil, and 24 hr at ambient temperature is too soon to receive benefits from the formation of calcium silicates. However, when the specimens were moist cured at 140 F for 24 hr, the strength was doubled.

One percent lime did not improve the soil from the standpoint of strength even with prolonged curing. The pH for the 1 percent lime soil mixture after one hour was 11.85, showing that all of the lime had reacted; therefore, calcium silicates could not be formed. As the pH test for one hour had indicated, 3 percent lime killed plasticity. There was also an increase in the unconfined compressive strength from 42 psi for the 6-hr room cured samples to 211 psi for the samples cured at 140 F for 72 hours.

As shown by Figures 1 and 2 for each percentage of lime added to the glacial till A-6(9) soil there was a maximum strength increase, which was related to a definite curing time. The chemical reactions of the soil and lime after the initial reaction are dependent on time and temperature. Five percent lime gave a greater strength for 24 hr curing at 140 F than 3, 7, and 9 percent. Likewise, 7 percent lime exceeded the strength of 5 and 9 percent after 96 hr curing, and 9 percent exceeded 7 percent at 192 hr curing. In every case the specimens gained strength as the pH decreased. Three percent lime was adequate for stabilizing the soil, and increased the strength four-fold, but additional lime resulted in a longer reaction with additional strength.

Figures 3, 4, and 5 show the relationship of pH and unconfined compressive strength for Porter Creek clay. Seven percent gave a pH of 12.30 for 48 hr when cured at room temperature; therefore, 6 percent is probably enough to kill the plasticity (Fig. 3). A plot of the unconfined compressive strength for specimens cured for 72 hr at 140 F shows 6 percent should give a strength of about 200 psi. However, if strength was the aim of the stabilization, lime in a percentage greater than 10 would be reacted in 72 hr at a temperature of 140 F, since 10 percent gave a pH of only 11.10 (Fig. 4). In most cases, additional strength above that obtained from the reaction resulting from the amount of lime required to hold the pH at 12.40 for one hour at room temperature is not needed.

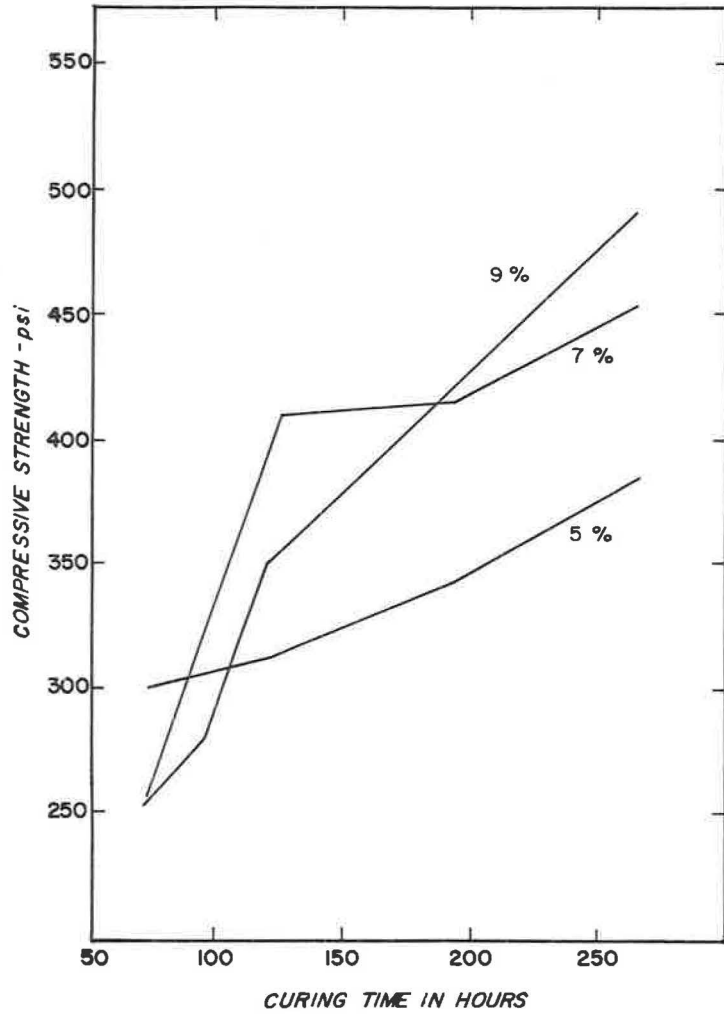


Figure 2. Lime-treated glacial till moist cured at 140 F.

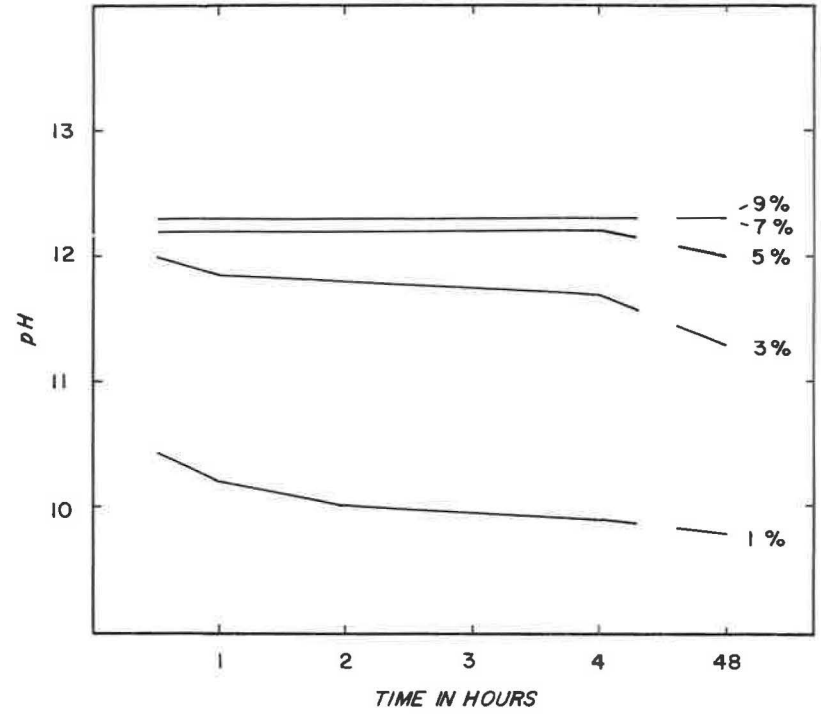


Figure 3. Porter Creek clay and lime, pH results at room temperature.

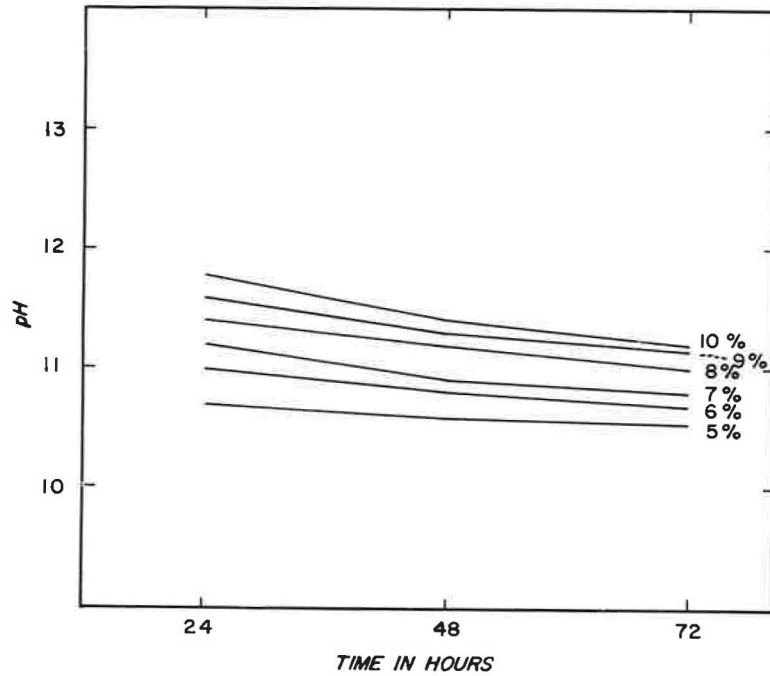


Figure 4. Porter Creek clay and lime, pH results moist cured at 140 F.

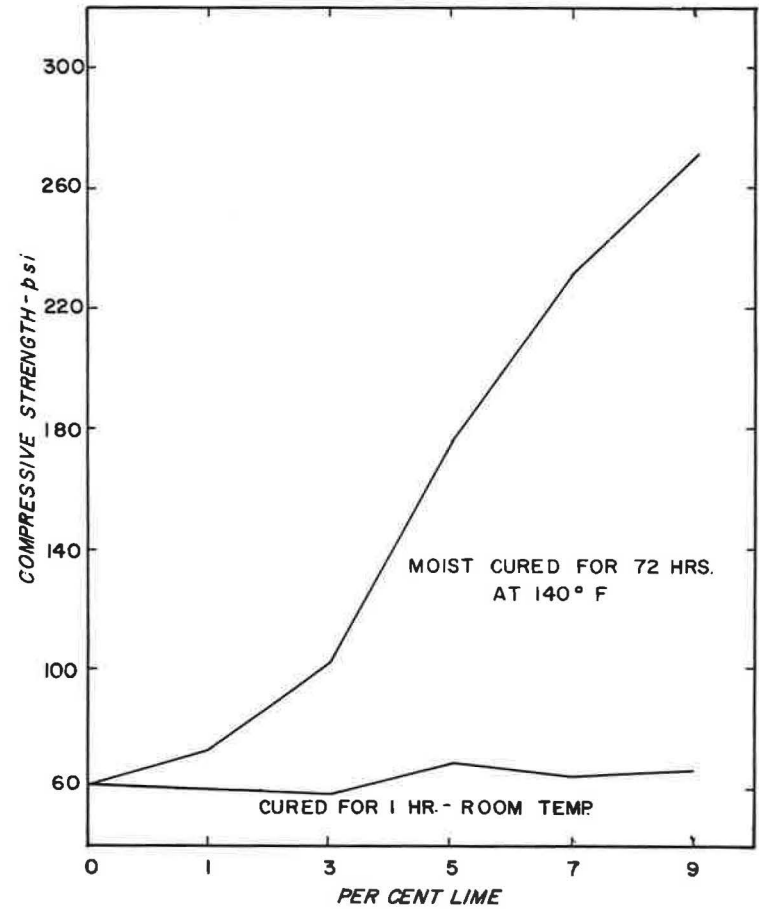


Figure 5. Lime-treated Porter Creek clay, room temperature vs oven cured.

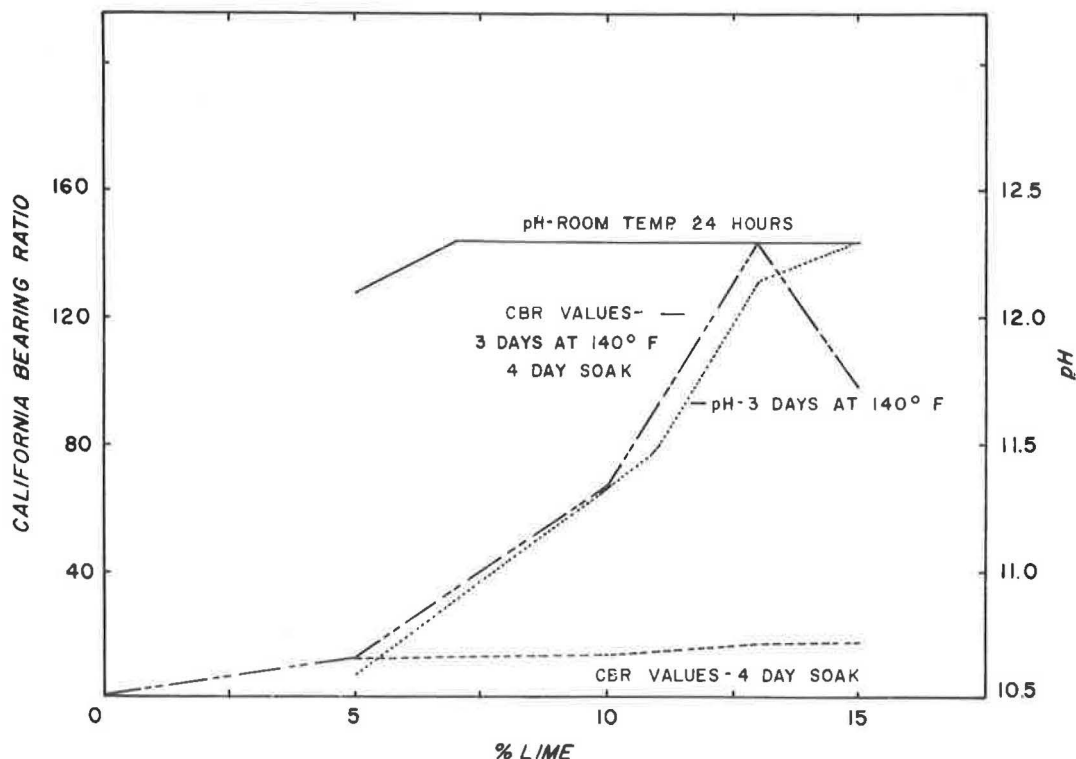


Figure 6. Lime-treated Pierre shale.

Figure 6 shows the relationship of pH results for room-cured slurries, slurries treated at 140 F for 72 hr, CBR values for specimens cured at 140 F for 72 hr, and CBR values for specimens cured only by the 4-day soak period. Nine percent lime is needed to hold the pH at 12.30 for one hour, but 15 percent is needed to hold the pH at 12.30 for 72 hr at 140 F; i.e., at 140 F just under 15 percent lime would be consumed in the soil reactions in 72 hr. This does not mean the greatest strength would be obtained from 15 percent in 72 hr. The greatest CBR value was obtained after 72 hr cure at 140 F with 13 percent lime (Fig. 6). However, the lime requirement of 9 percent as determined by the one-hour pH test increased the CBR value from 2 percent to 45 percent after curing for 72 hr at 140 F. A CBR value of 12 percent was recorded for specimens treated with 9 percent lime and cured only by soaking for 4 days at room temperature.

Figure 7 shows the relationship between pH values for slurries of Pierre shale kept at room temperature and those kept at 140 F for 3 yr. The pH for the slurry treated at 140 F for 72 hr was within a 0.05 pH of the sample maintained at room temperature for 365 days. This correlates with field results of Eades, Grim, and Nichols (4).

Research by Eades and Grim (3), and Diamond (2), offered proof that the clay minerals are destroyed with calcium silicate hydrates, resulting in new minerals when clays and soils are treated with lime. It seems reasonable to assume that a gel phase precedes the crystalline phase as reported by Brand (1). Therefore, if the pH of a soil-lime mixture is below 12.40, and the unconfined compressive strength increases until the pH is around 11.00, it must be due to the crystallization of the gels and the liberation of excess Ca^{++} . The released calcium would combine with silica ions which would be in equilibrium with the high pH of the system and increase strength.

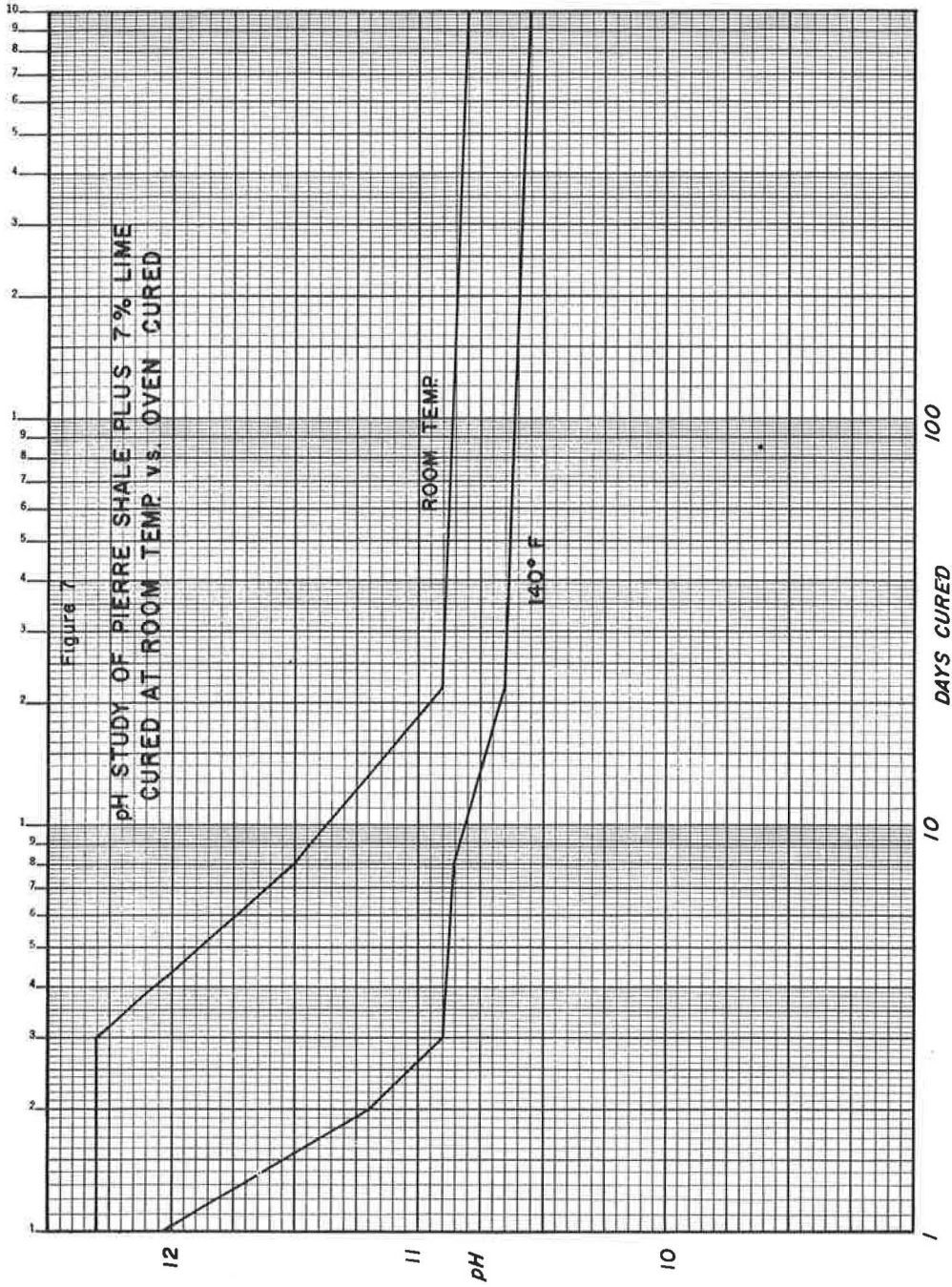


Figure 7.

SUMMARY OF TEST PROCEDURE

1. Representative samples of air-dried, minus No. 40 soil to equal 20 gm of oven-dried soil are weighed to the nearest 0.1 gm and poured into 150-ml (or larger) plastic bottles with screw tops.

2. Since most soils will require between 2 and 5 percent lime, it is advisable to set up five bottles with lime percentages of 2, 3, 4, 5, 6. This will insure, in most cases, that the percentage of lime required can be determined in one hour. Weigh the lime to the nearest 0.01 gm and add it to the soil. Shake to mix soil and dry lime.

3. Add 100 ml of CO₂-free distilled water to the bottles.

4. Shake the soil-lime and water until there is no evidence of dry material on the bottom. Shake for a minimum of 30 seconds.

5. Shake the bottles for 30 seconds every 10 minutes.

6. After one hour, transfer part of the slurry to a plastic beaker and measure the pH. The pH meter must be equipped with a Hyalk electrode and standardized with a buffer solution having a pH of 12.00.

7. Record the pH for each of the lime-soil mixtures. If the pH readings go to 12.40, the lowest percent lime that gives a pH of 12.40 is the percent required to stabilize the soil. If the pH did not go beyond 12.30 and 2 percent lime gives the same reading, the lowest percent which gives a pH of 12.30 is that required to stabilize the soil. If the highest pH is 12.30 and only 1 percent lime gives a pH of 12.30, additional test bottles should be started with larger percentages of lime.

CONCLUSIONS

A measure of the amount of lime consumed by a soil after one hour affords a quick method of determining the percent lime required for lime stabilization. A pH meter with a Hyalk glass electrode can be used to determine the percent lime which has reacted with a soil. This is accomplished by measuring the pH of lime-soil slurries to determine the minimum percent lime required to maintain a pH of 12.40.

The use of the pH meter is also applicable for determining the required percentage of dolomitic or magnesium limes for soil stabilization.

The one-hour pH or "quick test" can only be used to determine the lime requirements of a soil for stabilization. Since strength gains are related to the formation of calcium silicates, and their formation varies with the mineralogical components of the soil, a strength test is necessary to show the percentage of strength increase.

REFERENCES

1. Brand, Walter. Der Einfluss von Kalziumhydroxyd auf die Eigenschaften Schuffigon Boden in Straussenbau. Thesis. Reinisch-Westfälischen Technischen Hochschule Aachen, 1962.
2. Diamond, S., White, J. L., and Dolch, W. L. Transformation of Clay Minerals by Calcium Hydroxide Attack. Clay and Clay Minerals, Proc. of the Twelfth Nat'l. Conf., 1963.
3. Eades, J. L., and Grim, R. E. Reaction of Hydrated Lime with Pure Clay Minerals in Soil Stabilization. HRB Bull. 262, pp. 51-63, 1960.
4. Eades, J. L., Nichols, F. P., and Grim, R. E. Formation of New Minerals with Lime Stabilization as Proven by Field Experiments in Virginia. HRB Bull. 335, pp. 31-39, 1962.
5. Hilt, H. G., and Davidson, D. T. Lime Fixation in Clayey Soils. HRB Bull. 262, pp. 20-32, 1960.
6. Jackson, M. L. Soil Chemical Analysis. Prentice Hall, 1960.
7. Maclean, D. J., and Sherwood, P. T. Study of the Occurrence and Effect of Organic Matter in Relation to the Stabilization of Soils with Cement. Proc. Fifth Internat. Conf. on Soil Mech. and Found. Eng., Paris. Vol. 2, pp. 269-275, 1961.