

MODIFICATION OF LOESS BY THE
USE OF HYDRATED LIME

by 6408

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INTRODUCTION

Statement of Problem

Many areas throughout the midwestern United States are covered with a fine-grained, wind-deposited material which is classified as loess. The thickness of deposits is highly variable, ranging from a few inches in thickness to deposits which will exceed one-hundred feet in depth. Figure 1 shows the approximate boundaries of existing loess deposits. It should be noted that the major portion of the loess deposits are located within areas in which thirty inches of rainfall occurs annually and freezing conditions exist for several months of the year.

Failure of pavements on loess soils occurs almost universally as a result of frost heave, shear failure, and/or compressibility when high moisture conditions and freezing temperatures exist. For this reason, highway construction customarily involves thick bases and subbases for protection against pavement failure.

Purpose of the Study

The purpose of the study was to investigate a possible reduction in capillarity head in loess through the addition of hydrated lime. Agglomeration, a result of

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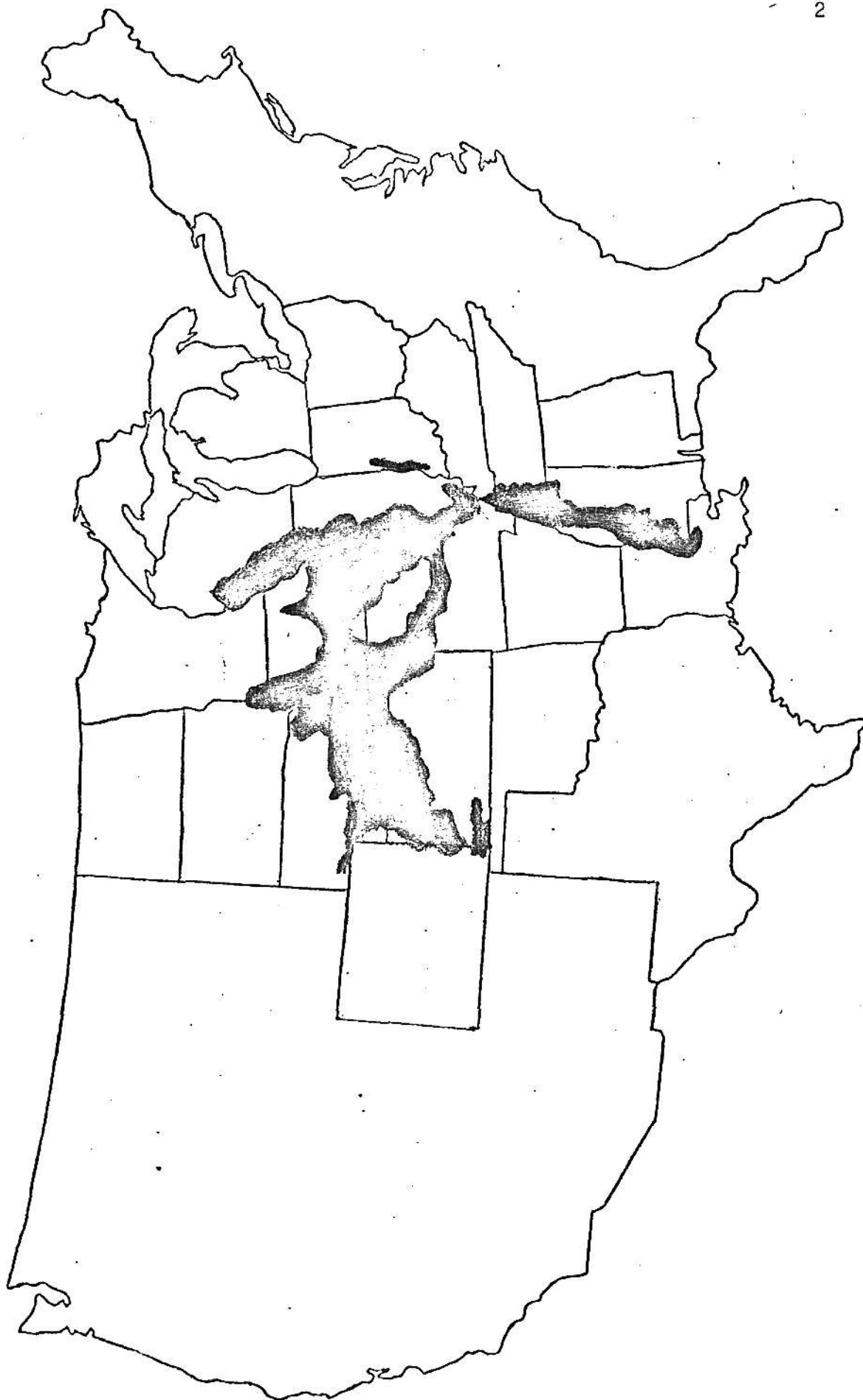


FIGURE 1. Loess Deposits of the Midwest

lime-soil reactions, increases the pore diameter of soil particles thereby causing an increase in permeability and attendant decrease in capillarity head. In the midwestern United States where the number and severity of freeze-thaw cycles are high, the reduction of capillarity head is of great importance.

Another potential benefit investigated was the possibility of increasing the shear strength by only the agglomeration of the soil particles. An increase in shear strength would make possible a reduction in subbase, base, and/or pavement thicknesses of highways.

The lime retention point theory was used as a basis for lime contents in order to insure that the minimum amount of lime will be used.

Scope of the Study

The scope of the study included a thorough review of the literature in the field of lime stabilization. The intent of the review will be to set forth the problems involved in lime stabilization of both clays and silts and the methods employed in solving the problems.

An experimental program was conducted on loessial soils obtained from various locations in the Midwest. The sampling locations are given by Table 1 and depicted by Figure 2.

The experimental program was conducted to determine whether the addition of hydrated lime would improve the performance characteristics of the various loessial soils.

<u>Approximate Location</u>	<u>County</u>	<u>State</u>	<u>Legal Description</u>
Atwood, Kansas	Rawlins	Kansas	S27, T 3S, R34W
Clinton, Iowa	Clinton	Iowa	S 6, T81N, R 7E
Kansas City, Kansas	Wyandotte	Kansas	S12, T11S, R23E
Omaha, Nebraska	Pottawattomie	Iowa	S34, T76N, R44W
St. Louis, Missouri	St. Louis	Missouri	S15, T45N, R 5E

TABLE 1. Sampling Locations

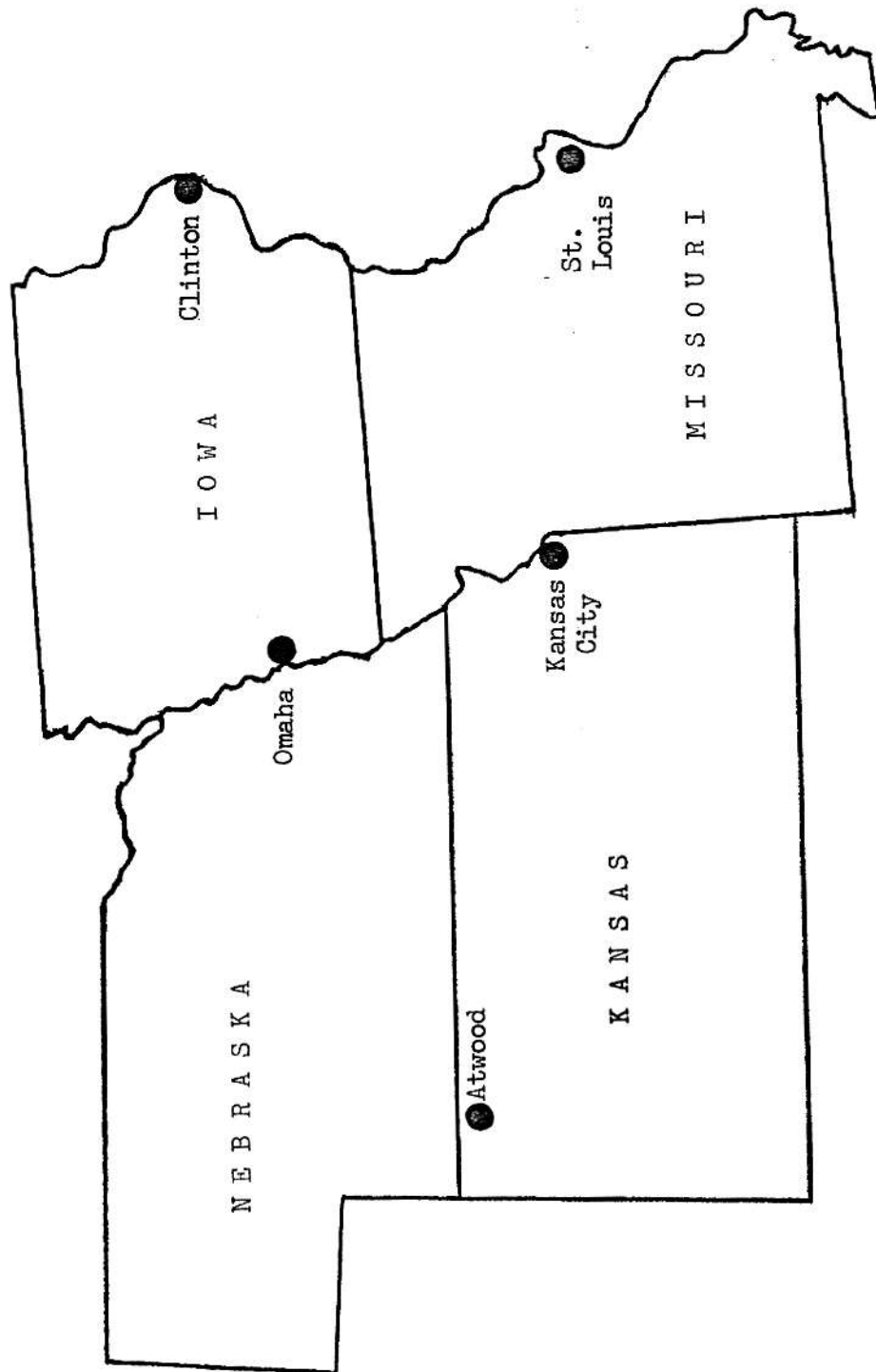


FIGURE 2. Sampling Locations

LITERATURE REVIEW

Loess

Although loess is quite common in the Midwestern United States, the published information relating to its engineering characteristics is scarce. Most of the work performed with loess has been of a geological nature dealing with its origin and development rather than its physical properties.

Origin

Loess is a buff-colored sedimentary deposit consisting of predominately silt size particles with some clay and traces of fine sand. The thickness of most loess deposits is only a few feet although some exist which have depths exceeding 160 feet. Usually, loess is nonstratified, homogeneous, calcareous, and porous and may have a weak vertical structure resembling jointing.

Mineralogically, loess is composed primarily of quartz (1,2). Smaller amounts of clay minerals, micas, feldspars, hornblende, and pyroxene also add to the composition which is so variable that little is known about the rocks from which the minerals originated.

Loess is thought to have been transported and deposited by the wind (3). Desert basins and active outwash plains have long been thought to be possible sources of loess

because they are areas in which fine sediment is exposed at the surface. The distribution of both sources are closely related to the distribution of loess deposits, thus leading to the theory regarding its general source areas.

Not all eolian deposits can be classified as loess, only wind-blown materials that are deposited directly on the ground surface from aerial transportation are defined as loess. Generally, loess deposits were formed in arid or semi-arid climates although the large deposits found in Alaska show that Arctic conditions were also favorable (4). The origin of the loess must evolve from a rather dry climate because winds are unable to transport intermixed wet silts and clays.

Physical Properties

The properties of loess are related to the distance of the deposits from their probable sources. The finer, clayey loess was deposited farther from the source while the coarse or more sandy is located nearer the source (5).

The unique feature of loess is the particle size which generally ranges from 0.01 to 0.05 mm (6) which is verified by data accumulated by the Bureau of Reclamation (7). Of 148 samples collected throughout the Midwest, 76 per cent were classified silty loess, 18 per cent clayey loess, and 6 per cent were found to be sandy loess.

The specific gravity of loess ranges from approximately 2.57 to 2.79 with 2.65 as an average value in the Midwest (7,8).

The Atterburg Limits of loess vary according to the gradation. In the case of silty loess the liquid limits range from 25-35 while corresponding plasticity index values are 5-15 (7,8).

In the natural state, loess is characterized by a relatively high permeability depending upon the gradation and density. Reworking and recompaction of loess almost always increased its density. In-place dry density varies from 66 to 104 pcf while standard density varies from 100 to 112 pcf (7,8,9).

The shear strength of loess varies as a function of moisture content, density, and amount of clay present. The angle of internal friction ranges from 28 to 36 degrees while the cohesion varies from 0 to 70 psi. The strength of loess in the natural state is attributable to the partial cementing of soil particles by calcareous elements present. Complete collapse of the soil structure may result when moisture is introduced to the highly soluble calcareous elements (8).

Consolidation is also affected by the amount of clay present, moisture content, and density of the loess. The resistance to consolidation is lowered greatly by the presence of moisture. Some authorities consider loess among the worst foundation materials for this reason. Initial density of the loess also governs its susceptibility to settlement (7,8).

Lime-Soil Reactions

The addition of lime to soil causes various effects, the degree of which is dependant upon the clay mineralogy and the plasticity of the soil.

The mixing of lime and soil results in two primary reactions; agglomeration of individual soil particles and cementation of the soil. The increase in grain size caused by the agglomeration of the soil particles results in plasticity index and density reduction and an increase in permeability.

Cementation or pozzolanic reaction, which is similar to the hydration of cement, is quite time consuming and results in appreciable strength gains of soil. Details of all reactions are given in the Appendix.

Lime-Loess Reactions

The stabilization of loess and silts through pozzolanic reactions has been found to be uneconomical as compared to other types of stabilization (10). The majority of the investigations involving lime have therefore been conducted with highly plastic soils.

Atterburg Limits

The effect of lime on loess is similar to that of lime with clays. The maximum reduction of plasticity index occurs with the addition of 1-4 per cent lime (10,11). Some investigations

have found that the effect of lime was dependent upon the soil horizon (11). The alteration of the Atterburg Limits of lime-treated B and C horizons vary as a function of the clay content whereas the A horizon shows no definite relationship to the amount of clay present. Although the upper horizons require lower lime quantities for maximum reduction of plasticity index, the C horizon is better suited for lime treatment. The decrease in plasticity index is accomplished by a substantial increase in plastic limit with little attendant change in the liquid limit.

Moisture-Density Relationships

The addition of lime to loess increases the optimum moisture content and lowers the dry density as measured by the Standard Proctor T-99 test (10,12).

Compressive Strength

The strength of loess is generally increased by the addition of lime, with the amount of strength gain dependent on the soil horizon. Regardless of the lime percentage, the C horizon is characterized by a relatively high strength gain, with the A horizon showing little significant gain (11).

As measured by the Standard Proctor T-99 test lime-loess mixtures have the characteristic of obtaining their maximum strength value at the optimum moisture content. Investigations have shown that even a slight deviation from the optimum moisture content results in substantial strength decreases

(10,12), with the relationship holding true regardless of the lime percentage.

Although the most common additive, fly ash, accelerates pozzolanic reactions in clay soils, little or no benefit is obtained in lime-loess mixtures unless the fly ash is of top quality (10). Investigations of various salts have shown strength benefits (13) although the extent of study in this area has been limited.

Lime Retention Point Theory

As stated previously, the mixing of lime and soil will result in agglomeration or flocculation of the soil particles and cementation or pozzolanic reaction of the soil. The reactions are the result of various chemical reactions occurring in the lime-soil mixture.

Flocculation

Flocculation is the result of either of two chemical reactions or a combination of both. The first reaction consists of cation exchange in which the calcium ions from the lime replace the weaker metallic ions, such as sodium and hydrogen, on the surface of the soil particles (14). The second process consists of attachment of additional calcium cations onto the surface of the soil particles. The reactions change the electrical charge of the soil particles causing an increase in bond between any two soil particles thereby resulting in agglomeration.

The flocculation process occurs mainly in the minus two hundred (-200) material and ceases only when the pH of the soil attains an approximate value of 11.0. Upon the attainment of a critical pH, the soil will cease assimilating any appreciable quantities of calcium ions. Therefore, irrespective of the quantity of lime used, the soil will assimilate only a certain number of calcium ions. The critical pH, which is also the lime percentage at which the greatest increase in plastic limit is observed, is termed the lime retention point (15). Lime-soil mixtures containing lime quantities less than the lime retention point exhibit little or no pozzolanic reaction. The lime causes a reduction in plasticity index and density, with an attendant increase in optimum moisture content and permeability. The lime-soil mixture becomes friable in texture in a matter of a few hours.

Cementation

Once the lime retention point has been reached, the free calcium ions are available for the pozzolanic reaction or cementation process. The cementation process is believed to be the result of aluminous and siliceous minerals of the soil reacting with lime to produce a gel of calcium silicates and calcium aluminates. The gel tends to cement the soil particles together in a manner similar to the hydration of portland cement, thereby resulting in a strength gain (16).

DESIGN OF EXPERIMENT

The experimental program included testing which was performed in the following order:

1. Raw soil
2. Lime retention point determination
3. Lime-treated soil

Raw Soil

Soil samples were obtained from the locations identified previously in the scope of the study. The raw soils were subjected to all of the tests shown hereinafter in Table 2.

Lime Retention Point Determination

Determination of the lime retention point required ten Atterburg Limits test per raw soil. Samples were prepared containing 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, and 5.0% hydrated lime by dry weight. The lime-soil mixtures were mixed for three minutes at a moisture content of optimum plus five per cent. After mixing, the samples were placed in a controlled temperature of $140 \pm 3^{\circ}\text{F}$. for a period of 24 ± 1 hours, after which they were subjected to both the liquid limit and plastic limit tests.

Lime-Treated Soil

All tests shown in Table 1 were performed on the lime-treated soils. The tests were performed at a lime content

corresponding to the lime retention point and at one-half of the lime retention point percentage. The hydrated lime used in the laboratory research was a high-calcitic type lime, the characteristics of which are shown in Table 2.

<u>Commercial Name</u>	<u>Type</u>	<u>Available Lime Index as $\text{Ca}(\text{OH})_2$</u>	<u>Residue Retained</u>	
			<u>#30</u>	<u>#200</u>
Ash Grove	High Calcitic	93.7%	0.16%	1.20%

TABLE 2. Characteristics of the Hydrated Lime Used in the Experimental Research

Procedure

Various soil properties tests were performed on the soils throughout the experimental program. All of the apparatus required for the tests was available in the laboratory. The tests that were employed, along with their designation numbers and sample sizes are shown in Table 3.

<u>Type of Test</u>	<u>Designation</u>	<u>Sample Size</u>
Grain Size Analysis	ASTM D422	0.5 kg.
Atterburg Limits	ASTM D423	0.3 kg.
Specific Gravity	ASTM D854	0.2 kg.
Standard Compaction	ASTM D698	5.0 kg.
Direct Shear Test	Bureau of Public Roads	Remolded Sample 2.5" diameter

TABLE 3. Tests Employed in the Experimental Program

All soils were crushed to pass the Number 4 sieve prior to being subjected to the Compaction Test and prior to molding for the Direct Shear Test. The following criteria were used with respect to lime treated soil:

1. The initial cure consisted of a controlled temperature of $140 \pm 3^{\circ}\text{F.}$ for a period of 24 ± 1 hours.
2. The final cure consisted of a controlled temperature of $140 \pm 3^{\circ}\text{F.}$ for a time period of 120 ± 3 hours. (The only test in which a final cure was utilized was the Direct Shear.)
3. All gradation samples were soaked for 24 ± 1 hours before washing.

The Direct Shear Test was conducted utilizing a controlled strain rate of 0.052 inches per minute.

Total Testing Required

The total number of soils tested that were required for the experimental program are shown in Table 4.

<u>Soil Condition</u>	<u>Atterburg Limits</u>	<u>Grain Size Analysis</u>	<u>Specific Gravity</u>	<u>Standard Compaction</u>	<u>Direct Shear</u>
Raw Soil	5	5	5	5	15
Retention Point Determination	50	0	0	0	0
50% Retention Point	5	5	5	5	15
Retention Point	5	5	5	5	15
TOTAL TESTS	65	15	15	15	45

TABLE 4. Total Tests Required

PRESENTATION OF DATA

All data is presented in such a manner so that the raw soil, the lime retention point percentage, and one-half the lime retention point percentage can be easily compared, i.e., for any one location the information, at the three different lime contents, is given simultaneously.

Data is presented in graphical form hereafter in this section. Data is also presented in tabular form in Appendix A.

Lime retention point determination is shown in Figures 3-7.

Grain size analysis results are depicted in Figures 8-12, while compaction characteristics are shown in Figures 13-17. Figures 18-22 show the direct shear test results.

Table 5 shows the collective results presented in Figures 3-22.

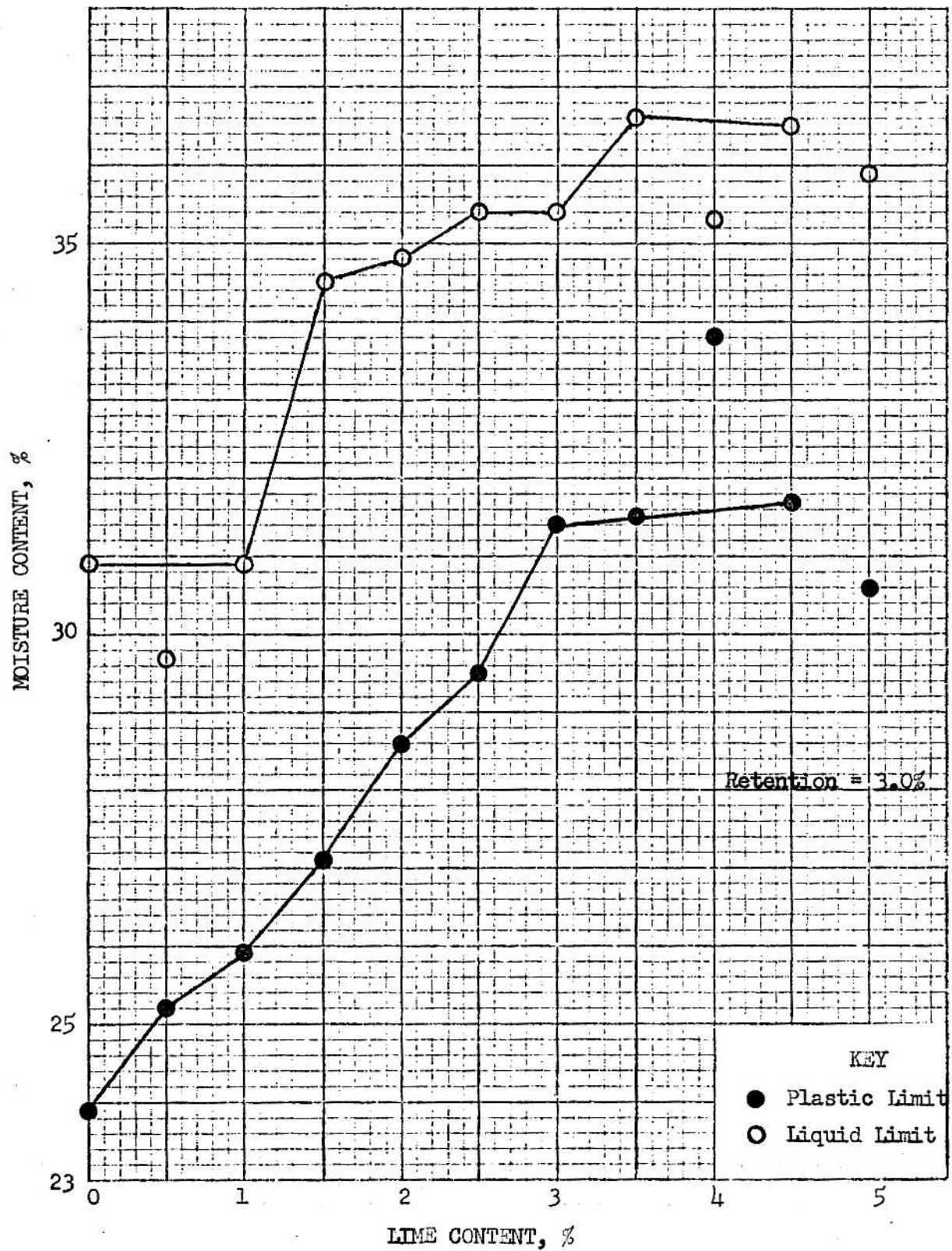


FIGURE 3. Lime Retention Point Determination for the Atwood Loess.

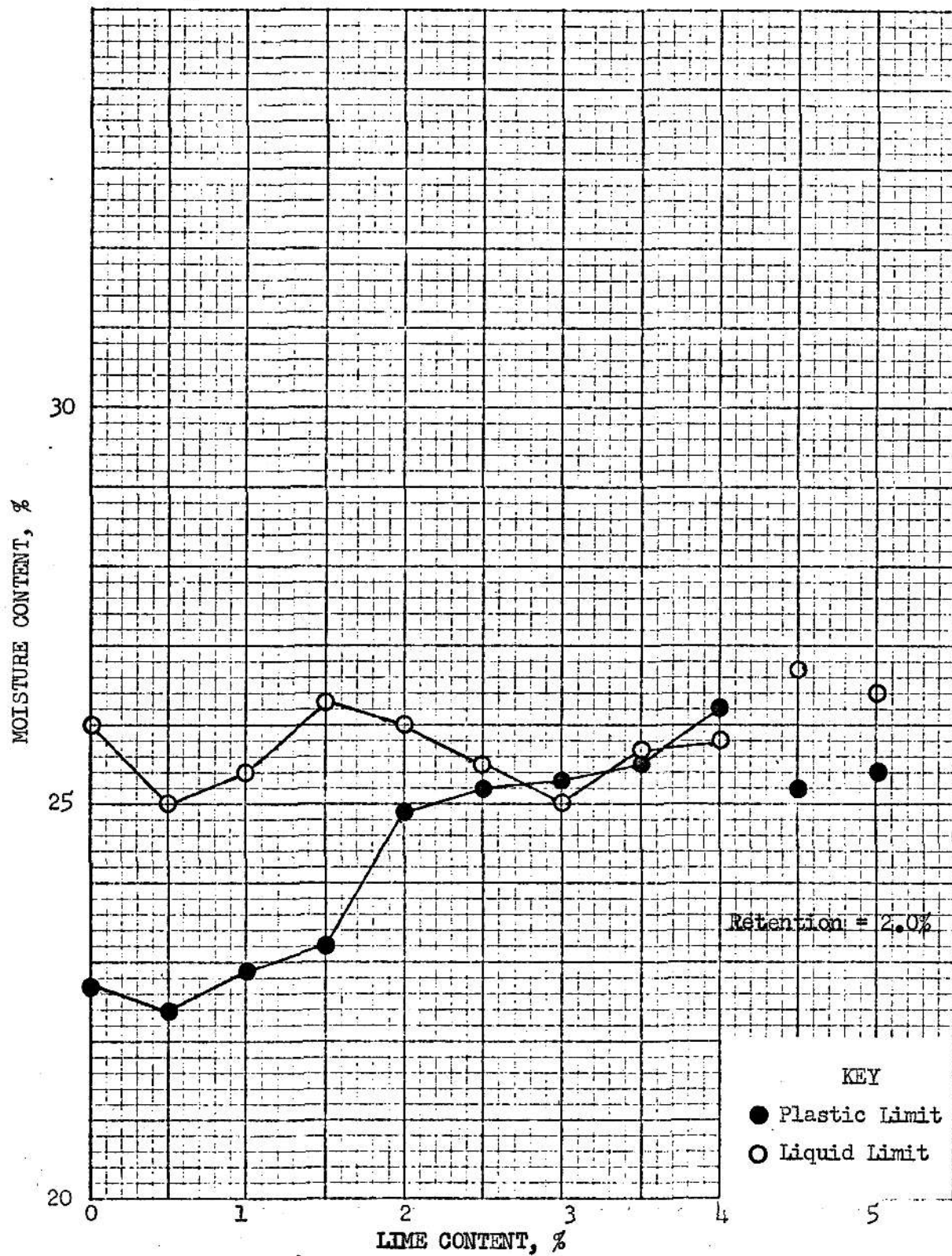


FIGURE 4. Lime Retention Point Determination for the Clinton Loess.

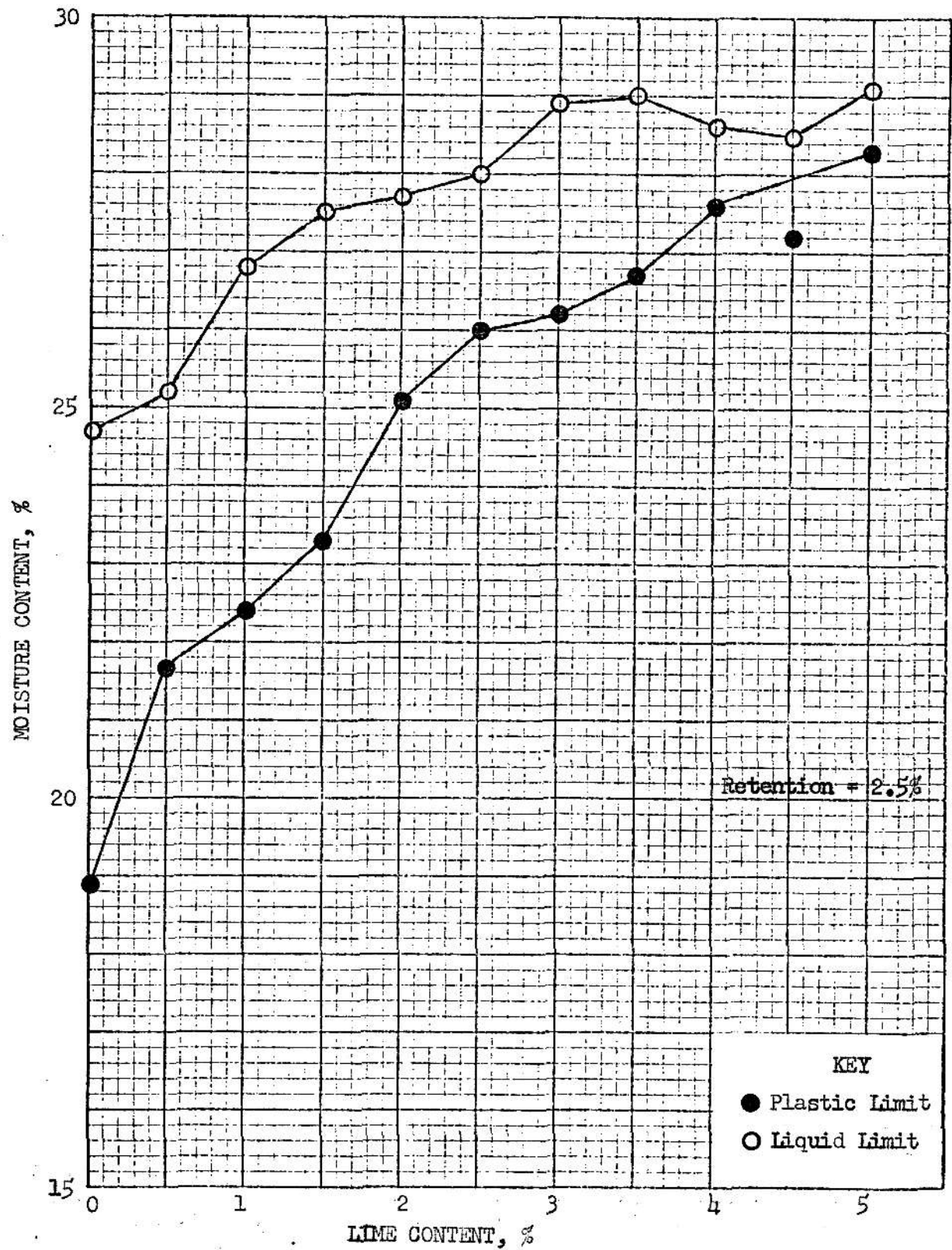


FIGURE 5. Lime Retention Point Determination for the Kansas City Loess.

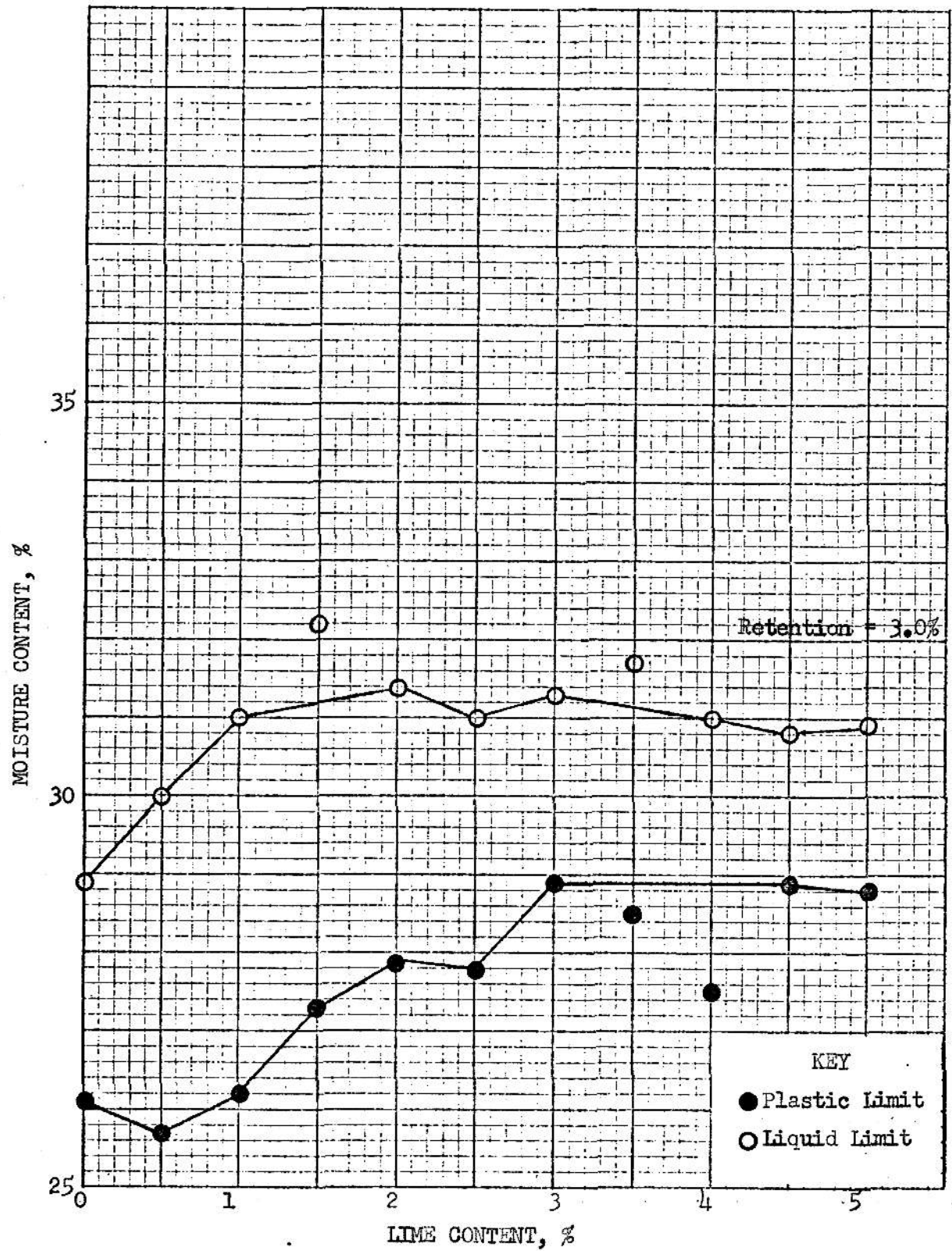


FIGURE 6. Lime Retention Point Determination for the Omaha Loess.

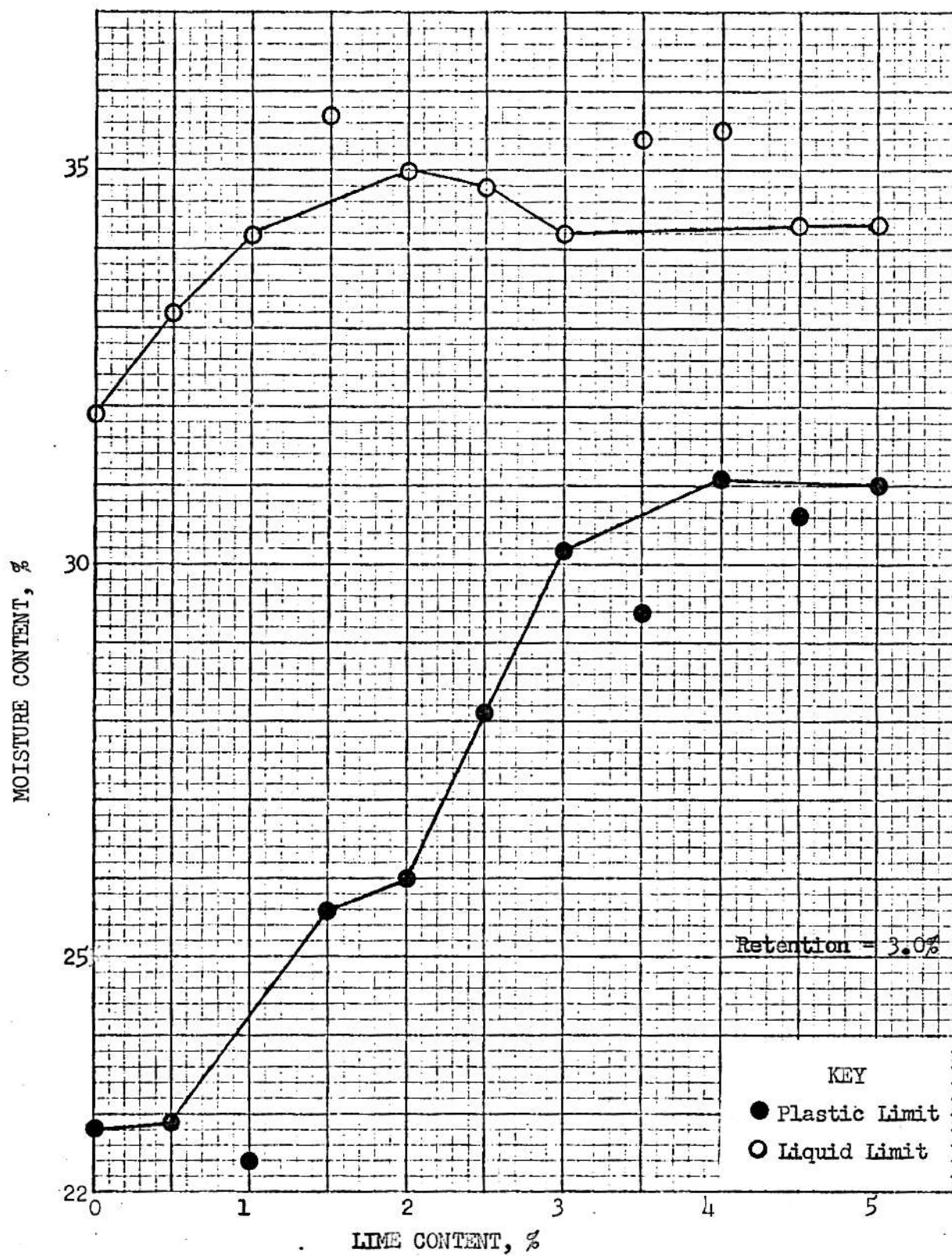


FIGURE 7. Lime Retention Point Determination for the St. Louis Loess.

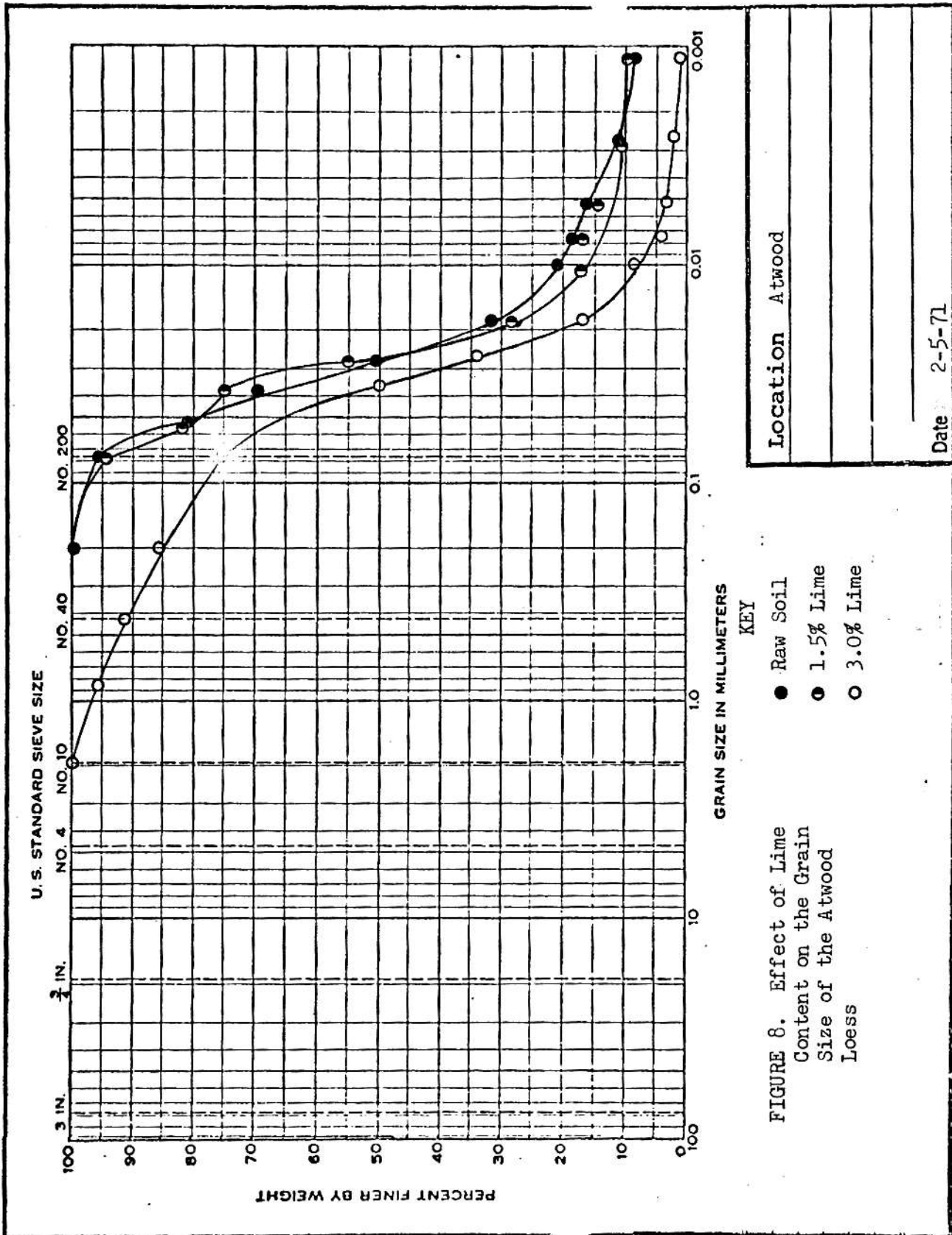


FIGURE 8. Effect of Lime Content on the Grain Size of the Atwood Loess

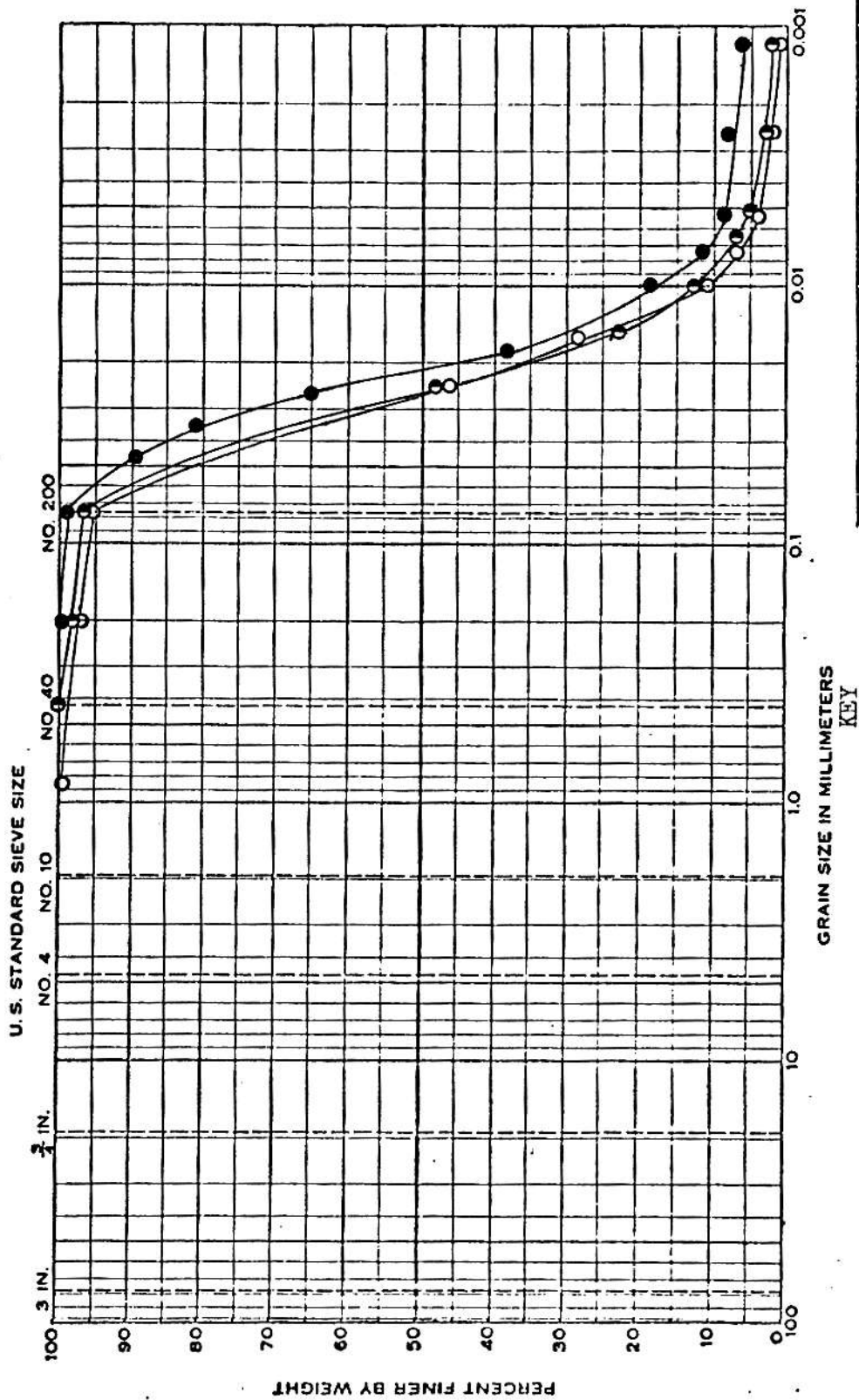


FIGURE 9. Effect of Lime Content on the Grain Size of the Clinton Loess

● Raw Soil
○ 1.0% Lime
○ 2.0% Lime

Location Clinton

Date 2-5-71

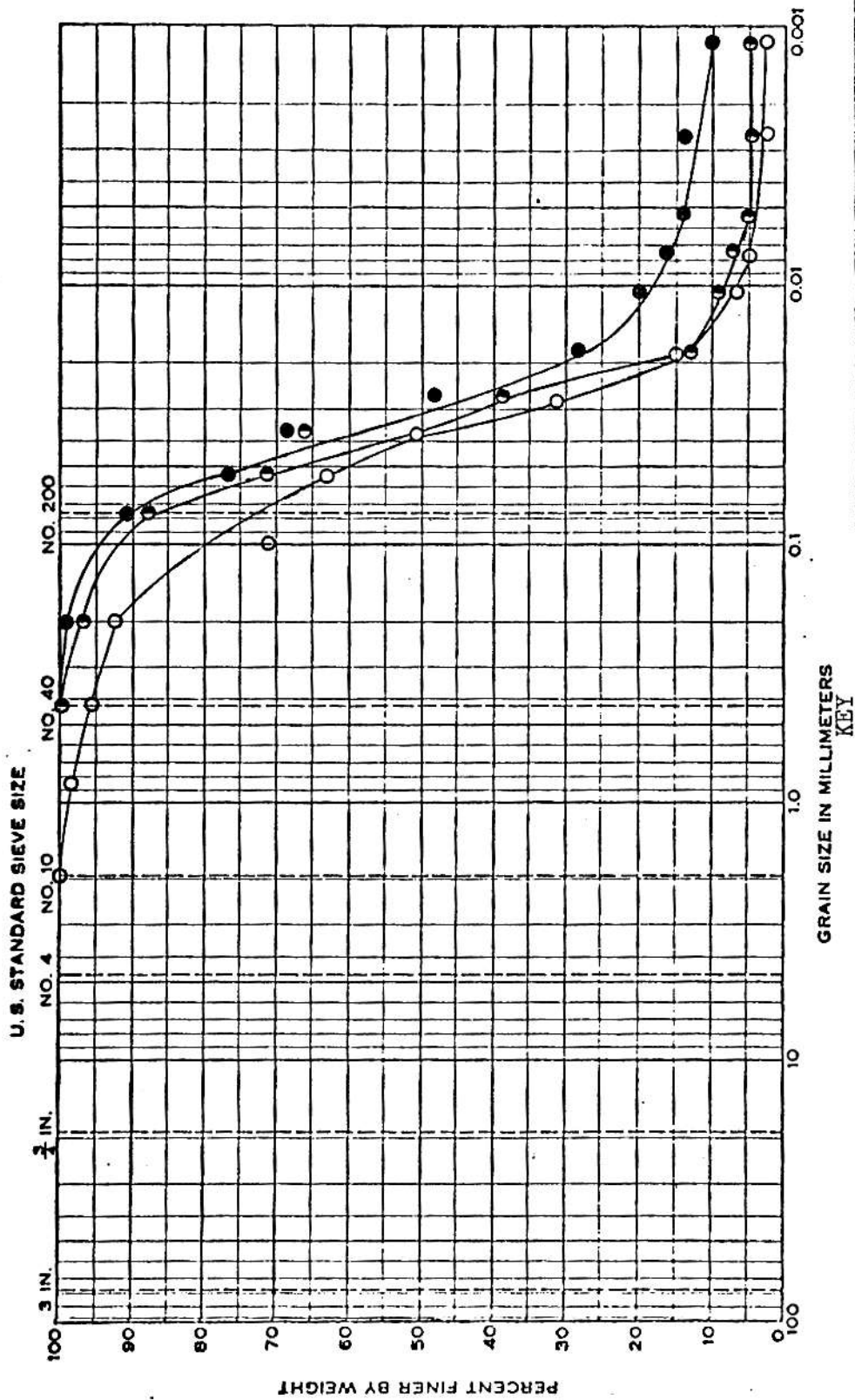


FIGURE 10. Effect of Lime Content on the Grain Size of the Kansas City Loess

Location Kansas City

Date 2-5-71

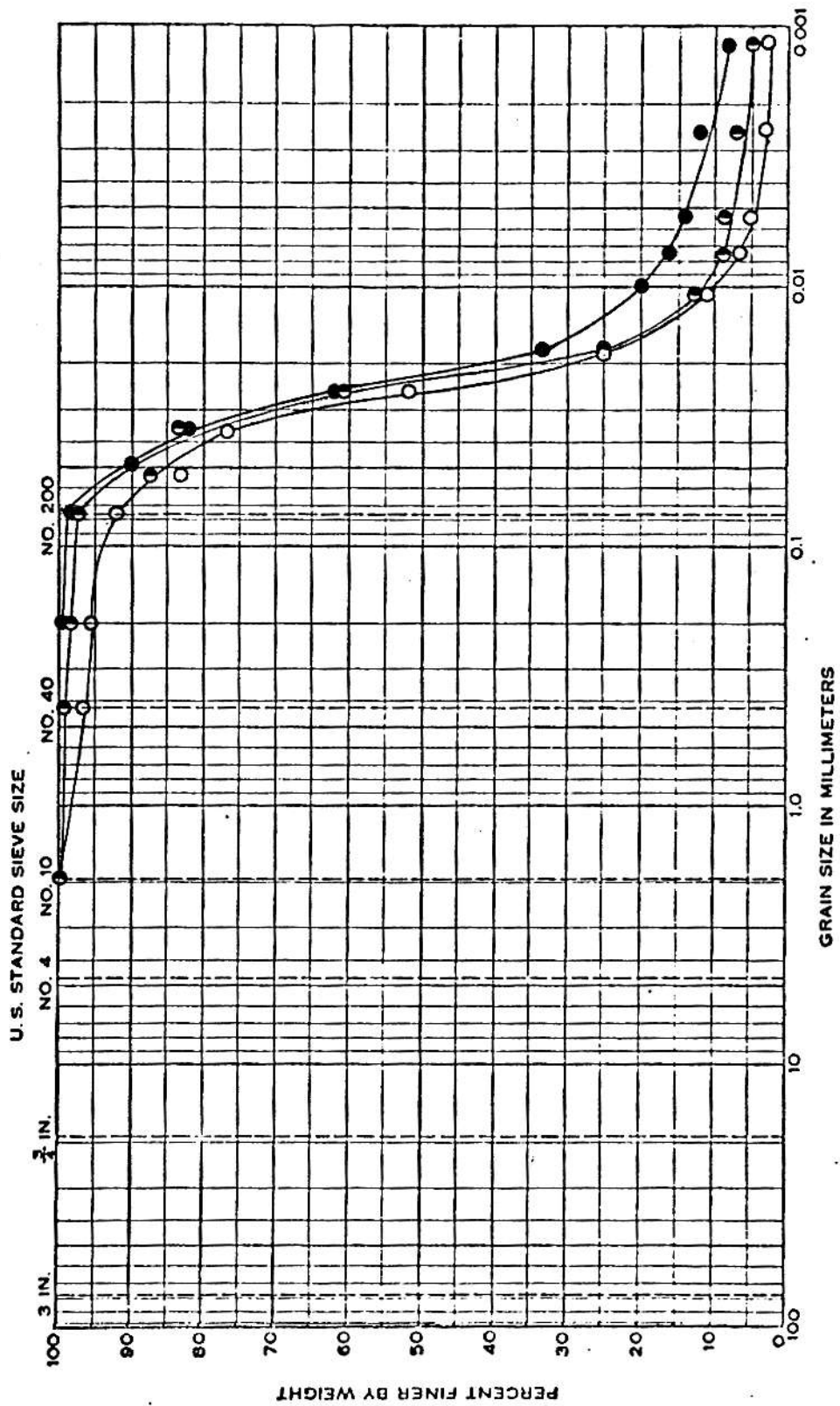


FIGURE 11. Effect of Lime Content on the Grain Size of the Omaha Loess

Location Omaha

Date 2-5-71

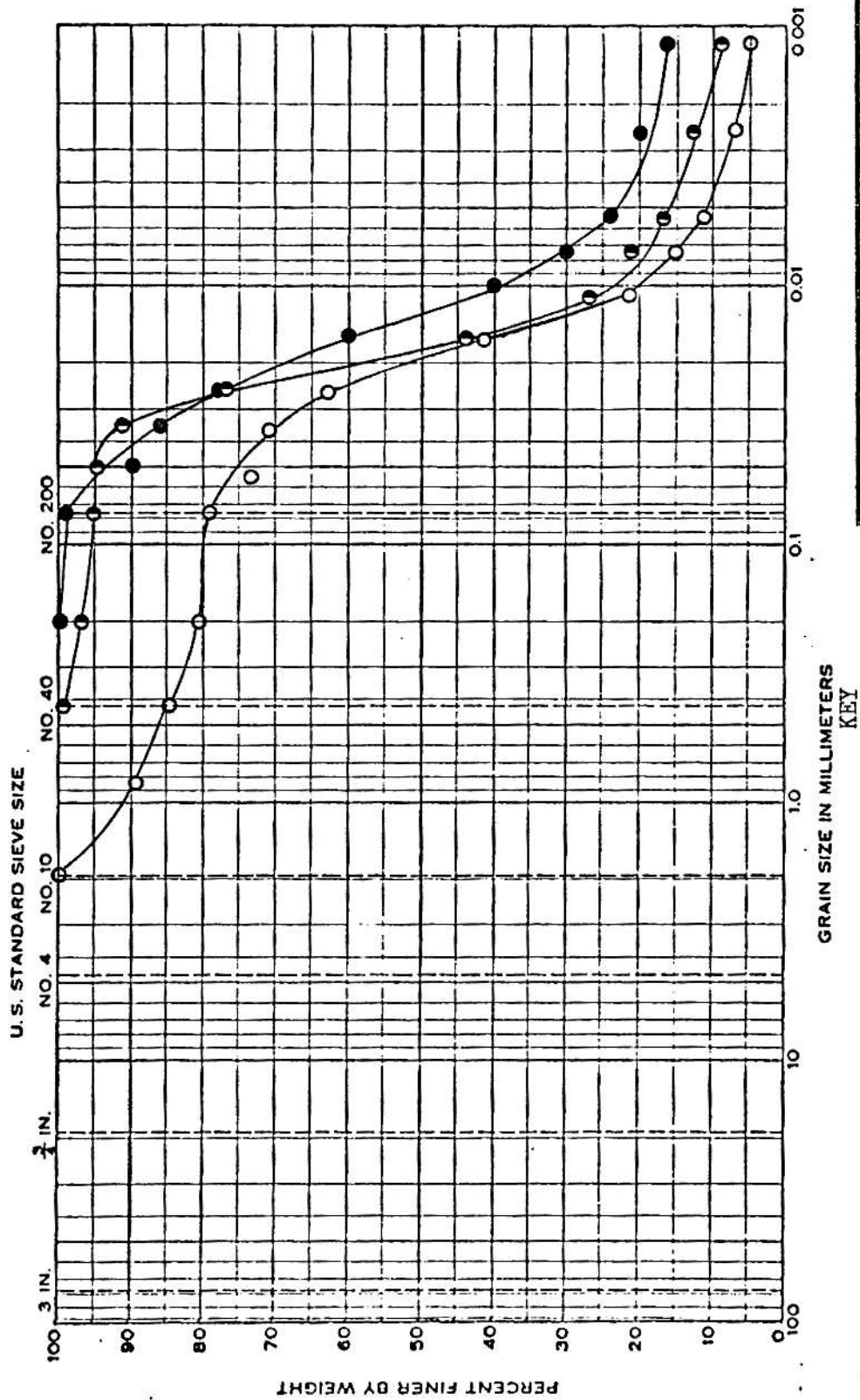


FIGURE 12. Effect of Lime Content on the Grain Size of the St. Louis Loess

Location St. Louis

Date

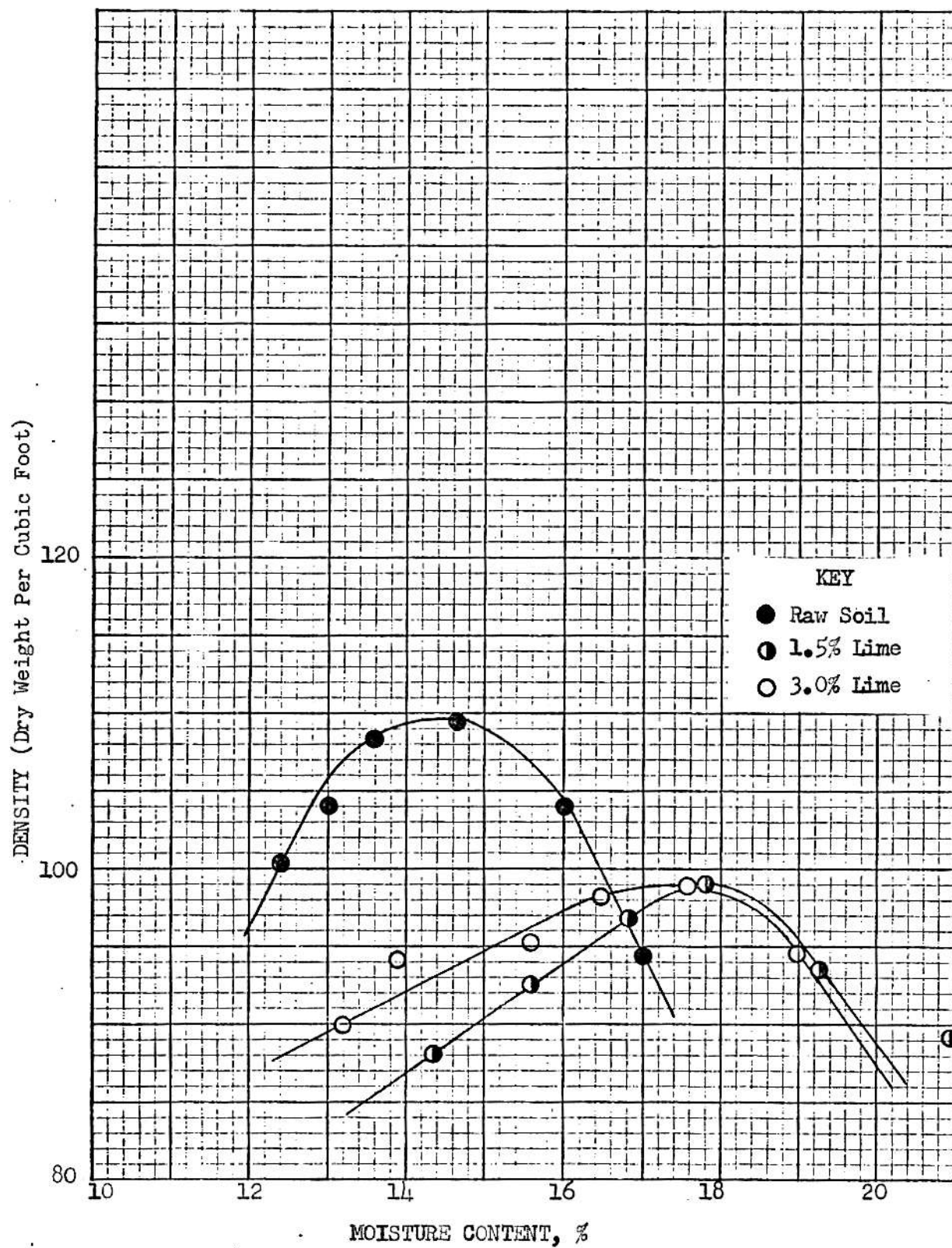


FIGURE 13. Effect of Lime Content on the Moisture-Density Relationships of the Atwood Loess.

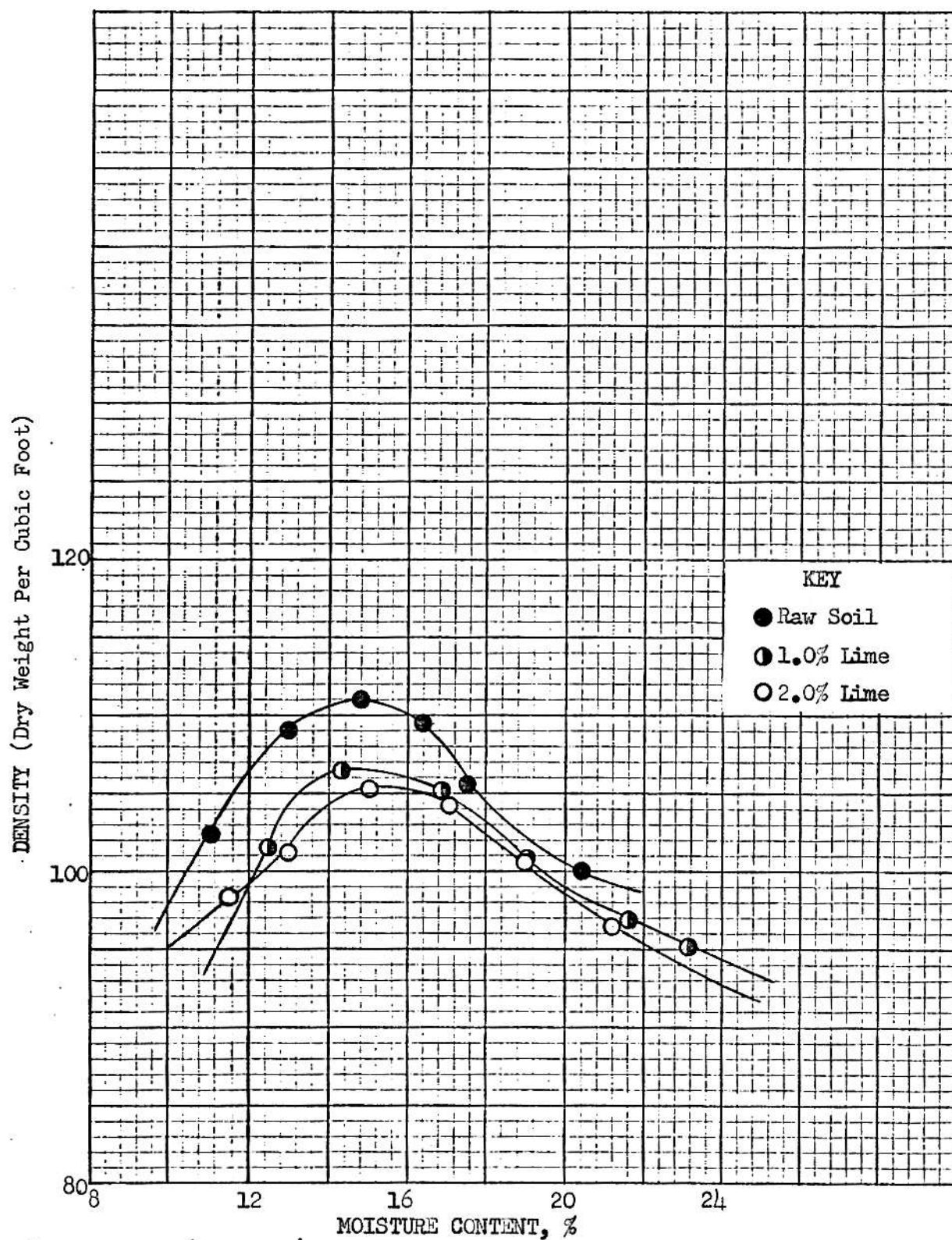


FIGURE 14. Effects of Lime Content on the Moisture-Density Relationships of the Clinton Loess.

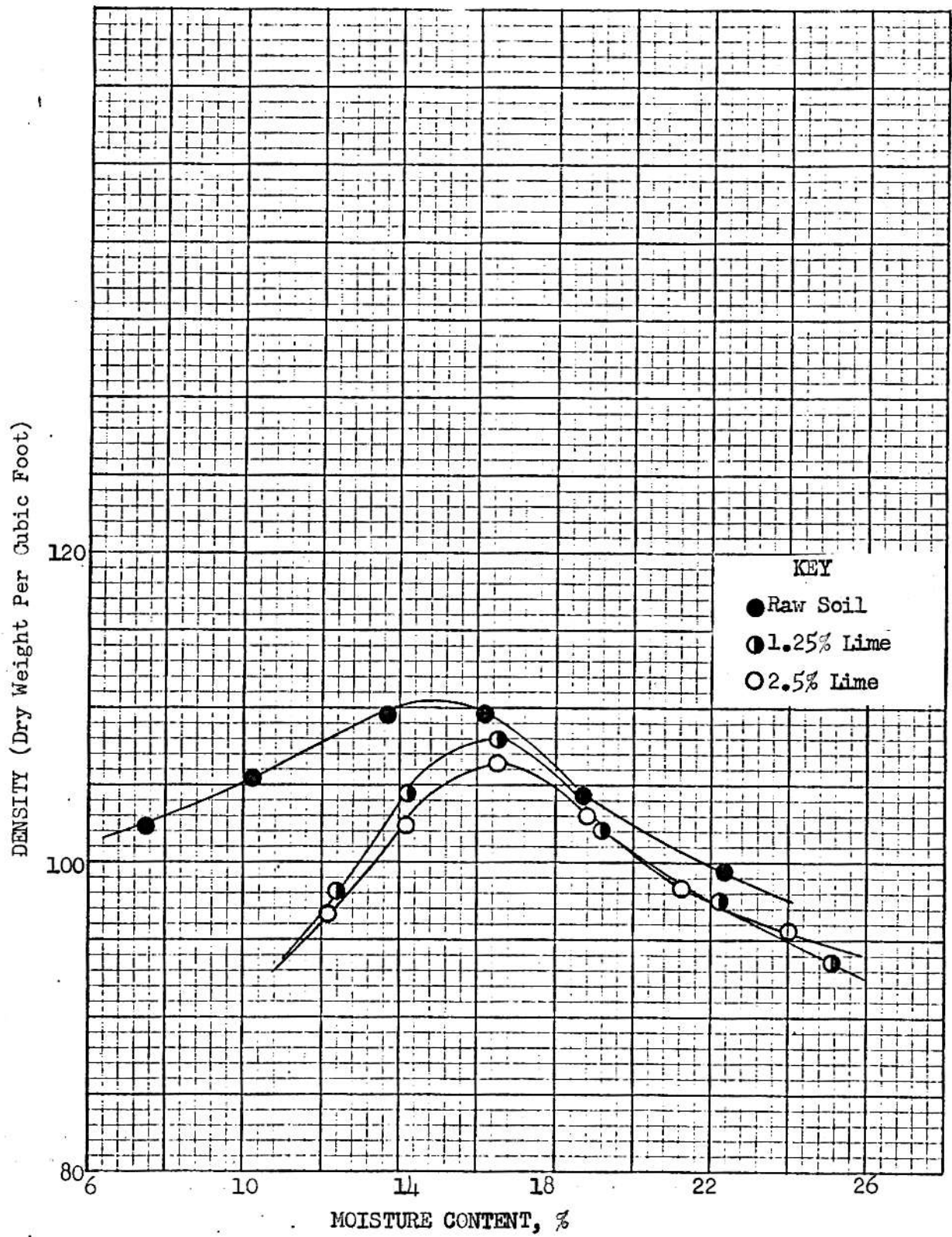


FIGURE 15. Effect of Lime Content on the Moisture-Density Relationships of the Kansas City Loess.

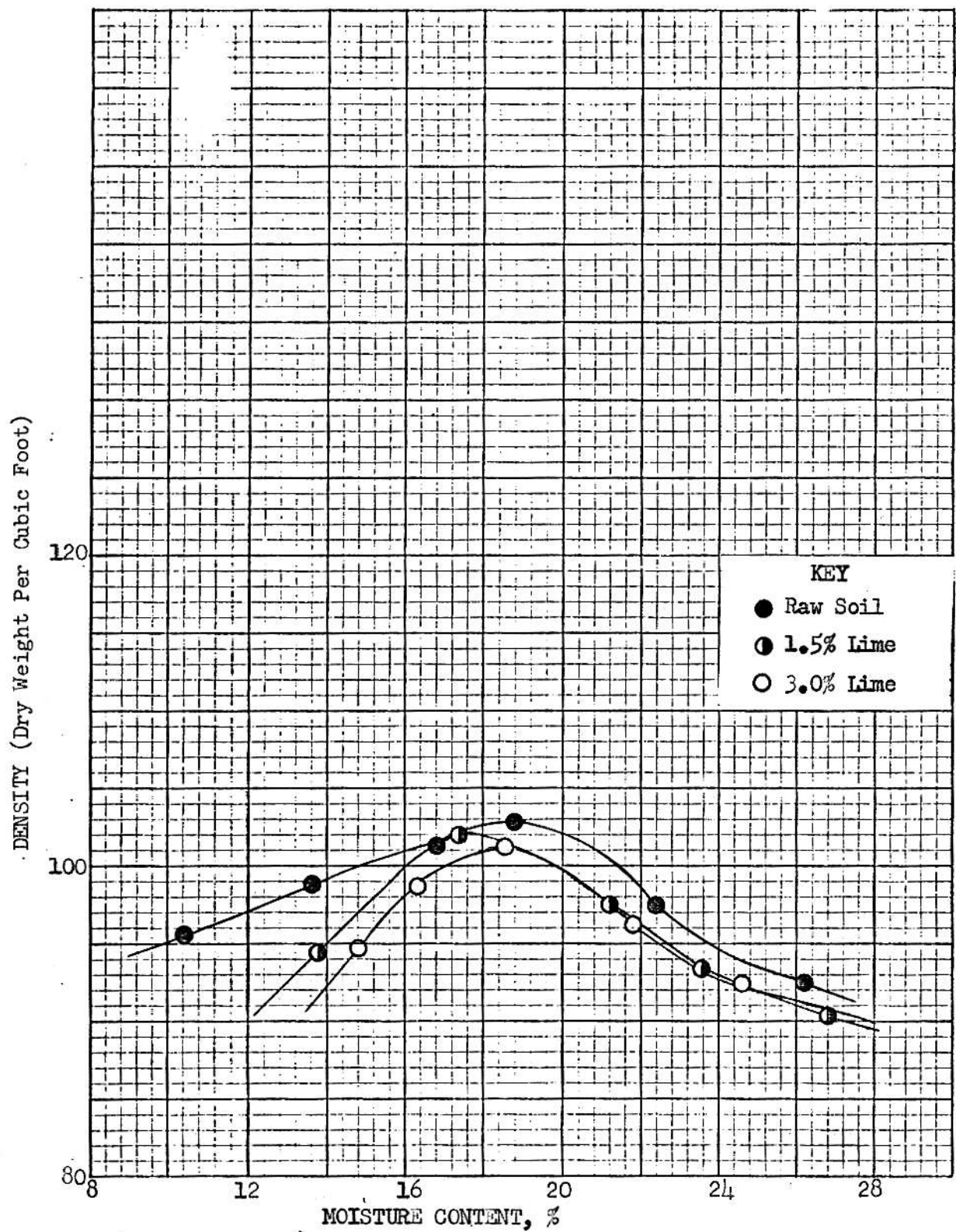


FIGURE 16. Effect of Lime Content on the Moisture-Density Relationships of the Omaha Loess.

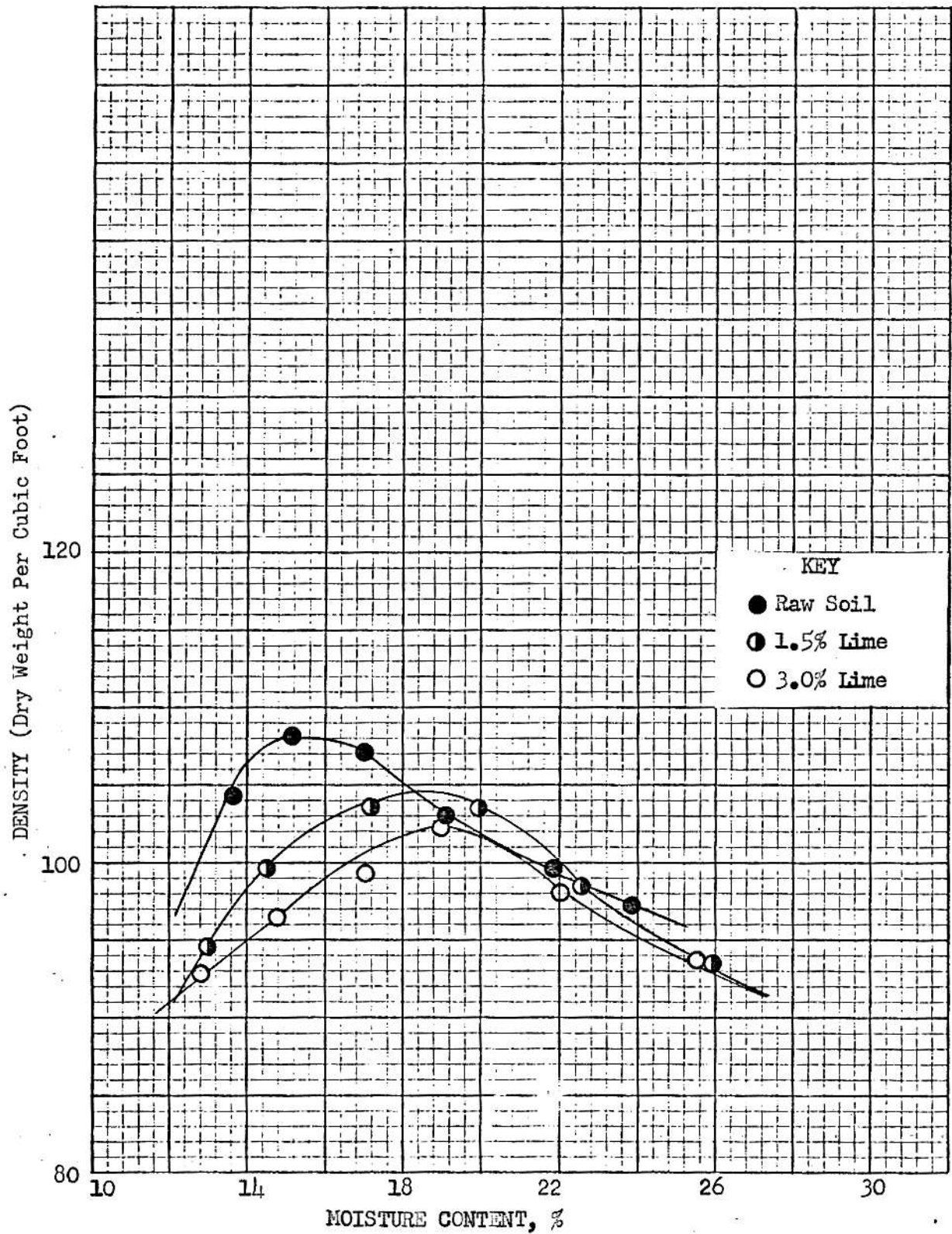


FIGURE 17. Effect of Lime Content on the Moisture-Density Relationships of the St. Louis Loess.

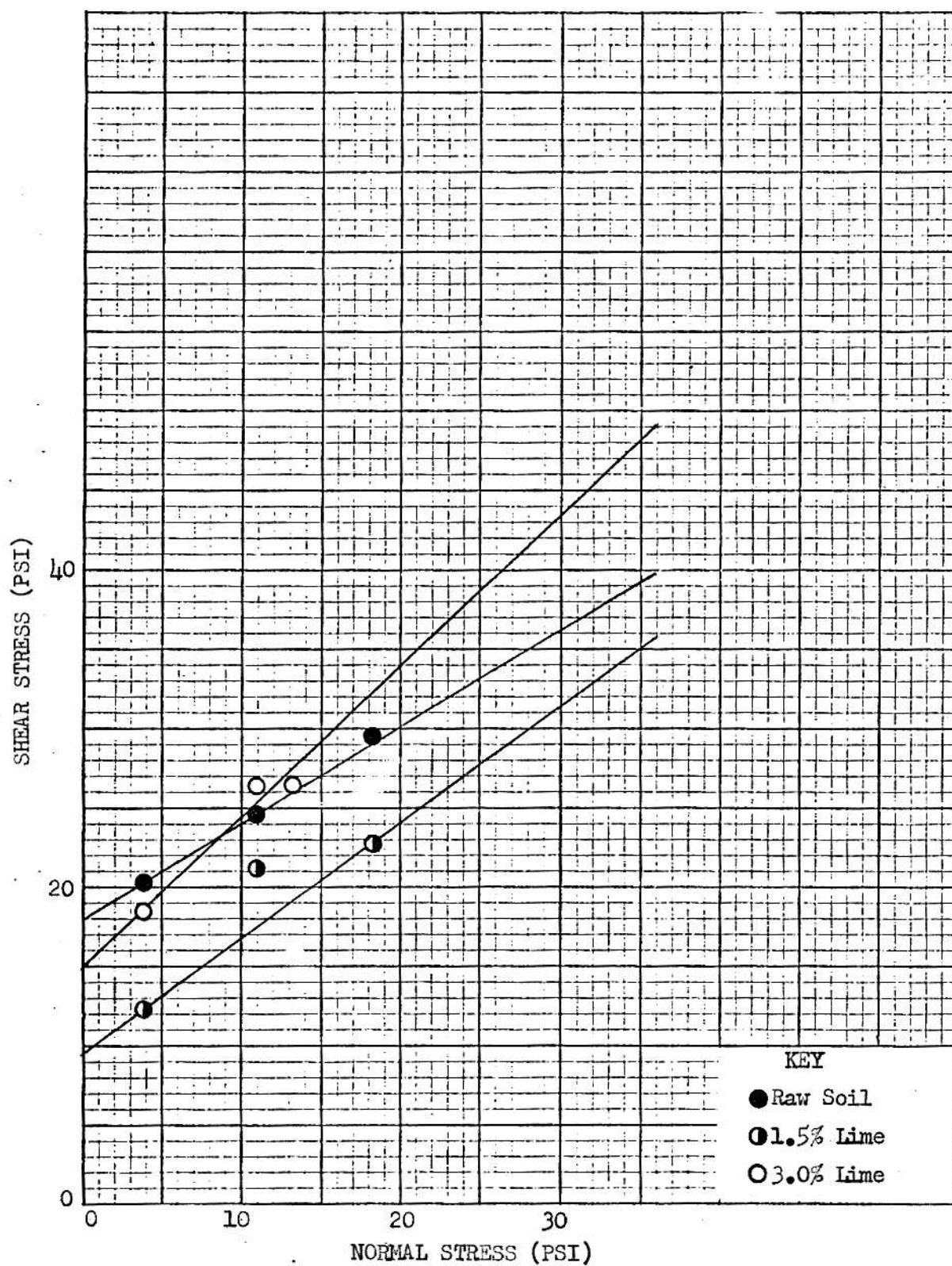


FIGURE 18. Effect of Lime Content on the Shear Strength Parameters of the Atwood Loess.

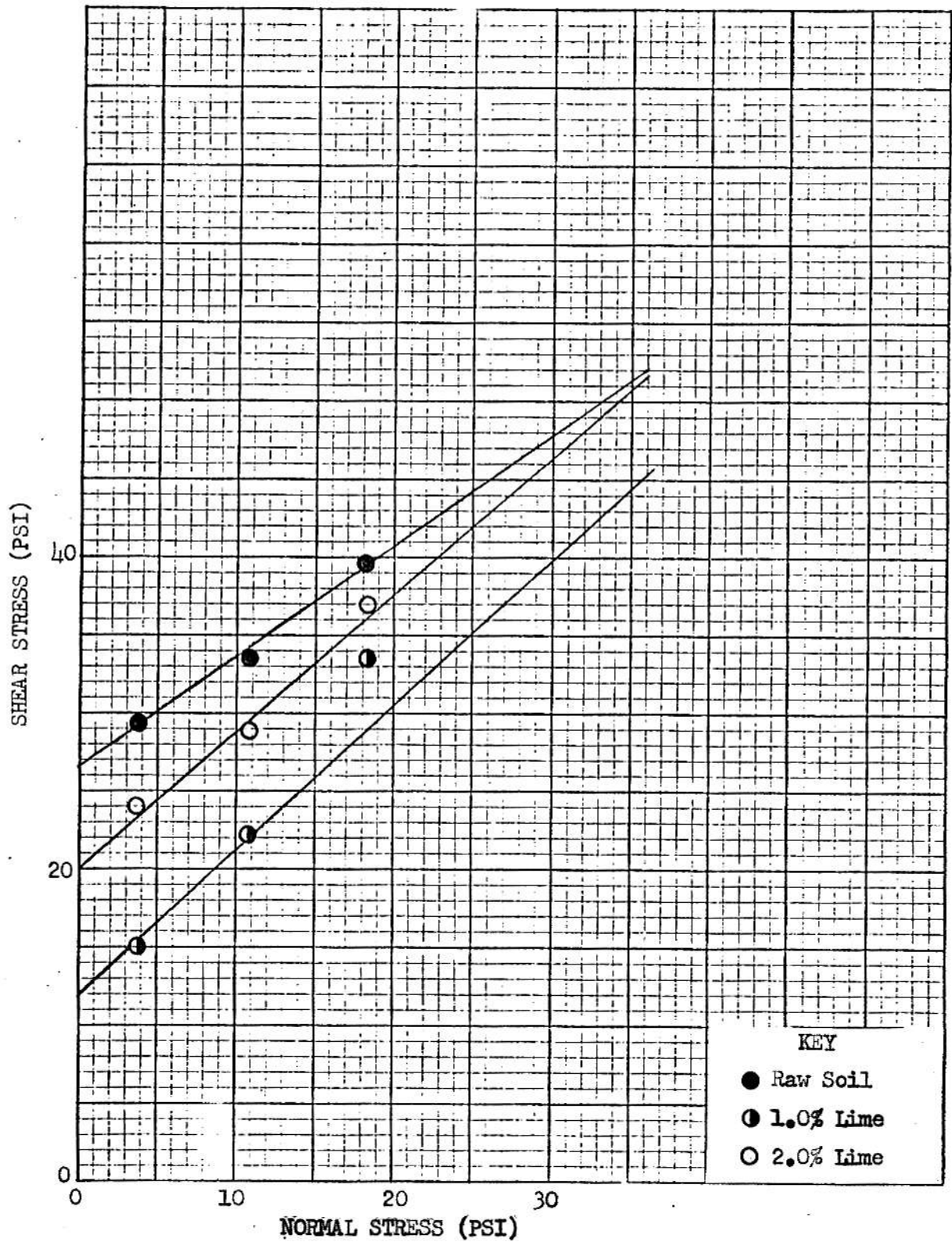


FIGURE 19. Effects of Lime Content on the Shear Strength Parameters of the Clinton Loess.

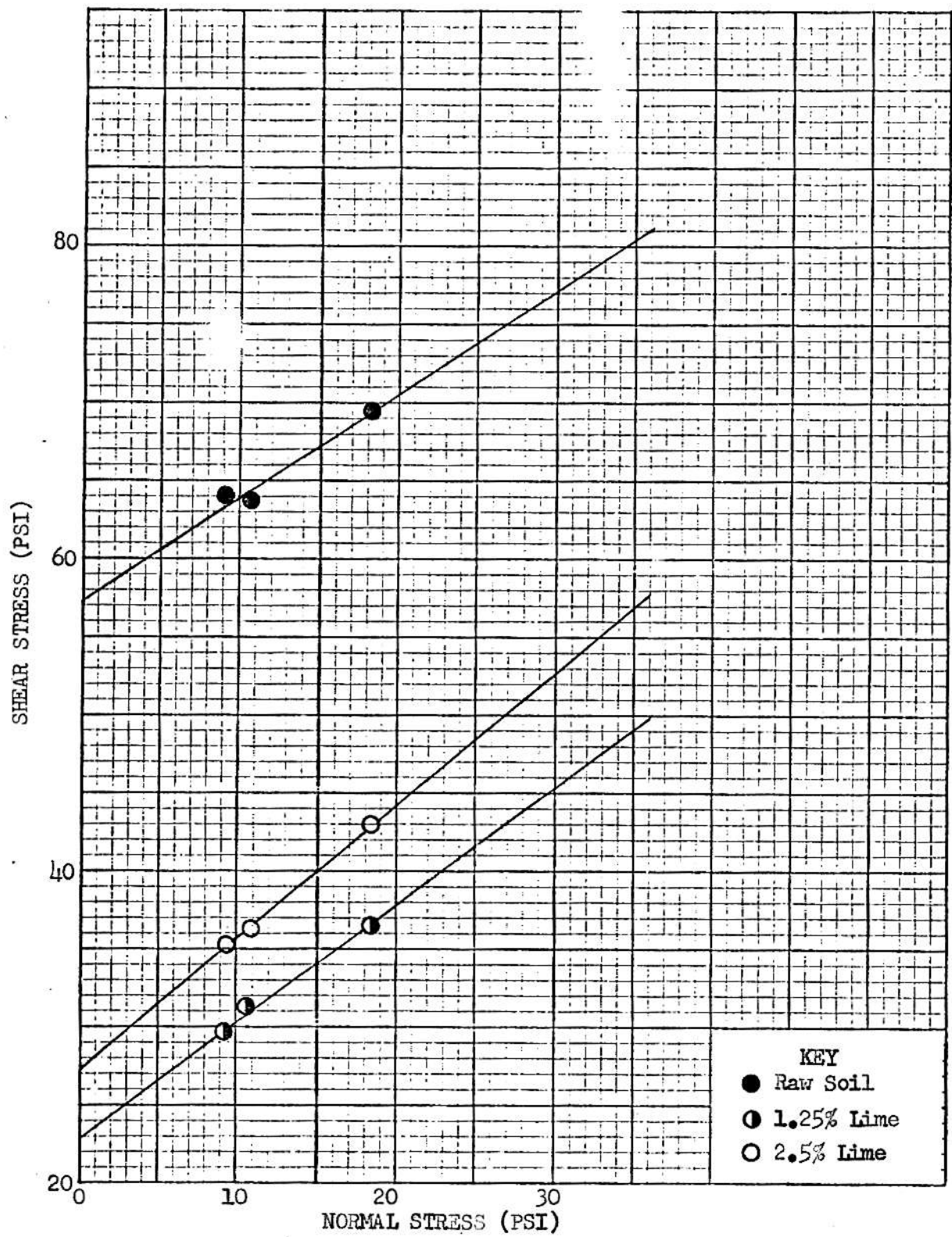


FIGURE 20. Effect of Lime Content on the Shear Strength Parameters of the Kansas City Loess.

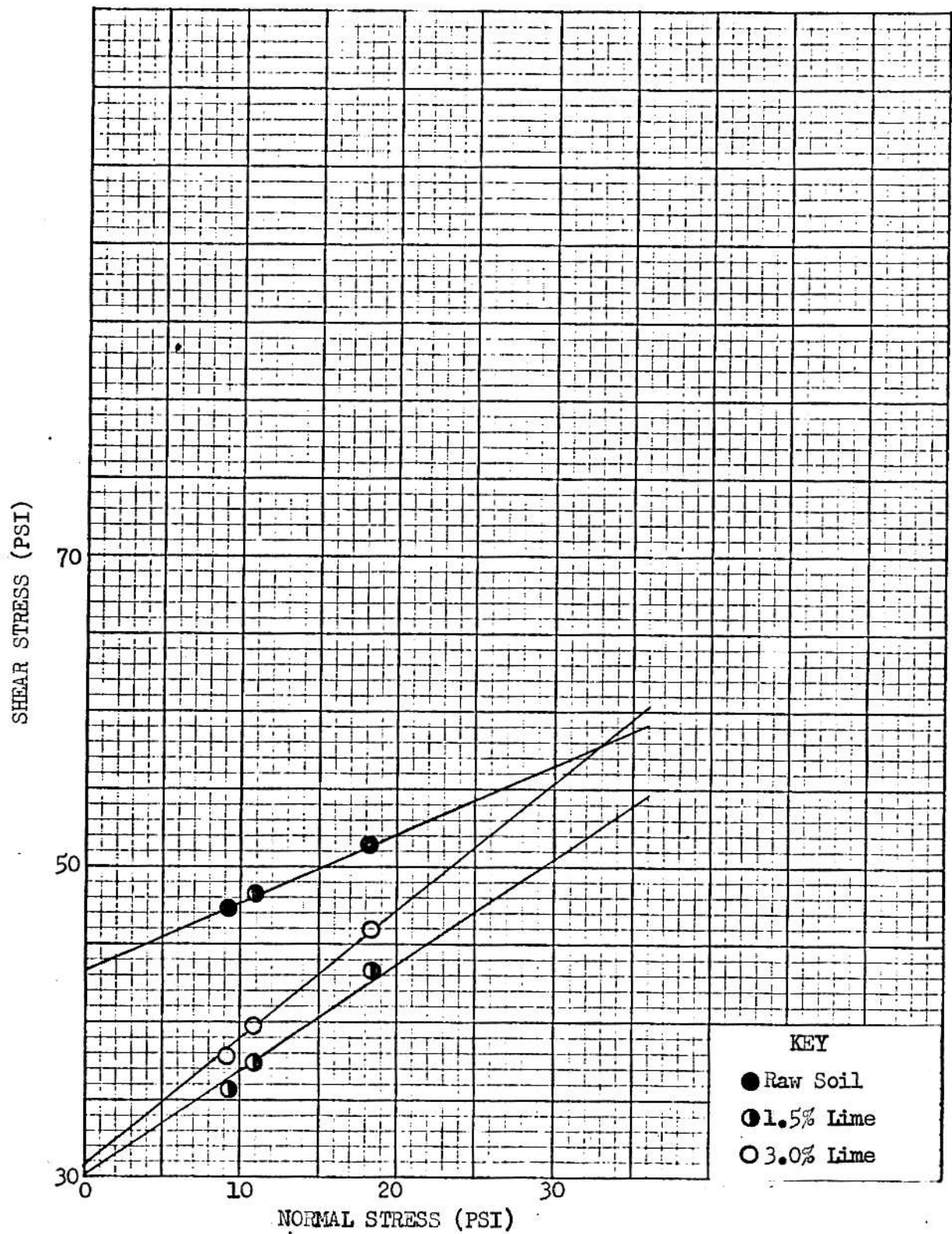


FIGURE 21. Effect of Lime Content on the Shear Strength Parameters of the Omaha Loess.

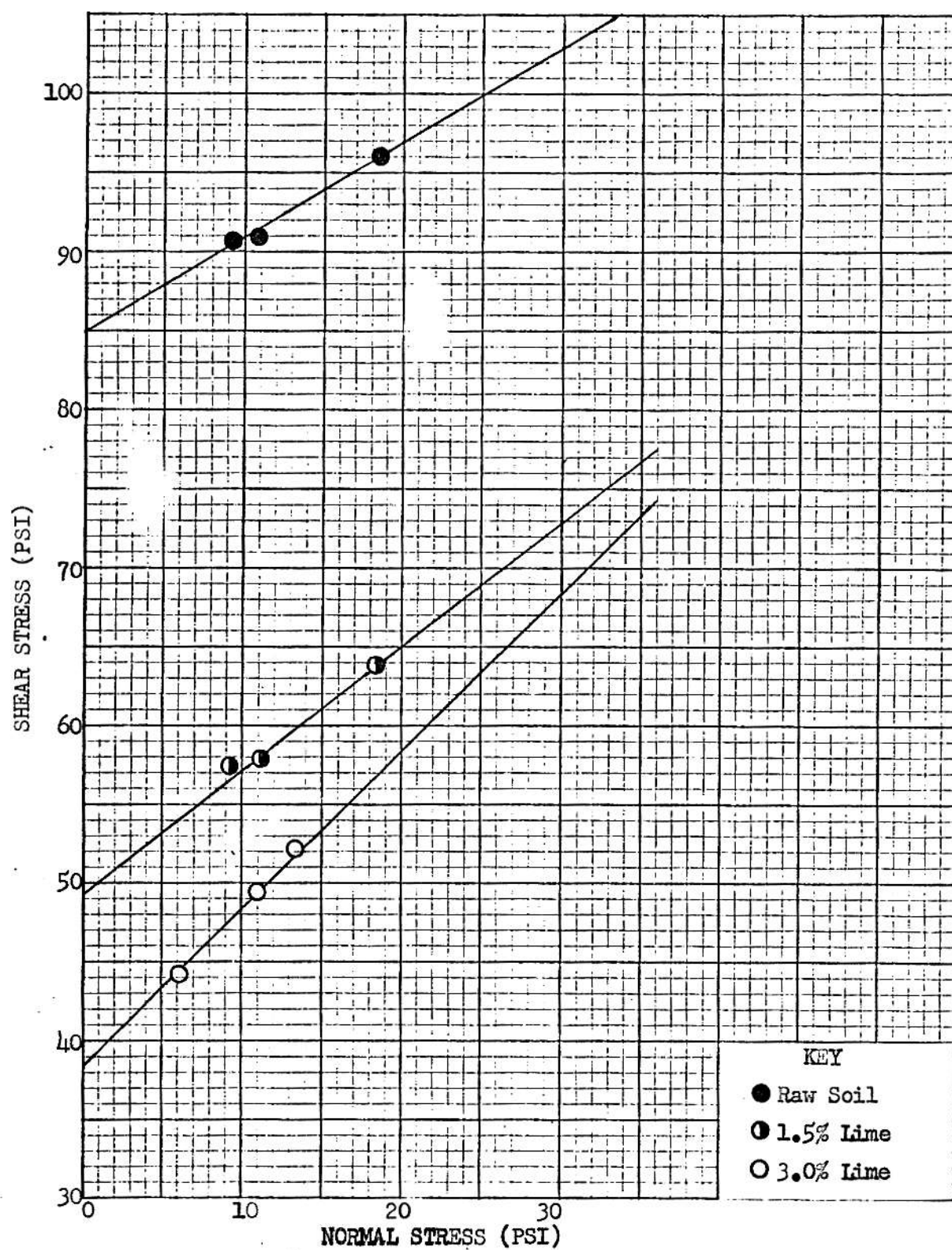


FIGURE 22. Effect of Lime Content on the Shear Strength Parameters of the St. Louis Loess.

RESULTS AND CONCLUSIONS

The results obtained from the laboratory study indicate that hydrated lime reacts with loess in a manner which is similar to its reaction with clay. The plasticity index and standard density were reduced while the optimum moisture content and the plastic limit were increased as shown in Table 1.

Agglomeration of the soil particles occurred with the small lime amounts used and resulted in substantial grain size increases, especially the loesses from the Atwood, Kansas City, and St. Louis areas, as shown by Figures 12-16. The increase in grain size would reduce the possibility of a capillary head and hence reduce the possibility of capillary action occurring in the subgrade.

The grain size increase also increased the angle of internal friction. Under heavy loading the increase would result in a higher shear strength, which was true in all of the locations except St. Louis. The loess from St. Louis had a higher clay content than did any of the other locations. The addition of the hydrated lime greatly reduced the cohesion of the loess which was much larger than the increase derived from the increase of the angle of internal friction. The net result was a substantial reduction in strength, which is not concluded to be critical because the Direct Shear Test was conducted on the samples in an oven-dried state to accelerate reactions and to simulate possible field conditions resulting from

long dry periods. Saturation of the samples would, for the most part, eliminate the cohesion of the loessial soils in both the raw and treated state. The result would be an increase in shear strength with lime treatment when the moisture content approaches saturation. This increase would occur in all of the loessial soils treated.

Location	Soil Condition	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Optimum Moisture (%)	Standard Density (pcf)	Specific Gravity	Angle of Internal Friction (degrees)	Dry Cohesion (psi)
Atwood Kansas	Raw	30.9	23.9	7.0	14.2	109.6	2.59	31.4	38.0
	1.5% Lime	34.5	27.1	7.4	21.5	98.8	2.62	36.3	29.5
	3.0% Lime	35.4	31.4	4.0	21.5	98.8	2.69	43.5	35.0
Clinton Iowa	Raw	26.0	22.7	3.3	14.6	110.8	2.69	35.4	26.6
	1.0% Lime	25.4	22.9	2.5	14.2	106.2	2.78	42.6	12.0
	2.0% Lime	26.0	24.9	1.1	15.2	105.2	2.75	42.0	20.0
Kansas City Kansas	Raw	24.7	18.9	5.8	15.0	110.4	2.63	33.5	57.2
	1.25% Lime	25.6	21.3	4.3	16.5	108.0	2.63	36.5	23.0
	2.5% Lime	28.0	26.0	2.0	16.5	106.5	2.63	40.0	27.4
Omaha Nebraska	Raw	28.9	26.1	2.8	18.6	102.9	2.66	24.0	43.2
	1.5% Lime	32.2	27.3	4.9	17.5	102.5	2.66	34.3	30.0
	3.0% Lime	31.3	28.9	2.4	18.6	101.2	2.65	39.5	30.9
St. Louis Missouri	Raw	31.9	22.8	9.1	15.3	108.0	2.62	31.0	85.0
	1.5% Lime	35.7	25.6	10.1	18.5	104.5	2.65	38.5	49.2
	3.0% Lime	34.2	30.2	4.0	19.0	102.2	2.62	45.0	38.5

TABLE 5. Results Obtained from the Experimental Program

SUGGESTED RESEARCH PROJECTS

The results of the experimental program encourage further research in hydrated lime-loessial soil modification. A test section constructed as a part of a highway project is suggested. Possible additional tests of interest for this test road are as follows:

1. Permeability Test
2. Triaxial Shear Test
3. California Bearing Ratio Test
4. Unconfined Compression Test
5. X-Ray Diffraction Test
6. Infrared Spectrometer Test.

The tests mentioned above should be correlated to the tests conducted in this experimental research.

ACKNOWLEDGEMENTS

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The most sincere recognition is also given to Dr. Jack B. Blackburn, Head of the Civil Engineering Department, for his aid and valuable advice.

Recognition also goes forth to the Materials Department of the State Highway Commission of Kansas, especially C. R. Fricke, Engineer of Tests, and B. K. Merritt, Operations Supervisor of the Soils Laboratory, for the permission to conduct testing with their facilities. Sincere thanks also goes forth to J. C. Freer, Keith Haxton, J. E. Bowman, III, Gordon Wing, Lyle Baker, and Paul Strucker for their technical aid and assistance in the laboratory. Special recognition is also given to L. D. Hamilton, Operations Supervisor of the Physical Test Section, for his guidance and assistance in the compilation of this thesis.

APPENDIX A

Location	Percent Lime																	
	0						1.0						1.5					
	L.L.	P.L.	P.I.	L.L.	P.L.	P.I.	L.L.	P.L.	P.I.	L.L.	P.L.	P.I.	L.L.	P.L.	P.I.	L.L.	P.L.	P.I.
Atwood	30.9	23.9	7.0	29.7	25.2	4.5	30.9	25.9	5.0	34.5	27.1	7.4	34.8	28.6	6.2	35.4	29.5	5.9
Clinton	26.0	22.7	3.3	25.0	22.4	2.6	25.4	22.9	2.5	26.3	23.2	3.1	26.0	24.9	1.1	25.5	25.2	0.3
Kansas City	24.7	18.9	5.8	25.2	21.7	3.5	26.8	22.4	4.4	27.5	23.3	4.2	27.7	25.1	2.6	28.0	26.0	2.0
Omaha	28.9	26.1	2.8	30.0	25.7	4.3	31.0	26.2	4.8	32.2	27.3	4.9	31.4	27.9	3.5	31.0	27.8	3.2
St. Louis	31.9	22.8	9.1	33.2	22.9	10.3	34.2	22.4	11.8	35.7	25.6	10.1	35.0	26.0	9.0	34.8	28.1	6.7
Location	Percent Lime																	
	3.0						4.0						4.5					
	L.L.	P.L.	P.I.	L.L.	P.L.	P.I.	L.L.	P.L.	P.I.	L.L.	P.L.	P.I.	L.L.	P.L.	P.I.	L.L.	P.L.	P.I.
Atwood	35.4	31.4	4.0	36.6	31.5	5.1	35.3	33.8	1.5	36.5	31.7	4.8	35.9	30.6	5.3			
Clinton	25.0	25.3	NP	25.7	25.5	0.2	25.8	26.2	NP	26.7	25.2	1.5	26.4	25.4	1.0			
Kansas City	28.9	26.2	2.7	29.0	26.7	2.3	28.6	27.6	1.0	28.5	27.2	1.3	29.1	28.3	0.8			
Omaha	31.3	28.9	2.4	31.7	28.5	3.2	31.0	27.5	3.5	30.8	28.9	1.9	30.9	28.8	2.1			
St. Louis	34.2	30.2	4.0	35.4	29.4	6.0	35.5	31.1	4.4	34.3	30.6	3.7	34.3	31.0	3.3			

TABLE A-1. Lime Retention Point Determination

Location	Soil Condition	% Retained for Sieve No.					
		10	20	40	60	100	200
Atwood	Raw	0	0	0	0	0.2	3.6
	1.5% L	0	0	0.2	0.2	0.4	5.2
	3.0% L	0	4.3	8.6	10.9	14.5	24.7
Clinton	Raw	0	0	0.2	0.2	0.4	1.0
	1.0% L	0	0.1	0.5	0.8	1.3	2.9
	2.0% L	0	0.3	1.0	1.6	2.1	4.4
Kansas City	Raw	0	0	0.2	0.4	0.8	9.0
	1.25% L	0	0	0.6	1.4	2.6	12.4
	2.5% L	0	1.4	3.8	5.4	7.6	29.0
Omaha	Raw	0	0	0	0	0	0.8
	1.5% L	0	0	0.2	0.4	0.6	1.6
	3.0% L	0	1.6	2.8	3.4	4.0	7.6
St. Louis	Raw	0	0	0	0	0.2	0.4
	1.5% L	0	0	0.2	0.6	3.2	4.6
	3.0% L	0	10.6	15.4	17.8	19.4	21.0

TABLE A-2. Effect of Lime Content on +200 Grain Sized Soil

Location and Soil Condition	Sedimentation Time (min.)	Maximum Grain Size (m.m.)	Actual Grain Size (m.m.)	% in Suspension
Atwood (Raw) Sp. G. = 2.59	0.0	---	---	--
	0.5	.112	.052	81
	1.0	.078	.037	69
	2.0	.055	.028	51
	5.0	.035	.018	32
	15.0	.020	.010	22
	30.0	.014	.0076	18
	60.0	.010	.0054	16
	250.0	.005	.0027	12
	1440.0	.002	.0011	8
Atwood 1.5% Lime Sp. G. = 2.62	0.0	---	---	--
	0.5	.112	.052	83
	1.0	.078	.038	75
	2.0	.055	.028	55
	5.0	.035	.018	27
	15.0	.020	.011	17
	30.0	.014	.0077	17
	60.0	.010	.0054	15
	250.0	.005	.0027	11
	1440.0	.002	.0011	9
Atwood 3.0% Lime Sp. G. = 2.69	0.0	---	---	--
	0.5	.112	---	--
	1.0	.078	.035	50
	2.0	.055	.026	34
	5.0	.035	.018	17
	15.0	.020	.010	8
	30.0	.014	.0076	4
	60.0	.010	.0053	3
	250.0	.005	.0026	2
	1440.0	.002	.0011	1

TABLE A-3. Effect of Lime Content on -200 Grain Sized Atwood Loess

Location and Soil Condition	Sedimentation Time (min.)	Maximum Grain Size (m.m.)	Actual Grain Size (m.m.)	% in Suspension
Clinton (Raw) Sp. G. = 2.69	0.0	---	---	---
	0.5	.112	.049	89
	1.0	.078	.035	81
	2.0	.055	.026	65
	5.0	.035	.017	38
	15.0	.020	.010	18
	30.0	.014	.0075	12
	60.0	.010	.0053	8
	250.0	.005	.0026	8
	1140.0	.002	.0011	6
Clinton 1.0% Lime Sp. G. = 2.78	0.0	---	---	---
	0.5	.112	---	---
	1.0	.078	---	---
	2.0	.055	.024	48
	5.0	.035	.016	25
	15.0	.020	.010	13
	30.0	.014	.0073	7
	60.0	.010	.0052	5
	250.0	.005	.0026	3
	1140.0	.002	.0011	2
Clinton 2.0% Lime Sp. G. = 2.75	0.0	---	---	---
	0.5	.112	---	---
	1.0	.078	---	---
	2.0	.055	.024	47
	5.0	.035	.017	28
	15.0	.020	.010	12
	30.0	.014	.0074	7
	60.0	.010	.0053	4
	250.0	.005	.0026	2
	1140.0	.002	.0011	1

TABLE A-4. Effect of Lime Content on -200 Grain Sized Clinton Loess

Location and Soil Condition	Sedimentation Time (min.)	Maximum Grain Size (m.m.)	Actual Grain Size (m.m.)	% in Suspension
Kansas City (Raw) Sp. G. = 2.63	0.0	---	---	---
	0.5	.112	.052	76
	1.0	.078	.037	68
	2.0	.055	.027	48
	5.0	.035	.018	28
	15.0	.020	.011	20
	30.0	.014	.0075	16
	60.0	.010	.0053	14
	250.0	.005	.0026	14
	1440.0	.002	.0011	10
Kansas City 1.25% Lime Sp. G. = 2.63	0.0	---	---	---
	0.5	.112	.053	71
	1.0	.078	.038	67
	2.0	.055	.028	39
	5.0	.035	.019	13
	15.0	.020	.011	9
	30.0	.014	.0078	7
	60.0	.010	.0055	5
	250.0	.005	.0027	5
	1440.0	.002	.0011	5
Kansas City 2.5% Lime Sp. G. = 2.63	0.0	---	---	---
	0.5	.112	.054	63
	1.0	.078	.039	51
	2.0	.055	.029	31
	5.0	.035	.019	15
	15.0	.020	.011	7
	30.0	.014	.0078	5
	60.0	.010	.0055	5
	250.0	.005	.0027	3
	1440.0	.002	.0011	3

TABLE A-5. Effect of Lime Content on -200 Grain Sized Kansas City Loess

Location and Soil Condition	Sedimentation Time (min.)	Maximum Grain Size (m.m.)	Actual Grain Size (m.m.)	% in Suspension
Omaha (Raw) Sp. G. = 2.66	0.0	---	---	--
	0.5	.112	.049	90
	1.0	.078	.035	82
	2.0	.055	.026	62
	5.0	.035	.018	34
	15.0	.020	.010	20
	30.0	.014	.0074	16
	60.0	.010	.0053	14
	250.0	.005	.0026	12
	1440.0	.002	.0011	8
Omaha 1.5% Lime Sp. G. = 2.66	0.0	---	---	--
	0.5	.112	.051	87
	1.0	.078	.037	83
	2.0	.055	.027	61
	5.0	.035	.018	25
	15.0	.020	.011	13
	30.0	.014	.0077	9
	60.0	.010	.0054	9
	250.0	.005	.0027	7
	1440.0	.002	.0011	5
Omaha 3.0% Lime Sp. G. = 2.65	0.0	---	---	--
	0.5	.112	.051	83
	1.0	.078	.037	77
	2.0	.055	.027	53
	5.0	.035	.018	25
	15.0	.020	.011	11
	30.0	.014	.0078	7
	60.0	.010	.0055	5
	250.0	.005	.0027	3
	1440.0	.002	.0011	3

TABLE A-6. Effect of Lime Content on -200 Grain Sized Omaha Loess

Location and Soil Condition	Sedimentation Time (min.)	Maximum Grain Size (m.m.)	Actual Grain Size (m.m.)	% in Suspension
St. Louis (Raw) Sp. G. = 2.62	0.0	---	---	---
	0.5	.112	.050	90
	1.0	.078	.035	86
	2.0	.055	.026	78
	5.0	.035	.017	60
	15.0	.020	.010	40
	30.0	.014	.0073	30
	60.0	.010	.0053	24
	250.0	.005	.0026	20
	1440.0	.002	.0011	16
St. Louis 1.5% Lime Sp. G. = 2.65	0.0	---	---	---
	0.5	.112	.050	95
	1.0	.078	.036	91
	2.0	.055	.026	77
	5.0	.035	.018	43
	15.0	.020	.011	27
	30.0	.014	.0075	21
	60.0	.010	.0054	17
	250.0	.005	.0027	13
	1440.0	.002	.0011	9
St. Louis 3.0% Lime Sp. G. = 2.62	0.0	---	---	---
	0.5	.112	.053	73
	1.0	.078	.037	71
	2.0	.055	.026	63
	5.0	.035	.018	41
	15.0	.020	.011	21
	30.0	.014	.0077	15
	60.0	.010	.0055	11
	250.0	.005	.0027	7
	1440.0	.002	.0011	5

TABLE A-7. Effect of Lime Content on -200 Grain Sized St. Louis Loess

Location	Point Number	1		2		3		4		5		6	
		Wt. (pcf)	Moist. (%)	Wt. (pcf)	Moist. (%)	Wt. (pcf)	Moist. (%)	Wt. (pcf)	Moist. (%)	Wt. (pcf)	Moist. (%)	Wt. (pcf)	Moist. (%)
Atwood	Raw	100.4	10.7	104.0	12.0	108.4	13.2	109.5	15.4	104.0	18.0	99.3	19.9
	1.5% L	88.0	14.7	92.6	17.2	96.6	19.7	98.9	21.6	93.6	24.5	89.3	28.0
	3.0% L	90.0	12.4	94.0	13.8	95.3	17.3	98.3	19.0	98.9	21.2	94.9	24.0
Clinton	Raw	102.5	11.1	109.0	12.9	110.9	14.8	109.5	16.4	105.8	17.5	100.0	20.4
	1.0% L	101.6	12.5	106.3	14.3	105.3	16.9	100.6	19.0	96.9	21.6	95.1	23.1
	2.0% L	98.1	11.7	101.3	13.0	105.3	15.1	104.6	17.3	100.3	19.2	96.6	21.3
Kansas City	Raw	102.2	7.5	105.4	10.2	109.5	13.7	109.5	16.2	104.3	18.7	99.4	22.4
	1.25% L	98.3	12.3	104.6	14.2	108.0	16.5	102.0	19.1	97.6	22.2	93.6	25.1
	2.50% L	96.9	12.1	102.6	14.5	106.6	16.4	102.9	18.6	98.0	21.2	95.3	24.0
Omaha	Raw	95.2	10.5	98.8	13.6	101.3	16.8	102.8	18.7	97.3	22.4	92.7	26.2
	1.5% L	94.6	13.8	98.6	16.3	102.6	17.3	97.6	21.1	93.3	23.5	90.3	26.8
	3.0% L	94.6	14.6	98.6	16.2	101.3	18.5	96.3	21.8	92.3	24.6	90.6	27.0
St. Louis	Raw	104.0	13.5	108.1	15.3	107.2	16.9	103.0	19.1	99.6	21.8	97.2	23.8
	1.5% L	94.6	13.0	99.6	14.5	103.6	17.1	103.6	19.8	98.3	22.5	93.3	25.9
	3.0% L	92.9	12.7	96.3	14.8	99.3	16.9	102.3	19.0	98.0	22.0	93.3	25.5

TABLE A-8. Effect of Lime Content on Moisture-Density Relationships

Location	Soil Condition	Std. Density (psf)	grams mold	Opt. Moist (%)	c.c. H ₂ O sample
Atwood	Raw	109.6	141.2	14.2	20.1
	1.5% L	98.8	127.3	21.5	27.4
	3.0% L	98.8	127.3	21.5	27.4
Clinton	Raw	110.8	142.7	14.6	20.8
	1.0% L	106.2	136.8	14.2	19.4
	2.0% L	105.2	135.5	15.2	20.6
Kansas City	Raw	110.4	142.2	15.0	21.3
	1.25% L	108.0	139.1	16.5	23.0
	2.5% L	106.5	137.2	16.5	22.6
Omaha	Raw	102.9	132.5	18.6	24.6
	1.5% L	102.5	132.0	17.5	23.1
	3.0% L	101.2	130.3	18.6	24.2
St. Louis	Raw	108.0	139.1	15.3	21.3
	1.5% L	104.5	134.6	18.5	24.9
	3.0% L	102.2	131.6	19.0	25.0

TABLE A-9. Direct Shear Molding Data

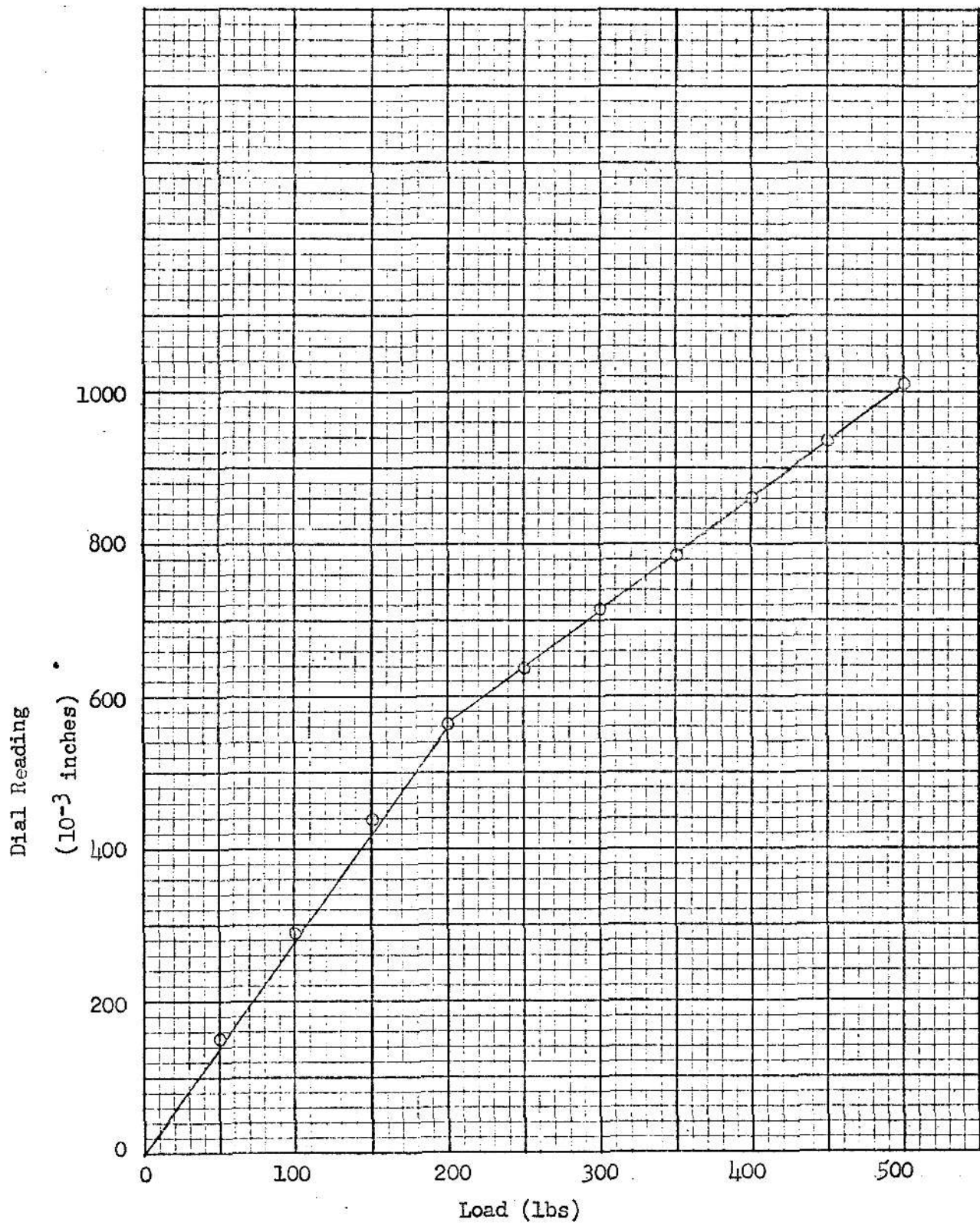


Figure A-1. Direct Shear Proving Ring Calibration Curve

Calibration

Direct Shear Proving Ring

1/21/71
S. E. Swartz
E. G. Zerr

<u>Load</u>	<u>Dial Reading</u>
0	0
50	146
100	289
150	437
200	566
250	638
300	711
350	786
400	861
450	935
500	1010

Factor 0-200 0.3540 Lb./Div.
Factor 200-500 0.6780 Lb./Div.

TABLE A-10. Proving Ring Calibration Data

DIRECT SHEAR TEST DATA

Sample No. Atwood - RawDate 1-27-71

Original Proving Ring Reading = $\frac{22}{10^{-3}} \times 10^{-3} \text{ in. lbs}$
 Proving Ring Calibration Factor = $\frac{0.3540}{10^{-3}} \times 10^{-3} \text{ in.}$
 Cross-sectional Area = $\frac{\pi D^2}{4} = 0.7854 \text{ D}^2 = 4.91 \text{ in.}^2$

Normal Load (psi)	Deflection Reading (10 ⁻³ in.)	Proving Ring Reading (10 ⁻³ in.)	Correct. Ring Reading (10 ⁻³ in.)	Shear	
				lbs.	p.s.i.
3.79	10	34	12	4.25	0.87
"	20	36	14	4.60	0.94
"	30	42	20	7.08	1.44
"	40	330	308	109.03	22.21
"	50	550	528	186.91	38.07
"	54	580	558	197.53	40.23

10.95	10	33	11	3.89	0.79
"	20	82	60	21.24	4.33
"	30	95	73	25.84	5.26
"	40	111	189	66.91	13.63
"	50	332	310	109.74	22.35
"	60	592	570	203.39	41.42
"	62	616	594	219.66	44.74

18.12	10	80	58	20.53	4.18
"	20	140	118	41.77	8.51
"	30	203	181	64.07	13.05
"	40	215	193	68.32	13.91
"	50	651	629	243.39	49.57
"	60	676	654	260.34	53.02

TABLE A-11. Direct Shear Test Data

DIRECT SHEAR TEST DATA

Sample No. Atwood - 1.5 LDate 2-1-71

Original Proving Ring Reading = 22 x 10⁻³ in. lbs
 Proving Ring Calibration Factor = 0.3540 10⁻³ in. 2
 Cross-sectional Area = $\frac{\pi D^2}{4} = 0.7854 D^2 =$ 4.91 in. 2

Normal Load (psi)	Deflection Reading (10 ⁻³ in.)	Proving Ring Reading (10 ⁻³ in.)	Correct. Ring Reading (10 ⁻³ in.)	Shear	
				lbs.	p.s.i.
3.79	10	41	19	6.73	1.37
"	20	82	60	21.24	4.33
"	30	253	231	81.77	16.65
"	36	468	446	157.88	32.16

10.95	10	64	42	14.87	3.03
"	20	76	54	19.12	3.89
"	30	87	65	23.01	4.69
"	40	95	73	25.84	5.26
"	50	290	268	94.87	19.32
"	59	590	568	202.03	41.15

18.12	10	118	96	33.98	6.92
"	20	192	170	60.18	12.26
"	30	214	192	67.97	13.84
"	40	218	196	69.38	14.13
"	50	448	426	150.80	30.71
"	60	593	571	204.07	41.56
"	70	601	579	209.49	42.67

TABLE A-12. Direct Shear Test Data

DIRECT SHEAR TEST DATA

Sample No. Atwood - 3.0 LDate 2-1-71 and 2-15-71

Original Proving Ring Reading = $\frac{22 \times 10^{-3} \text{ in. lbs}}{0.3540} = 6.21 \times 10^{-3} \text{ in. lbs}$
 Proving Ring Calibration Factor = $\frac{10^{-3} \text{ in.}}{0.3540} = 2.82 \times 10^{-3} \text{ in.}$
 Cross-sectional Area = $\frac{\pi D^2}{4} = 0.7854 D^2 = 4.91 \text{ in.}^2$

Normal Load (psi)	Deflection Reading (10 ⁻³ in.)	Proving Ring Reading (10 ⁻³ in.)	Correct. Ring Reading (10 ⁻³ in.)	Shear	
				lbs.	p.s.i.
3.79	10	44	22	7.79	1.59
"	20	55	33	11.68	2.38
"	30	61	39	13.81	2.81
"	40	116	124	43.90	8.94
"	50	555	533	188.68	38.43

10.95	10	74	52	18.41	3.75
"	20	118	96	33.98	6.92
"	30	162	140	49.56	10.09
"	40	363	341	120.71	24.59
"	50	590	568	202.03	41.15
"	59	628	606	227.80	46.40

13.19	10	75	53	18.76	3.82
"	20	97	75	26.55	5.41
"	30	153	131	46.37	9.44
"	40	407	385	136.29	27.76
"	50	607	585	213.56	43.50
"	54	629	607	228.48	46.53

TABLE A-13. Direct Shear Test Data

DIRECT SHEAR TEST DATA

Sample No. Clinton - RawDate 1-27-71

Original Proving Ring Reading = $\frac{22}{10^{-3}} \times 10^{-3} \text{ in. lbs}$
 Proving Ring Calibration Factor = $\frac{0.3540}{10^{-3} \text{ in.}^2}$
 Cross-sectional Area = $\frac{\pi D^2}{4} = 0.7854 D^2 = \frac{4.91}{10^{-3} \text{ in.}^2}$

Normal Load (psi)	Deflection Reading (10 ⁻³ in.)	Proving Ring Reading (10 ⁻³ in.)	Correct. Ring Reading (10 ⁻³ in.)	Shear	
				lbs.	p.s.i.
3.79	10	27	5	1.77	0.36
"	20	67	45	15.93	3.24
"	30	436	414	146.56	29.85

10.95	10	61	39	13.81	2.81
"	20	70	48	16.99	3.46
"	30	193	171	60.53	12.33
"	38	488	466	164.96	33.60

18.12	10	31	9	3.19	0.65
"	20	117	95	33.63	6.85
"	30	294	272	96.29	19.61
"	40	516	494	174.88	35.62
"	50	575	553	195.76	39.87

TABLE A-14. Direct Shear Test Data

DIRECT SHEAR TEST DATA

Sample No. Clinton - 1.0 LDate 2-1-71Original Proving Ring Reading = 22 x 10⁻³ in. lbsProving Ring Calibration Factor = 0.3540 10⁻³ in. 2Cross-sectional Area = $\frac{\pi D^2}{4} = 0.7854 D^2 =$ 4.91 in. 2

Normal Load (psi)	Deflection Reading (10 ⁻³ in.)	Proving Ring Reading (10 ⁻³ in.)	Correct. Ring Reading (10 ⁻³ in.)	Shear	
				lbs.	p.s.i.
3.79	10	43	21	7.43	1.51
"	20	47	25	8.85	1.80
"	30	139	117	41.42	8.44
"	40	230	208	73.63	15.00

10.95	10	39	17	6.02	1.23
"	20	258	236	83.54	17.01
"	28	332	310	109.74	22.35

18.12	10	123	101	35.75	7.28
"	20	190	168	59.47	12.11
"	30	204	182	64.43	13.12
"	40	380	358	126.73	25.81
"	51	490	468	165.67	33.74

TABLE A-15. Direct Shear Test Data

DIRECT SHEAR TEST DATA

Sample No. Clinton - 2.0 L Date 2-1-71

Original Proving Ring Reading = 22 x 10^{-3} in. lbs

Proving Ring Calibration Factor = 0.3540 10^{-3} in. 2

Cross-sectional Area = $\frac{\pi D^2}{4} = 0.7854 D^2 =$ 4.91 in. 2

Normal Load (psi)	Deflection Reading (10^{-3} in.)	Proving Ring Reading (10^{-3} in.)	Correct. Ring Reading (10^{-3} in.)	Shear	
				lbs.	p.s.i.
3.79	10	41	19	6.73	1.37
"	20	203	181	64.07	13.05
"	26	354	332	117.53	23.94

10.95	10	61	39	13.81	2.81
"	20	73	51	18.05	3.68
"	30	130	108	38.23	7.79
"	40	378	356	126.02	25.67
"	46	423	401	141.95	28.91

18.12	10	95	73	25.84	5.26
"	20	300	278	98.41	20.04
"	30	515	493	174.52	35.54
"	34	535	513	181.60	36.99

TABLE A-16. Direct Shear Test Data

DIRECT SHEAR TEST DATA

Sample No. Kansas City - RawDate 2-4-71Original Proving Ring Reading = 22 x 10^{-3} in. lbsProving Ring Calibration Factor = 0.3540 10^{-3} in. 2Cross-sectional Area = $\frac{\pi D^2}{4} = 0.7854 D^2 =$ 4.91 in. 2

Normal Load (psi)	Deflection Reading (10^{-3} in.)	Proving Ring Reading (10^{-3} in.)	Correct. Ring Reading (10^{-3} in.)	Shear	
				lbs.	p.s.i.
9.15	10	60	38	13.45	2.74
"	20	80	58	20.53	4.18
"	30	247	225	79.65	16.22
"	33	756	734	314.58	64.07

10.95	10	87	65	23.01	4.69
"	20	90	68	24.07	4.90
"	30	200	178	63.01	12.83
"	37	753	731	312.55	63.66

18.12	10	128	106	37.52	7.64
"	20	139	117	41.42	8.44
"	30	157	135	47.79	9.73
"	40	680	658	263.05	53.58
"	43	794	772	340.35	69.32

TABLE A-17. Direct Shear Test Data

DIRECT SHEAR TEST DATA

Sample No. Kansas City - 1.25 L

Date 2-2-71

Original Proving Ring Reading = 22 x 10⁻³ in. lbs

Proving Ring Calibration Factor = 0.3540 10⁻³ in. 2

Cross-sectional Area = $\frac{\pi D^2}{4} = 0.7854 D^2 =$ 4.91 in. 2

Normal Load (psi)	Deflection Reading (10 ⁻³ in.)	Proving Ring Reading (10 ⁻³ in.)	Correct. Ring Reading (10 ⁻³ in.)	Shear	
				lbs.	p.s.i.
9.15	10	53	31	10.97	2.23
"	20	70	48	16.99	3.46
"	30	81	59	20.89	4.25
"	40	95	73	25.84	5.26
"	50	330	308	109.03	22.21
"	60	426	404	143.02	29.13
"	65	432	410	145.14	29.56

10.95	10	86	64	22.66	4.59
"	20	122	100	35.40	7.21
"	30	432	410	145.14	29.56
"	43	457	435	153.99	31.36

18.12	10	116	94	33.28	6.78
"	20	130	108	38.23	7.79
"	30	168	146	51.68	10.53
"	40	390	368	130.27	26.53
"	50	530	508	179.83	36.63

TABLE A-18. Direct Shear Test Data

DIRECT SHEAR TEST DATA

Sample No. Kansas City - 2.5 LDate 2-2-71

Original Proving Ring Reading = 22 x 10^{-3} in. lbs
 Proving Ring Calibration Factor = 0.3540 10^{-3} in.
 Cross-sectional Area = $\frac{\pi D^2}{4} = 0.7854 D^2 =$ 4.91 in.²

Normal Load (psi)	Deflection Reading (10^{-3} in.)	Proving Ring Reading (10^{-3} in.)	Correct. Ring Reading (10^{-3} in.)	Shear	
				lbs.	p.s.i.
9.15	10	74	52	18.41	3.75
"	20	92	70	24.78	5.05
"	30	120	98	34.69	7.07
"	40	310	288	101.95	20.76
"	50	502	480	169.92	34.61
"	55	512	490	173.46	35.33

10.95	10	53	31	10.97	2.23
"	20	69	47	16.64	3.39
"	30	85	63	22.30	4.54
"	40	365	343	121.42	24.73
"	50	522	500	177.00	36.05
"	51	526	504	178.42	36.34

18.12	10	84	62	21.95	4.47
"	20	148	126	44.60	9.08
"	30	148	126	44.60	9.08
"	40	400	378	133.81	27.25
"	50	602	580	210.17	42.81
"	58	634	612	231.87	47.22

TABLE A-19. Direct Shear Test Data

DIRECT SHEAR TEST DATA

Sample No. Omaha - RawDate 2-4-71

Original Proving Ring Reading = $\frac{22 \times 10^{-3} \text{ in. lbs}}{0.3540} = 62.175 \text{ in. lbs}$
 Proving Ring Calibration Factor = $\frac{10^{-3} \text{ in.}}{0.3540} = 2.825 \times 10^{-3} \text{ in.}$
 Cross-sectional Area = $\frac{\pi D^2}{4} = 0.7854 D^2 = 4.91 \text{ in.}^2$

Normal Load (psi)	Deflection Reading (10 ⁻³ in.)	Proving Ring Reading (10 ⁻³ in.)	Correct. Ring Reading (10 ⁻³ in.)	Shear	
				lbs.	p.s.i.
9.15	10	43	21	7.43	1.51
"	20	80	58	20.53	4.18
"	30	155	133	47.08	9.59
"	39	633	611	231.19	47.09

10.95	10	81	59	20.89	4.25
"	20	108	86	30.44	6.20
"	30	125	103	36.46	7.43
"	40	160	138	48.85	9.95
"	51	642	620	237.29	48.34

18.12	10	70	48	16.99	3.46
"	20	106	84	29.74	6.06
"	30	154	132	46.73	9.52
"	40	500	478	169.21	34.46
"	47	665	643	252.85	51.50

TABLE A-20. Direct Shear Test Data

DIRECT SHEAR TEST DATA

Sample No. Omaha - 1.5 LDate 2-2-71

Original Proving Ring Reading = 22 $\times 10^{-3}$ in. lbs
 Proving Ring Calibration Factor = 0.3540 10^{-3} in.²
 Cross-sectional Area = $\frac{\pi D^2}{4} = 0.7854 D^2 =$ 4.91 in.²

Normal Load (psi)	Deflection Reading (10^{-3} in.)	Proving Ring Reading (10^{-3} in.)	Correct. Ring Reading (10^{-3} in.)	Shear	
				lbs.	p.s.i.
9.15	10	75	53	18.76	3.82
"	20	112	90	31.86	6.49
"	30	392	370	130.98	26.68
"	40	515	493	174.52	35.54

10.95	10	65	43	15.22	3.10
"	20	85	63	22.30	4.54
"	30	100	78	27.61	5.62
"	40	492	470	166.38	33.89
"	47	542	520	184.08	37.49

18.12	10	61	39	13.81	2.81
"	20	140	118	41.77	8.51
"	30	308	286	101.24	20.62
"	40	578	556	196.82	40.09
"	50	607	585	213.56	43.50

TABLE A-21. Direct Shear Test Data

DIRECT SHEAR TEST DATA

Sample No. Omaha - 3.0 LDate 2-2-71

Original Proving Ring Reading = 22 x 10^{-3} in. lbs
 Proving Ring Calibration Factor = 0.3540 10^{-3} in. 2
 Cross-sectional Area = $\frac{\pi D^2}{4} = 0.7854 D^2 =$ 4.91 in. 2

Normal Load (psi)	Deflection Reading (10^{-3} in.)	Proving Ring Reading (10^{-3} in.)	Correct. Ring Reading (10^{-3} in.)	Shear	
				lbs.	p.s.i.
9.15	10	55	33	11.68	2.38
"	20	70	48	16.99	3.46
"	30	102	80	28.32	5.77
"	40	280	258	91.33	18.60
"	50	546	524	185.50	37.78

10.95	10	60	38	13.45	2.74
"	20	85	63	22.30	4.54
"	30	119	97	31.34	6.99
"	40	576	554	196.12	39.94

18.12	10	83	61	21.59	4.40
"	20	145	123	43.54	8.87
"	30	316	294	104.08	21.20
"	40	573	551	195.05	39.73
"	48	635	603	225.76	45.98

TABLE A-22. Direct Shear Test Data

DIRECT SHEAR TEST DATA

Sample No. St. Louis - Raw Date 2-4-71

Original Proving Ring Reading = $\frac{22}{10^{-3}} \times 10^{-3} \text{ in. lbs}$

Proving Ring Calibration Factor = $\frac{0.3540}{10^{-3} \text{ in.}^2}$

Cross-sectional Area = $\frac{\pi D^2}{4} = 0.7854 D^2 = \frac{4.91}{\text{in.}^2}$

Normal Load (psi)	Deflection Reading (10^{-3} in.)	Proving Ring Reading (10^{-3} in.)	Correct. Ring Reading (10^{-3} in.)	Shear	
				lbs.	p.s.i.
9.15	10	35	13	4.60	0.94
"	20	90	68	24.07	4.90
"	30	101	79	27.97	5.70
"	40	107	85	30.09	6.13
"	50	252	230	81.42	16.58
"	56	948	926	444.76	90.58

10.95	10	80	58	20.53	4.18
"	20	84	62	21.95	4.47
"	30	118	96	33.98	6.92
"	40	682	660	265.52	54.08
"	53	952	930	447.47	91.14

18.12	10	83	61	21.59	4.40
"	20	134	112	39.65	8.08
"	30	152	130	46.02	9.37
"	40	500	478	169.21	34.46
"	50	1038	1016	505.78	103.01

TABLE A-23. Direct Shear Test Data

DIRECT SHEAR TEST DATA

Sample No. St. Louis - 1.5 LDate 2-3-71

Original Proving Ring Reading = 22 x 10^{-3} in. lbs
 Proving Ring Calibration Factor = 0.3540 10^{-3} in. 2
 Cross-sectional Area = $\frac{\pi D^2}{4} = 0.7854 D^2 =$ 4.91 in. 2

Normal Load (psi)	Deflection Reading (10^{-3} in.)	Proving Ring Reading (10^{-3} in.)	Correct. Ring Reading (10^{-3} in.)	Shear	
				lbs.	p.s.i.
9.15	10	64	42	14.87	3.03
"	20	70	48	16.99	3.46
"	30	95	73	25.84	5.26
"	40	255	233	82.48	16.80
"	50	660	638	249.49	50.81
"	52	708	686	282.04	57.44

10.95	10	73	51	18.05	3.68
"	20	133	111	39.29	8.00
"	30	136	114	40.36	8.22
"	40	144	122	43.19	8.80
"	50	670	648	256.27	52.19
"	53	712	690	284.75	57.99

18.12	10	68	46	16.28	3.32
"	20	131	109	38.59	7.86
"	30	135	113	40.00	8.15
"	40	135	113	40.00	8.15
"	50	650	628	242.71	49.43
"	57	755	733	313.90	63.93

TABLE A-24. Direct Shear Test Data

DIRECT SHEAR TEST DATA

Sample No. St. Louis - 3.0 L Date 2-15-71

Original Proving Ring Reading = $\frac{22 \times 10^{-3} \text{ in. lbs}}{0.3540} = 6.24 \times 10^{-3} \text{ in.}^2$

Proving Ring Calibration Factor = $\frac{10^{-3} \text{ in.}^2}{0.3540} = 2.82 \times 10^{-3} \text{ in.}^2$

Cross-sectional Area = $\frac{\pi D^2}{4} = 0.7854 D^2 = 4.91 \text{ in.}^2$

Normal Load (psi)	Deflection Reading (10^{-3} in.)	Proving Ring Reading (10^{-3} in.)	Corrected Ring Reading (10^{-3} in.)	Shear	
				lbs.	p.s.i.
6.03	10	54	32	11.33	2.31
"	20	59	37	13.10	2.67
"	30	61	39	13.81	2.81
"	40	415	393	139.12	28.33
"	50	612	590	216.95	44.19

10.95	10	74	52	18.41	3.75
"	20	106	84	29.74	6.06
"	30	118	126	44.60	9.08
"	40	248	226	80.00	16.29
"	50	603	581	210.85	42.94
"	55	652	630	244.07	49.71

13.19	10	87	65	23.01	4.69
"	20	138	116	41.06	8.36
"	30	167	145	51.33	10.45
"	40	171	149	52.75	10.74
"	50	532	510	180.54	36.77
"	60	672	650	257.63	52.47

TABLE A-25. Direct Shear Test Data

APPENDIX B

ADDITIONAL LITERATURE REVIEW

Exhaustive investigations requiring much time and effort have greatly contributed to the popularity and importance of lime in the stabilization field today. These investigations, beginning approximately twenty-five years ago, have continued to the present.

History

In a broad sense, the use of lime stabilization dates back many years. One of the first known cases is the Shensi pyramids in the Tibetan-Mongolian Plateau, which were constructed over 5,000 years ago.

China and India have also used lime stabilization for many years to aid in construction of dams, underground chambers, and massive bridge footings. Records over 2,000 years old show the Romans' use of lime in subbases of roads. Although the methods were rather primitive, these and other countries have continued the stabilizing practices through the present (17,18,19).

The use of lime as a stabilizing agent has only gained prominence in this country since World War II. The Texas Highway Department is credited with being the pioneer in this field. The first dozen years met with little success, but as more knowledge was obtained through laboratory and field testing, benefits of lime stabilization began to be realized.

The lack of success in the earlier years can be attributed to improper controls of mixing, compacting, and curing in field construction. It is believed that disappointing results from some of the earlier Texas jobs delayed the development of lime stabilization in that state for a period of ten years. Later test results caused a tremendous increase in the use of lime as a stabilizing agent in this state and many other states soon followed the lead.

Because lime stabilization is still a relatively young field, the long range benefits or detrimental effects are still to be fully recognized. Satisfactory performances have been observed from several projects that were constructed 12 to 15 years ago. Since this time approximates the life of many pavement designs today, experts feel that lime stabilization has a very promising future.

Eades, Nichols, and Grim (16) go even further in stating, "If portland cement concrete is considered permanent and it depends on hydrated calcium silicates for its strength, then with proof that the same type of minerals are formed, it must follow that lime stabilization is permanent." Further study may prove these investigators correct.

Limes

Lime types are categorized as function of their parent material. Calcitic limes are derived from a high calcium limestone, dolomitic lime from dolomitic limestone, and rejuvenated lime as a by-product of different manufacturing processes. Rejuvenated lime cannot be accurately analyzed in the general sense because of the many different types. Figure B-1 shows the properties of the high calcitic and dolomitic limes (20).

(a) Quicklime			
<u>Chemical Composition</u>	<u>High Calcium, %</u>	<u>Dolomitic, %</u>	
CaO	92.25 - 98.00	55.50 - 57.50	
MgO	0.30 - 2.50	37.60 - 40.80	
CO ₂	0.40 - 1.50	0.40 - 1.50	
SiO ₂	0.20 - 1.50	0.10 - 1.50	
Fe ₂ O ₃	0.10 - 0.40	0.05 - 0.40	
Al ₂ O ₃	0.10 - 0.50	0.05 - 0.50	
H ₂ O	0.10 - 0.90	0.10 - 0.90	
Specific gravity	3.2 - 3.4	3.2 - 3.4	
Specific heat at 100 F, Btu per lb	0.19	0.21	
Bulk density (pebble lime), pcf	55 - 60	55 - 60	
(b) Hydrates			
	<u>High Calcium</u>	<u>Monohydrated Dolomitic</u>	<u>Dihydrated Dolomitic</u>
Principal chemical composition	Ca(OH) ₂	Ca(OH) ₂ + MgO	Ca(OH) ₂ + Mg(OH) ₂
Specific gravity	2.3 - 2.4	2.7 - 2.9	2.4 - 2.6
Specific heat at 100 F, Btu per lb	0.29	0.29	0.29
Bulk density, pcf	25 - 35	25 - 35	30 - 40

from Highway Research Board Bulletin 304, pg. 102

Figure B-1. Properties of commercial limes

The term "lime" is used very loosely. It not only refers to the oxide of calcium, but to its many other associated chemical derivatives. In the past, this has been confusing to many engineers.

Quicklime

Lime is produced by calcination or "limeburning" of limestone. In the case of calcitic lime, the carbonates of calcium are replaced with oxides.



The production of dolomitic lime is very similar to that of calcitic lime except that the magnesium is present in sufficient quantity to be of importance.



The oxide state produced in both cases is termed quicklime or unslaked lime. In this state, it is very reactive with moisture and carbon dioxide. Reaction with carbon dioxide results in a non-reactive powder. For this reason, precautions must be taken in shipping and handling to reduce exposure of quicklime to moisture and air.

Hydrated or Slaked Lime

As mentioned previously, quicklime is very reactive with moisture. Occurrence of this reaction results in a state known as hydrated or slaked lime. The calcitic lime reaction results in calcium hydroxide.



The dolomitic reaction is more involved.

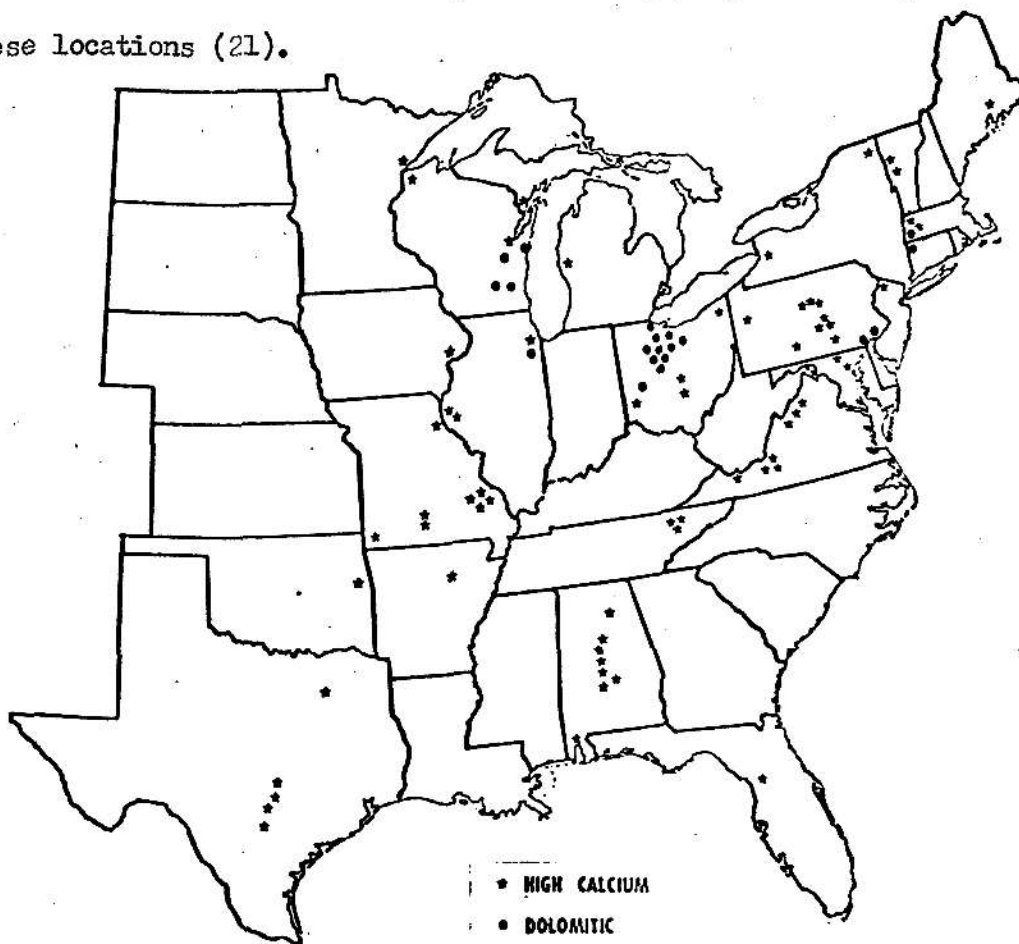


The end result of this reaction is termed monohydrated dolomitic lime. Addition of pressure to this end result gives dihydrated dolomitic lime, $\text{Ca}(\text{OH})_2 + \text{Mg}(\text{OH})_2$. Very little information could be found concerning the dihydrated dolomitic lime as its use in stabilization is limited due to the manufacturing costs.

Rejuvenated lime usually takes the form of hydrated or slaked lime. Lack of uniformity in quality is the major criticism of this type of lime (18,20).

Sources and Sizes Available

Lime is produced throughout most of the United States although calcitic limestone deposits are located mainly in the eastern and midwestern states. Dolomitic limestone deposits are found in the midwest and New England States. Figure B-2 depicts these locations (21).



From National Lime Association Lime, pg. 513

Figure B-2. Limestone sources of the Eastern and Midwestern United States

Quicklime is available in either bulk form or paper sacks. In bulk form, the following sizes are generally available:

- Lump lime - maximum size of eight inches in diameter.
- Crushed or pebble lime - sizes from 2 1/2 to 1/4 inches.
- Ground lime - all sizes passing a No. 8 sieve and 40-60 per cent passing a No. 100 sieve.
- Pulverized lime - all sizes passing a No. 20 sieve and 85-95 per cent passing a No. 100 sieve.

The two finest sizes are the result of grinding larger sized material. In special cases, lime can be ground much finer than shown with as much as 99.5 per cent passing the No. 325 sieve.

Hydrated lime is usually shipped in 50-pound bags but can also be shipped in bulk. Bulk hydrated lime generally has 95 per cent or more passing the No. 200 sieve although it can be ground as fine as quicklime if the application requires it be (18).

Quicklime does display some economical advantages over hydrated lime since it contains about 25 per cent more CaO and MgO than does hydrated lime and costs are reduced by 15-20 per cent. The resulting saving is on the order of 60 per cent. The saving, however, may be offset by the cost of storage and handling because of the extra precautionary measures required for quicklime.

Soil-Lime Reactions

Several chemical reactions occur in soil-lime mixtures, e.g., flocculation and cation exchange, pozzolanic reaction, and carbonation. Although the pozzolanic process has been investigated for some time, little is known regarding what actually occurs (20,22).

Flocculation and Cation Exchange

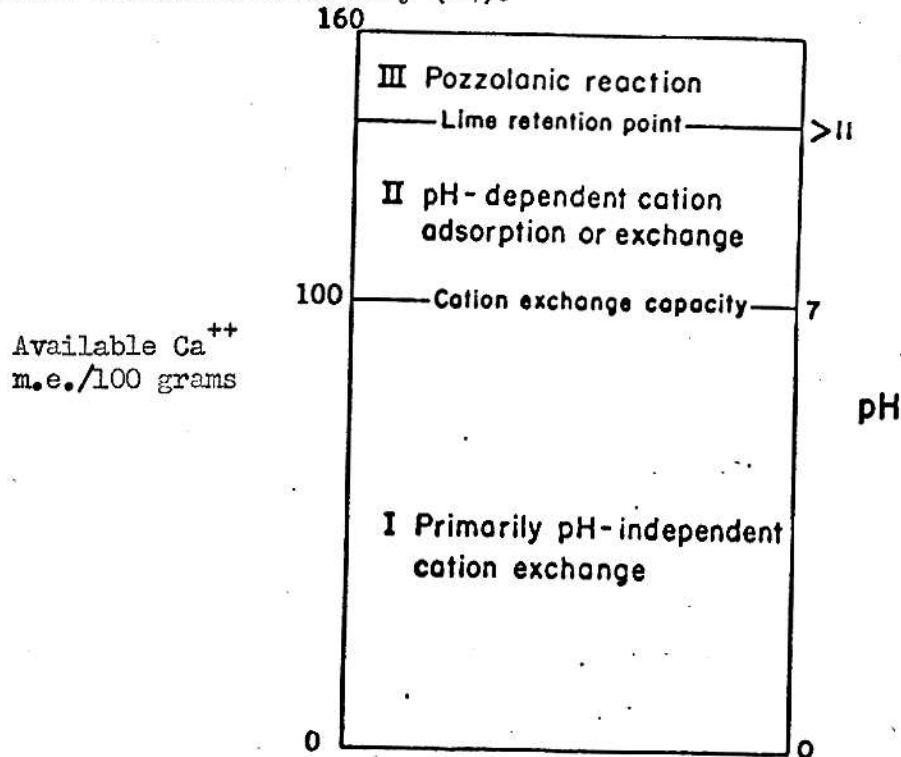
Addition of lime to plastic soil causes the soil to become friable and silt-like in texture. This is the result of either of two reactions or a combination of both. The first is the cation exchange in which the calcium ions from the lime replace the weaker metallic ions, such as sodium and hydrogen, on the surface of the soil particles (14). The other process is the crowding of additional calcium cations from the lime onto the soil particles.

Both of these reactions change the electrical charge of the soil particles causing an increase in the bond between any two soil particles. The result is the flocculation and friable condition of the soil. Flocculation occurs in the minus two hundred (-200) material. The result of this process is an increase in the grain size of the soil with a resulting increase in the angle of internal friction. The increase in the angle of internal friction will, potentially, increase the shear strength of the soil.

The number of calcium ions which a soil will assimilate is dependent not only upon the type of soil, but upon the pH of the soil as well. Irrespective of the quantity of lime used, the soil will only accept a certain number of calcium ions. Maximum flocculation results in the maximum plastic limit increase which is usually equal to the maximum plasticity index reduction (15). Lime-soil mixtures have been investigated regarding the percentage of lime required for this effect. This lime percentage is termed the lime retention point (14,23).

In a case of an Iowan montmorillonite clay, completion of the ion exchange occurred at a pH of 10.5 to 11.5 (14,24).

Figure B-3 depicts the relationship of cation exchange and pH in a montmorillonite clay (14).



From Highway Research Board Record 29, pg. 67

Figure B-3. Relationship of pH and cation exchange

Only after this retention is reached can pozzolanic reaction proceed.

Pozzolanic Reaction

The cementing action ensuing when lime and soil are mixed is termed pozzolanic reaction. This is the "unexplained" because the reaction which occurs is not fully understood. It is believed that the aluminous and siliceous minerals of the soil react with lime to produce a gel of calcium silicates and

calcium aluminates. The gel tends to cement the soil particles together in a manner similar to the hydration of portland cement, thus new minerals are formed. The strength-gaining process is very slow in most cases and varies with the type of soil. The action may be hastened considerably with the addition of other chemicals although the cementing requires much more time than that of concrete (25,26).

The reaction is believed to occur only after a critical pH of the soil has been reached thus indicating the importance of the amount of lime added. If the addition is less than that of the retention point, very little or no pozzolanic reaction can occur.

Carbonation

When exposed to air, lime will react to form calcium carbonate. The reaction is a reversal of the lime-producing process. Calcium carbonate is a very weak cementing agent and hinders both the pozzolanic reaction and the rate of strength gain.

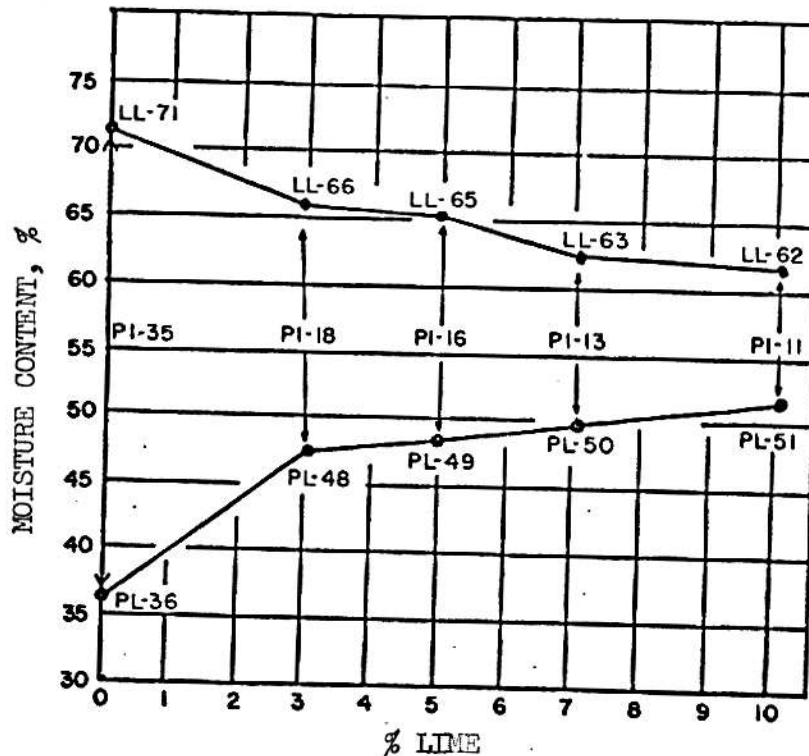
Effects of Soil-Lime Mixtures

The addition of lime to plastic soils has various effects on the physical characteristics of the soils. The degree of the effects vary with the amount of lime added and the type of minerals present in the soil.

Plasticity Index

Given a soil characterized by a relatively high plastic index, 15 or higher (27), the addition of lime will cause a

substantial reduction in the plastic index in almost all cases. This is generally the result of an increase in plastic limit and a decrease in the liquid limit. The typical effect is illustrated by Figure B-4 (15).



From Highway Research Board Record 29, pg. 3

Figure B-4. Lime's effect on plastic properties

It is not uncommon, however, for the liquid limit to increase (28). In either case, the change in the liquid limit is slight. The plastic limit change is the critical change, for it is a much larger deviation than that of the liquid limit. The importance of this change is demonstrated by its use as the basis of the retention point theory.

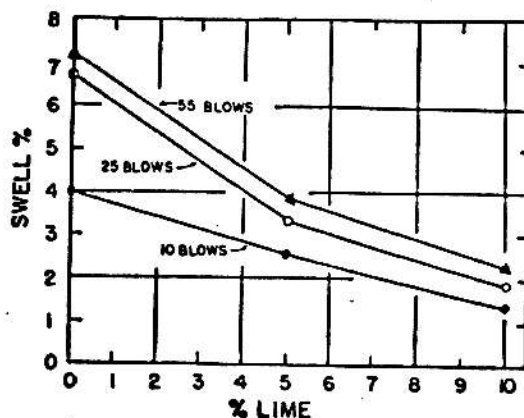
The lowering of the plasticity index is an almost immediate reaction (29). It generally occurs within the first 48

hours after lime has been added to the soil. There are some cases in which soils have become non-plastic after just 48 hours (30). Such a reaction is excellent where a soil is used as a mineral filler or a binder. To be used satisfactorily in bituminous mixes, a binder must be friable and easily pulverized to insure proper coating of the soil particles. The lime-soil reaction permits this condition to exist.

The reduction of plasticity index by addition of lime to soil has been utilized in many ways. Addition of a small percentage of lime has enabled construction to proceed at a much earlier time in the wet spring season than would have been otherwise possible (31). This is typical of many jobs where lime is used not as a long-term stabilizing agent, but as an aide to construction on a short-term basis. The "semi-stabilization" or soil modification results solely from P.I. reduction as the quantity of lime added is not sufficient to cause an appreciable strength gain.

Swell and Shrinkage

A marked decrease in swell and shrinkage usually occurs with lime-soil mixtures, as illustrated in Figure B-5 (15).



From Highway Research Board Record 29, pg. 11

Figure B-5. Effect of lime on swell

This is thought to be mainly the result of an increase in the shrinkage limit and a decrease in shrinkage ratio and is particularly true in the case of expansive soils (32).

The change in swell and shrinkage potential seems to be related to the change in plasticity index. As the plasticity index drops with increasing lime content, the shrinkage limit increases. As the change in P.I. becomes less, the increase in shrinkage limit becomes less (20).

Adsorption failure time is the time required for over-dried soil briquettes to fall apart or slack down when partially immersed in water. Investigations performed by Nebraska showed that with addition of just three per cent lime the time was increased from a few minutes to over two hours. Indications are that the addition of lime increases the resistance of a plastic soil to slaking. The result is a capillarity change which appears to be affected by the decrease in both the shrinkage ratio and plasticity index (33) causing the soil to become less adsorptive.

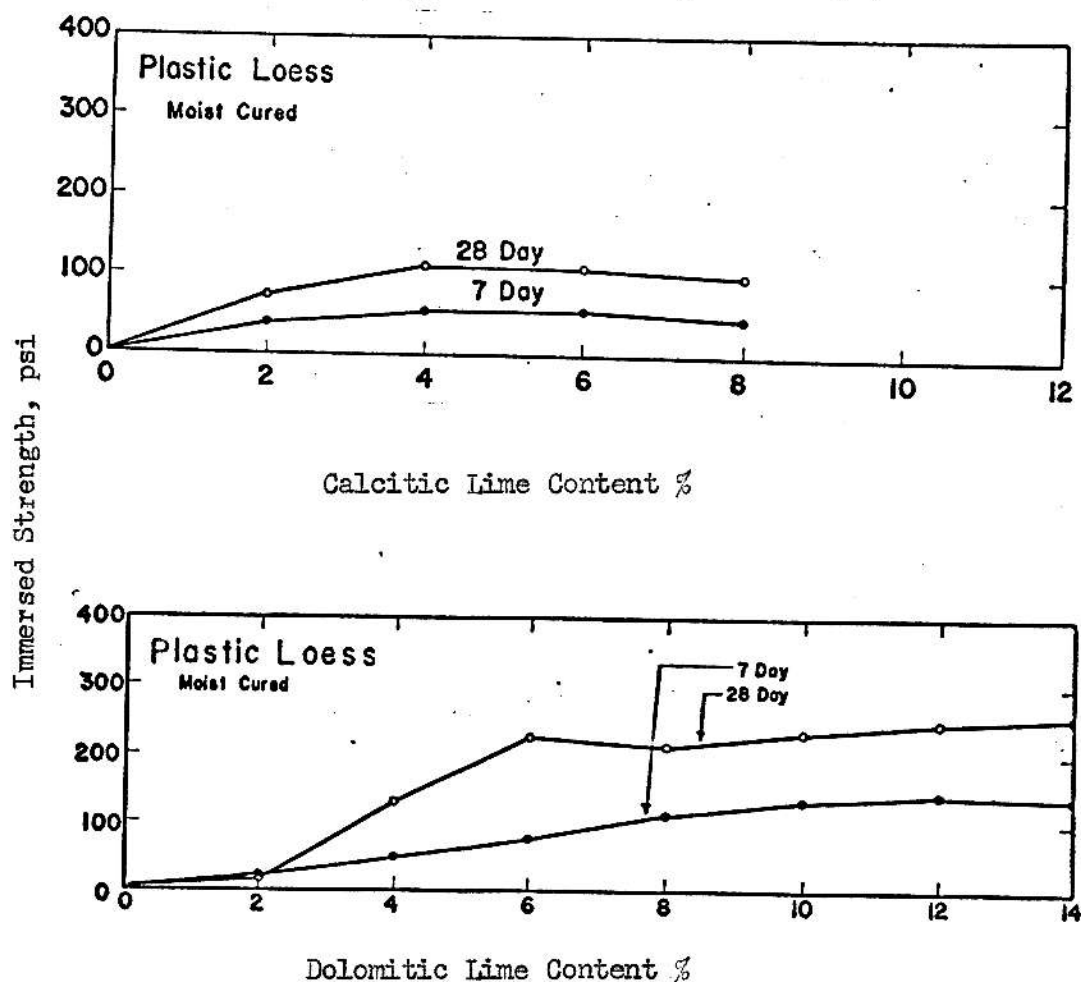
Strength

The strength of a plastic soil is generally increased with the addition of lime (34,35). The amount of strength gain is dependent upon many factors, e.g., per cent of lime added, type of soil, temperature, and thickness of treatment to list a few. Since strength gain is the result of pozzolanic reaction, temperature plays a very important role, as the strength gain rate varies as a function of temperature. Very little or no gain can be attained if the lime content is below the retention point.

The type of lime used is also a critical factor.

Comparisons of hydrated limes showed higher strength gains for dolomitic monohydrated limes than for the calcitic limes (36,37,38).

The comparison held true, especially where a lime content of three per cent or more is used (10), as shown in Figure B-6 (38).



From Highway Research Board Bulletin 262, pg. 38-39

Figure B-6. Comparison of lime types to compressive strength gains

Strength gain comparisons of quicklime, hydrated lime, and re-juvenated lime indicated some interesting variations. Quicklime developed the most strength and was thought to be a direct result

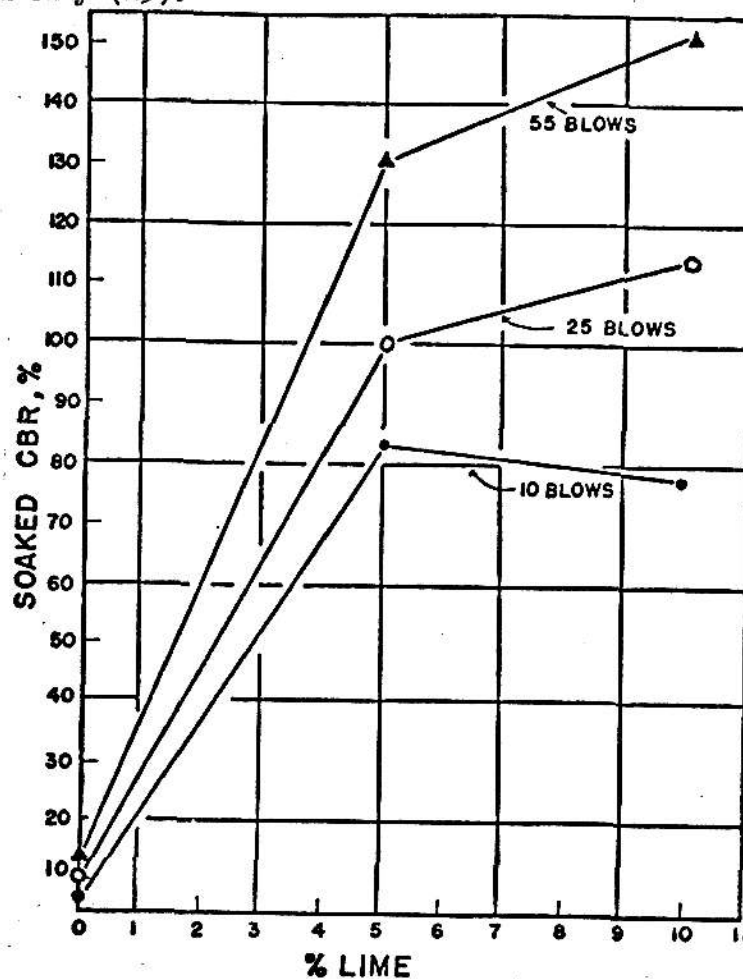
of heat generated by its reaction with water during the process (39,40). The temperature increase permitted the pozzolanic reaction to accelerate at a higher rate than either the hydrated or rejuvenated limes.

Of the three lime types, the hydrated lime gave the second highest strength gain and rejuvenated lime the lowest strength. In fact, the quantity of rejuvenated lime required to attain strengths similar to the other two lime types was doubled. Investigations have pointed out that the rejuvenated lime can be used satisfactorily in a base or subbase of a road carrying very light traffic (40,41). Although this type of lime is least effective of all the limes, its use is still very promising where a low lime content is required such as in soil modification. The use of rejuvenated lime could be very economical because it is a waste product, therefore, its cost might offset its relative inefficiency.

Studies have been conducted to compare powder lime application and slurry lime application. The findings showed that quicklime in a slurry form gave the best results. Hydrated lime was equally effective in either the slurry or powder form. Results also showed that the dolomitic limes provided higher strengths than did the calcitic limes (42).

The relationship existing between compaction and strength has been the source of numerous studies. Some investigators (43) have found that strengths were greatly increased by increased compaction. Overcompaction hindered strength gains in soils with

short curing periods, but did not harm soils with longer curing periods. The conclusion drawn was that overcompaction shear planes of the soil-lime mixture tended to heal through longer curing times. Figure B-7 shows the effect of compaction on a lime-treated clay (15).



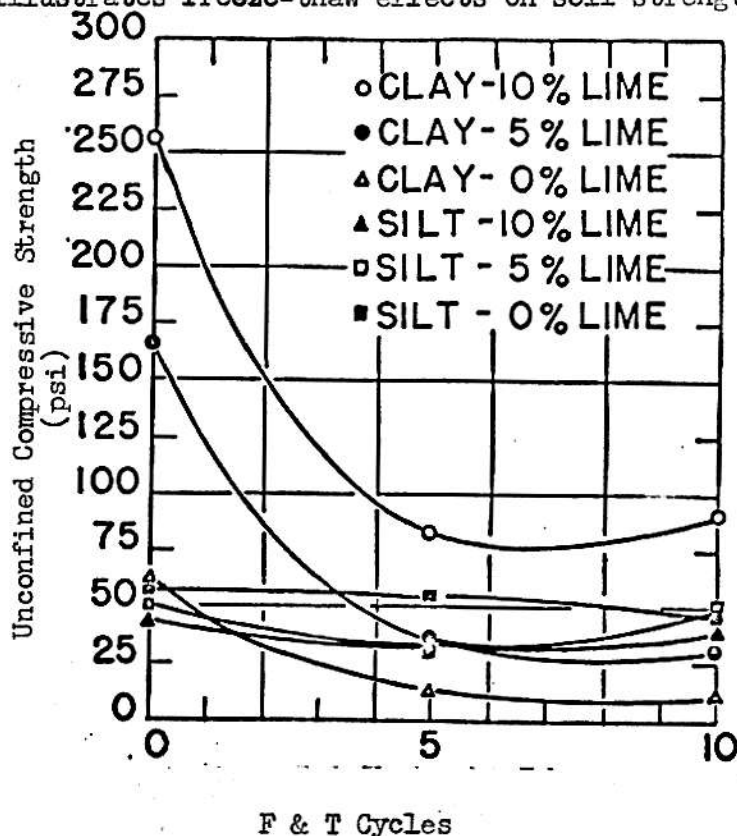
From Highway Research Board Record 29, pg. 11

Figure B-7. Compaction characteristics of a lime-treated clay

One of the most adverse effects on soil strength is the action of freezing and thawing, which investigators (33,44, 45,46) have found to occur most markedly during the first five

cycles. Resistance to this loss is substantially increased with the addition of lime (34). From cycles five through ten, the loss noticeably was decreased. The severity of this effect is not based solely upon the type of soil, but the type of lime used and elapsed time between mixing and compaction as well.

Figure B-8 illustrates freeze-thaw effects on soil strength (33).



From Highway Research Board Record 92, pg. 6

Figure B-8. Freeze-thaw effects on lime-treated soils

Frost effect studies have been conducted varying compactive efforts and types of lime on both kaolinite and montmorillonite minerals (45). With standard compactive efforts, kaolinite being the predominant clay mineral present, addition of lime of any type resulted in only a slight increase in frost.

resistance. When compaction was delayed, the increase in frost resistance was much higher.

Where montmorillonite was predominant, and standard compactive efforts were used, addition of high calcitic lime improved resistance to frost action more than did the dolomitic lime. Here, any delay in compaction proved to be unsatisfactory.

Investigations (33), comparing freeze-thaw effects of lime-treated clay soils and lime-treated silt soils, have shown that strength losses incurred were much greater in the lime-clay mixtures than those in the lime-silt mixtures. Even with the relatively large losses incurred by the lime-clay mixtures their final strength values still were substantially larger than the lime-silt mixtures due to the higher initial strengths of the lime-clay mixture.

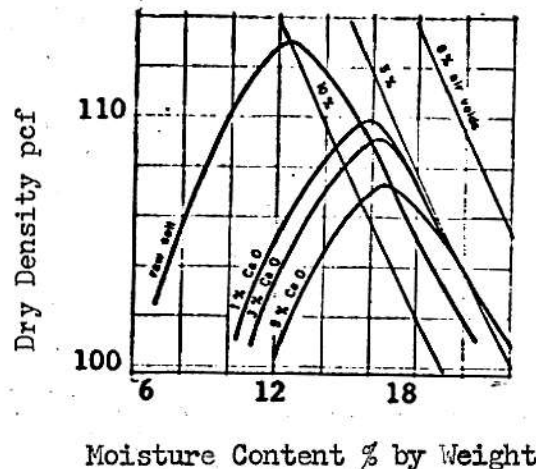
Using ten per cent high calcitic lime, one source (44) showed that after five freeze-thaw cycles the lime-soil mixture evidenced a strength approximately equal to that of the raw soil which was not subjected to any freeze-thaw cycles. Utilizing the California Bearing Ratio Test, a comparison of lime-treated samples soaked in water for five days and raw soil that was not subjected to moisture was made. The test results indicate a decrease in permeability is obtained by lime treatment of the soil.

Dry Density and Optimum Moisture Content

Addition of lime to plastic soils has a definite effect on the dry density and optimum moisture content of the soil. For a given compactive effort, the dry density is usually decreased.

and shows the optimum moisture increased. The changes are the result of flocculation of the soil particles.

In the usual case, the percentage of increase in optimum moisture content is somewhat higher than the percentage of decrease of dry density. Figure B-9 shows a typical compaction curve (47).



From Highway Research Board Bulletin 231, pg. 21

Figure B-9. Effect of lime on moisture-density relationships

Reactive Soil Minerals for Lime Stabilization

Among the soil minerals that have plagued highway builders for many years are montmorillonite, kaolinite, and illite all of which can be improved through lime treatment.

Montmorillonite

Montmorillonite reacts most satisfactorily of all minerals when mixed with lime. Even a small addition will cause a marked increase in the plastic limit. Although montmorillonite is a very reactive pozzolanic agent, opinions vary as to the lime retention percentage. Some investigators (48) have found that

2-3 per cent lime is required while other sources (49) contend that 4-6 per cent is required. The reaction of lime on a mineral is dependent upon the structure of the mineral. In the case of montmorillonite, a three layer clay mineral, pozzolanic reaction can proceed only after the space between the layers are saturated with Ca^+ .

Kaolinite

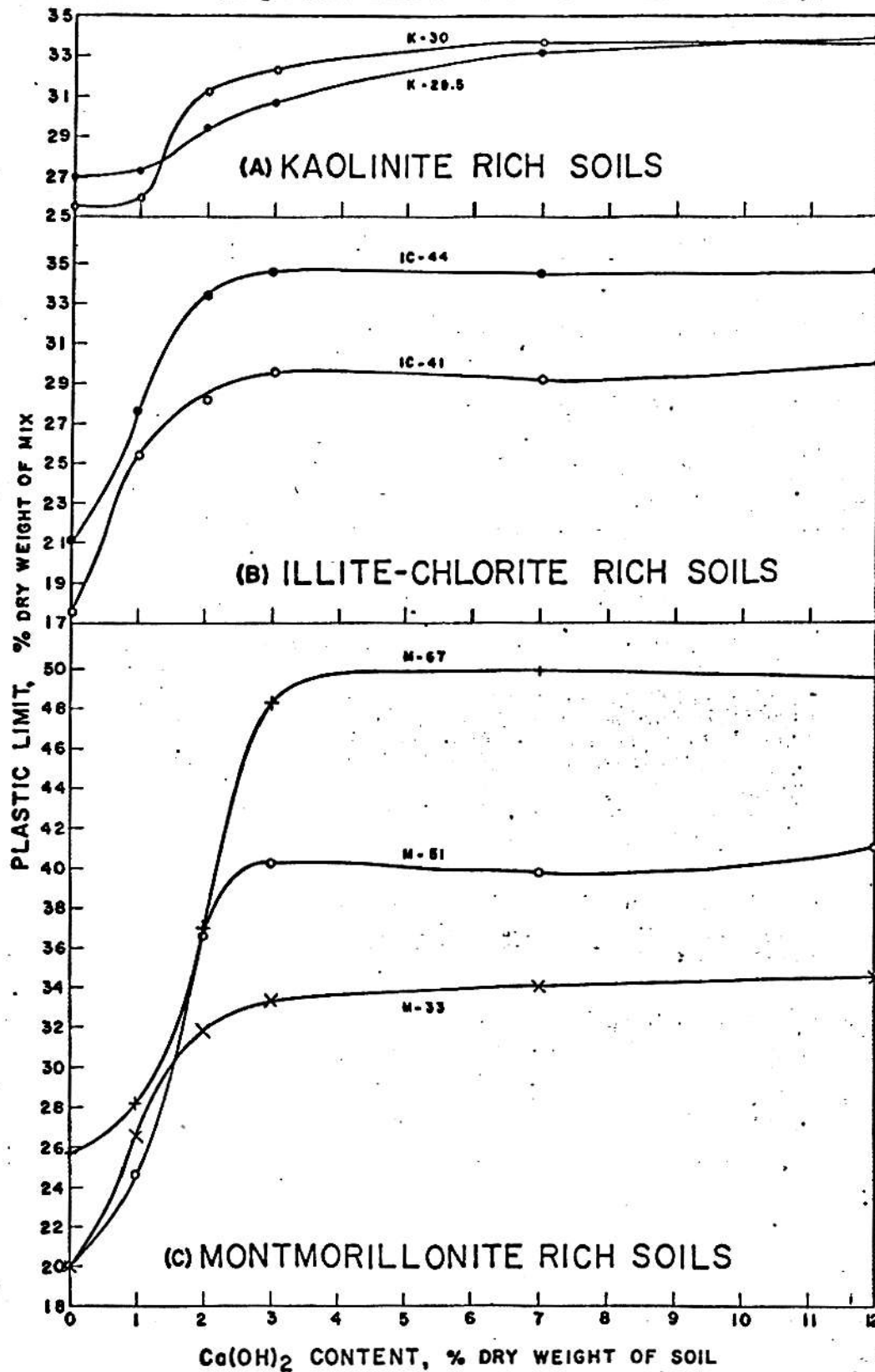
Kaolinite is an effective pozzolanic reagent which differs from the other two minerals in that pozzolanic reaction begins with the addition of even the slightest amount of lime. Another difference is the observed delay in the plastic limit increase. Here again, the structure of the mineral is the explanation as kaolinite is a double layered mineral. Unlike the other minerals, pozzolanic reaction is not retarded until the spacing is saturated with Ca^+ , but begins when the lime attacks the outer edges of the particle. The result is a delay in plastic limit increase because the major portion of the Ca^+ is being used for the pozzolanic reaction (49).

Investigations have shown that the optimum lime content for the maximum increase in plastic limit for both kaolinite and montmorillonite soils is also the lime retention capacity (48).

Illite

Illite is the third of the reactive minerals considered herein. It also is a three layer mineral having characteristics similar to those of montmorillonite and exhibits the most retarded pozzolanic reaction of the three minerals. Figure B-10 shows the

effect of lime on the plastic limits of the three minerals (48).



From Highway Research Board Bulletin 262, pg. 26

Figure B-10. Effect of lime and mineralogy on plasticity properties

Effective Factors on Reactions

Several factors in lime-soil mixtures are important to stabilization, they are: lime content, temperature, time, moisture, and treatment thickness. No one factor can be said to be of greater importance in producing proper stabilization.

Lime Percentage

The importance of the correct lime percentage in a lime-soil mixture cannot be overstressed. The percentage required depends upon the results that are desired. If swell control and plasticity index reduction are desired, the lime content should not exceed that of the lime retention point. That amount of lime will be adequate for soil modification.

If strength increase due to pozzolanic reaction is required, the lime content should be greater than that of the lime retention point. The amount of lime required depends upon the strength desired, providing the strength is less than the maximum attainable.

Temperature

Temperature also plays a critical role in lime stabilization. Although many investigations have been conducted, no optimum stabilization temperature has been determined for the general case, and it is unlikely that one can be determined because of the variables encountered in lime types and soil mineralogy.

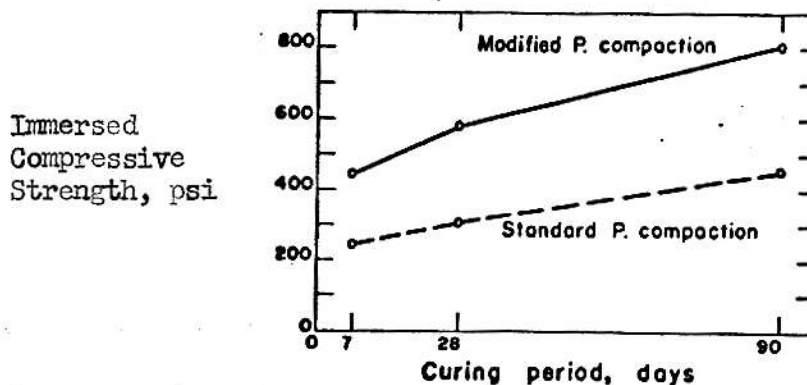
Investigations, correlating field and laboratory curing temperatures showed that comparisons could be made. The

studies showed that a two-day laboratory cure at 120°F. obtained approximately the same strength as 40-45 day old field-cured samples at summer temperatures.

The rate of pozzolanic reaction is highly dependent upon temperature although opinions vary on the minimum temperature required for pozzolanic reaction to proceed. Some investigators contend that little or no pozzolanic reaction occurs at temperature of less than 50°F. (50) while others give a minimum value of 40°F. (51,52).

Time

Time is a factor which is critical to all phases of stabilization, especially during the curing phases (53). Care should be taken to allow proper time during the initial curing or "rotting" period because flocculation and plasticity index reduction occur during this phase. Strength gain is highly dependent upon the time allotted to the final cure (54). The importance of time in curing is shown by Figure B-11 (12).



From Highway Research Board Record 29, pg. 35

Figure B-11. Time-strength curve developed for an alluvial clay

Heavy traffic should be limited on stabilized roadways to allow time for sufficient strength gains (55). Time is also critical in the mixing and compaction phases to insure that each is performed completely and accurately (56).

Moisture

The importance of moisture content throughout the stabilization process cannot be overemphasized. It is most critical in the mixing and compaction phases but is also essential in the curing phases. Not only is the moisture necessary for chemical reactions but moisture movement through the soil may serve as a vehicle for lime diffusion (24).

Treatment Thickness

The treatment thickness in stabilization is dictated by whether the lime-soil mixture is to be a base, subbase, or subgrade. The National Lime Association recommends that a thickness of six inches be the minimum for a base course (17). No strict rule for subbase or subgrade thicknesses has been set although customarily, six to ten inches is used.

Recently, investigations (57) have been conducted pertaining to deep in-situ soil stabilization by high-pressure injection of a lime-slurry. Data compiled one year after treatment, indicated an increase in shear strength although all other benefits usually derived from lime treatment were lacking. Since the lime movement was in the vertical direction only, incomplete mixing of the lime and soil resulted in the lack of the usual benefits. Stabilization of this type may require more time for

proper lime migration to occur. The use of deep in-situ slurry injections in high embankments has possibilities which would warrant further study.

Success has been reported in Oklahoma and Illinois where subbase soil was stabilized to a depth of twenty-four inches (58). The process utilized a large plow which was used in both the scarification and mixing processes.

Construction Procedures

The sequence of steps performed in lime stabilization is nearly standard, variation occurs only in the manner in which the steps themselves are performed. According to the National Lime Association (52), absence of the initial mixing and initial curing phases differentiate base treatment from subbase and subgrade treatment. Hence, base treatment has five phases while subbase and subgrade treatment are comprised of seven phases.

Once the roadway has been brought to the proper line and grade, the treatment phase may begin. The treatment procedure follows the recommendations of the National Lime Association (17,52).

Scarification

The soil in the roadway should be scarified to the depth of desired stabilization. Scarification utilizes the use of disc plows and grader scarifiers. Care should be exercised to insure that all deleterious and foreign material, i.e., rocks, stumps, tree limbs, etc., are removed. The soil should then be

partially pulverized so that no particle size exceeds three inches in diameter. Soil pulverized to this size insures proper lime migration.

Lime Application

The method of lime application used varies from project to project with bag application being the most popular method for smaller jobs. Sacked lime, usually fifty-pound bags, is placed at specified intervals along the roadway, emptied, and the lime spread evenly throughout the area. Bag lime application is the most expensive because of the labor costs involved, but it eliminates the necessity of large capital outlay needed for heavy equipment required by the other methods.

Dry bulk application is another method and is the fastest method of application because it utilizes self-unloading trucks. Loss of lime during application restricts the use of the dry bulk method to rural areas. Nonuniformity of application is also a problem when applying lime with the dry bulk method. Safety dictates that both the dry bulk method and the bag method be restricted to the application of hydrated lime.

Lime may be placed on roadways in slurry form although this method has its limitations. Economically, slurry application is limited to jobs where a lime requirement of four per cent or less is indicated thereby causing the rate of application to be appreciably less than that of the dry bulk method. Slurry application is undesirable in soil where the moisture content is near optimum because of the extra water added from

the slurry itself.

Although the cost of the batching plants cause many contractors to be less than receptive to the slurry method, it does have the advantage of safely utilizing both hydrated lime and quicklime (52). Slurry application is preferable in residential areas because it allows dust problems to be held to a minimum.

Preliminary Mixing

Preliminary mixing, a practice used only in the subbase and subgrade stabilization processes, utilizes disc harrows and grader scarifiers. To allow for maximum flocculation and cation exchange, the moisture content during this process should be at least optimum plus five per cent.

Initial Cure

Subbase and subgrade stabilization process includes an initial cure or "rotting" phase, named so because of the odor resulting from the lime-soil reaction. The lime-soil mixture is usually placed in windrows and allowed to "mellow" or age. Although the recommended time allotted for this process is from 0-48 hours the high side of the range is preferred (59). Longer cure is advisable if a very difficult soil is to be treated. Care should be taken to maintain a moisture content of optimum plus five per cent throughout the process so that maximum chemical action can occur. A satisfactory initial cure will result in a soil that is friable and easily pulverized due to the improvement of the plastic properties.

Final Mixing

To insure maximum lime migration and pozzolanic reaction, rigid construction controls must be maintained during the final mixing process. Utilizing high speed rotary mixers, mixing should be continued until all of the soil is pulverized sufficiently to pass the one-inch screen with at least sixty per cent passing the No. 4 sieve (52,60). Optimum moisture content of lime-soil mixture should be maintained throughout the process. It cannot be overemphasized that proper mixing cannot be satisfactorily accomplished by the use of a grader.

Compaction

Compaction is accomplished through the use of sheep's-foot, pneumatic, and vibratory rollers. With the AASHO T-99 compactive effort as a standard, base material should be compacted to a minimum of 98 per cent of the maximum density while subbase and subgrade should have a minimum of 95 per cent. The elapsed time between the final mixing phase and the compaction phase should be kept to a minimum (12,52,60,61).

Final Curing

The last phase of lime stabilization is the final cure which is usually three to seven days in length but can be extended if necessary. There are two methods of final curing:

- (a) **Moist Cure**, consists of maintaining an excess of free moisture on the treated surface.
- (b) **Membrane Cure**, utilization of an impermeable bituminous seal to preclude moisture loss in the lime-soil mixture.

No heavy equipment or traffic, with the exception of rollers, should be allowed on the compacted soil during this time.

Construction Limitations

1. Satisfactory lime stabilization can occur only during the warmer months of the year. The recommended period is the time period between the spring thaw and mid-September (17) although the limits could vary, depending upon the type of lime used. Dolomitic monohydrated limes react more readily at cooler temperatures than do the other lime types (58) thereby offering the possibility of its use during cooler periods.

2. Once "mellowing" of the lime-soil mixture has started, reworking the mixture is not advisable without additional lime (17,52). Caution should be exercised to avoid application of excess lime because of the potentially detrimental effects to soil strength (26,39).

Testing Methods

Various methods of testing are employed to determine benefits derived from lime stabilization. Some tests are basic to soils engineering and used by all investigators while other tests are oriented toward more specific information. Most tests are standard insofar as they are taken from or are variations of ASTM or AASHTO procedures.

X-Ray Diffraction

The use of x-ray diffraction is a valuable tool in documenting the changes resulting from the addition of lime to

soil (62). Not only can the predominant minerals of the raw soil be determined (15) but the identification and progress of the various reactions of the lime-soil mixtures may be observed (49). With the knowledge of the minerals present in the soil, a satisfactory prediction can be made as to the possibility of stabilization with lime.

Plasticity Tests

Atterburg Limits and the swell test constitute the tests conducted in the determination of the plastic properties of a soil.

Liquid limit, plastic limit, shrinkage limit, and plasticity index are the soil properties determined by Atterburg Limits tests. The Atterburg Limits can be used in prediction of swell, flocculation capabilities, and percentage of lime required to stabilize a soil. Combined with the pH test, Atterburg Limits are essential in the determination of the lime retention point.

Of all the tests used in control of lime stabilization in the United States, the swell test procedure has been subject to the most variation among investigators.

Although the swell test does not indicate plastic properties as do the Atterburg Limits, it produces a good approximation of the swell and shrinkage potential of a soil. The swell test has been used by some investigators as a basis for lime content in soil modification.

pH Test

While the pH test is a measure of the acidity or alkalinity of a substance, its primary use in lime stabilization and lime modification is the determination of the lime retention point. Once the pH of the raw soil has been determined, the amount of lime required to raise the pH to the lime retention point can be determined in the laboratory. The determination is especially important in the lime modification process because the lime retention point establishes the maximum amount of lime to be used. In the stabilization process a quantity of lime in excess of the retention point percentage must be used to insure adequate Ca^+ for the pozzolanic reactions. The increase above the retention point varies as a function of soil mineralogy and lime types.

Durability Tests

At the present, the most popular durability indicators of lime-soil mixtures are the freeze-thaw and wet-dry cycle tests. Strength is usually the evaluation criterion of these tests, although moisture distribution, visual inspection, and unit length change have also been examined (46). The procedure is nearly standard in these tests, but variables in the cycles such as number, time, and temperature extremes of cycles and moisture content of the samples make comparison of results among sources impossible.

The Benkelman Beam Deflection test might also be placed in the category of durability tests, as shown recently by its use

in a Nebraska investigation (30,63). Deflection tests might be used more extensively in the future as a means of field durability determination due to the difficulty encountered in obtaining undisturbed samples (64).

Moisture-Density Relationships

The basis of evaluating relative density of soils varies from the use of Standard Proctor Density (AASHTO T-99) and Modified Proctor Density (AASHTO T-180) to numerous similar methods. Difficulty is encountered in correlation of strength values reported by investigators because of the variation of sample dimensions, moisture content, and per cent standard density to which the lime-soil mixtures are compacted.

Strength Tests

The Unconfined Compressive Strength test is the most used method of evaluation of strength of lime-soil mixtures although the Triaxial Shear Test and the California Bearing Ratio test appear to be gaining in popularity. The choice of the test used by any investigator is a matter of preference because all of the methods give satisfactory results. The major problem which is characteristic of all methods of strength determination is the obtaining of undisturbed samples, a problem particularly typical of lime-treated soils. Here too, the test results are difficult to correlate among sources not only because of the various methods used, but because of the variables encountered in the molding procedures.

Other Considerations

Possibly, the major problem encountered in lime stabilization is the problem of stabilization of a soil that cannot be feasibly or economically stabilized with lime, a problem which could be eliminated by proper investigation. Other factors also affect the lime stabilization process and will now be reviewed.

Costs

The expense of lime treatment varies with geographic location, method of application, soil mineralogy, quantity of lime used, labor costs, and thickness of treatment. Statistics on projects completed 15-20 years ago show an average cost of \$.35 per square yard, which is based on a treatment thickness of six inches (17).

A recently constructed project in Missouri illustrated that lime treatment is still, in most cases, economical (55). Under the contract prices for this particular project, comparisons showed that a five-inch lime treated base thirty feet wide cost approximately the same as a four-inch rolled stone base. Although no strength comparisons were given, the indicated performance of the lime-soil was better than that of the rolled stone base. Cracking appeared in the pavement over the rolled stone base within a relatively short time after construction while the pavement above the lime-soil mixture showed no indication of failure.

Costs of lime, relative to other materials, have not increased disproportionately. Labor costs and transportation of lime from the manufacturer should be the major price influencing factors on lime stabilization in the future.

Safety

Because hydrated lime is the most popular lime form used today, safety is much less a problem than in years past. In the earlier days of stabilization quicklime in the dry form was used extensively, resulting in severe burns to unprotected workmen when body perspiration came in contact with lime dust.

Workmen handling lime in the field should be properly clothed with long sleeve shirts, high-lace shoes or boots, gloves, hats, and eye goggles or safety glasses. Proper clothes should be the safety criterion regardless of the type of form of lime being used (52). Extreme caution should also be exercised where wind conditions exist, especially in residential areas where powdered lime may be carried by the wind.

The most critical safety problem today is eye protection although most accidents are the result of neglect of safety standards. "Safety varies directly as a function of safety measures practiced" should be the criterion used.

Climatic Conditions

The climatic conditions are very important to the overall stabilization process because they dictate whether or not a soil needs treatment. Severity and frequency of freezing

and thawing and fluctuations in soil moisture content are factors contributing to need for soil stabilization.

Excessive moisture during and after the completion of the lime stabilizing process does not have any harmful effect on the treated soil unless erosion occurs (31,52). The major problems caused by climatic conditions are those resulting from freezing and thawing. The most critical time period which a lime-soil mixture undergoes is the spring season following treatment (51). At this time both the lime-soil mixture strength and the subgrade support strength are at a minimum. For this reason, lime treatment should be completed as early in the year as possible in order to allow maximum time for the pozzolanic reaction to proceed before exposure to any low temperatures. Traffic should be held to an absolute minimum during the spring season following treatment because any loading applied to the roadway at this time has a detrimental effect on the cementing action.

Abrasion and Cracking

Lime treated roadways are susceptible to abrasion and cracking when left unprotected. Cracking is caused by volume changes in either the subgrade or the lime-soil mixture and heavy traffic on the roadway prior to the attainment of adequate curing (20). Cracking can be minimized by rigid moisture and compaction controls, use of proper rollers and rolling procedures, and prohibition of heavy traffic during the curing period. If heavy equipment or traffic is unavoidable, a light membrane seal

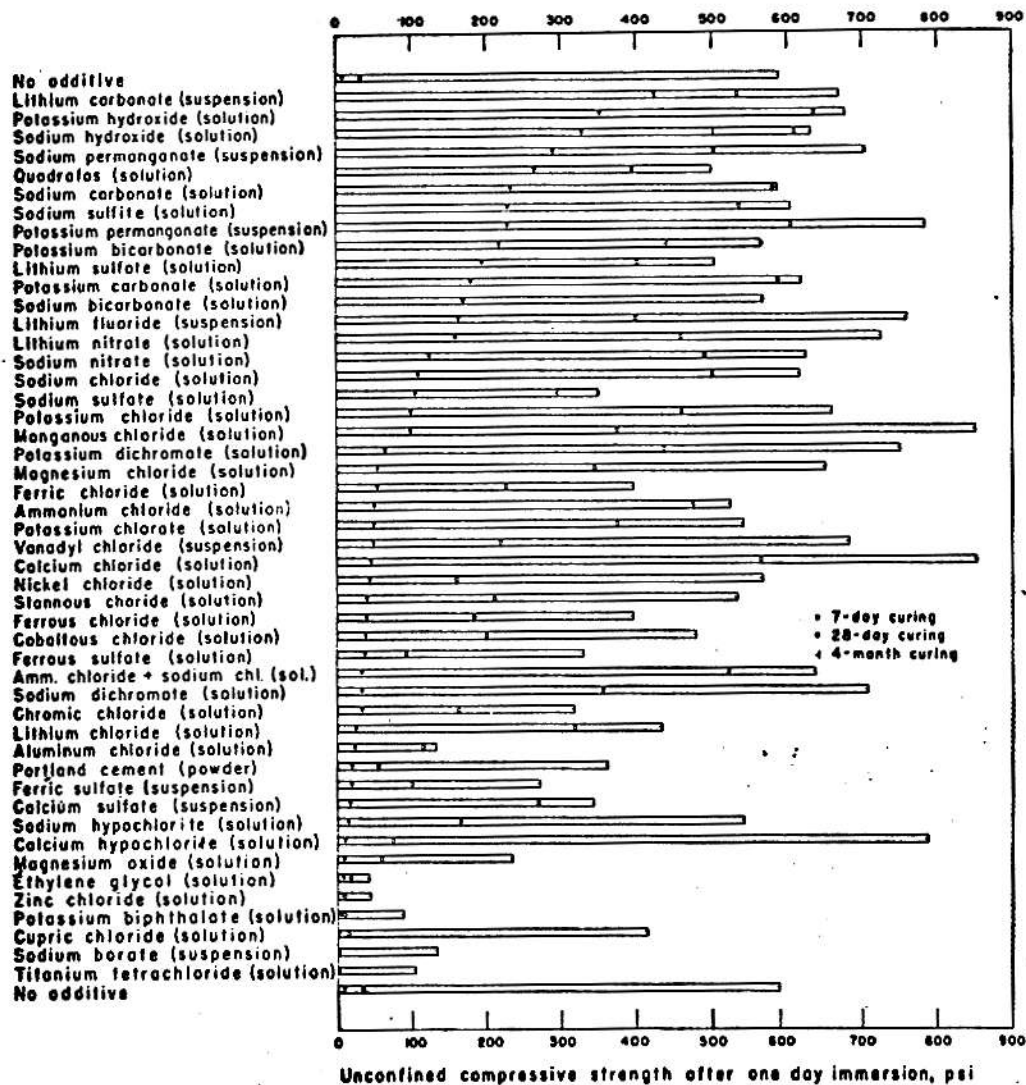
can be used with reasonable satisfaction.

Abrasion, also the result of traffic on the unprotected roadway, may be eliminated by use of a light bituminous seal.

Additives

When lime is added to some soils, satisfactory pozzolanic reactions do not proceed because of the absence of pozzolans in the soil. Addition of a pozzolan has the effect of increasing the rate of strength gain as well as the ultimate strength of the treated soil. In the United States, fly-ash is the most widely used pozzolan (12,25,34,35,43,64,65,66). Numerous investigations have been conducted in seeking new materials which could be used as pozzolans, but results have been disappointing. One investigation reported adequate strength gains resulted with use of expanded shale dust as a pozzolan additive (67).

Additions of various chemicals to lime-flyash-soil mixtures are being investigated regarding the possibility of accelerating the pozzolanic reaction even more. Figure B-12 shows the comparisons of an investigation where chemicals were used as additives (68). Here, 0.5 per cent of the listed chemicals were added to 75: 2.5: 22.5 Ottawa sand-lime-flyash mixtures which were compacted to Standard Proctor Density. More investigations utilizing chemicals as additives may result in combinations of chemicals which could maximize the rate of pozzolanic reaction.



From Highway Research Board Bulletin 231, pg. 70

Figure B-12. Effect of trace minerals on lime's strength gain.

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MODIFICATION OF LOESS BY THE
USE OF HYDRATED LIME

by

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ABSTRACT

Exhaustive investigations requiring much time and effort have greatly contributed to the popularity and the importance of hydrated lime in the stabilization field today. Hydrated lime has been used for approximately thirty years in the stabilization of medium to highly plastic soils. The purpose of this thesis was to investigate the effects of mixing hydrated lime with a low plastic loessial soil, especially on the grain size and shear strength parameters. Sampling locations were selected near Atwood, Kansas; Clinton, Iowa; Kansas City, Kansas; Omaha, Nebraska; and St. Louis, Missouri.

The quantities of hydrated lime added to the soil samples were determined by utilizing the lime retention point theory. Laboratory research was conducted on raw soil, one-half the retention point percentage, and at the retention point percentage. The samples were subjected to standard compaction, specific gravity, Atterburg Limits, grain size analysis, and direct shear tests in the laboratory.

The laboratory research showed that the results of hydrated lime-loessial soil mixtures were very similar to those of hydrated lime-plastic soil mixtures. Reduction in plasticity index, standard density, and dry cohesion were accompanied with increases in plastic limit, grain size, optimum moisture, and angle of internal friction. The losses in the dry cohesions are not critical because they were

obtained from samples in an oven dried state. Saturation of the samples would essentially reduce the cohesion of the soils to zero. Therefore, the increase in the angle of internal friction did increase the shear strength. The grain size increase would also reduce the effect of capillary action.