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Transportation Research Record 1295

Contents

Foreword

The 12 papers included in this Record on soil stabilization are of interest to researchers and practicing engineers alike. The first group of four papers is related to cement stabilization; the second group of five papers is related to lime stabilization; and the third group of three papers is related to chemical soil stabilization.

Okamoto et al. report on the use of a Clegg impact tester and a Proceq Type PT test hammer for nondestructive evaluation of cement-stabilized soils. They present correlations between impact hammer test values and compressive strength of the stabilized material. Maher et al. report on the use of municipal solid waste (MSW) ash as a stabilization material. They investigated MSW ash-cement-sand composite mixtures for strength characteristics at various ages. Scavuzzo presents a method for determining the cement content of soil-cement that correlates the heat of neutralization with the cement content. Hadley reports on the results of a material characterization and analysis test program conducted to establish the magnitude, scope, and expected variation in engineering properties measured in the laboratory and in the field of cement-stabilized base and subbase materials. Variation analysis was conducted on modulus, Poisson's ratio, tensile stress, tensile strain, and fatigue cycles to failure.

McCallister and Petry studied the effects of continuous water leaching on the engineering and physical properties of a lime-treated expansive clay. They found that property changes are related to lime content and initial moisture content. Ferris et al. investigated the use of barium hydroxide and barium chloride as a pretreatment for sulfate-bearing soils to prevent the formation of ettringite, a highly expansive mineral that may develop in lime-stabilized sulfate-bearing soils. The laboratory study included evaluation of several soils using the California bearing ratio (CBR) and potential volume change tests. Basma and Tuncer studied the effect of lime on the volume change and compressibility of two highly expansive clay soils. Ghazali et al. present the results of an experimental investigation on the consolidation and shear strength properties of kaolin clay stabilized with hydrated lime and phosphoric acid. Tuncer and Basma studied the effect of lime treatment on the strength and stress-strain characteristics of a cohesive soil. They evaluated untreated and lime-treated specimens in the laboratory.

Fletcher and Humphries evaluated the effect on the CBR value of a micaceous silt caused by the inclusion of discrete polypropylene fibers in the compacted soil. The laboratory study parameters included various configurations and lengths of fibers. Shepard et al. report on the use of calcium chloride as an additive in secondary road rehabilitation. The process involves recycling and blending the asphalt surface course and granular base course and adding the calcium chloride to improve bearing capacity and frost susceptibility characteristics. Ajayi-Majebi et al. evaluated the use of epoxy-based systems to improve the CBR value of fine, poorly graded soil found at low-duty airport sites. They developed stabilization models on the basis of statistical analysis of the test data.

Nondestructive Tests for Determining Compressive Strength of Cement-Stabilized Soils

PAUL A. OKAMOTO, BRIAN T. BOCK, AND PETER J. NUSSBAUM

Nondestructive impact hammer tests were made on cement-stabilized noncohesive soils using a Clegg impact soil tester and a Proceq Type PT test hammer. Compressive strength of the stabilized soils varied from about 5 to 1,000 psi. Curves correlating impact hammer values with cement-stabilized soil compressive strength were developed for each of the test hammers. These curves can be used to correlate in situ nondestructive test values with the compressive strength of cement-stabilized soils.

Normally, tests for acceptance of the quality of soil-cement or cement-treated base course construction involve determination of cement content, compaction, and layer thickness.

In some areas, agencies have attempted to base acceptance on the compressive strength of drilled cores (see ASTM D1633–84 and ASTM C39–86). This procedure has led to difficulties, especially where relatively low-strength material is specified (300 to 400 psi). Often, it is not possible to obtain good core recovery from cement-stabilized bases at early ages; this difficulty can lead to unwarranted conclusions about the quality of construction. In addition, core drilling and testing of cores for compressive strength are both costly and time consuming. Thus, a need exists for a simple nondestructive testing device that can be used in the field to provide a more meaningful evaluation of the quality of cement-stabilized materials.

OBJECTIVE AND SCOPE

The investigation was undertaken to develop a nondestructive test method for determining the compressive strength of cement-stabilized soil. The objective was accomplished within the following scope:

1. Two commercially available impact hammers, a Clegg impact soil tester and a Proceq Type PT test hammer were selected to evaluate correlations between test hammer values and cement-stabilized soil compressive strength.
2. Six cohesionless soil materials were selected to evaluate impact hammer responses for a range of soil materials.
3. Each of the six soils was stabilized with different amounts of cement to develop a range of compressive strength.
4. Cement-stabilized soil test blocks were molded for impact hammer testing. Companion cylindrical specimens were compacted to the same density as the test blocks to determine compressive strength.
5. Correlation curves of cement-stabilized soil compressive strength versus impact hammer test values were developed for each of the two instruments.

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TEST EQUIPMENT AND SOILS

Two commercially available impact hammers were selected for the test program. Selection criteria included transportability, ease of operation, and cost. The Clegg impact soil tester

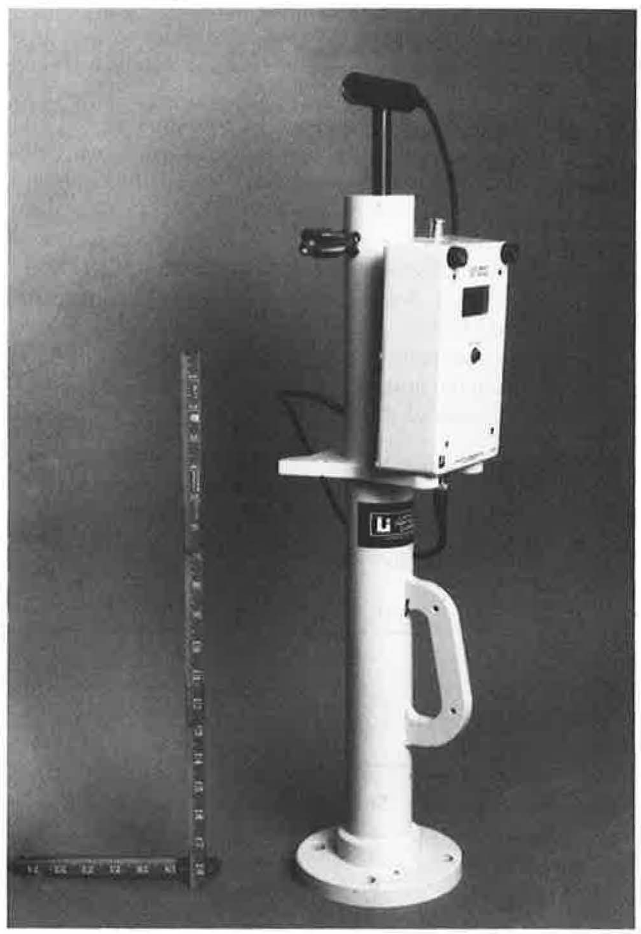

FIGURE 1 Clegg impact soil tester.

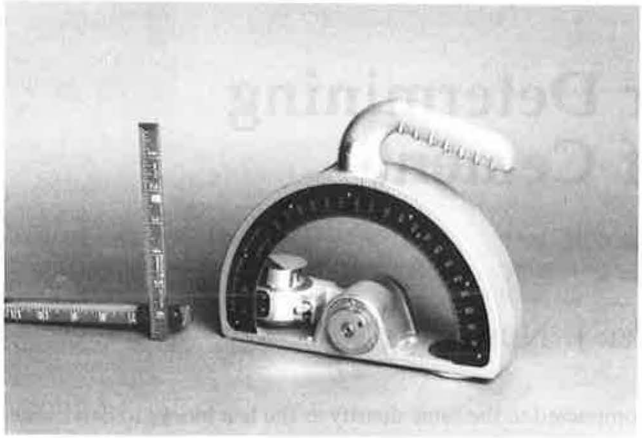

FIGURE 2 Proceq Type PT hammer.

including an approximately 10 × 10 × 31-in. carrying case weighs about 15 lb. The Proceq Type PT test hammer weighs about 5½ lb and is carried in an approximately 3 × 6 × 10-in. case. The test hammers are shown in Figures 1 and 2.

The Clegg impact soil tester consists of a 10-lb compaction hammer together with a guide tube and an electronic meter. The meter signal is provided by an accelerometer fastened to the hammer. Meter maximum deceleration readout is in digital form.

The Proceq Type PT hammer is a pendulum-type impact tester. Impact energy is 0.65 ft-lb with a hammer surface area of about 1.95 in.^{2} Impact hammer rebound values are read from a scale on the instrument.

Six soils encompassing the range of cohesionless soil materials commonly used for soil-cement construction were obtained for this laboratory test program. Soil classifications and descriptions are presented in Table 1. AASHTO soil classifications vary from A–1–a to A–3(*1*). Standard density and optimum moisture determined in accordance with ASTM D558–82, *Moisture-Density Relations of Soil-Cement Mixtures*, varied from 106 to 142 lb/ft^3 and from 6 to 13.5 percent by weight of dry soil plus cement, respectively. Moisture-density tests were made using the amount of cement estimated to produce soil-cement for each of the six soils.

TEST PROCEDURES

To evaluate the response of the impact hammers over a hardness range of compressive strength of about 5 to 1,000 psi, cement-stabilized soil test blocks were molded with varying amounts of cement. Amounts of cement used for test blocks made with each of the six soils are presented in Table 2. The amount of water used for batching the cement-stabilized soil was equivalent to the optimum moisture content determined from the moisture-density tests.

The mix was placed in a 12 × 24 × 8-in. container and statically compacted to a thickness of about 6 in. On the basis of experience with both impact hammers, the boundary support of the container has an insignificant effect on surface hardness impact values when specimens are at least 6 in. deep and have

TABLE 2 CEMENT CONTENTS AND DENSITIES

Soil No.	Cement Content, percent	Dry Density, pcf
1	2	86.3
	5	95.9
	8	101.4
	10	99.5
	12	99.5
	16	99.5
2	2	100.5
	5	100.5
	8	99.5
	10	100.5
	12	105.7
	16	105.7
3	5	108.2
	10	108.2
	15	106.2
4	5	104.1
	10	102.0
	15	102.0
5	8	117.4
	12	122.0
	16	122.0
6	4	128.3
	7	128.3
	10	118.5

TABLE 1 SOIL CLASSIFICATION AND DESCRIPTION

Soil No.	AASHTO Class	Max. Dry Density, pcf	Optimum Moisture, percent	Sieve Analysis, percent passing 1/2 in.	no. 4	no. 10	no. 40	no. 200
1	A-1-b	118	11.0	100	96	75	38	11
2	A-3	106	11.5	100	100	100	96	0
3	A-1-b	128	9.5	100	96	78	44	10
4	A-2-4	118	13.5	100	99	89	61	15
5	A-1-b	127	8.5	100	97	76	28	1
6	A-1-a	142	6.0	83	60	49	24	4

a compressive strength of less than 1,000 psi. For specimens with compressive strengths in excess of approximately 1,000 psi, the within-test variability of impact values exceeds any differentiation of specimen support and boundary type.

Companion 2.8 × 5.6-in. cylindrical specimens were made in general compliance with ASTM D1632–87, *Making and Curing Soil-Cement Compression and Flexure Test Specimens in the laboratory.* However, the cylinders were compacted to the same density as the companion block specimens and curing was under wet burlap at 72°F.

Impact hammer tests on the cement-stabilized blocks were made after 1, 2, 3, 5, 7, 10, 14, and 17 days of curing under wet burlap. Three impact instrument readouts were averaged for each test to provide an impact value. The hammers were moved between each impact readout to prevent impact footprint superposition. Companion cylindrical specimens were tested each day that impact values were obtained according to ASTM D1633–84, *Compressive Strength of Molded Soil-Cement Cylinders.* A best-fit curve of compressive strength versus curing age was determined for each block's companion cylinders. Thus, normalized compressive strength data were available for each of the impact hammer values.

TEST RESULTS

Individual Soil Analysis

A linear regression of compressive strength on impact values for the Clegg impact soil tester and the Proceq Type PT test hammer was done for each of the six different soils. Outlier and leverage points identified in each linear regression were eliminated. The regression analysis for the six individual soils is presented in Table 3. The linear regression form for the compressive strength and impact value data is the log-log equation. For the six equations derived for the Clegg data, the coefficients of determination, R^2, ranged from 0.75 to 0.94 and averaged 0.86. For the Proceq data, the values of R^2 ranged from 0.25 to 0.97 and averaged 0.75.

Combined Analysis

The analysis of individual soil data indicated that a single log-log relationship for all soils combined was feasible for both impact testers. Impact values versus compressive strength for the Clegg and Proceq testers are plotted in Figures 3 and 4, respectively. The values of R^2 for the Clegg and Proceq hammer log-log relationships are 0.90 and 0.84, respectively. The resulting equations are presented in Table 4.

Confidence Intervals of Strength Prediction

Confidence intervals at the 5 percent level of significance for the prediction of the actual value resulting from a given impact value were computed as shown in Figures 5 and 6 for the Clegg and Proceq hammers, respectively. The 95 percent confidence intervals represent the uncertainty of the probability density function about compressive strength, given an impact value. The conditional expectation, also computed for the two data sets, is presented in Table 5. The 95 percent confidence levels for the conditional expectation represent the uncertainty of the probability density function about the expected, or average, compressive strength for a given impact value. The conditional expectation confidence intervals are nonlinear functions of the impact values. The half-prediction interval as a percentage of compressive strength ranged from approximately 5 to 11 percent for both hammers.

Within-Test Variability

The within-test coefficient of variation, equal to the standard deviation divided by the mean, can be estimated from the within-test range of the three impact values (see ACI 214-77). The soil-cement specimen surface is assumed to be uniform in hardness and compressive or impact strength. Any impact value variation, therefore, follows from equipment and test method variability. The range in impact values cal-

TABLE 3 SUMMARY OF LINEAR REGRESSION ANALYSIS ON INDIVIDUAL SOILS

Impact Hammer	Soil No.	Soil Class	No. of Tests	Compressive Strength, psi Minimum	Maximum	Average	log (f'c) = "a" + "b" x log (Impact) "a"	"b"	R-sq.
Clegg	1	A-1-b	40	60	270	150	0.949	0.762	0.75
	2	A-3	39	40	770	300	0.015	1.372	0.90
	3	A-1-b	23	120	540	320	0.279	1.176	0.91
	4	A-2-4	24	100	490	230	0.326	1.173	0.94
	5	A-1-b	9	440	940	690	-0.263	1.522	0.75
	6	A-1-a	23	210	780	490	1.093	0.800	0.89
Proceq	1	A-1-b	25	90	270	190	0.952	0.857	0.86
	2	A-3	27	120	770	370	-2.327	2.742	0.97
	3	A-1-b	22	120	540	330	-0.960	1.981	0.94
	4	A-2-4	24	100	490	230	-1.024	2.049	0.93
	5	A-1-b	9	440	940	690	-0.304	1.703	0.25
	6	A-1-a	23	210	780	490	0.806	1.058	0.54

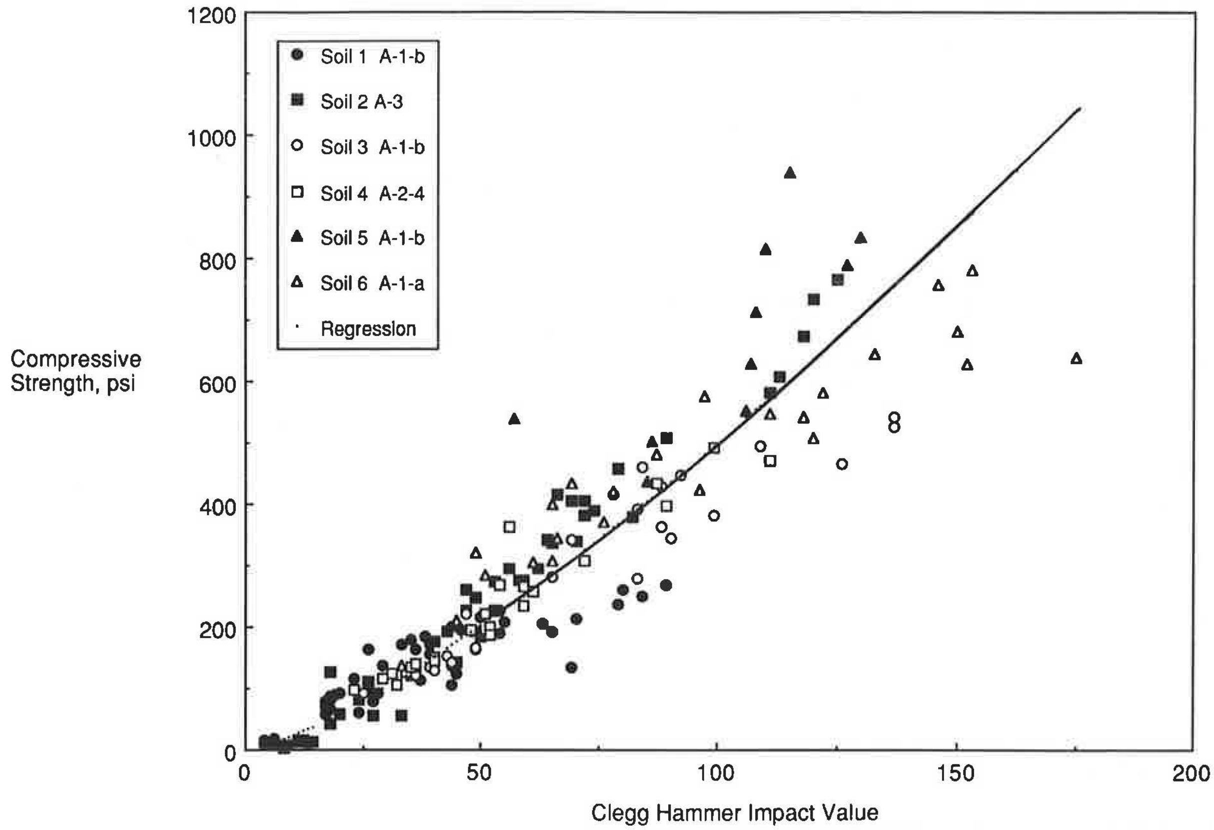

FIGURE 3 Clegg hammer data.

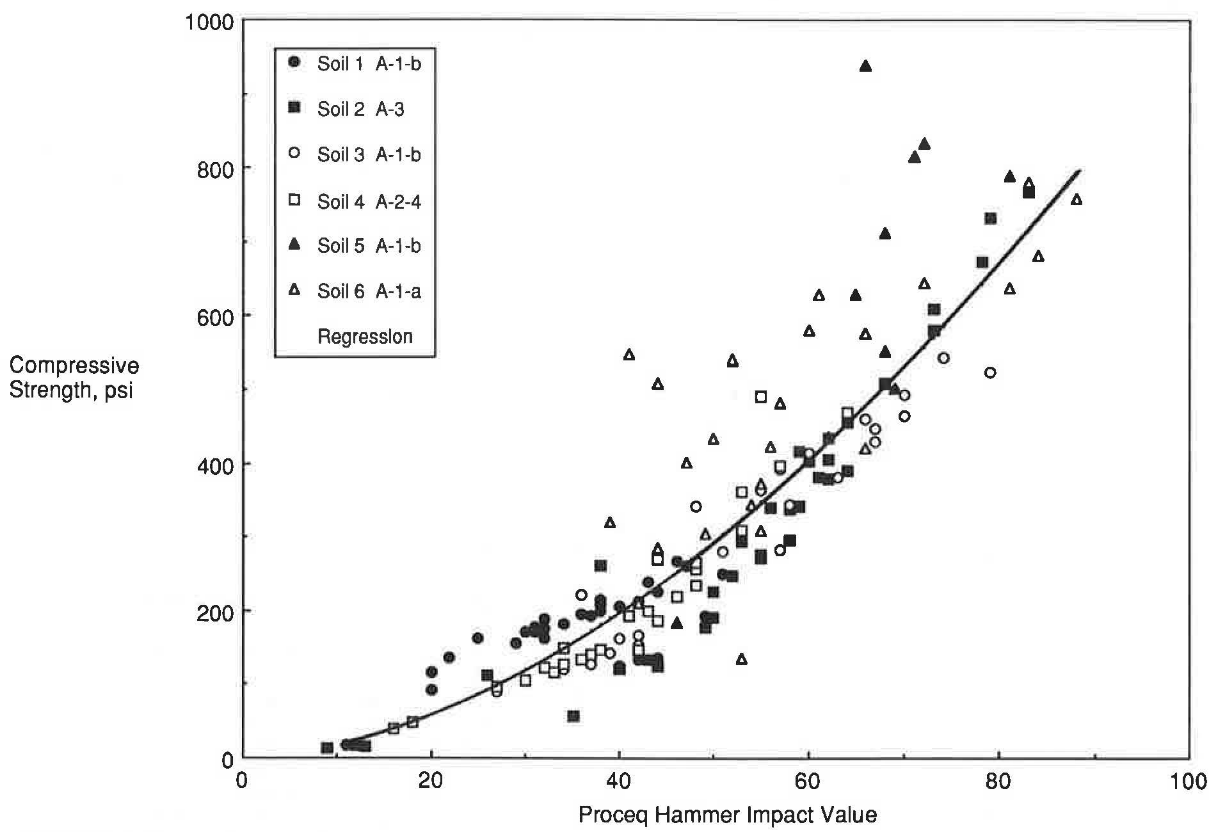

FIGURE 4 Proceq hammer data.

TABLE 4 SUMMARY OF LINEAR REGRESSION ANALYSIS

Impact Hammer	No. of Tests	Compressive Strength, psi Minimum	Maximum	Average	log (f'c) = "a" + "b" x log (Impact) "a"	t-statistic	"b"	t-statistic	Standard Error	R-sq.
Clegg	179	5	940	275	0.081	1.4	1.309	39.5	0.157	0.90
Proceq	145	15	940	325	-0.516	-4.8	1.757	27.1	0.133	0.84

culated for each soil, cement content, and test age were also assumed to represent the true ranges, because a single set of tests may be statistically insufficient.

The coefficient of variation calculated for the three impact values can be compared to that expected from compressive strength tests. The coefficient of variation of within-test compressive strength testing was calculated from published data of 115 sets of duplicate specimens molded from 35 different soil materials (*2,3;* ASTM D1633–84).

The resulting distributions of within-test coefficient of variation are shown in Figures 7 and 8. Average within-test coefficients of variation are 13.7, 9.7, and 10.5 percent for the Clegg hammer, Proceq hammer, and compressive strength tests, respectively. For coefficients of variation between 5 and 20 percent, the impact hammers exhibit a slightly larger variation than that expected from compressive strength testing. The percentages with coefficients of variation less than 20 percent are 80, 93, and 85 for the Clegg, Proceq, and compressive strength data, respectively. The analysis indicates that similar within-test scatter of data exists between the impact hammers and compressive strength testing.

ADDITIONAL RESEARCH

The research program evaluated testing equipment to nondestructively determine compressive strength of soil-cement. Testing of the equipment in the field should be done to verify or modify the compressive strength versus impact value correlation developed from laboratory data. During field testing, the following issues should be addressed:

1. The number of impact tests averaged at a given test location may have to be increased to obtain similar coefficients of variation in the field as those expected from compressive strength testing.

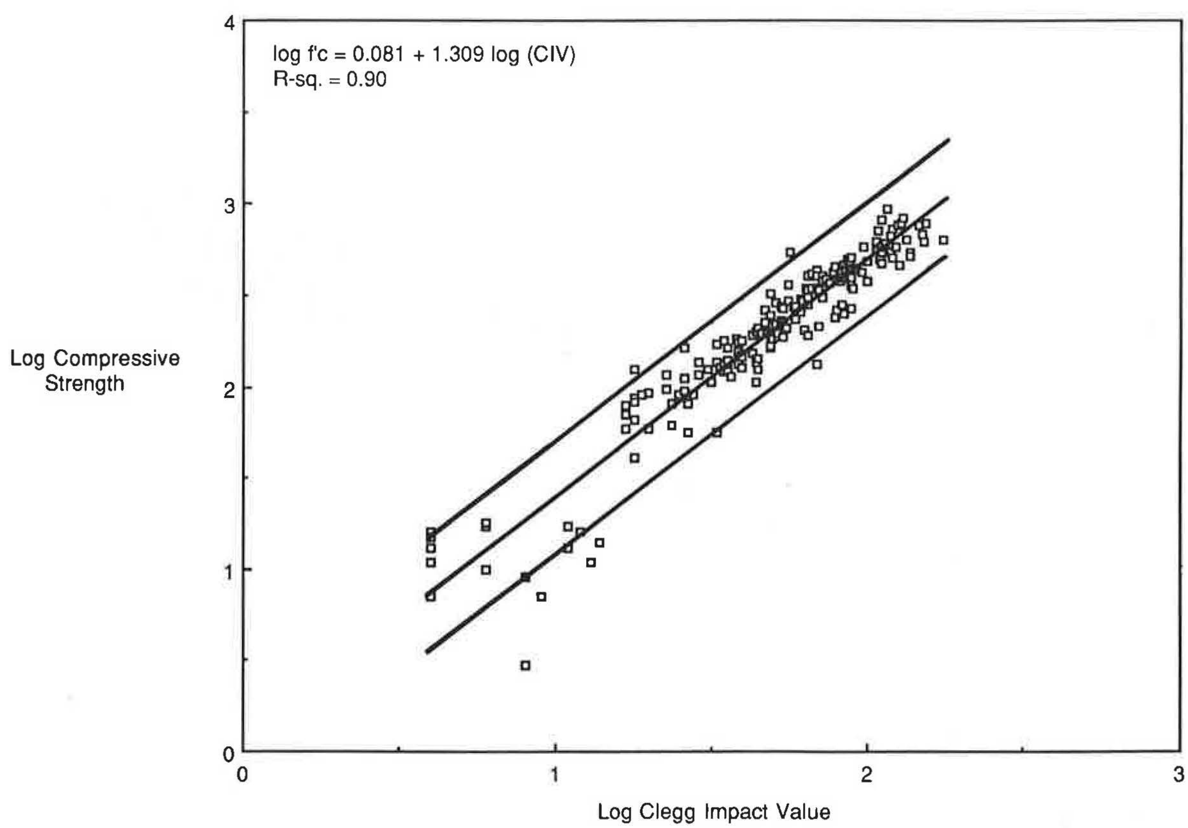

FIGURE 5 Clegg hammer 95 percent confidence intervals.

TABLE 5 CONFIDENCE INTERVALS FOR STRENGTH PREDICTION

f'c, psi	Clegg at 95% Level of Significance				Proceq at 95% Level of Significance			
	Low f'c, psi	High f'c, psi	Plus or minus, psi	Range, percent	Low f'c, psi	High f'c, psi	Plus or minus, psi	Range, percent
100	94	106	6	6.1	92	109	9	8.5
200	190	211	11	5.3	190	211	11	5.3
300	283	318	18	5.8	285	316	15	5.1
400	375	427	26	6.6	377	424	24	5.9
500	465	538	37	7.3	467	536	34	6.9
600	554	650	48	8.0	555	649	47	7.8
700	643	762	60	8.5	641	764	61	8.7
800	731	876	73	9.1	727	880	76	9.6
900	818	990	86	9.6	812	997	93	10.3
1000	905	1105	100	10.0	896	1116	110	11.0

NOTE: Prediction interval of the conditional expectation.

Range percent is one half the prediction interval as a percentage of compressive strength.

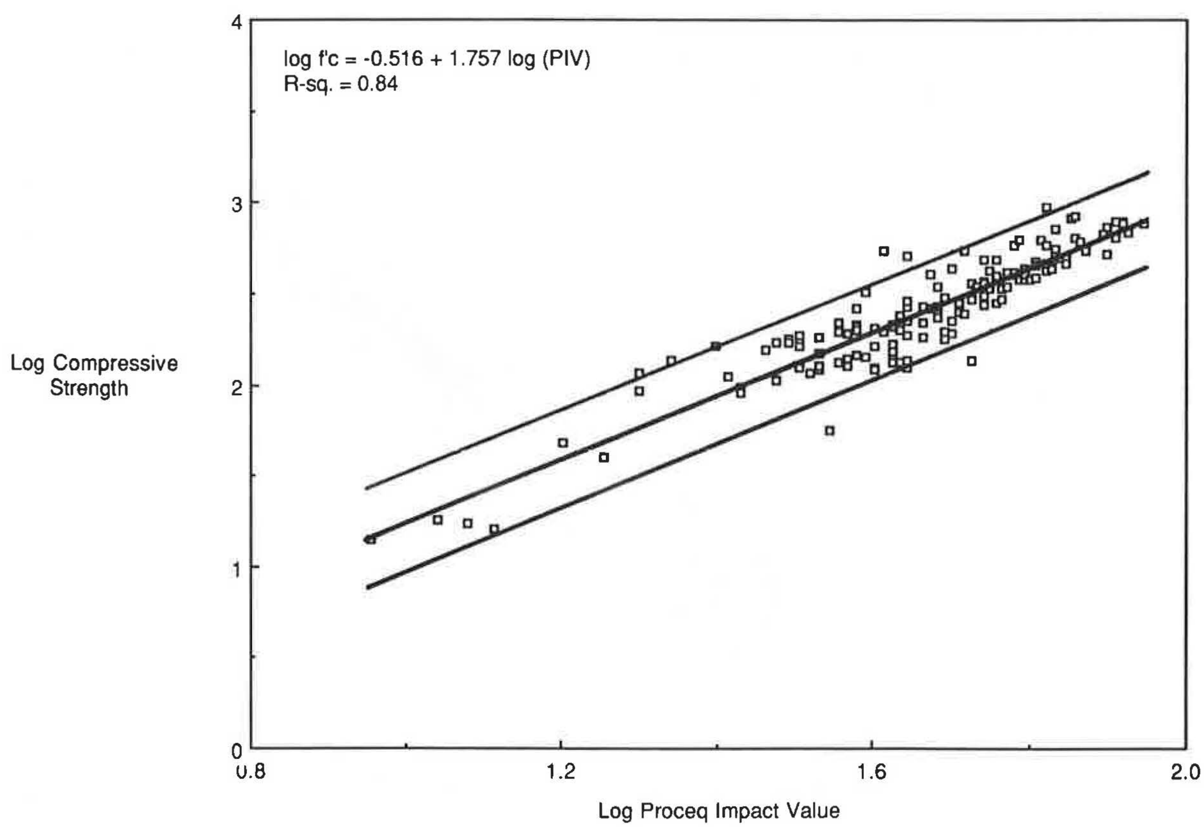

FIGURE 6 Proceq hammer 95 percent confidence intervals.

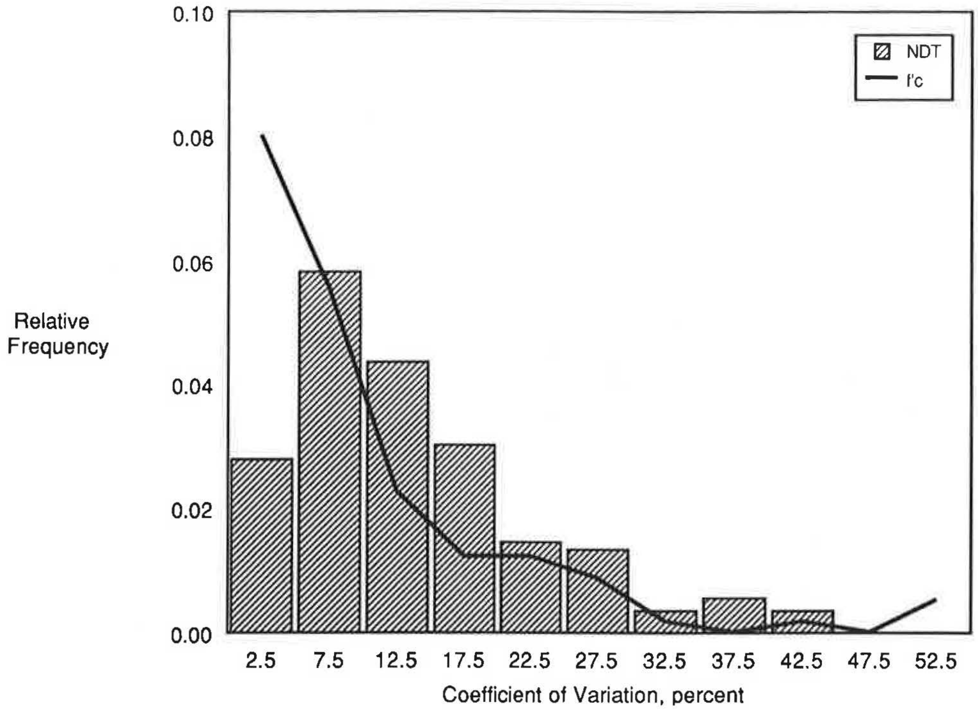

FIGURE 7 Relative frequency histogram for Clegg hammer impact values.

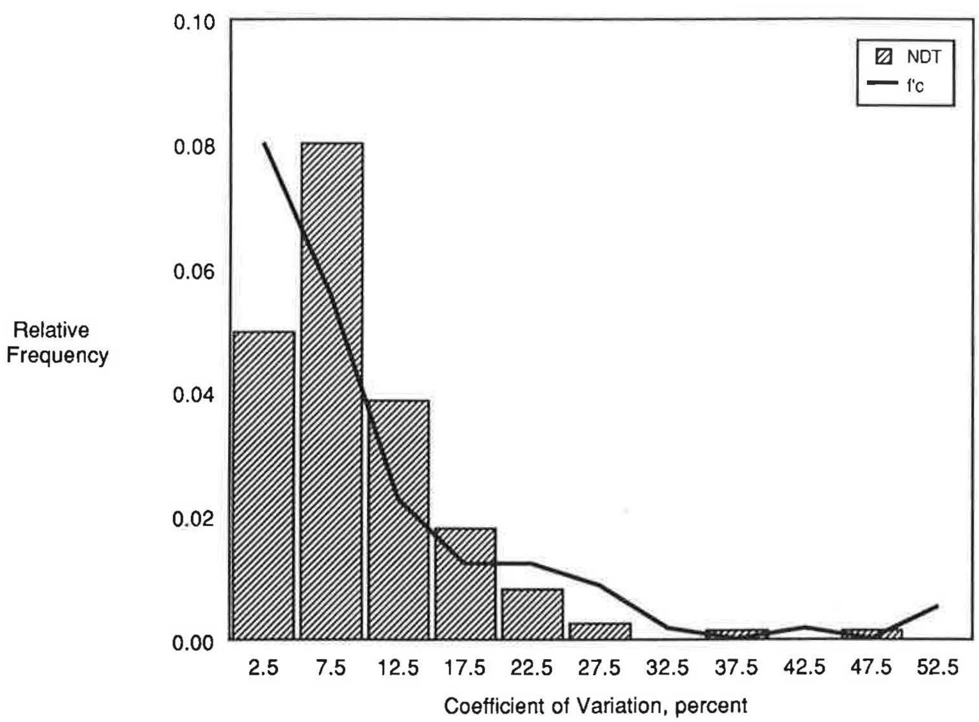

FIGURE 8 Relative frequency histogram for Proceq hammer impact values.

2. The effects of specimen damage during coring need to be established. Correlations between strength and impact value may need to be modified to incorporate specimen damage.

3. The study incorporated only six different cohesionless parent soils. Field testing should encompass a variety of soils (with differing degrees of cohesion) and cement contents.

CONCLUSIONS

Correlation curves were developed permitting the use of results from impact hammer nondestructive tests to determine the compressive strength and rate of strength gain of in situ cement-stabilized soil construction. This nondestructive method avoids the testing difficulties associated with drilling cores. It is recommended that five hammer readings, each on an untested area of the surface, be averaged.

ACKNOWLEDGMENTS

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Properties of Municipal Solid Waste Ash-Cement Composite

M. H. MAHER, D. FLOOD, AND P. N. BALAGURU

Incineration, which is a viable alternative for processing of municipal solid waste (MSW), reduces the volume of the waste destined for landfills. Results from a recent study on physical, chemical, and engineering properties of MSW mixed with top and bottom ash are presented. These results can be used to assess the possibility of using MSW incinerator ash for certain construction purposes and soil stabilization applications. Some of the test results are compared with those of fly ash (coal burning) cement composites for assessment of their potential use. The MSW ash-cement composite was investigated for strength characteristics at various stages of maturity. The independent variables considered were MSW ash content, sand content, and water cementitious ratio. The ash-cement ratio was varied from 10 to 20. Water-cementitious ratios of 0.45 and 0.6 were investigated. The response variables in this study included compressive strength and splitting tensile strength. Strength tests were conducted at 3, 7, and 28 days of maturity. Tests were also performed to determine the maximum density and optimum moisture content of the MSW ash as needed for field applications.

In the past few years, a number of studies have been done on municipal solid waste (MSW) incinerator ash—generally to investigate toxic waste properties, resource recovery equipment and techniques, and relative costs for hazardous waste incineration. Little, if any, research has been carried out on the construction properties of MSW ash, mainly because of the potential toxicity of the fly ash portions. Although the resulting top (fly) and bottom mixture of MSW ash is usually nontoxic, the bottom ash still contains deleterious materials that might reduce strengths over time. For these reasons, most of the research done on construction applications of ashes has focused on nontoxic coal fly ash, which currently is used in a wide variety of concrete mixes as a pozzolan.

The bulk of the research on MSW solid waste in the 1970s and the early 1980s pursued the feasibility of converting the solid waste materials to power and energy sources or supplementary fuels. Additional studies during this period investigated various resource recovery techniques and processing equipment (*1*).

In the mid-1970s, it became evident that hazardous waste incineration was the most practical method of hazardous waste disposal. The volume reduction achieved made incineration more attractive than landfill disposal, ocean dumping, and deep-well injections. By the early 1980s, the incineration of MSW was more prevalent, and concern was rising about the toxic condition of the incinerated MSW. The incoming amount of MSW was increasing rapidly, and various studies were done on hazardous waste incineration costs (*2*).

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Recently, the projected quantities of MSW ash have been substantial enough to warrant further studies to determine disposal methods. In New Jersey, for example, landfill area is diminishing; and disposal of MSW in these landfills must be monitored because of the effects of heavy metals' leaching into the subsurface groundwater.

Research for recovery of lead, cadmium, and chromium has recently been done at Rutgers University (*3*), using electrochemical plating techniques. Kinetic studies were done on ashes from two different incinerators to determine the time when the ash extraction reaction could be stopped to remove peak quantities of these heavy metals.

Municipalities have also required analysis of MSW incinerator ash to determine appropriate landfill design. The city of Sheboygan, Wisconsin, prompted this type of study in 1989 (*4*). A characterization was done on the MSW ash produced from that city's fluidized-bed furnace by obtaining numerous samples from the bottom ash and the incinerator's sludge lagoon on various dates. Individual and composite samples were taken of the bottom, sludge, and fly ashes; and comparisons were made with the bottom and fly ashes produced the same day. Although the resulting chemical testing done indicated a possible groundwater impact requiring a designed landfill, physical tests were also performed on the bottom ash and extremely low specific gravities and high organic contents were noted.

Further work has been done at Rutgers University (*5*) on the chemical and some physical properties of MSW ash obtained from various types of incinerators in Massachusetts, New Jersey, and Canada. The Canadian bottom ash had over half of the total specimen over 2.01 mm in size. This portion of the sample contained such materials as broken glass, slag, metal fragments, and pebbles as the principal components. An analysis of the metal distribution of these ashes showed that the largest concentrations of lead, cadmium, and chromium appeared on the smallest fly ash particles. Other parameters determined from this analysis were particle surface areas, morphology, and densities.

Although MSW incineration is probably the most feasible method of disposal, the process has drawbacks. Certain materials are not easily incinerable and will not sustain combustion. Organic materials, especially those containing chlorine, yield products of incomplete combustion (PICs) that can evidence toxicity (*6*). Insufficient turbulence during combustion can result in the development of inclusions, which results in various materials' escaping incineration. Certain substances also require that the incinerator be supplied with supplementary fuels to maintain combustion temperatures. These circumstances have generated a number of articles that describe the combustion and incineration process (*7*).

As previously mentioned, most of the research done on incinerator ashes for use in the construction industry has incorporated coal fly ash into concrete and grout mixes. Prior work has been accomplished, however, on the use of coal fly ash as structural fill (*8*). This investigation determined the engineering properties of New Jersey fly ash and found it to function satisfactorily as a structural fill with a design pressure of 5 tons/ft^2.

Further research for incinerator ashes for construction purposes has resulted in the use of a wastewater sludge ash and clay mixture to produce construction bricks. (Coal fly ash has also been used for this purpose.) The maximum sludge percentage used to produce the bricks was 40 percent (*9*).

Sludge ash was also experimented with in Singapore for use as an aggregate in lightweight concrete (*9*). The sludge was incinerated at temperatures exceeding 1,000°C and the ash was then crushed and graded, which resulted in a porous (66 percent) aggregate with a specific gravity of 2.90 and a pH value of 9.0. When used in a lightweight concrete mix, the 28-day strengths were comparable to those of other aggregates.

So far, there has been little research accomplished on MSW ash for construction use. The studies that have been done are generally chemical analyses or characteristic determinations for specific landfill designs. The concept of compressive and tensile strength testing of an MSW ash-cement composite is a new approach to a possible solution for incinerator ash disposal.

OBJECTIVE

Physical properties and strength characteristics of MSW ash cement-sand composites are described. The test results presented are compared to those of coal fly ash cement composites for the purpose of assessing the potential use of MSW incinerator ash in applications such as soil stabilization, structural and nonstructural fills, etc.

EXPERIMENTAL PROGRAM

Materials Tested

The MSW incinerator ash used in this study was obtained from a mass-burn MSW incinerator in New Jersey. The schematic diagram of this type of incinerator is shown in Figure 1. The furnace or the combustion chamber of these incinerators generally burns within a temperature range of 600°C to 1,000°C (1,100°F to 1,800°F), while maintaining turbulence to ensure maximum incineration. The ash used in this study was a combination of top and bottom ash after the top, or fly ash, had been previously sprayed with lime. This mixture is generally the end product of most incinerators and carries a high pH value as a result. The chemical composition of the ash is presented in Table 1. The principal metal constituents are aluminum and zinc.

In addition to MSW ash, ASTM Type 1 portland cement and concrete sand were used for preparation of the composite mixes. The particle size distribution of the concrete sand is shown in Figure 2.

Mixture Proportions

A summary of the mix designations and proportions is presented in Table 2. The first 10 mixes use a constant portion of cement while varying the ash, sand, and water-cementitious ratio. This ratio is the fraction of the water content to cement, plus the ash content, and was kept at 0.6 for Mixes 1 through

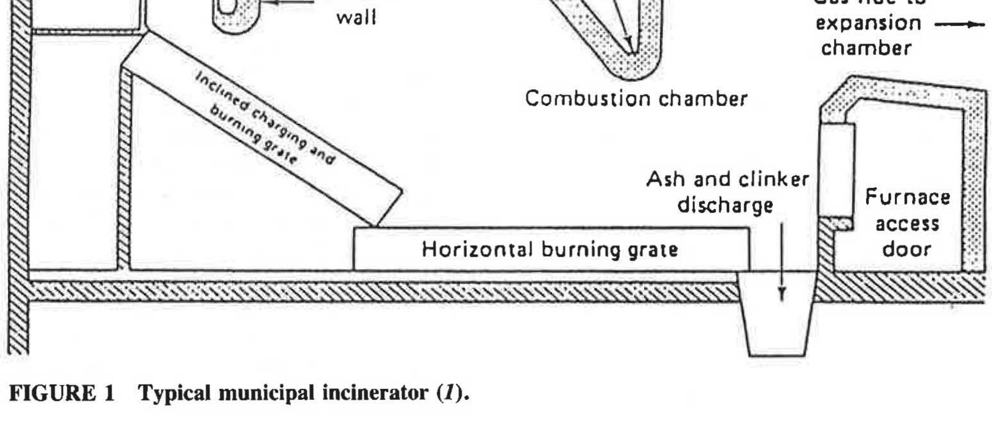

FIGURE 1 Typical municipal incinerator (*1*).

TABLE 1 CHEMICAL COMPOSITION OF NEW JERSEY MSW INCINERATOR ASH

Chemical Composition	mg/kg	Blank[1] (control)	MDL[2] mg/kg
Aluminum	10354.00	N.D[3]	0.1
Antimony	6.9	N.D	0.1
Arsenic	13.73	N.D	0.1
Barium	61.26	N.D	0.1
Cadmium	39.30	N.D	0.1
Chromium	23.78	N.D	0.1
Copper	315.00	N.D	0.1
Lead	701.4	N.D	0.1
Mercury	0.81	N.D	0.02
Molybdenum	14.10	N.D	0.1
Nickel	21.76	N.D	0.1
Selenium	1.2	N.D	0.1
Silver	0.68	N.D	0.1
Zinc	2238.00	N.D	0.1
Total Residue	735000.0	N.D	N/A
Volatile Residue on Total Residue	6500	N/A	N/A

1 Water blank, indicating the initial element amount in the solution prior to testing the ash.

2 Minimum Detection Limit

3 No Detection

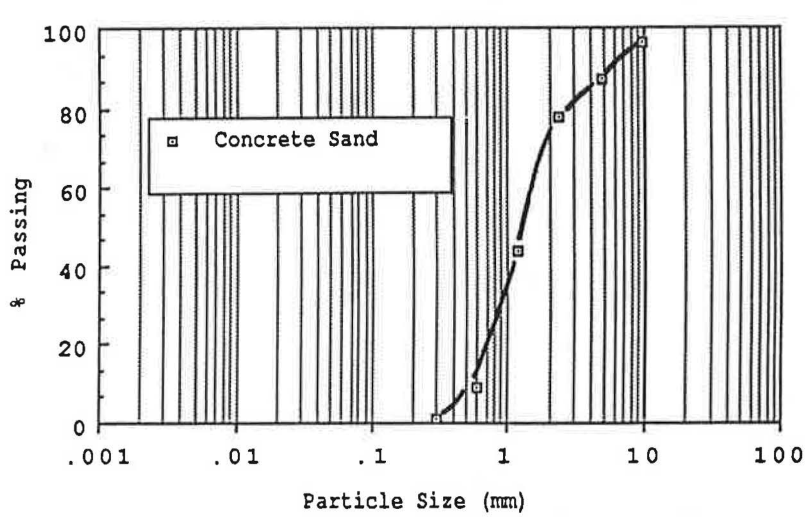

FIGURE 2 Grain size distribution of concrete sand.

TABLE 2 MIX DESIGNATION AND PROPORTIONS

MIX DESIGNATION	CEMENT:MSW-ASH:SAND	WATER/CEMENTITIOUS
1	1-6-10	0.6
2	1-8-10	0.6
3	1-10-10	0.6
4	1-6-15	0.6
5	1-8-15	0.6
6	1-10-15	0.6
7	1-6-20	0.6
8	1-8-20	0.6
9	1-10-20	0.6
10	1-6-10	0.45
11	1-1-2	0.6
12	4-1-5	0.6

9. Mix 10 was prepared using a water-cementitious ratio of 0.45. These mix proportions were specially chosen for comparison of the test results with data obtained from a study on the properties of high volume coal fly ash cement composite (flowable mixtures).

Mixes 11 and 12 (low-volume ash content) were prepared with lower ash and higher cement contents to investigate the possibilities of using the ash for various grouts (such as driller's grout) and soil stabilization applications.

TEST RESULTS AND DISCUSSION

Index Properties of MSW Incinerator Ash

Numerous physical tests were conducted on the ash to determine its index properties. A summary of the results for particle size distribution, Atterberg limits, and maximum dry density–optimum moisture content tests is given in Table 3. The particle size distribution for MSW ash and that of a coal

TABLE 3 COMPARISON OF PHYSICAL PROPERTIES OF NEW JERSEY MSW INCINERATOR ASH AND COAL FLY ASH

Properties	NJ MSW Ash	NJ Coal Fly Ash
Specific Gravity	2.39	2.54
Max. Dry Density (pcf)	104.6	103.2
Opt. Moist. Content	13.2	13.6
Uniformity Coeff.	> 10	2.5
Liquid Limit	35.10	16.8
Plastic Limit	34	-
Average pH Value	12.07	-

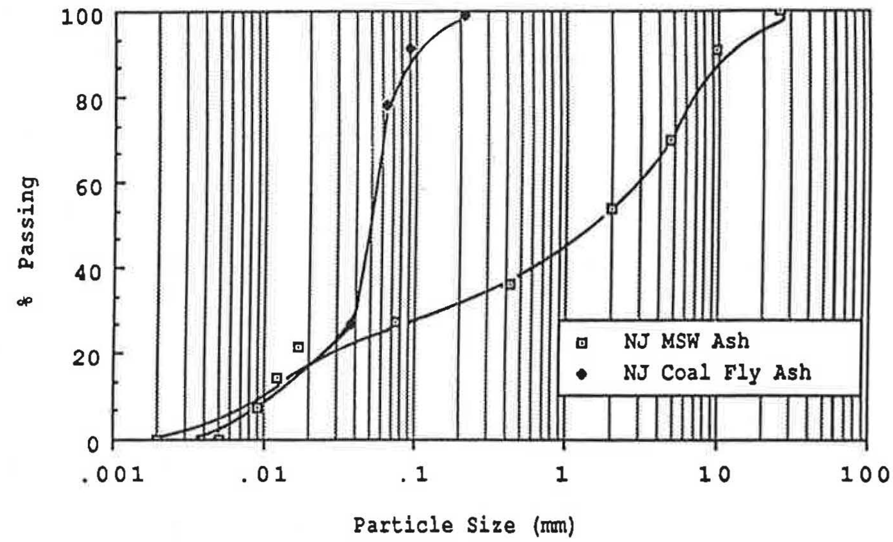

FIGURE 3 Grain size distribution, New Jersey MSW ash and New Jersey coal fly ash.

fly ash for comparison are presented in Figure 3. A significant difference is observed between the gradation of these two types of ash. Because of its heterogeneity, MSW ash is far less uniform in size than coal fly ash.

The maximum dry density and optimum moisture content were determined using ASTM-1557-C, a modified Proctor compaction test. The results of compaction tests for the New Jersey MSW ash as well as those for coal fly ash from New Jersey and other regions are shown in Figure 4.

Compressive and Tensile Strength of MSW Incinerator Ash Cement-Sand Composite

The cured specimens of MSW ash cement-sand composites were tested for determination of their compressive (ASTM D1633) and tensile (ASTM D3967-86) strength. A summary of the compressive and tensile strength test results (28-day) for high-volume MSW ash cement-sand composites, and those for coal fly ash cement-sand composites (for comparison), is presented in Table 4. Compressive and tensile strengths of MSW ash composites tend to decrease with increasing ash and sand content (all other factors constant). This trend was opposite to that observed for coal fly ash composites. A possible explanation is that the coal fly ash acts as a pozzolan in the cement-sand composites. Thus, increasing its content significantly enhances the composite's strength.

Although the MSW ash strength test results (Table 4) compare well with those of coal fly ash composites, particularly in lower ash content range, the high-volume mix proportions are not recommended for practical use, mainly because of deterioration of the test samples after 28-day tests. Deterioration was caused by excessive cracking caused by expansion and leaching of gel from within the sample. This may be a result of calcium hydroxide (lime) presence in the ash and its effect on the composite durability (*10*). As expected, deterioration was more evident in specimens with high ash content. It was therefore decided to (a) reduce the water-cementitious

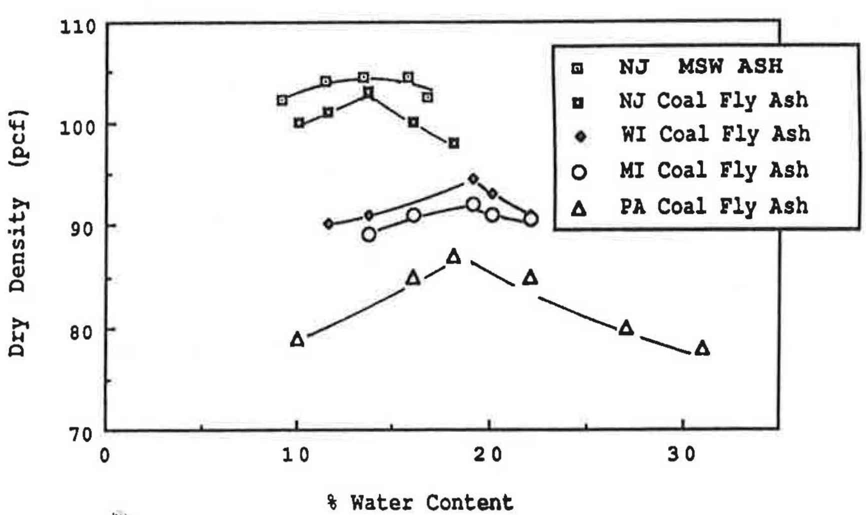

FIGURE 4 Density-moisture relationship.

TABLE 4 COMPRESSIVE AND TENSILE STRENGTH OF ASH CEMENT-SAND COMPOSITES WITH HIGH-VOLUME ASH

Mix Proportions by Parts (Cement-Ash-Sand)	NJ MSW Ash		NJ Coal Fly Ash	
	Compressive Strength (psi)	Tensile Strength (psi)	Compressive Strength (psi)	Tensile Strength (psi)
1-6-10	166	28	198	18
1-8-10	167	28	159	17
1-10-10	106	22	-	-
1-6-15	127	25	255	27
1-8-15	125	24	250	21
1-10-15	110	22	276	18
1-6-20	139	24	212	22
1-8-20	117	22	301	31
1-10-20	93	22	636	30

ratio of the mix or (b) increase the cement content of the mixtures, or both, to determine a suitable mix proportion that gives a stable mix.

Although reduction of the water-cementitious ratio significantly enhanced the strength of high-volume ash composites (Table 5), sample deterioration was still a problem. This problem was greatly reduced by increasing the cement content of the mix. Two mix proportions (one part cement, one part ash, two parts sand, 1–1–2; and the other 4–1–5) were tested for observing the effect of increase in cement content. The results of compressive and tensile strength tests (strength versus curing time) for these mixes are shown in Figure 5. As for mix stability, no long-term deterioration has been observed to date (in 8 months).

The stiffness of the composite was reduced by increase in the ash content. Comparison of the stress-strain relationships for the various mix proportions tested is shown in Figures 6–8.

CONCLUSIONS

1. MSW incinerator ash used in this study is a relatively light and nonuniform (high-C_u) material with a specific gravity slightly lower than that of New Jersey coal fly ash.
2. MSW incinerator ash used in this study has somewhat lower optimum moisture content and higher maximum dry density than those of New Jersey coal fly ash.
3. Compressive and tensile strengths of high-volume MSW ash (>40 percent ash by weight) low cement (<6 percent by weight)–sand composites are in general lower than those of coal fly ash mixes and decrease significantly with increasing ash or sand content. High-volume MSW ash mixes are not recommended for practical use because of the deterioration of the composite.
4. Compressive and tensile strengths of low-volume MSW ash (<40 percent ash), relatively high-volume (25 percent by

TABLE 5 EFFECT OF WATER-CEMENT RATIO ON THE STRENGTH OF MSW INCINERATOR CEMENT-SAND COMPOSITE

Water-Cement Ratio	Compressive Strength (psi)	Tensile Strength (psi)
0.6	166	28
0.45	276	51.3

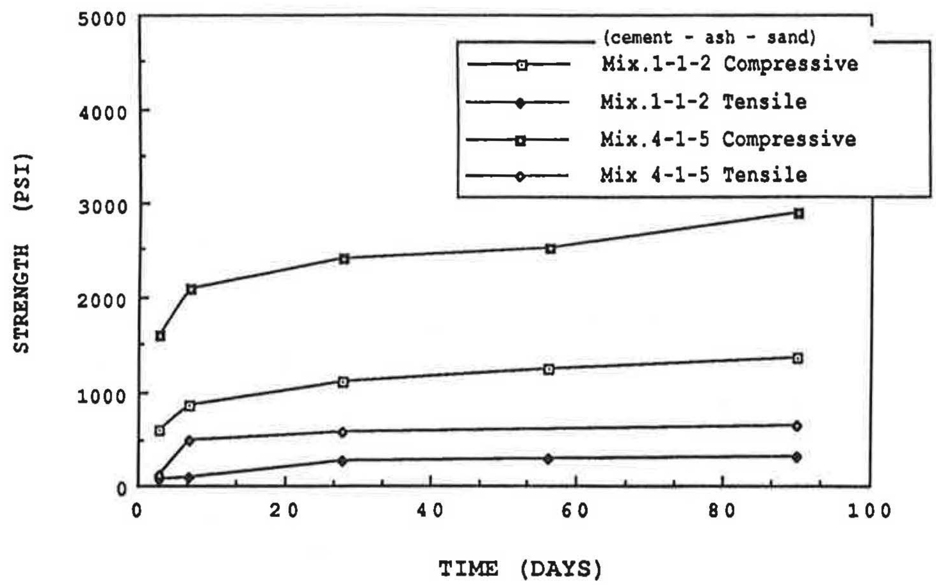

FIGURE 5 Strength of low-volume MSW ash, high-cement-content–sand composites.

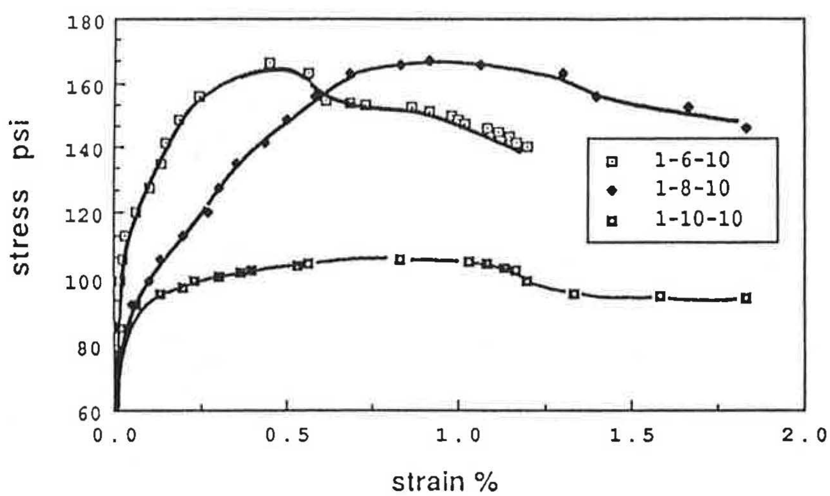

FIGURE 6 Variation in the stress-strain relationship of the composites as a function of MSW ash content (1 part cement and 10 parts sand).

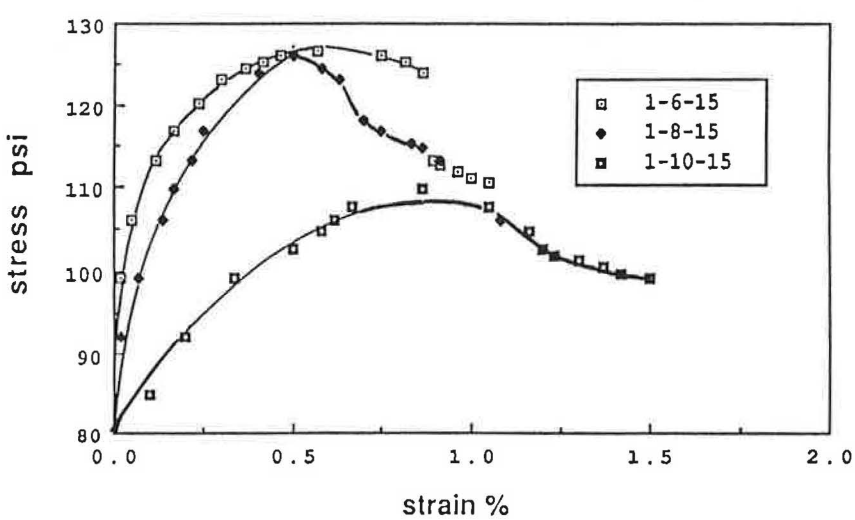

FIGURE 7 Variation in the stress-strain relationship of the composites as a function of MSW ash content (1 part cement and 15 parts sand).

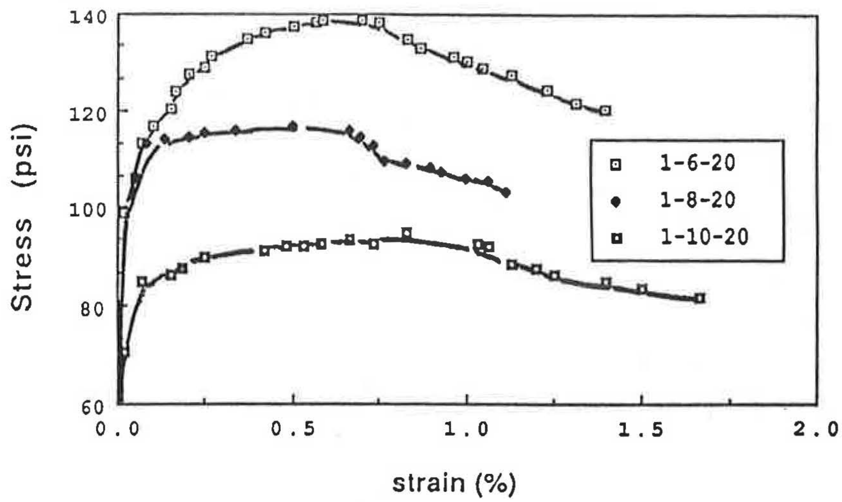

FIGURE 8 Variation in the stress-strain relationship of the composites as a function of MSW ash content (1 part cement and 20 parts sand).

weight) cement-sand composites compare well with those of typical soil-cement mixtures and are in general higher than those of strongly cemented soils. The mix proportion 1-1-2, which corresponds to 25 percent cement, 25 percent ash, and 50 percent sand, is a stable mix with no deterioration.

5. Use of MSW incinerator ash for construction will require a proper specification for mix proportion. To design optimum mix proportions for a particular application, two factors should be considered: first, mix proportions, particularly the cement content, should be such that no deterioration shall take place; and second, mix proportions should meet the requirements for compressive and tensile strength.

6. Stiffness of the composite decreases with increase in ash content irrespective of the mixture proportions.

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Determining Cement Content of Soil-Cement by Heat of Neutralization

ROBERT SCAVUZZO

A heat of neutralization test method (as modified by the Bureau of Reclamation) for determining cement content of freshly mixed soil-cement is discussed. Once a calibration curve is established, a cement content determination can be made in approximately 15 min, which in field applications has been found to be accurate to within ±1 percentage point of actual cement content. A sodium acetate–glacial acetic acid buffer solution is used to initiate an exothermic reaction with the calcium hydroxide in the soil-cement test specimen. During construction, test specimens are obtained directly from the spreader as soil-cement is placed, and separation of plus No. 4 (4.75-mm) sieve size material is not required. Results from an interlaboratory testing program were analyzed to establish precision and bias of the suggested test method.

In 1988, the Bureau of Reclamation was placing soil-cement for slope stabilization at Jackson Lake Dam, Wyoming. Because of gradation characteristics of borrow material, a soil-cement mix design having 50 percent plus No. 4 (4.75-mm) sieve size material was the most economical mixture. There was concern that typical titration methods for determining cement content of soil-cement for construction control would be difficult to use and too slow to perform. Titration methods typically require separating the plus No. 4 (4.75-mm) sieve size material and include the time-consuming task of screening and washing the soil-cement mix to obtain test specimens of appropriate size and gradation characteristics. This circumstance prompted initiation of a program to develop a test method for determining cement content of soil-cement with the following objectives: (a) separation of plus No. 4 (4.75-mm) sieve size material not required for obtaining test specimens; (b) cement content determinations in a timely manner, i.e., within 15 to 20 min; (c) durable apparatus for field use; (d) ease of performing test method by field personnel; and (e) accuracy of ±1 percentage point of actual cement content for soil-cement mix designs having cement contents of 3 to 15 percent and up to 50 percent plus No. 4 (4.75-mm) sieve size material.

TEST METHOD Q116B–1978

Test methods used for determining cement content of soil-cement mixtures were reviewed and evaluated against the five requirements. Test Method Q116B–1978, *Cement Content of Cement Treated Materials (Heat of Neutralization)* (*1*), was the test method that came closest to satisfying the five requirements.

In the test method, an acidic buffer solution of glacial acetic acid and sodium acetate with an initial pH of approximately 2 is used to react with the calcium hydroxide in a soil-cement test specimen. This reaction is exothermic and the heat produced is proportional to the quantity of cement present in the test specimen. The nature of this proportionality is linear, which makes it conducive to establishing a calibration curve and determining a corresponding line equation for a given material. Soil-cement test specimens of unknown cement contents can then be tested by obtaining the heat of neutralization, and the cement content can be determined from the calibration curve. Heat of neutralization is defined as the temperature increase resulting from the exothermic reaction in the acidic buffer solution when mixed with a soil-cement test specimen.

The Australian test method specified that a 5-kg soil-cement test specimen be mixed with 1 L of buffer solution consisting of 150 g of sodium acetate, 240 g of glacial acetic acid, and distilled water. A calibration curve is prepared using seven different percentages of cement contents that bracket the cement content specified in the mix design, i.e., if the target figure is 2 percent cement, percentages of cement of 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, and 3.5 are used. Calibration test specimens are prepared at the water content to be used during soil-cement placement. Duplicate heat of neutralization tests are performed at each of the seven cement contents. Temperature increase corresponding to each of the 14 tests is plotted versus cement content and a calibration line is established. Subsequent soil-cement test specimens of unknown cement contents are tested, the heat increase obtained is evaluated using calibration data, and the cement content is determined.

RECLAMATION PROCEDURAL CHANGES

Test Method Q116B–1978 was originally developed for soil-cement mix designs having cement contents of 3.5 percent or less. At cement contents greater than 3.5 percent, the mixture of 1 L of buffer solution with a 5-kg test specimen gels into a solid mass, preventing proper mixing. Recommendations by the Main Roads Department for cement contents greater than 3.5 percent were (a) add additional buffer solution, up to 3 L, to the 5-kg soil-cement test specimen; (b) adjust the mass ratio of buffer solution to test specimen from 1 L to 5 kg for cement contents less than 3.5 percent, to 2.5 L to 4 kg for cement contents between 6.0 and 7.5 percent. It was also stated that any required reduction of test specimen mass should

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be kept to a minimum and under no circumstances should the soil-cement test specimen be less than 4 kg.

After performing a series of tests, the Bureau of Reclamation made the following changes to the original Australian heat of neutralization test method: (a) reduce the number of cement contents required for obtaining a calibration curve from seven to three; (b) perform three heat of neutralization tests at each of the three cement content values selected, to construct a calibration curve using 9 data points, reduced from 14; and (c) use 1.5 L of buffer solution mixed with a 1.5-kg soil-cement test specimen for mix designs consisting of up to 15 percent cement and 50 percent plus No. 4 (4.75-mm) sieve size material.

INTERLABORATORY TEST PROGRAM

After the changes were incorporated into the test method, an interlaboratory test program was performed in accordance with ASTM C802–87, *Standard Practice for Conducting an Interlaboratory Test Program to Determine the Precision of Test Methods for Construction Materials.* This test program was performed to validate the procedural changes outlined, solicit user comments on the revised heat of neutralization test procedure, and provide reliable information from which precision statements of the type prescribed in ASTM C670–88, *Standard Practice for Preparing Precision and Bias Statements for Test Methods for Construction Materials*, could be developed.

The interlaboratory test program was performed on two soil-cement mix designs in two separate phases. Phase I consisted of testing soil-cement specimens of a fine-grained mix design and Phase II used a coarse-grained mix design. Fine- and coarse-grained mix designs were selected to determine if the changes made to the test method were effective both on a fine-grained mix and on one in which a significant percentage of plus No. 4 (4.75-mm) sieve size material was present. The test method changes investigated were (a) reducing the number of cement contents and calibration test specimens required to construct a calibration curve, and (b) reducing the test specimen size from 5 to 1.5 kg.

Testing laboratories from federal, state, and private sectors participated in the test program. Ten testing laboratories participated in Phase I of the test program and eight testing laboratories participated in Phase II. Participating laboratories were provided with a written procedure and individual test specimens of appropriate soil mass, each with a corresponding amount of cement. Each laboratory was required to thoroughly mix the soil, cement, and water, as specified, for each test specimen before performing the heat of neutralization test. A moisture content of 8 percent was specified for each test specimen.

Phase I

For the first phase of the test program, each participating laboratory performed a total of 15 cement content determinations on soil-cement test specimens. The specimens consisted of 5 percent minus ⅜-in. and 95 percent minus No. 4 (4.75-mm) sieve size material. Nine determinations were performed on test specimens of known cement contents—three at 5, three at 7, and three at 9 percent cement—to develop a calibration curve and determine a calibration line equation. The remaining six cement content determinations were performed on duplicate test specimens designated for each laboratory as Test Specimens A, B, and C, but of cement content unknown to the participating laboratories. Test Specimens A were prepared at 6 percent cement, B at 7 percent cement, and C at 8 percent cement.

Phase II

The purpose of the second phase of the interlaboratory test program was to determine the effect that a significant percentage of plus No. 4 (4.75-mm) sieve size material would have on the reliability of the revised heat of neutralization procedure in determining cement content.

Phase II of the test program consisted of the same number of test specimens at the same cement contents as in Phase I. The gradation of Phase II test specimens was as follows:

Material Size	*Percent Retained*
¾ to 1½ in.	14
⅜ to ¾ in.	20
No. 4 to ⅜ in.	16
Minus No. 4	50

Calibration Test Specimen Results

Results obtained from Phase I calibration test specimens (5 percent plus No. 4) are shown in Figure 1. Statistical analysis performed on data from the 90 heat of neutralization tests, 30 at each of the three cement contents, resulted in a correlation coefficient of 0.981 with a corresponding R^2 value of 96 percent. The solid line shown in Figure 1 represents the best-fit line obtained performing a linear regression through all data points. The narrow set of dashed lines indicates the 95 percent confidence limit within which the average value of all data points would fall. The wider set of dashed lines indicates the 95 percent confidence limit within which any data point would fall.

Results obtained from Phase II calibration test specimens (50 percent plus No. 4) are shown in Figure 2. Statistical analysis performed on data from the 72 heat of neutralization tests, 24 at each of the three cement contents, resulted in a correlation coefficient of 0.974 with a corresponding R^2 value of 95 percent. The solid, narrow, and wide sets of dashed lines represent the best-fit line, 95 percent confidence limits for average data points, and 95 percent confidence limits for any data point, respectively, as described for Phase I calibration test specimen results.

Tables 1 and 2 present summaries of Phase I and Phase II calibration test specimen results, respectively. Average, maximum, and minimum values of temperature increase in degrees Celsius, as well as calculated within- and between-laboratory standard deviations, are provided.

Test Specimen Results

Interlaboratory test program results for Test Specimens A, B, and C (5 percent plus No. 4) and Test Specimens D, E,

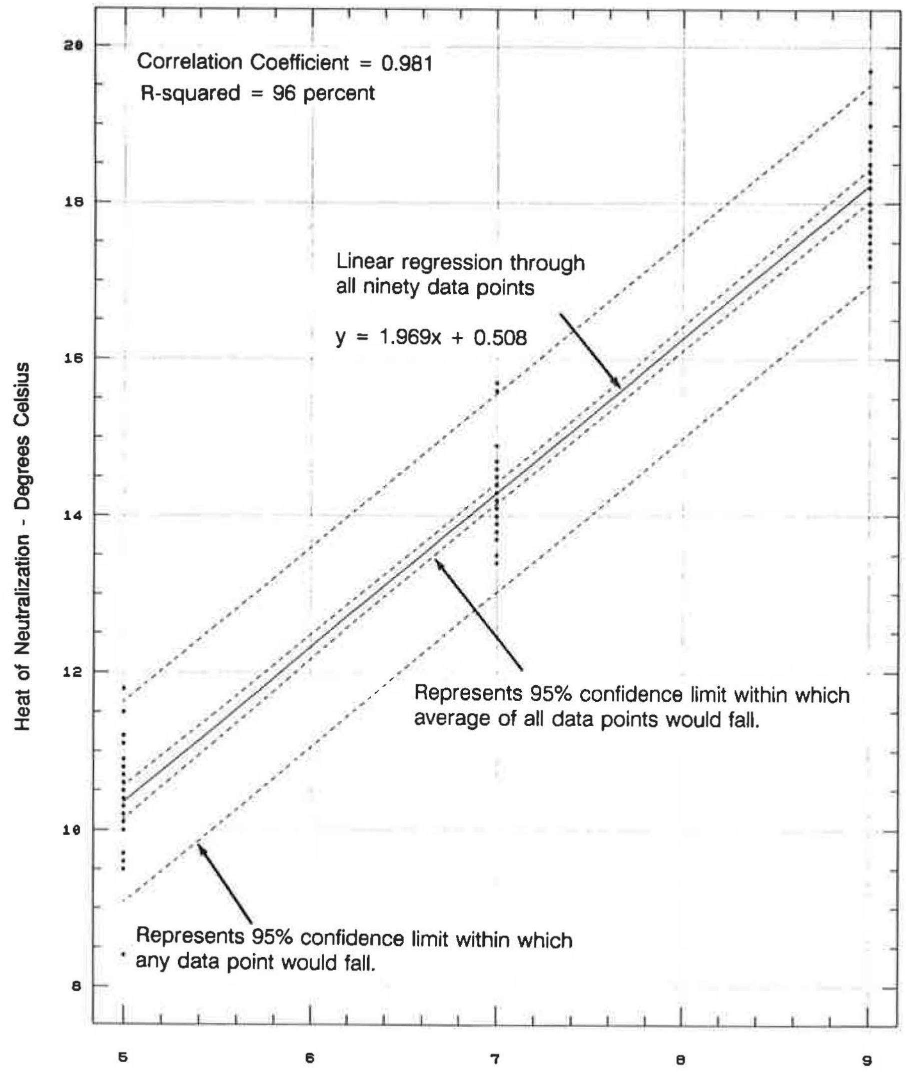

FIGURE 1 Phase I calibration test specimens.

and F (50 percent plus No. 4) are presented in Tables 3 and 4, respectively. Average, maximum, and minimum values of cement content, as well as calculated within- and between-laboratory standard deviations, are provided.

Precision and Bias Statements

Results obtained from the interlaboratory test program were used to develop precision and bias statements as prescribed in ASTM C670–88 for the heat of neutralization test procedure as revised by the Bureau of Reclamation. Table 5 presents a summary of single-operator and multilaboratory precision and bias values obtained for the calibration test specimens. Values shown in Rows 2, 3, and 5 of Table 5 are calculated in accordance with ASTM C670–88 and are in degrees Celsius.

Table 6 presents a summary of single-operator and multilaboratory precision and bias values obtained for Test Specimens A, B, C (5 percent plus No. 4) and Test Specimens D, E, F (50 percent plus No. 4). Values shown in Rows 2, 3, and 5 of Table 6 are calculated in accordance with ASTM C670–88 and are in percentage points of cement content.

From the single-operator and multilaboratory precision and bias values presented in Tables 5 and 6, precision statements can be written for specimens containing from 3 to 15 percent cement and up to 50 percent plus No. 4 (4.75-mm) sieve size material as described in the following section.

CALIBRATION TEST SPECIMENS

Single-Operator Precision

The single-operator standard deviation of a single test result (a test result defined in this procedure as the average of three separate measurements) has been found to be 0.5°C. Therefore, results of two properly conducted tests by the same operator (each consisting of the average of three calibration test specimens of the same cement content) should not differ

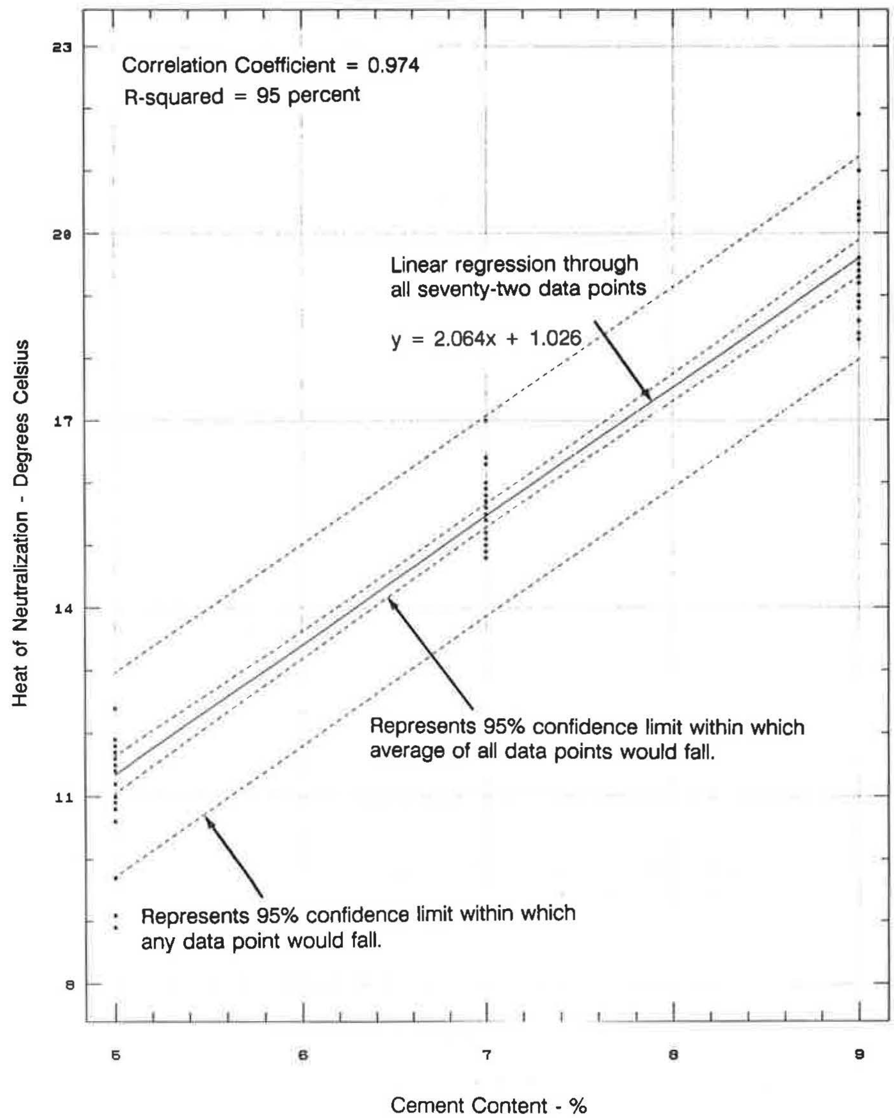

FIGURE 2 Phase II calibration test specimens.

TABLE 1 SUMMARY OF INTERLABORATORY TEST PROGRAM FOR 5 PERCENT PLUS NO. 4 CALIBRATION TEST SPECIMENS

Cement Content %	Temperature Rise Average °C	Maximum °C	Minimum °C	Standard Deviation within-lab	between-lab
5	10.4	11.8	8.4	0.512	0.659
7	14.3	15.7	13.4	0.321	0.613
9	18.2	19.7	17.2	0.338	0.684
			Average:	0.390	0.652

TABLE 2 SUMMARY OF INTERLABORATORY TEST PROGRAM FOR 50 PERCENT PLUS NO. 4 CALIBRATION TEST SPECIMENS

Cement Content %	Temperature Rise Average °C	Maximum °C	Minimum °C	Standard Deviation within-lab	between-lab
5	11.2	12.4	8.9	0.573	0.912
7	15.7	17.0	14.8	0.423	0.603
9	19.5	21.9	18.3	0.585	0.879
			Average:	0.527	0.798

TABLE 3 SUMMARY OF INTERLABORATORY TEST PROGRAM FOR 5 PERCENT PLUS NO. 4 TEST SPECIMENS A, B, AND C

Test Specimen	Cement Content Average %	Cement Content Maximum %	Cement Content Minimum %	Standard Deviation within-lab	Standard Deviation between-lab
A	6.0	6.4	5.7	0.164	0.232
B	7.1	7.5	6.4	0.130	0.313
C	8.2	8.6	7.7	0.130	0.235
			Average:	0.141	0.260

TABLE 4 SUMMARY OF INTERLABORATORY TEST PROGRAM FOR 50 PERCENT PLUS NO. 4 TEST SPECIMENS D, E, AND F

Test Specimen	Cement Content Average %	Cement Content Maximum %	Cement Content Minimum %	Standard Deviation within-lab	Standard Deviation between-lab
D	5.9	6.5	5.3	0.095	0.298
E	7.1	7.4	6.5	0.167	0.270
F	8.0	8.5	7.8	0.126	0.173
			Average:	0.129	0.247

TABLE 5 SUMMARY OF INTERLABORATORY TEST PROGRAM FOR PRECISION DETERMINATION OF CALIBRATION TEST SPECIMENS

		Phase I 5% plus No. 4	Phase II 50% plus No. 4
Single-Operator	Standard deviation of a single test result [1]	0.39 [4]	0.53
Single-Operator	Maximum allowable difference between two test results [2]	1.1	1.5
Single-Operator	Maximum allowable difference between the highest and lowest individual temperature determinations [3]	2.2	3.0
Multilaboratory	Standard deviation of a single test result [1]	0.65	0.80
Multilaboratory	Maximum allowable difference between two test results from different laboratories	1.8	2.3

[1] A test result is defined as the average of three separate measurements

[2] Each consisting of the average of three calibration test specimens at the same cement content

[3] The three individual temperature determinations used in calculating a test result

[4] Numbers in table are in degrees Celsius

TABLE 6 SUMMARY OF INTERLABORATORY TEST PROGRAM FOR PRECISION DETERMINATION OF TEST SPECIMENS

		Phase I - 5% plus No. 4 Test Specimens A, B, and C	Phase II - 50% plus No. 4 Test Specimen D, E, and F
Single Operator	Standard deviation of a single test result [1]	0.14 [4]	0.13
Single Operator	Maximum allowable difference between two test results [2]	0.40	0.37
Single Operator	Maximum allowable difference between the highest and lowest cement content determinations [3]	0.55	0.51
Multilaboratory	Standard deviation of a single test result [1]	0.26	0.25
Multilaboratory	Maximum allowable difference between two test results from different laboratories	0.74	0.71

[1] A test result is defined as the average of two separate measurements

[2] Each consisting of the average of two cement content determinations

[3] The two cement content determinations used in calculating a test result

[4] Numbers in table are in percentage points of cement content

by more than 1.5°C. The range (difference between the highest and lowest) of the three individual temperatures used in calculating a test result should not exceed 3.0°C.

Multilaboratory Precision

The multilaboratory standard deviation of a single test result (a test result is defined in this procedure as the average of three separate measurements) has been found to be 0.8°C. Therefore, results of two properly conducted tests in different laboratories on the same soil-cement mix should not differ by more than 2.3°C.

TEST SPECIMENS

Single-Operator Precision

The single-operator standard deviation of a single test result (a test result is defined in this procedure as the average of two separate measurements) has been found to be 0.14 of a percentage point of cement content. Therefore, results of two properly conducted tests by the same operator (each consisting of the average of two cement content determinations) should not exceed 0.40 percentage points of cement content. The range (difference between the highest and the lowest) of the two individual cement content determinations used in calculating a test result should not exceed 0.55 percentage point of cement content.

Multilaboratory Precision

The multilaboratory standard deviation of a single test result (a test result is defined in this procedure as the average of two separate measurements) has been found to be 0.26 of a percentage point of cement content. Therefore, results of two properly conducted tests in different laboratories on the same soil-cement mix should not differ by more than 0.74 percentage point of cement content.

CONCLUSIONS

Results of the interlaboratory test program indicate that changes made to Test Method Q116B–1978 by The Bureau of Reclamation produce single-operator and multilaboratory precision values that are well within acceptable limits for producing a high-quality end product. The Bureau of Reclamation's revised heat of neutralization test method can be used to accurately and quickly determine the cement content of freshly mixed soil-cement containing from 3 to 15 percent cement and up to 50 percent plus No. 4 (4.75-mm) sieve size material. The test method does not require separation of the plus No. 4 (4.75-mm) sieve size material for obtaining test specimens, and cement content determinations can be made in 15 to 20 min. Required apparatus is durable for field use, the test procedure is easily performed by field personnel, and in field applications performed to date the method is accurate to within ±1 percentage point of actual cement content.

A written test procedure has been prepared for use on Bureau of Reclamation projects. The written test method will soon be submitted to ASTM for approval and publication through its normal consensus standards procedure.

REFERENCE

1. *Cement Content of Cement Treated Materials (Heat of Neutralization).* Test Method Q116B–1978, Main Roads Department, Queensland, Brisbane, Australia, 1978.

Publication of this paper sponsored by Committee on Soil–Portland Cement Stabilization.

Material Characterization and Inherent Variation Analysis of Soil-Cement Field Cores

William O. Hadley

A knowledge of the variation in fundamental engineering properties of the various construction materials is essential for a comprehensive evaluation of the performance of various road sections. Support is provided for the Louisiana Experimental Base Project, an in-service experimental road project constructed to aid in an evaluation of design performance characteristics of a number of experimental test sections. The expected variation in the static and resilient (fatigue) properties of the materials that make up the layers of the pavement structure can provide an inherent variation data base from which continuing evaluation and analysis of pavement behavior and performance of these layers can be undertaken. Results are presented of material characterization and inherent analysis of a resilient (fatigue) test program undertaken to establish the magnitude, scope, and expected variation in fundamental engineering properties of laboratory-prepared specimens and field cores of the cement-stabilized materials used in the base and subbase layers of some of the test sections of the Base Project. Variation analyses were completed for such fundamental properties as modulus, Poisson's ratio, tensile stress, tensile strain, and fatigue cycles to failure. Regression analysis techniques were also used to quantify those factors that significantly affect the fatigue life of the various construction materials used in the Base Project. This information, when combined with the in-service performance results from the Base Project, should produce a better knowledge of the important mix variables affecting the fundamental engineering performance properties, which should result in changes and improvements in quality control measures.

Knowledge of the magnitude, scope, and expected variation in the fundamental engineering properties of the various construction materials used in pavement structural sections is essential for a comprehensive evaluation of the performance of roadway sections. Support is provided to the Louisiana Experimental Base Project, an in-service experimental road project constructed to aid in an evaluation of design-performance characteristics of a number of experimental test sections.

A repetitive (fatigue) testing program was undertaken (see Table 1) to establish the magnitude and scope of inherent variation in the fundamental resilient properties of field cores representative of in-service conditions and to develop material characterization information from field cores and laboratory specimens (see Table 2) of the three types of cement-stabilized base materials (sandy soil, sandy loam, and sand-clay-gravel) used in the Base Project.

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Regression analysis techniques were used to quantify those factors that significantly affect the fatigue life of the various construction materials used in the Base Project. This information, when combined with the in-service performance results from the Base Project, should produce a better knowledge of the important mix variables affecting the fundamental engineering performance properties and could lead to changes and improvements in quality control measures.

The project is situated on a portion of US-71–167 that accommodates a moderate volume of mixed vehicular traffic. To ensure that the flow of traffic would not be affected by its experimental status, the Base Project was completed as a part of a construction project upgrading US-71–167 to a four-lane facility.

The terrain at the Base Project is generally flat with poor drainage. The subgrade material is basically a fine-grained soil ranging from a silty clay loam to a heavy clay. The range in mean ambient air temperatures is from approximately 39°F (40°C) to 84°F (29°C); the mean annual rainfall is approximately 55 to 60 in. (140 to 150 cm).

The projected average daily traffic at the time of construction was 7,990 vehicles, including approximately 15 percent trucks. All test sections were included in a portion of a newly constructed two-lane roadway adjacent to an existing two-lane highway.

The Base Project consisted of 18 test sections—14 experimental sections and 4 control sections (see Figure 1). The factors investigated in the project included three different base types, four different pavement design lives, and two surface thicknesses. The Control Sections C1 through C4 and Test Sections 2, 4, 6, 8, 9, 10, 12, and 13 all included soil-cement base and subbase layers.

Each test section is approximately 550 ft (168 m) long with a 50-ft (15-m) transition zone interconnecting each adjacent test section. The randomization scheme for locating the various test sections included a complete randomization of the 10- and 15-year design sections, but limited randomization of the 5-year sections. The latter sections were grouped together to allow for maintenance of the 5-year design sections at the same time. Detailed information on the construction of the Base Project is available from the Louisiana Department of Transportation and Development (LA DOTD).

The coring program established for soil-cement base test sections of the Base Project is presented in Table 3. The program was structured to provide for an investigation of the variability in material properties of the various pavement and materials used throughout the Base Project. The coring plan

TABLE 1 RESILIENT (FATIGUE) TESTING PROGRAM—SOIL-CEMENT FIELD CORES

PAVEMENT LAYER	TEST TYPE	VARIATION EVALUATED	FUNDAMENTAL MATERIAL PROPERTIES ESTIMATES
Soil Cement	Repetitive	Longitudinal - 10' spacing	N_f, Cycles to Failure
Bases and	Indirect	Longitudinal - 1' spacing	E_r, Resilient Modulus
Cement Stabilized	Tensile	Lateral - 4' spacing	μ_r, Resilient Poisson's Ratio
Sand-Clay-Gravel	Test	Lateral - 1' spacing	ϵ_r, Resilient Tensile Strain
Base		Depth - vertical	S_r, Resilient Applied Tensile
		Stress levels -	Stress
		sandy soil 45 to 75 psi	
		sandy loam 35 to 65 psi	
		sand-clay-gravel 20 to	
		50 psi	

allowed for variational analysis in the longitudinal (along the road), lateral (across the road), and vertical (depth into pavement) directions. In addition, the plan included various spacings of the coring locations to provide for evaluation of inherent variation within close spacings (±1 ft) as well as larger spacings (±10 ft).

The fundamental engineering properties investigated in this phase included resilient modulus, resilient Poisson's ratio, and cycles to failure (Log N_f). Because the specimens were field cores, the only controllable variable that could actually be varied was the tensile stress repeatedly applied to the specimens during the fatigue test. Properties associated with the cores, such as percent clay balls, percent flushing (concentration or migration of cement to crack or flaw), and total percent flaws, could only be measured sample by sample and could not be established as fixed values. All tests were conducted at a test temperature of 75°F (24°C) and at 1 Hz (sinusoidal waveform).

CONSTRUCTION CONSIDERATIONS

The soil-cement construction procedure used in the construction of the Base Project apparently does not provide a uniform

TABLE 2 RESILIENT (FATIGUE) TESTING PROGRAM—LABORATORY-PREPARED SOIL-CEMENT SPECIMENS

PAVEMENT LAYER	TEST TYPE		FACTORS EVALUATION TYPE EVALUATED	FUNDAMENTAL MATERIAL CHARACTERISTICS ESTIMATES
Soil Cement	Repetitive	Extended mix cond.	3.6-0.9% sandy soil	N_f, Cycles to Failure
Base:	Indirect	a) Cement	5.3-8.8% sandy loam	E_r, Resilient Modulus
Sandy soil	Tensile	content	3.4-6.6% S/C/G	μ_r, Resilient Poisson's Ratio
Sandy loam	Test	b) Moisture	10.9-17.9% sandy soil	ϵ_r, Resilient Tensile Strain
Sand-Clay-		content	12.0-19.0% sandy loam	S_r, Resilient Applied Tensile
Gravel			12.5-17.3% S/C/G	Stress
		c) Age at test	11 to 365 days	
		d) Delay	0 to 240 minutes	
		e) Stress level	73 to 87% of static strength	

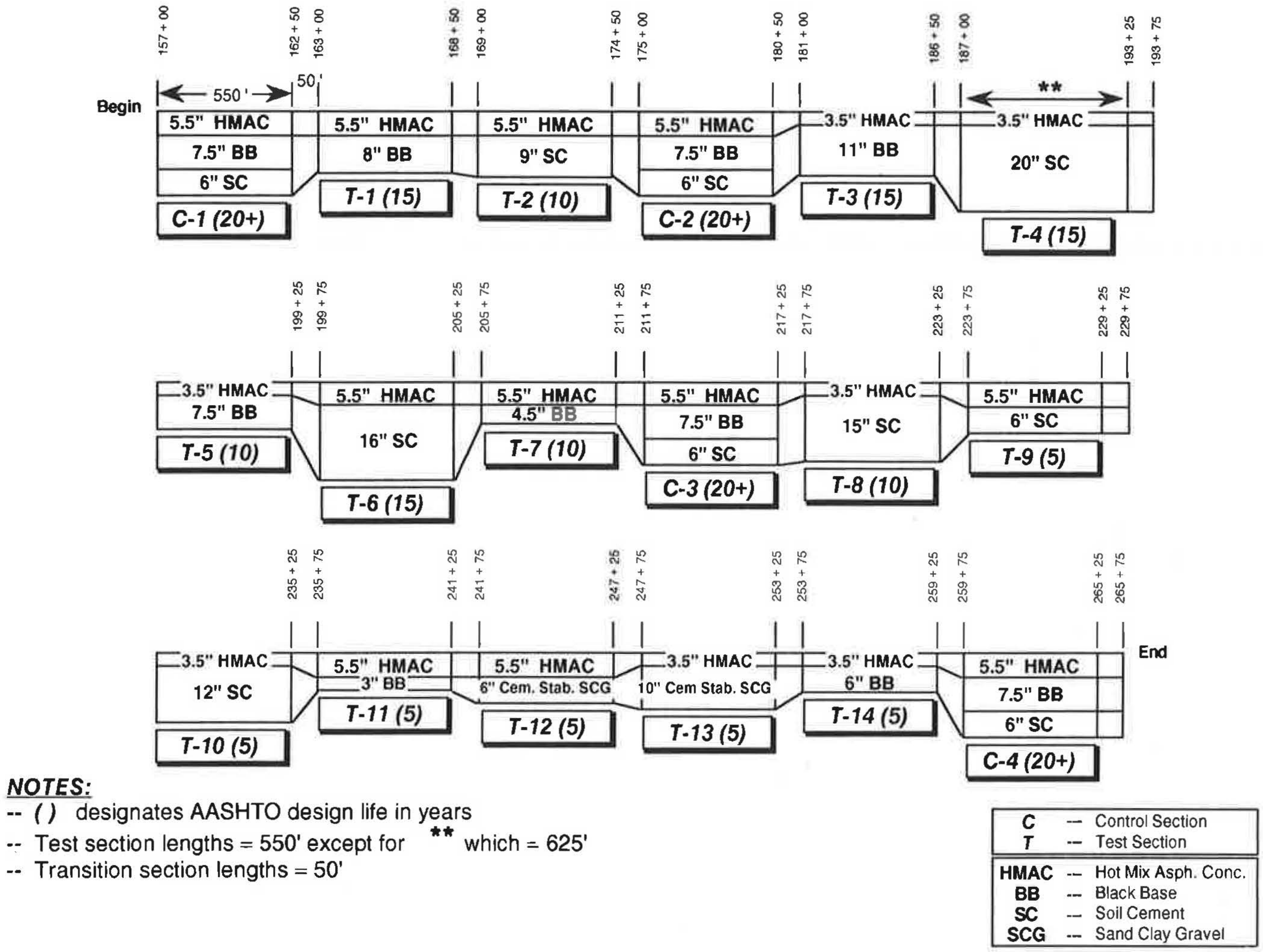

FIGURE 1 Experimental base project layout.

product; in fact the procedure apparently resulted in a number of flaws in the various cement-stabilized base layers. Evidence of this condition can be drawn from the initial LA DOTD coring operation established to obtain 7 and 28 days' cores for job verification purposes. During this coring operation, only 60 intact cores could be obtained from 187 different boring operations (a recovery rate of 32 percent).

Similar results were obtained in the coring program presented in Table 1 where 65 good test specimens were obtained from a total of 154 test specimens (a recovery rate of 43 percent). From this information, it can be postulated that over one-half of soil-cement areas would have internal flaws. Some flaws encountered in the cement-stabilized material include lamination, cracks, compaction planes, cutter planes, flushing (migration of cement to flawed areas), and clay balls. As a result of this condition, the field cores were separated into two groups. One group, called "clear specimens" (having minimum flaws), is representative of those specimens with the percent of cross section of specimen composed of flaws less than approximately 20 percent. The other group, called "flawed specimens," represented those specimens with a percent flawed area exceeding approximately 20 percent. In most cases, the flaws in the cores could not be observed by the naked eye but would become apparent after only a few cycles of the fatigue-resilient test.

INHERENT VARIATION ANALYSIS OF FIELD CORES

The inherent variation information for the clear and flawed specimens is presented in Table 4. In these data, the cement content variability as well as modulus and Poisson's ratio variation are similar for the clear and flawed specimens. In addition, the repeated applied stress and resulting tensile strain are also similar in magnitude. The principal differences, of course, are in the amount of flushing, clay balls, total flaws (percent flushing plus percent clay balls), and, more important, Log N_f (fatigue). There are obvious significant differences between clear and flawed specimens.

The inherent variation information associated with longitudinal, lateral, and vertical directions is presented in Tables 5 and 6 for clear and flawed specimens, respectively. The combined inherent variation is given in the bottom row of each table. Statistical F tests indicated, as expected, a significant difference in variances for resilient modulus, resilient

TABLE 3 RANDOMIZED BORING PLAN—LOUISIANA EXPERIMENTAL BASE PROJECT: SOIL-CEMENT SUBBASES AND BASES

SECTION DESIGNATION	DESIGN LIFE YRS	SAMPLING DIRECTION AND GROUPING	STATION	LANE	COMMENTS
Control 1	20	longitudinal	160 + 24	outside	outside
		within	160 + 25		wheel path
			160 + 26		
Test 1	15	longitudinal	166 + 74	inside	outside
		within	166 + 75		wheel path
			166 + 76		
Test 2	10	longitudinal	171 + 15	outside	outside
		among	171 + 25		wheel path
			171 + 35		
Control 2	20	longitudinal	178 + 15	outside	outside
		among	178 + 25		wheel path
			178 + 35		
Test 4	15	lateral	192 + 25	outside	9' Rt of CL
		within	192 + 25		10' Rt of CL
			192 + 25		11' Rt of CL
Test 6	15	lateral	202 + 00	inside	2' Lt of CL
		among	202 + 00		6' Lt of CL
			202 + 00		10' Lt of CL
Control 3	20	lateral	213 + 50	inside	2' Lt of CL
		among	213 + 50		6' Lt of CL
			213 + 50		10' Lt of CL
Control 3 (cont.)	20	lateral	213 + 50	outside	2' Rt of CL
		among	213 + 50		6' Rt of CL
			213 + 50		10' RT of CL
Test 8	10	longitudinal	221 + 40	outside	outside
		among	221 + 50		wheel path
			221 + 60		
Test 9	5	longitudinal	227 + 99	inside	outside
		within	228 + 00		wheel path
			228 + 01		
Test 10	5	longitudinal	231 + 49	outside	outside
		within	231 + 50		wheel path
			231 + 51		
Test 12	5	lateral	245 + 00	outside	2' Rt of CL
		among	245 + 00		6' Rt of CL
			245 + 00		10' Rt of CL
Test 13	5	longitudinal	251 + 99	outside	outside
		within	252 + 00		wheel path
			252 + 01		
Control 4	20	lateral	261 + 50	outside	9' Rt of CL
		within	261 + 50		10' Rt of CL
			261 + 50		11' Rt of CL

TABLE 4 INHERENT VARIATION DATA FOR CEMENT-STABILIZED FIELD CORES, CLEAR AND FLAWED SPECIMENS

	FLAWED SPECIMENS (69 SPECIMENS)			CLEAR SPECIMENS (65 SPECIMENS)			
VARIABLE	MEAN VALUE	STANDARD DEVIATION	COEFFICIENT* OF VARIATION, %	MEAN VALUE	STANDARD DEVIATION	COEFFICIENT OF VARIATION	RANGE
Cement, %	8.93	1.00	11.25	9.14	1.00	10.92	8 - 10
E, 10^5 psi	4.7817	2.0421	42.7	5.4561	1.8829	34.5	0.54 - 9.687
μ	0.2686	0.1053	39.2	0.2739	0.1062	38.8	0.107 - 0.500
ϵ_{tm}, μin/in	774.36	658.43	85.0	623.48	487.85	78.2	256 - 4073
STS	121.75	18.99	15.6	123.15	15.52	12.6	79.3 - 155.5
Flushing, % Area	41.4	33.9	81.9	4.9	10.9	222.5	0 - 60
Clay balls, % Area	4.7	8.1	171.9	4.6	4.6	100.0	0 - 22
Flaws, % Area	46.1	35.5	77.0	9.5	11.7	122.7	0 - 69
Log N_f	1.8892	1.151	60.9	4.4356	1.7283	39.0	

E - Resilient modulus

ϵ_{tm} - Resilient tensile strain

μ - Resilient Poissons Ratio

STS - Applied tensile stress psi

Log (N_f) - Logarithmn (Cycles to Failure)

$$* \text{ Coefficient of Variation} = \frac{\text{Standard Deviation}}{\text{Mean}} \times 100$$

Poisson's ratio, and Log N_f. On the other hand, there were no significant differences in the mean values of resilient modulus and Poisson's ratio for the clear and flawed specimens. As indicated previously, there was a significant difference in the fatigue life of clear and flawed specimens. In terms of cycles to failure at a given applied stress (e.g., 122-psi tensile stress), the clear specimen on the average exhibited a life of 27,300 cycles, whereas the flawed specimen exhibited a life of 77 cycles. These values represent a great difference in the fatigue lives of the clear and flawed specimens. This fact, combined with low recovery of good cores from the cement-stabilized base layers, leads to the conclusion that there are two levels (or populations, in statistical terms) of base materials that have the same basic behavioral response (i.e., E and μ are similar) but drastically different performance characteristics (i.e., consideration of fatigue in producing the fracture distress mode).

Comparisons between field core results for the cement-stabilized sandy soil (Type A) and sandy loam (Type B) base materials were completed for the clear specimen group (Table 7) to ascertain whether or not there were significant differences in the means and variances of the three engineering properties. If there were no significant differences, then the results could be pooled and any subsequent use of the data would be simplified.

Table 7 indicated that there were no significant differences in either the variances or means for the two base materials. These results were combined to provide fundamental engineering property estimates compatible with the cement-stabilized sandy soil and sandy loam base materials. From these analyses the following information was developed:

Property	*Mean Value*	*Standard Deviation*	*Degrees of Freedom*
Resilient modulus, 10^5 psi	5.457	1.909	64
Resilient Poisson's ratio	0.274	0.108	64

which could be applied to either the cement-stabilized sandy (Type A) or sandy loam (Type B) base materials. In addition, the fatigue results for these two material types could be combined because no significant difference was found in the fatigue life of these two materials.

TABLE 5 INHERENT VARIATION IN FUNDAMENTAL MATERIAL PROPERTIES FROM FATIGUE-RESILIENT INDIRECT TENSILE TEST RESULTS FOR SOIL-CEMENT FIELD CORES, CLEAR SPECIMENS

		RESILIENT MODULUS		RESILIENT POISSON'S RATIO		LOG OF CYCLES TO FAILURE	
TYPE VARIATION	DESIGN LEVEL	MEAN SQUARES	DEGREES FREEDOM	MEAN SQUARES	DEGREES FREEDOM	MEAN SQUARES	DEGREES FREEDOM
A - longitudinal	1,2	1.679945	1	0.000924	1	0.923435	1
direction - 1'	3						
spacing	4	5.912301	3	0.001804	3	0.763232	3
B - longitudinal	1,2	1.129505	1	0.001800	1	0.720361	1
direction - 10'	3						
spacing	4						
C - lateral	1,2	1.404053	8	0.005158	8	0.602582	8
direction (outside	3						
lane)-2' spacing	4						
D(O) - lateral	1,2	0.939809	3	0.010016	3	0.293517	3
direction (outside	3						
lane) 4' spacing	4						
D(i) - lateral	1,2	0.618001	4	0.000663	4	0.737277	4
direction (inside							
lane)-4' spacing							
H - longitudinal	1,2						
direction - spacing	3						
greater than 50'	4	2.598060	1	0.017672	1	0.254222	1
E - vertical	1,2	2.078457	11	0.008254	11	2.818147	1
	3	0.146395	2	0.000833	2	1.162119	2
	4	0.322401	3	0.004150	3	0.726061	3
Combined results		1.771980	37	0.005686	37	1.000291	37

TABLE 6 INHERENT VARIATION IN FUNDAMENTAL MATERIAL PROPERTIES FROM FATIGUE-RESILIENT INDIRECT TENSILE TEST RESULTS FOR SOIL-CEMENT FIELD CORES, FLAWED SPECIMENS

TYPE VARIATION	DESIGN LEVEL	RESILIENT MODULUS MEAN SQUARES	RESILIENT MODULUS DEGREES FREEDOM	RESILIENT POISSON'S RATIO MEAN SQUARES	RESILIENT POISSON'S RATIO DEGREES FREEDOM	LOG OF CYCLES TO FAILURE MEAN SQUARES	LOG OF CYCLES TO FAILURE DEGREES FREEDOM
A - longitudinal direction @ 1' spacing	4	1.635848	5	0.004797	5	0.785509	5
B - longitudinal direction @ 10' spacing	1,2	4.629924	1	0.000005	1	0.016989	1
	3	4.057256	9	0.012570	9	6.322710	9
C - lateral direction (outside lane)-2' spacing	4	2.609158	2	0.028428	2	0.038738	2
D(O) - lateral direction (outside lane)-4' spacing	4	0.013448	1	0.057970	1	0.113822	1
D(i) - lateral direction (inside lane)-4'spacing	1,2	4.835846	3	0.000311	3	0.477943	3
G - longitudinal direction @ 10-50' spacing	4	0.056785	1	0.000050	1	4.699919	1
H - longitudinal direction @ > 50'	1,2	0.154013	1	0.005000	1	0.195573	1
	4	6.399537	6	0.032275	6	4.727870	6
E - vertical	1,2	8.278179	10	0.013283	10	0.505976	10
	3	8.864214	4	0.010393	4	3.772197	4
	4	1.628167	6	0.006172	6	2.169734	6
Combined results		4.909988	49	0.011045	49	1.290111	49

LABORATORY SPECIMENS VERSUS FIELD CORES

A perplexing problem associated with experimental analysis and evaluation based on laboratory-prepared specimens is the uncertainty of the premise that laboratory results are applicable to field conditions. In order to obtain information concerning the existence of a correlation between field and laboratory core results, statistical comparisons were completed between the resilient properties of laboratory specimens and field cores for the combined results of cement-stabilized sandy soil and sandy loam base materials (Table 8).

Table 8 indicated that significant differences occurred for resilient modulus, Poisson's ratio, and particularly fatigue life. There may not have been a practical difference in the modulus values (4.711 versus 5.456 × 10^5 psi), but there were surely differences in Poisson's ratio (0.101 versus 0.274) and Log N_f (2.2103 versus 4.4356). Therefore, there are two separate groupings (or populations).

A subsequent comparison was made between the laboratory results and flawed field core results (Table 9) to check any correlation that may have existed. This comparison indicated that there were no significant differences in mean modulus values, whereas significant differences did exist in Poisson's ratio (0.101 versus 0.269) and Log N_f (2.2103 versus 1.8892). It is, however, believed that the difference between the Log N_f values is not of practical significance (162 cycles versus 77 cycles). Differences in ages at time of test between the field cores (approximately 2½ years) and laboratory specimens (1½ weeks to 1 year) could affect comparison of the results. However, this time difference may not be critical, and the laboratory fatigue-resilient data developed in this study are generally compatible with the results from the flawed core specimens.

Consequently, the laboratory fatigue data may well represent the majority of the soil-cement material in place at the Base Project. The low recovery rate of good field cores and the high moisture contents present in the cement-stabilized base layers are apparently correlated, as was found in the laboratory fatigue-resilient testing program for cement-stabilized sandy (Type A) and sandy loam (Type B) base materials.

Only a few fatigue tests were completed for field cores of the cement-stabilized sand-clay-gravel base material because few good core specimens could be extracted during the coring operation. Consequently, the resilient-fatigue results for laboratory-prepared specimens (Table 10) will have to be considered as representative of in-service conditions. When considering the earlier results, this premise seems reasonable.

ANALYSIS OF VARIANCE

The analysis of variance results for Log N_f of the combined cement-stabilized sandy (Type A) and sandy loam (Type B) field cores are presented in Tables 11 and 12 for clear field cores and in Tables 13 through 15 for flawed field cores.

Resilient Modulus for Clear Field Cores

The resilient modulus was significantly affected by a combination of percent tensile stress applied, measured percent tensile strain, and cement content. In general, a higher resilient modulus is generally associated with a lower strain value and a higher applied percent stress. In addition, higher values of cement content and tensile strain correspond to higher modulus values. The main effect of tensile strain ε can then be ameliorated by the addition of higher amounts of cement (Table 11).

Resilient Poisson's Ratio for Clear Field Cores

No factors were found to significantly affect the resilient Poisson's ratio; therefore, an overall mean value of 0.274 should be used as the best estimate of this property.

Log N_f for Clear Field Cores

The analysis of variance (Table 12) presents the main effects that significantly influence the fatigue life of the combined results. The approximate unit variation (AUV) values indicate that a longer life would be associated with a higher modulus, higher cement content, higher flushing (up to 20 percent), and lower stress levels.

Log N_f for Flawed Field Cores

No significant factors affected the resilient modulus (at a 5 percent level); however, the variable resilient modulus was significant at the 10 percent level (Table 11). In this instance, it was considered practical to include this variable in a later regression analysis. Increases in Log E would then result in higher fatigue lives.

Resilient Poisson's Ratio and Resilient Modulus for Flawed Field Cores

There were two variables that had a significant impact both on the resilient Poisson's ratio and on the resilient modulus of the flawed cement-stabilized field cores (Tables 14 and 15). Increases both in resilient tensile strain and applied stress levels would produce higher values of Poisson's ratio. On the other hand, higher resilient moduli values would be associated with lower resilient strain at higher stress levels.

REGRESSION ANALYSIS

The centered data technique was used in this study to develop regression equations by a stepwise regression technique. The terms included in each equation correspond to those factors and interactions found to be of practical engineering significance in the analysis of variance. The resulting equations can provide estimates of the various dependent parameters measured in the study within some standard errors. Included with the equations are the standard errors of estimate, $\hat{S}r$, and the coefficients of determination, R^2. Explanations of the centered data values are included in the legends of the appropriate tables.

TABLE 7 COMPARISON OF INHERENT VARIATION IN CEMENT-STABILIZED SANDY (TYPE A) AND SANDY LOAM (TYPE B) FIELD CORES—CLEAR SPECIMENS

MATERIAL PROPERTY	SANDY SOIL (A)			SANDY LOAM (B)			SIGNIFICANT DIFFERENCE IN	
	MEAN	STANDARD DEVIATION	DEGREES FREEDOM	MEAN	STANDARD DEVIATION	DEGREES FREEDOM	VARIANCE	MEAN
Modulus, 10^5 psi	5.397	2.042	37	5.536	1.679	28	No	No
Poisson's Ratio	0.280	0.100	37	0.266	0.115	28	No	No
Log of cycles to failure	4.2567	1.7913	37	4.6721	1.6430	28	No	No

Clear Field Cores

The regression equations for the clear field cores follow. There are two forms of the equation for Log N_f. The first form provides for the effects of all the independent or associated variables. The second equation is one that provides for the effect of the single variable most correlated with Log N_f. All the equations presented have been checked for fit and can be considered as having adequate predictive capabilities.

$$\text{Log } N_f = 4.4354 + 1.1052\ (\text{CEM}) + 1.0796\ (E) - 1.0330\ (\text{STL}) + 0.3278\ (\text{FLG})$$

$R^2 = 0.3961$ $\quad \hat{S}r = 1.3985$

For the single most correlated variable:

$$\text{Log } N_f = 4.4356 - 5.3471\ (\text{Log } \varepsilon_t - 2.7433)$$

$R^2 = 0.3145$ $\quad \hat{S}r = 1.4422$

Resilient Poisson's ratio:

$\mu_r = 0.274$ $\quad \hat{S}r = 0.1062$

Resilient modulus:

$$E_r = 5.3081 - 2.016(\varepsilon_t) + 1.1030(\text{STS}) + 1.8396(\text{CEM} \times \varepsilon_t)$$

$R^2 = 0.5238$ $\quad \hat{S}r = 1.3310$

TABLE 8 COMPARISON OF INHERENT VARIATION IN FUNDAMENTAL PROPERTIES FROM FATIGUE-RESILIENT INDIRECT TENSILE TEST RESULTS—LABORATORY SPECIMENS VERSUS CLEAR FIELD CORES

FUNDAMENTAL PROPERTY	COMBINED RESULTS SANDY SOIL (A) & SANDY LOAM (B) LABORATORY PREPARED RESULTS			COMBINED RESULTS SANDY SOIL (A) & SANDY LOAM (B) CLEAR FIELD CORES			SIGNIFICANT DIFFERENCE IN	
	MEAN	STANDARD DEVIATION	DEGREES FREEDOM	MEAN	STANDARD DEVIATION	DEGREES FREEDOM	VARIANCE	MEAN
Resilient Modulus, 10^5 psi	4.711	2.7580	128	5.456	1.8830	65	Yes	Yes
Resilient Poisson's Ratio	0.101	0.0317	128	0.274	0.1060	65	Yes	Yes
Log of cycles to failure	2.2103	1.0828	128	4.4356	1.7283	65	Yes	Yes

TABLE 9 COMPARISON OF INHERENT VARIATION IN FUNDAMENTAL PROPERTIES FROM FATIGUE-RESILIENT INDIRECT TENSILE TEST RESULTS—LABORATORY SPECIMENS VERSUS FLAWED FIELD CORES

FUNDAMENTAL PROPERTY	COMBINED RESULTS SANDY SOIL & SANDY LOAM LABORATORY PREPARED SPECIMENS			COMBINED RESULTS SANDY SOIL & SANDY LOAM FIELD CORES			SIGNIFICANT DIFFERENCE IN	
	MEAN	STANDARD DEVIATION	DEGREES FREEDOM	MEAN	STANDARD DEVIATION	DEGREES FREEDOM	VARIANCE	MEAN
Resilient Modulus, 10^5 psi	4.711	2.758	128	4.782	2.042	69	Yes	No
Resilient Poisson's Ratio	0.101	0.0317	128	0.269	0.1053	69	Yes	Yes
Log of cycles to failure	2.2103	1.0828	128	1.8892	1.1510	69	No	Yes

TABLE 10 INHERENT VARIATION IN FUNDAMENTAL MATERIAL PROPERTIES FROM FATIGUE-RESILIENT INDIRECT TENSILE TEST FOR CEMENT-STABILIZED SAND-CLAY-GRAVEL, SOIL-CEMENT LABORATORY MIX DESIGN CONDITIONS

MATERIAL PROPERTY	MEAN VALUE	DEGREES FREEDOM	DUPLICATE VARIATION WITHIN SPECIMEN		DUPLICATE VARIATION WITHIN BATCH		REPLICATE VARIATION BATCH TO BATCH			
			MEAN SQUARES	DEGREES FREEDOM	MEAN SQUARES	DEGREES FREEDOM	MEAN SQUARES	DEGREES FREEDOM	RECOMMENDED VARIANCE	DEGREES FREEDOM
Resilient Modulus, 10^5 psi	3.9785	55	2.30357	35	0.036856	1	4.568606	8	2.790735	42
Resilient Poisson's Ratio	0.201	55	0.017023	35	0.000861	1	0.013544	8	0.016786	42
Log of cycles to failure	2.1344	55	0.645534	35	0.105363	1	1.241348	8	0.776901	42

TABLE 11 ANALYSIS OF VARIANCE FOR RESILIENT MODULUS FROM FATIGUE-RESILIENT INDIRECT TENSILE TEST RESULTS FOR CLEAR SOIL-CEMENT FIELD CORES, SANDY (TYPE A) AND SANDY LOAM (TYPE B) SOILS

SOURCE OF VARIATION	DEGREES FREEDOM	MEAN SQUARES	F VALUES	SIGNIFICANCE LEVEL	APPROXIMATE UNIT VARIATION*
ϵ_L	1	50.60315	27.76	0.01	-1.715
STL	1	59.46342	32.63	0.01	+1.103
cemxϵ_L	1	8.78757	4.82	5.0	+0.693
Residual	41	1.746721			
Error	35	1.822608			

* The Approximate Unit Variation represents the effect on the dependent variable (i.e. resilient modulus) of a change in an independent variable (e.g. tensile strain) equal to one standard deviation from mean value.

Factor Legend

ϵ_L - % ultimate elastic tensile strain, μin/in

STL - % ultimate tensile strength

CEM - % cement

TABLE 12 ANALYSIS OF VARIANCE FOR LOG N_f FROM FATIGUE-RESILIENT INDIRECT TENSILE TEST RESULTS FOR CLEAR SOIL-CEMENT FIELD CORES, SANDY (TYPE A) AND SANDY LOAM (TYPE B) SOILS

SOURCE OF VARIATION	DEGREES FREEDOM	MEAN SQUARES	F VALUES	SIGNIFICANCE LEVEL	APPROXIMATE UNIT VARIATION*
E	1	40.7394	29.24	0.01	+0.57
STL	1	22.2706	16.12	0.05	-0.93
CEM	1	6.9428	5.03	5.0	+0.69
FLG	1	4.2179	3.05	10.0	+0.30

* The Approximate Unit Variation represents the effect on the dependent variable (i.e. resilient modulus) of a change in an independent variable (e.g. tensile strain) equal to one standard deviation from mean value.

Factor Legend

E - Modulus of elasticity, 10^5 psi

STL - % of ultimate tensile strength

CEM - % cement

FLG - % flushing (migration of cement)

TABLE 13 ANALYSIS OF VARIANCE FOR LOG N_f FROM FATIGUE-RESILIENT INDIRECT TENSILE TEST RESULTS FOR FLAWED SOIL-CEMENT FIELD CORES

SOURCE OF VARIATION	DEGREES FREEDOM	MEAN SQUARES	F VALUES	SIGNIFICANCE LEVEL	APPROXIMATE UNIT VARIATION*
Log E	1	10.48272	3.90	<5%	+0.39
Residual	47	0.55102			
Error	48	2.68549			

* The Approximate Unit Variation represents the effect on the dependent variable (i.e. resilient modulus) of a change in an independent variable (e.g. tensile strain) equal to one standard deviation from mean value.

Factor Legend

E = resilient modulus

TABLE 14 ANALYSIS OF VARIANCE FOR POISSON'S RATIO FROM FATIGUE-RESILIENT INDIRECT TENSILE TEST RESULTS FOR FLAWED SOIL-CEMENT FIELD CORES

SOURCE OF VARIATION	DEGREES FREEDOM	MEAN SQUARES	F VALUES	SIGNIFICANCE LEVEL	APPROXIMATE UNIT VARIATION*
ϵ_{tm}	1	0.076179	5.52	5.0	+0.029
SL	1	0.069516	5.03	5.0	+0.030
Residual	46	0.007205			
Error	48	0.013813			

* The Approximate Unit Variation represents the effect on the dependent variable (i.e. resilient modulus) of a change in an independent variable (e.g. tensile strain) equal to one standard deviation from mean value.

Factor Legend

ϵ_{tm} - Measured tensile strain

SL - Stress Level:

$$SL = \frac{\text{Applied tensile stress}}{\text{Ultimate tensile strength}}$$

TABLE 15 ANALYSIS OF VARIANCE FOR RESILIENT MODULUS FROM FATIGUE-RESILIENT INDIRECT TENSILE TEST RESULTS FOR FLAWED SOIL-CEMENT FIELD CORES

SOURCE OF VARIATION	DEGREES FREEDOM	MEAN SQUARES	F VALUES	SIGNIFICANCE LEVEL	APPROXIMATE UNIT VARIATION*
ϵ_{tm}	1	74.89069	15.25	0.05	-1.15
SL	1	43.14413	8.79	0.5	+0.81
Residual	46	1.463888			
Error	48	4.910197			

* The Approximate Unit Variation represents the effect on the dependent variable (i.e. resilient modulus) of a change in an independent variable (e.g. tensile strain) equal to one standard deviation from mean value.

Factor Legend

ϵ_{tm} - Measured tensile strain

SL - Stress Level: $SL = \dfrac{\text{Applied tensile stress}}{\text{Ultimate tensile strength}}$

where

CEM = (cement − 9.1385)/1.9590;
Cement = (percent cement − 9.1385)/1.595;
flg = percent of flushing or cement migration (internal flaw);
FLG = (flg − 0.4908)/0.11005;
STS = (applied tensile stress − 123.15)/15.519;
ε_t = (ε_t measured − 623.48)/573.60;
μ_r = resilient Poisson's ratio;
E_r = resilient modulus (10^5 psi);
ε_{tm} = measured tensile strain (microinch/inch);
STL = (stress level − 0.75110)/0.11110;
SL = stress level, defined by (applied tensile stress)/(ultimate tensile strength); and
E = (E_r − 5.4561)/18,829.

The regression equation for resilient modulus represents the effects of strain, applied tensile stress, and cement content on the resilient modulus of cement-stabilized sandy (Type A) and sandy loam (Type B) field cores.

As noted in the analysis of variance section, no significant factors affected Poisson's ratio; therefore, no regression equation could be developed. A value of 0.274 should then be used as a best estimate of Poisson's ratio for a cement-stabilized sandy (Type A) and sandy loam (Type B) base materials.

Flawed Field Cores

The regression equations for Log N_f, resilient modulus E_r, and resilient Poisson's ratio μ_r follow. The coefficient of determination, R^2, values range from 0.116 to 0.416. All three equations exhibit adequate fit and should be considered adequate for estimating the fatigue life, resilient modulus, and Poisson's ratio for cement-stabilized sandy (Type A) and sandy loam (Type B) base materials.

$$\text{Log}\,N_f(\text{mean}) = 1.8892 + 1.7792\,(\text{Log}\,E - 0.6324)$$

$R^2 = 0.1164 \qquad \hat{S}r = 1.0900$

Resilient modulus:

$$E_r = 4.7815 - 1.0663\,\varepsilon_{tm} + 5.1140\,(\text{STS})$$

$R^2 = 0.4162 \qquad \hat{S}r = 1.5838$

Resilient Poisson's ratio:

$$\mu_r = 0.2687 + 0.0260\,\varepsilon + 0.1903\,(\text{STS})$$

$R^2 = 0.1831 \qquad \hat{S}r = 0.097$

where

S = standard deviation,
ε_{tm} = [measured tensile strain (microinch/inch) − 774.36]/592.86,
STS = [applied tensile stress (psi) − 121.76]/120.20, and
E = resilient modulus, 10^5 psi.

CONCLUSIONS

As in any controlled experimentation, the findings and conclusions resulting from this study are limited to the range of variables considered. On the basis of the data and the analysis described earlier in this report, the following general conclusions are offered.

A series of regression equations has been developed that includes the significant variables that have statistically significant effects on the appropriate fundamental engineering property. They can be used to provide estimates of the various dependent parameters within the standard error of estimate, $\hat{S}r$. All regression equations included in the report have been checked to ensure adequate fit of the data and have been judged to have adequate predictive capabilities.

The inherent variations in the fundamental properties of a cement-stabilized base material in longitudinal (along the road), lateral (across the road), and vertical (depth) directions could be combined individually for the clear and flawed field cores.

The fundamental resilient engineering properties were essentially the same for cement-stabilized sandy (Type A) and sandy loam (Type B) base materials obtained from the field or laboratory.

The resilient modulus and Poisson's ratio values were found to be essentially the same for the clear (minimum flaws) and flawed field cores, whereas the fatigue lives for the two were found to be drastically different. Therefore, the two field conditions (i.e., clear and flawed specimens) form two separate material groups (i.e., statistical populations) that must be considered in any subsequent analyses.

On the bases of the low recovery rate in good field cores for job verification; observation of various cracks, lamination, compaction planes, and layer separations in field cores obtained during the second major coring operation to cores for fatigue-resilient testing; and the subsequent low number of clear core specimens found to exist during the material characterization study, the mixed-in-place soil-cement construction procedure presently used apparently does not provide the quality and uniformity expected in a cement-stabilized base layer. The flawed field cores exhibited an average fatigue life of 77 cycles, whereas the clear field cores displayed an average fatigue life of 27,300 cycles.

The results of the resilient-fatigue test program for mix design conditions were compatible with the flawed field core resilient-fatigue test program. It was therefore concluded that the laboratory results could adequately predict the fundamental resilient engineering properties of the flawed portions of the cement-stabilized base layer.

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Physical Property Changes in a Lime-Treated Expansive Clay Caused by Leaching

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The effects that continuous water leaching has on the engineering and physical properties of a lime-treated expansive clay in north-central Texas were determined. Seventy laboratory-prepared lime-treated clay samples were subjected to continuous accelerated leaching for periods of 45 and 90 days in large-diameter, flexible-wall leach cylinders. The soils' physical properties were measured before and after leaching, then graphically and statistically analyzed for significant changes. Results indicated that leaching does have detrimental impact on the physical properties of lime-treated expansive clays. The property changes are related to lime content and initial moisture content. Permeability of all samples increased dramatically with the addition of lime. Maximum detrimental changes generally occurred at lime contents at or less than the lime modification optimum. At lime contents at or above the lime stabilization optimum, the detrimental effects of leaching were minimized or eliminated. Changes to properties upon leaching varied depending on their compaction water content relative to the optimum.

Expansive clays exhibit high potential for volume change because of changes in soil moisture. Jones and Jones (*1*) estimated that the annual cost of damage to facilities built on expansive clays in the United States exceeded $9 billion.

One of the most common and effective physiochemical treatments of expansive clay is to add lime, either calcium hydroxide [$Ca(OH)_2$] or quicklime (CaO) to the soil. Lime treatment has been widely used for many years and is currently used in more than 40 states for stabilizing runways, buildings, roads, and parking lots (*2,3*). Although much is known about the phenomenon of soil-lime reactions in expansive clays, little work has been done to investigate leaching of these lime-treated soils. Of particular concern was determining whether the benefits of lime treatment were reduced through leaching over time.

PURPOSE

The long-term effects of continuous leaching on a lime-treated expansive clay from North Central Texas were studied by performing continuous leaching on laboratory-prepared specimens and analyzing the physical property changes that occurred.

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Seventy laboratory-compacted samples of various soil-lime mixtures were leached with distilled and demineralized water in specially designed large-diameter, flexible-wall cells for periods of 45 and 90 days. Physical property tests done before the start of the leach cycle, then repeated at the end of the leach cycle, provided indication of changes in the properties. Variables investigated included changes in lime content, initial compaction moisture content, and leach durations. Constants maintained were compactive effort, cure conditions, and leaching flow pressure.

Laboratory testing was conducted on three different expansive clays from the same geologic formation. Therefore, repeatability and conformation of results were enhanced.

BACKGROUND

Although soil-lime permanency has been frequently questioned by engineers, it is generally accepted that the physical changes occurring within the soil-lime mass remain relatively permanent. Eades and Grim (*4*) questioned the permanency of lime in the soil and tested pure clay minerals. They speculated that, if stabilization was caused by only flocculation or ion exchange, percolating ground water could possibly remove calcium in the soil-lime mixture. They concluded, without leach testing, that formation of crystalline silicate hydrates would be permanent and not susceptible to leaching. However, the percentage of lime added to the soil must be sufficient to generate large calcium silicate pozzolans (*5*).

Durability of Projects

The permanency of lime-treated soils is often upheld by discussing the numerous long-standing successful lime-stabilized construction projects. Gutschick (*6*) cited several successful projects throughout the United States that were built in the 1960s and 1970s. Kelly (*7*) undertook an extensive field investigation that verified the long-term effectiveness of roads at various military installations in the South that had been lime stabilized in the 1940s and 1950s.

Tests on the reconstructed lime-treated Friant-Kern Canal in California showed, after 7 years of cyclic immersion, that the effective lime content had decreased from the original 4 percent quicklime by approximately 1 percent. However, the dry unit weight remained constant and the strength increased (*8*).

The Dallas–Fort Worth (DFW) International Airport was completed in 1973 and contained 2,400,000 yd^2 of lime-treated subgrade material. The airport has provided 15 continuous years of service without major maintenance (*9*).

Barenberg (*10*) conducted a series of leach tests on lime-cement-fly-ash-aggregate (LCFAA) mixtures to study the migration of lime and cement under pavements. Test results indicated that less than 0.1 percent of the approximately 4.0 percent original lime plus cement content had been leached.

Stocker (*11*) percolated small amounts of water through lime and cement-treated samples to study the dissolution of cementatious material. Small amounts of dissolution did occur in his tests, but substantial cementation reduction was minimal.

Permeability Changes

The studies of permeability changes in lime-treated clays are widely varied. Townsend and Kylm (*12*) hypothesized and found through testing that lime stabilization would increase the pore volume caused by flocculation, thus increasing permeability. Ranganathan (*13*,p.331) found a 10-fold increase in permeability of a lime-treated expansive clay. Fossenberg (*14*,p.221), however, found a reduction in the permeability of a lime-treated clay. Gutschick (*6*) found that the permeability of a lime-fly-ash-aggregate lime mixture used on an irrigation channel initially increased but decreased with time to that of the natural clay.

Stocker (*11*) noted that for strong, advanced stages of lime modification in a soil mass, the permeability tended to decrease. However, for less modified soils, there may be an apparent reduction in permeability.

Summary

Extensive research has been conducted over the last 30 years to study the complex nature of soil-lime reactions. Much is known about those reactions and how they can affect the physical properties of an expansive clay. However, no comprehensive analysis has been undertaken to study what effects leaching would have on the long-term properties of lime-treated expansive clays. The research reported here focused on those property changes.

LABORATORY TEST PROGRAM

Soil Properties

The soils chosen for this research were the locally available well-documented (*9*) weathered clay shales from the Eagle Ford geologic formation. Three separate sites were chosen from this formation and the soils from each site were independently tested for repeatability of the leaching process. The soil sites will be referred to throughout this report as Sites 1, 2, and 3. Tests of the natural clays had plasticity index (PI) values ranging from 30 to 100. Swelling pressures ranged from 0.5 ton per square foot (tsf) to as high as 4 tsf. Unconfined compressive strengths for the soils from all sites averaged 5 tsf. Table 1 presents the average properties for the clays from all three sites.

TABLE 1 SOIL PROPERTIES FROM SITES 1, 2, AND 3

Property	Data Site		
	No.1	No.2	No.3
Silt and Clay (<0.002756in) (%)	85	90	98
Clay Fraction (<0.000079in) (%)	35	12	60
Specific Gravity	2.74	2.71	2.73
Maximum Dry Unit Weight (pcf)	103.5	101.0	100.0
Optimum Moisture (%)	22.5	22.5	24.5
Liquid Limit (%)	63	60	76
Plastic Limit (%)	33	27	31
Plasticity Index (%)	30	33	45
Linear Shrinkage (%)	22.0	17.7	24.4
Swell Pressure (psf)	977.6	1104.7	2117.9
Free Swell (%)	1.80	3.21	11.97
Unconfined Compressive Strength (tsf)	5.46	5.02	4.62
Permeability (ft/min x 10^{-9})	10.4	45.3	13.6

The Eagle Ford clays contained from 12 to 60 percent clay and 85 to 98 percent clay and silt size particles. The soils were classified as CH in the Unified Soil Classification System. Montmorrillonite and calcite were the most predominant minerals present in all clays tested.

Test Procedures

All laboratory physical property testing was done in accordance with the U.S. Army Corps of Engineers Manual (EM) 1110–2–1906 (*15*), except for the swell pressure and free swell testing. Swell pressure tests were conducted until no swelling occurred after 24 hr. Free swell tests (of 33 psf overburden) allowed samples to swell vertically until at least 48 hr. The Texas Test Method, Tex–107–E, was used to determine linear shrinkage. Unconfined compressive strengths were determined using a strain rate of 0.5 percent per minute.

All soils tested were first slaked through a No. 40 sieve, air-dried at 120°F, then lightly pulverized before testing. All testing before leaching was conducted on remolded samples. All swelling, strength, and permeability tests were conducted on samples compacted to meet 95 percent of standard Proctor maximum dry unit weight (ASTM D698) at optimum moisture content (OMC) for the various soil-lime mixtures.

Soil-Lime Reactivity

The soils were investigated for their hydrated lime reactivity before any leach tests were conducted, to identify the range of lime contents for use during leach testing and to establish baseline data for comparing changes in physical properties after leaching. Lime content values chosen for each site were based on their lime modification optimum (LMO) value using the Eades and Grim pH test (*16*), confirmed by Atterberg limits. The lime content values chosen included the lime stabilization optimum (LSO), at which maximum unconfined compressive strength occurs. Lime content values chosen for all testing were as follows: Site 1 Soil, 0, 1, 2, 3, 4, 6, and 8 percent; Site 2 Soil, 0, 1, 2, 3, 5, and 7 percent; and Site 3 Soil, 0, 1, 2, 3, 5, 7, and 9 percent. The LMO values were determined to be 4.0 percent for Site 1 soils and 3.0 percent for soils from Sites 2 and 3. The LSO values were found to be 6.0 percent for soils from Sites 1 and 3 and between 6.0 and 7.0 percent for Site 2 soils.

Moisture-dry unit weight curves were developed for all the soil-lime mixtures to determine optimum moisture contents and maximum dry unit weights for each. Because a Harvard miniature device was to be used, trial work revealed that compacting in the miniature device with four layers and 25 blows per layer (of 40 lbf from the Harvard tamper) approximated unit weights produced using standard Proctor equipment.

Physical property testing was conducted on samples at their respective OMC values. These property tests revealed that the expansive behavior of the Eagle Ford clays could be dramatically reduced with the addition of small amounts of hydrated lime. For example, with only 3.0 percent lime added, PI values decreased by up to 75 percent and vertical swells decreased by as much as 96 percent. Figure 1 shows the typical reduction of swell pressure achieved.

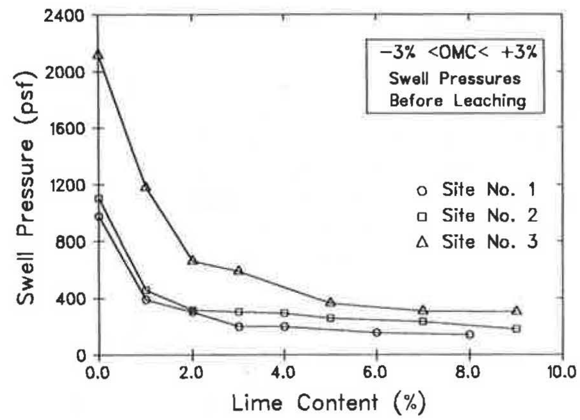

FIGURE 1 Effects of lime on swell pressures before leaching.

Leach Test Program

Leach Test Equipment

In order to model prolonged leaching on lime-treated clays, nine large-diameter, flexible-wall leach cells were designed and built. A triaxial membrane was placed on the inside of the cylinder that allowed an air-confining pressure to be applied to the sides of the sample, thus preventing leakage along the side of the sample. Pressurized water was applied to the sample from the top and was dispersed by flowing over a porous stone before reaching the sample. A porous stone was also placed on the bottom of the sample to prevent soil washout.

The nine cells were individually controlled to allow for continuous and simultaneous leach testing without halting all the tests to perform set up, take down, or maintenance operations on a single cell. The filtered distilled and demineralized leach water was supplied to the samples from a 38-gal tank that had regulated air pressure applied to it to generate the flow. Leachate was collected after it passed through the samples into 5-gal carboys, as shown in Figure 2.

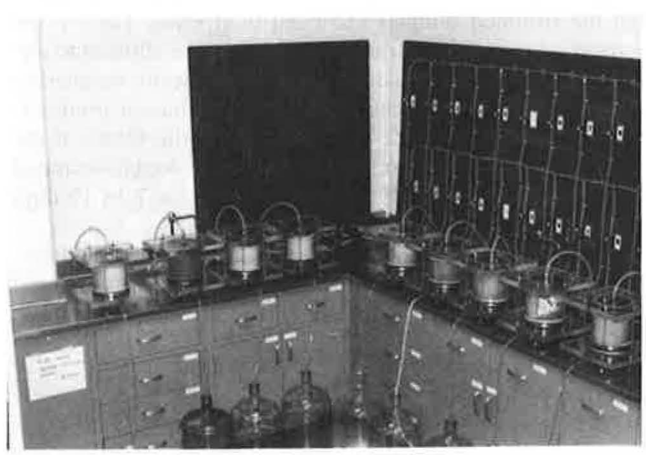

FIGURE 2 Multiple leach cell testing operation.

Leach Sample Preparation

Bulk quantities of the natural soils were air-dried at 120°F for a minimum of 5 days. The soils were then passed through a No. 4 sieve to remove rocks and organic material. Material not passing a No. 4 sieve was pulverized and then mixed with the material passing the No. 4 sieve. The required percentages of hydrated lime and water were mixed into the soil. The soil-lime-water mixture was covered with plastic wrap and mellowed for 24 hr.

The mellowed mixture was compacted in a 6-in.-diameter mold using the compactive effort required to obtain 95 percent of the standard Proctor maximum dry unit weight for the soils at their various lime contents. The compacted samples were double-wrapped in plastic wrap, coated with wax, and placed in an oven to cure for 48 hr at 120°F. This accelerated cure method was chosen as the standard for all lime-treated samples. Moisture content checks at the end of the cure period showed minimum decreases, generally less than 1 percent.

The cured samples were then placed in the test cylinders between the porous stones and sealed in the cells. A confining pressure of approximately 15 psi was applied and then the flow pressure valve was opened to start the leach process. Flow pressures of 10 psi were used throughout to minimize disturbance to the test specimens, yet allowing the water sufficient time to migrate through and interact with the soil-lime specimen. Leach test samples were duplicated for all lime and moisture content mixtures.

Postleach Testing

At the completion of the leach test, the cell was disconnected from the master control panel and disassembled. The sample was cut in half, so as to make top and bottom samples. A moisture content sample was obtained from each of these sample halves. The leach sample halves were then wrapped in a double layer of plastic wrap until ready for complete testing (2 days). Each half was tested under identical conditions. The results included, therefore, four tests that were averaged.

Samples to determine swell pressure, free swell, and unconfined compressive strength were trimmed from the leached sample halves. Because all testing was conducted at OMC, and the trimmed samples exceeded their OMC (nearly 100 percent saturated), the trimmed samples were allowed to air-dry for up to 6 hr. The samples were periodically weighed to determine weight loss caused by drying. Once a predetermined weight was reached on the basis of the OMC of the soil-lime mixture, the trimmed samples were double-wrapped in plastic wrap and allowed to equilibrate for 7 to 10 days before testing.

The remainder of the leaching soil samples was air-dried, pulverized, and passed through a No. 40 sieve. This material was used for Atterberg limits and linear shrinkage testing.

RESULTS

The leach test program was conducted to study the effects of leaching on compacted soil-lime samples with three variables considered, the amount of lime used, the moisture used at compaction, and the duration of leaching. Leaching was conducted for 45 and 90 days with the majority of testing being conducted at 45 days using a 10-psi flow pressure. The lime contents used have been discussed. Moisture contents used at the time of compaction were selected on the basis of three ranges: (a) ± 3 percent of optimum moisture (OMC); (b) -8 to -3 percent below optimum (dry or $-$OMC); and (c) $+3$ to $+8$ percent above optimum (wet or $+$OMC). Table 2 presents a summary of the 70 leach tests conducted.

Permeability

For all three soil sites, the permeabilities of the lime-treated clay increased with the addition of as little as 1 percent lime. The amount of increase ranged from a 7-fold increase to a maximum 342-fold increase. The maximum increase in permeability for all three soils appeared at or near the LMO value. At very low or very high lime contents, the increase in permeability was less pronounced, as shown in Figure 3a. However, even at very high lime contents permeability was still much greater than that of the natural soil.

During all leach testing, permeabilities of the lime-treated clays decreased during leaching. This behavior is consistent with that of other research (*6*). This decrease was fairly rapid at the start of the test, but after approximately 300 hr, the permeabilities reached a point where they were decreasing at a much slower rate, as shown in Figure 3b. This same leveling phenomenon has been noted when soils are leached with industrial fluids (*17*). It is speculated that this leveling of the permeabilities is a result of the samples' becoming saturated during the leach process. The flow rates became relatively steady during saturation and continued to slowly decrease with leaching.

Treated samples compacted wet of optimum displayed the lowest permeabilities. The highest permeabilities occurred in samples compacted dry of optimum with lime contents at their LMO.

Changes in permeability in lime-treated clays are, therefore, believed to be directly related to the ion complex within the clay soil. With the addition of small amounts of lime (less than the LMO value), clay soil particles flocculate to some degree and pozzolanic reactions do not occur on a large scale, resulting in a small increase in permeability. Maximum flocculation and agglomeration occur at the LMO value, opening up large flow channels and producing extremely large permeability increases. At lime contents exceeding the LMO value, the soil pH is sufficiently elevated that silica and alumina hydrates are formed, producing massive crystalline structures and effectively blocking flow channels. Permeabilities declined as lime percentages were increased, but sufficient channels remained so that permeabilities well exceeded that of the natural soil.

Atterberg Limits

Figure 4 shows a typical diagram of the effects of leaching on Atterberg limits of the lime-treated clay from Site 1. After leaching, plastic limit (PL) and liquid limit (LL) values de-

TABLE 2 SUMMARY OF VARIABLES USED DURING LEACH TESTING FOR ALL SOIL SITES

Soil Site	Variables					
	Percent Lime (%)	Leach Durations		Moisture Content (%) [a]		
		45 days	90 days [b]	-OMC	OMC	+OMC
1	0	X	X	-	X	X
	1	X	-	X	X	X
	2	X	-	-	X	-
	3	X	X	X	X	X
	4	X	-	X	X	X
	6	X	X	X	X	X
	8	X	-	-	X	-
2	0	X	X	-	X	X
	1	X	-	X	X	X
	2	X	-	X	X	X
	3	X	X	X	X	X
	5	X	-	X	X	X
	7	X	X	-	X	-
3	0	X	X	-	X	-
	1	X	-	-	X	-
	2	X	-	-	X	-
	3	X	X	-	X	-
	5	X	-	-	X	-
	7	X	-	-	X	-
	9	X	X	-	X	-

[a] -OMC = -8% to -3% below OMC; OMC = -3% < OMC < +3%; +OMC = +3% to +8% above OMC
[b] 90 Tests were conducted at OMC only
\- Not tested

creased, whereas PI values increased. Maximum increase in postleach PI value appeared in samples with 1 to 3 percent lime, for those leached 45 days and compacted at OMC. Samples leached 90 days had even larger increases in PI value. The largest increase in PI value occurred in Site 3 material with 3 percent lime, which increased from a PI value of 11 to 31. Maximum increases in PI values during leaching occurred in samples compacted wet of optimum with the minimum increase for samples compacted dry of (or at) optimum moisture.

The increase in PI value after leaching was less as more lime was added to the soils. When approximately 6 percent or more lime had been added to the soils from all three sites, the PI value after leaching was less than or equal to the PI value before leaching. The duration of the leach cycle did not have an adverse impact on the PI value of the soils when at

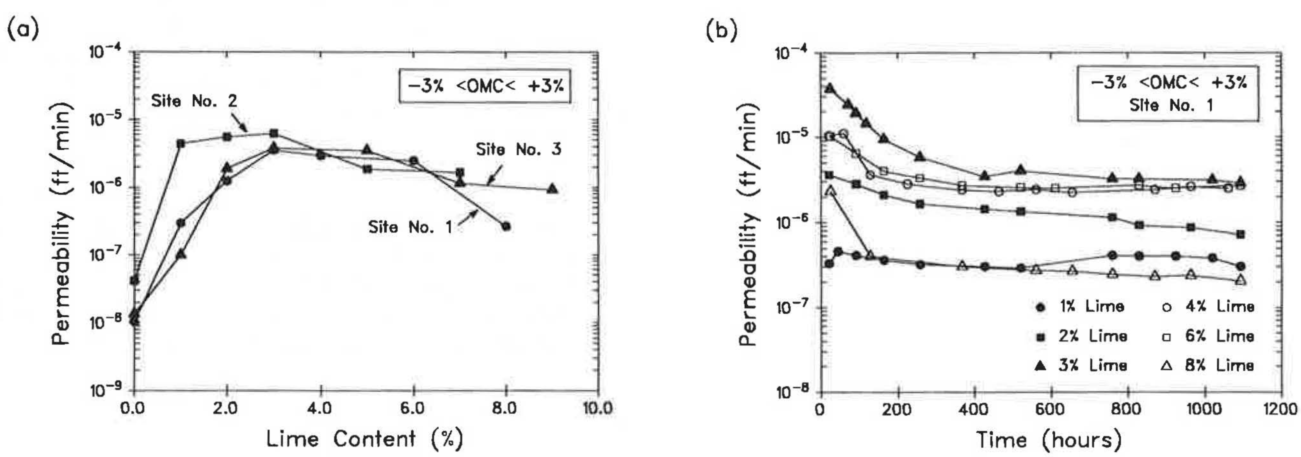

FIGURE 3 (a) Lime content versus permeabilities after 600 hr of leaching; (b) permeability changes during 45 days of leaching.

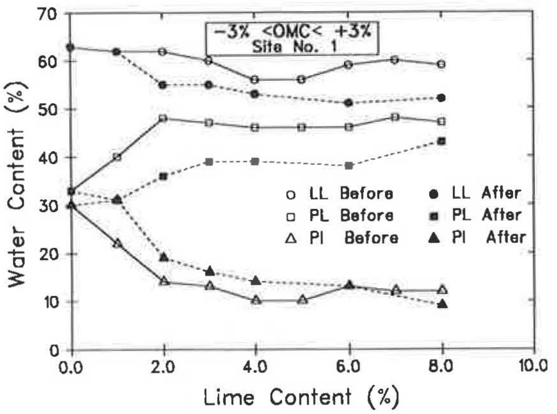

FIGURE 4 Atterberg limits before leaching and after leaching 45 days.

least 6 percent lime had been added. Increases in lime content did not appear to affect the PL and LL values, which were generally lower after leaching.

Linear Shrinkage

A typical diagram depicting the effects of leaching on linear shrinkage is shown in Figure 5 for Site 3 material compacted at OMC. Shrinkage increased after leaching with the maximum increases occurring at lime contents less than (or at) the LMO value. In Site 3 material, samples compacted wet of optimum had the least amount of increase in shrinkage, whereas those compacted dry had the highest shrinkage after leaching. Leaching 90 days appeared to have a more detrimental impact on the higher lime content samples than leaching 45 days. For the samples with the lower lime contents, 45 days of leaching generally appeared to be more detrimental.

As the lime was increased in the leached samples, the increase in postleach shrinkage was less. When 6 to 7 percent lime had been added to samples, this linear shrinkage was less than or equal to the shrinkage noted before leaching.

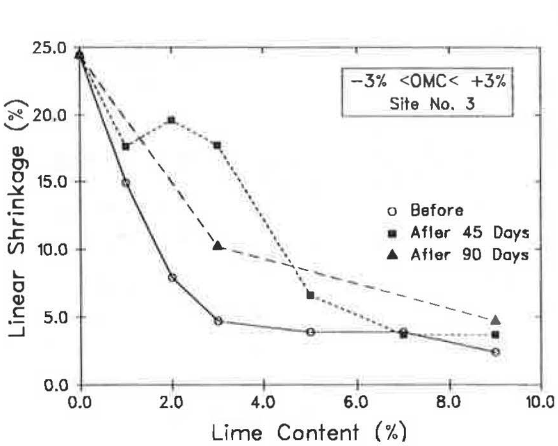

FIGURE 5 Linear shrinkage before leaching and after leaching 45 and 90 days.

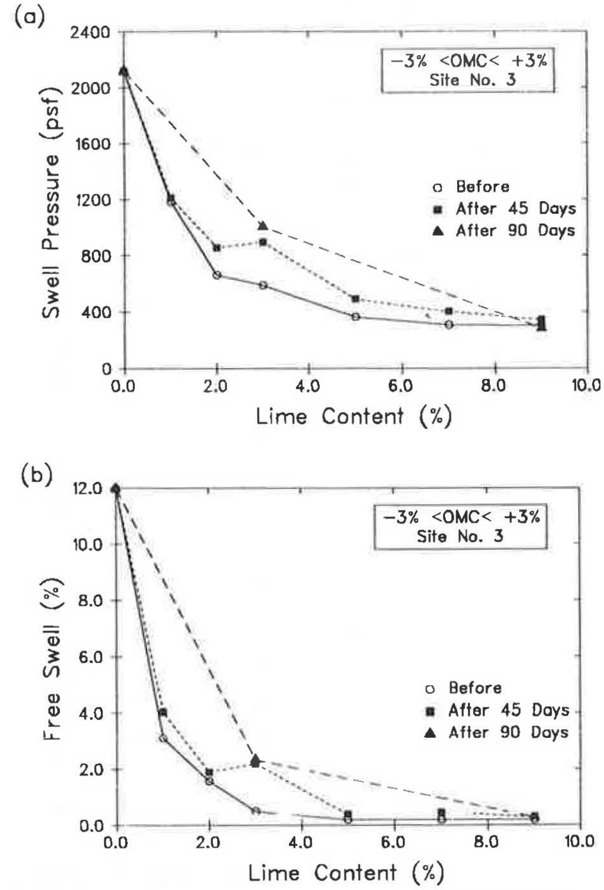

FIGURE 6 (a) Swell pressure and (b) free swell before leaching and after leaching 45 and 90 days.

Swell Properties

Swelling Pressure

The results of leaching on the swelling pressures of the clay tested from Site 3 are shown in Figure 6a for the samples tested at the OMC value. The results are typical for all three sites. Tests indicated increases in swell pressure after leaching at all lime contents. These increases ranged from as little as 13 percent to as high as 98 percent. The maximum increase in swell pressure occurred in samples treated with lime contents of 3 to 6 percent. When 7 to 8 percent lime had been added, the swelling pressure after leaching was approximately equal to that before leaching. Maximum increases of swelling pressure occurred in samples compacted wet of optimum, whereas those compacted dry of (and at) the OMC value produced lower increases in swelling pressures. Samples leached 90 days had slightly higher swell pressures at the lower lime contents than samples leached 45 days. However, at the higher lime contents, there was no significant difference in swell pressures caused by changes in leach duration.

Free Swell

In samples tested for free swell after leaching, there was an increase in swell for samples tested at all lime contents. Figure

6b shows typical changes in swell after leaching 45 and 90 days for Site 3 material compacted at OMC. Maximum increases in postleach free swell occurred in samples tested with 3 to 4 percent lime, with increases ranging from 115 to 340 percent. When approximately 6 percent lime had been added to all materials, free swell increases became negligible, although there still was some slight increase in swell. Maximum increases of postleach free swell occurred in samples compacted wet of optimum, whereas samples compacted dry had slightly lower increases in swell than those compacted at optimum moisture. Samples leached 90 days displayed little difference in swell increase over those leached 45 days for Site 3 soil. However, for the other two sites at lower lime contents, 90 days of leaching produced higher swell than did 45 days of leaching.

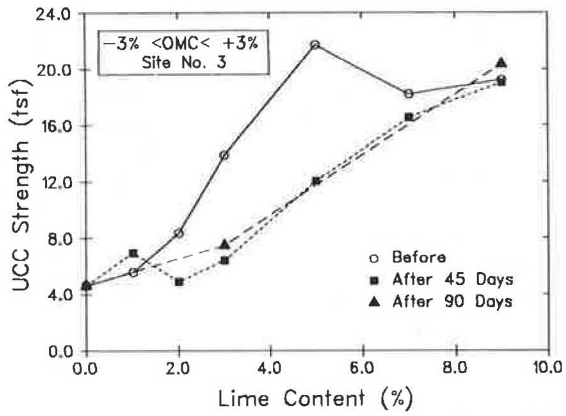

FIGURE 7 Unconfined compressive strength before leaching and after leaching 45 and 90 days.

Unconfined Compressive Strength

Unconfined compressive strength tests run on samples leached 45 and 90 days indicated that materials with very low lime content had considerable loss in strength during leaching. The maximum loss was a decrease of 76 percent for Site 1 material with 1 percent lime. As the amount of lime was increased in the soils, the loss of shear strength declined such that after 7 to 8 percent lime was added to the soils, the loss was negligible. After approximately 8 percent lime had been added to the soils, the postleach strength was actually higher. Samples leached 90 days had similar results to those leached 45 days, except in material from Site 1, which had a higher strength loss after leaching 90 days. Figure 7 shows a typical plot of the strengths measured after leaching 45 and 90 days for Site 3 material compacted at optimum moisture.

The maximum strength measured after leaching occurred in samples compacted at optimum moisture or slightly wet of optimum, although it was less than the preleach strength measured. Samples compacted dry of optimum displayed dramatic strength loss after leaching, averaging over 52 percent less strength when compared to samples compacted at the OMC value.

DISCUSSION OF PHYSICAL PROPERTY CHANGES

In all physical property tests conducted, postleach testing revealed that there were detrimental effects on the stabilizing attributes of lime-treated clays. The samples treated with lime contents of 1 to 4 percent displayed the largest detrimental changes during leaching. However, in all physical property tests conducted, there was a minimum lime content beyond which leaching was not significantly detrimental. This optimal lime content varied slightly between property tests. For Atterberg limits and linear shrinkage, the optimal lime content was found to be between 5 and 6 percent; for swelling properties, it varied between 6 and 8 percent; and for strength, it was found to be 7 to 8 percent.

These amounts of lime are approximately the lime contents established as the LSO values of the soils, i.e., the amounts of lime necessary to provide optimal pozzolanic reactions. It is believed that leaching greatly increases the water molecule concentration, causing the soil-lime water system to attempt to diffuse any loosely held calcium cations away from the clay particle surface while also increasing the adsorbed water layer surrounding the clay particle. This process ultimately results in increases in PI value, shrinkage, and swelling properties.

When the lime content of expansive clays is below the optimum concentration for maximum pozzolanic reaction, flocculation bonding is relatively weak and could possibly be removed by flowing water during leaching, as suggested by Diamond and Kinter (*18*). Therefore, longer leach durations on the samples mixed at the lower lime contents should result in more detrimental effects. This research supports that theory because many samples leached 90 days had larger increases in PI value, free swell, and swell pressure than those leached 45 days.

Furthermore, if flowing water breaks down flocculation, then it follows that a higher permeability would increase the rate of breakdown. The highest permeability in this research occurred in samples compacted at their LMO values. The largest increase in postleach linear shrinkage, PI value, and swelling properties occurred in samples compacted with 3 to 4 percent lime. When enough lime is added to the soil, it is believed that the increase in calcium concentration offsets the increase in water molecules caused by leaching, and pozzolanic reaction products will begin to close off flow channels and produce permanent interparticle bonding.

In order to minimize the effects of leaching on strength losses, it was necessary to increase the amount of lime added to the samples to at least 1 percent above the LSO value. The additional water added during leaching may prevent adequate pozzolan formation until the calcium present is sufficient to offset the disruptive presence of additional water.

A statistical analysis was performed on the data to determine if the changes in properties measured were statistically significant. The t statistic was used to test the significance of the differences in the means of the test results before leaching to the means of the test results after leaching.

Before testing for significant difference between the means, equal population variances were tested using the F distribution. All F tests indicated that there was insufficient evidence to refute the assumption of equal population variances.

All t statistics were tested at the $\alpha = 0.05$ primary level of significance and the $\alpha = 0.10$ secondary level of significance. All results were recorded as either a statistically significant difference between the means before and after leaching or as insufficient evidence (IE) to refute the null hypothesis of no difference in the means.

The statistical analysis indicated that the changes in the physical properties after leaching were statistically significant at lower lime contents and that at lime contents between 5 and 7 percent, the difference in means had less significance. The only exception was that the statistical analysis indicated that there was insufficient evidence to suggest that the LL value was significantly reduced. Statistical analyses on strength and linear shrinkage indicated that nondetrimental effects could be achieved at lime contents approximately ½ to 1 percent below those revealed by graphical comparisons.

CONCLUSIONS AND RECOMMENDATIONS

The intent of this research was to test the effects of continuous water leaching on lime-treated expansive clay. Physical property testing was conducted on the soils before and after leaching for changes in lime reactive characteristics. The results were analyzed graphically and statistically. Some specific conclusions follow:

1. The permeabilities of lime-treated samples were 7 to 340 times higher than those of the natural clays with the maxima occurring at the LMO values.
2. There are detrimental changes in soil-lime mixtures during continuous leaching, the maximum being in materials with below-LMO percentages of lime, for all physical properties measured.
3. The change to properties during leaching tended to be proportional to the duration of the leach cycle.
4. There was a range of lime contents, ±2 percent of the LSO value, for each physical property tested that minimized or eliminated the detrimental effects of leaching.
5. Samples compacted dry of optimum exhibited the highest increase in permeability and linear shrinkage and much lower strength. Samples compacted wet of optimum displayed the highest increase in postleach PI value and swell properties.
6. Statistical analysis of the physical changes after leaching confirmed that the detrimental effects were statistically significant, except for reduction of the LL value.
7. The statistical analysis of shrinkage and strength results indicated that ½ to 1 percent less lime could be used to minimize detrimental effects than the amount revealed by graphical analysis.

On the basis of the results of this study, the following recommendations are made:

1. For the Eagle Ford clays tested in this research, the detrimental effects caused by leaching can be minimized or eliminated if the lime content is at least 1 percent over the LSO value.
2. Lime-treated soils should be compacted within 1 percent of their OMC value.
3. Additional leach testing should be conducted on other expansive clays to develop a comprehensive analysis of the effects of leaching on different clays. Variables to consider should be longer leach cycles, various cure times, and compaction criteria.

ACKNOWLEDGMENTS

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Improved Characteristics in Sulfate Soils Treated with Barium Compounds Before Lime Stabilization

G. A. Ferris, J. L. Eades, R. E. Graves, and G. H. McClellan

Some sulfate-bearing soils stabilized with calcium hydroxide (lime) have developed heave over periods of time. This heave is thought to result from reactions of soluble sulfates, calcium hydroxide, and free aluminum in the soil or groundwater, or both, to form ettringite (3 $CaO \cdot Al_2O_3 \cdot 3CaSO_4 \cdot 32H_2O$), a highly water-expansive mineral. Laboratory testing, using the California bearing ratio (CBR) method, has indicated increased bearing strength values and decreased swell when barium hydroxide or barium chloride was added to sulfate-rich soils before lime application. A California soil containing sodium sulfate had increased strength values when either barium compound was used with lime as compared with specimens with lime only. A barium hydroxide treatment followed by lime application to a Texas soil containing sodium sulfate was successful, showing increased CBR values and a decrease in percent swell. Potential volume change tests were conducted on a Colorado soil and the California and Texas soils using lime only and lime added to soils treated with barium hydroxide or barium chloride. The barium hydroxide plus lime treatment showed a marked decrease in swell pressure when compared with lime-only treatment. The mix of barium chloride plus lime decreased in swell pressure, but not as significantly as the mix of barium hydroxide plus lime. The presence of ettringite in the treated soils was determined using scanning electron microscopy. Ettringite formation was not detected in the California or Colorado soils for either combination of barium hydroxide or barium chloride plus lime. The Texas soil contained an abundance of ettringite in the mix of barium chloride plus lime, and it was present, but sparse, in the mix of barium hydroxide plus lime.

Soluble sulfates react with calcium hydroxide and free aluminum to form ettringite ($3CaO \cdot Al_2O_3 \cdot 3CaSo_4 \cdot 32H_2O$) (*1*). Expansion caused by the growth of ettringite in sulfate soils treated with calcium hydroxide (lime) may produce severe problems in the construction and performance of pavement foundation systems (*2*). The amount and type of sulfates present, sodium sulfate or calcium sulfate, and the amount and type of clay material are properties believed to play key roles in the poststabilization expansion developed over time in lime-treated sulfate soils. The formation of ettringite also is responsible for the deterioration of concrete by sulfate attack (*3,4*).

The sulfate content is clearly the most important property to consider when evaluating such soils for construction purposes. The quantity of sulfates present generally dictates the extent to which ettringite will form. Simply, the greater the content of soluble sulfates in a soil, the greater the potential for the growth of ettringite.

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That sodium sulfate and calcium sulfate (gypsum) have different solubilities suggests that the form of sulfates present in a soil plays an active role in the degree to which ettringite will form. Gypsum is approximately 100 times less soluble than other sulfate minerals normally found in soils (*5*). Calcium and sodium sulfate commonly form evaporite minerals in arid to semiarid regions, because of little or no leaching, crystallizing when their concentrations exceed their solubility limits. Gypsum is the most common sulfate mineral found in soils because of its relatively low solubility.

The percentage and type of clay minerals present in a soil generally dictate the amount of lime required for stabilization. Soils with a high clay content or an initial high plasticity index (PI) and swell require greater amounts of lime to effectively reduce the plasticity, eliminate the swell, and stabilize the soil (*6*). The addition of lime to a sulfate-bearing soil provides calcium, which reacts with the sulfates to form gypsum, which may react with aluminum to form ettringite (*7*).

The type or types of clay present also are believed to be major factors in determining the strength and swell potential in lime stabilization (*8*). Smectites are three-layered clays that are highly expansive. Thus, a soil containing large amounts of smectite will require more lime to become stabilized (*9*). However, the two-layered structure of kaolinite may allow it to be a greater source of aluminum needed for the formation of ettringite in sulfate-bearing soils.

Tests have been conducted to determine if reactions that form ettringite could be minimized in sulfate-bearing soils by pretreating them with barium hydroxide or barium chloride in an effort to reduce the soluble sulfates before lime stabilization. Barium compounds should react to form less-soluble barium sulfates (*10*), thereby reducing the availability of calcium sulfates for ettringite formation. Another method involving a double-lime treatment of sulfate soils also was investigated in an effort to reduce detrimental sulfate reactions.

MATERIALS AND METHODS

Three soils were studied in this research project because of their high sulfate content and expansive nature. Soils from Orange County, California, Central Texas, and Denver, Colorado, were used in various aspects of the testing procedures. The soils vary in composition with the amount and type of sulfates, the amount and type of clay components, swell, and plasticity. The lime used in all tests was a calcium hydroxide [$Ca(OH)_2$] obtained through Fisher Scientific.

Initial properties that influence lime stabilization were determined by analyzing untreated soil samples. Soil mineral compositions were determined using X-ray diffraction (XRD) procedures (*11*) and microscopic techniques. Clay percentages were determined using a standard hydrometer test (ASTM D422). The plasticity indices were determined by a standard Atterberg limits test (*12*).

California Bearing Ratio (CBR) Testing

The optimum water contents for compaction of the soils were determined by a modified Proctor density test (ASTM D698). The soils were then compacted, using a standard CBR method (ASTM D1557), into 6-in.-diameter molds at their optimum water contents and soaked in water for periods of 4, 14, 40, and 60 days. After the soaking periods, the compacted soils were measured for percent swell and tested to determine bearing strength values.

Three types of treatment methods were conducted on the soils. Untreated soils and soils treated with 6 percent lime were tested for swell and strength values after 4-day soaking periods.

A double application of lime (*7*) was conducted where 3 percent lime was added followed by an uncompacted wet curing period of 7 days before the application of an additional 3 percent lime before compaction. The samples were then soaked for 60 days before being tested for swell and strength characteristics.

In the barium compound treatment method, soils were pretreated with 3 percent barium hydroxide or 3 percent barium chloride, compacted at their optimum water contents, soaked in water for 14 days, and tested for strength and swell values. The soils were then dried at 50°C, disaggregated, treated with 6 percent lime, compacted at their optimum water contents, and soaked for periods of 14 and 40 days before being tested for strength and swell values.

Potential Volume Change (PVC) Testing

The soils were compacted into 2.75-in. molds at their plastic limits and at 2.5 times standard Proctor compactive efforts and measured for swell pressures exerted against the restraining force of a proving ring over periods of 7 days using a PVC meter. The meter is used to perform swell index tests to determine the expansive nature of a soil and to give it a rating of either noncritical, marginal, critical, or very critical, depending on the amount of swell that is developed (*13*).

Two soil treatment methods were investigated in the PVC testing. In the first method, 6 percent lime was added to each soil, followed by mixing, compacting, and monitoring of swell pressures developed during 7-day soaking periods.

In the second method, 3 percent barium hydroxide or 3 percent barium chloride was added to each soil, followed by wet curing for 7 days, and drying at 50°C. They were then disaggregated, treated with 6 percent lime, compacted, and monitored for swell pressures developed during 7-day soaking periods.

RESULTS

Initial properties that influence lime stabilization and control the behavior of sulfate soils are the soluble sulfates content, clay percentage, plasticity index (Table 1), and soil mineral composition (Table 2). The Texas and Colorado soil properties were almost identical, except that the Colorado soil contained more kaolinite (Figure 1). The California soil had a soil mineral composition similar to that of the Texas soil, but the soluble sulfates content and clay percentage were much lower.

CBR Testing

Testing after a 4-day soaking period resulted in an increase in CBR values and a decrease in percent swell for both soils when 6 percent lime was added compared with the untreated soils (Table 3). Testing after a 14-day soaking period of soils pretreated with the two barium compounds indicated an increase in CBR values with the addition of lime to the pretreated soils (Table 3). The mix of barium hydroxide plus lime appeared to control the swell more effectively than the mix of barium chloride plus lime in the Texas soil. Comparing these data with tests previously conducted using lime only and untreated samples, the mix of barium hydroxide plus lime increased in CBR values for both soils and decreased in percent swell for the Texas soil (Table 3, Figure 2). Percent swell for the California soil may be considered negligible in all cases. The mix of barium chloride plus lime increased in CBR values for the California soil but had little to no improvement in CBR values or percent swell for the Texas soil (Table 3). The California soil was retested using an extended soaking period of 40 days. When both barium compounds were used, CBR values increased over those of the previous 14-day soaking test (Table 3).

The double application of lime using the California and Texas soils was relatively successful (Table 3). The results were somewhat improved over those for the mix of barium chloride plus lime but were not as successful as those for the mix of the barium hydroxide plus lime.

TABLE 1 INITIAL SOIL PROPERTIES

Soil Type	Soluble Sulfates	Clay %	Plasticity Index
Texas	8,870 ppm	67%	41
California	3,850 ppm	27%	13
Colorado	10,000 ppm *	80%	44

*-value from (14).

TABLE 2 ORIGINAL SOIL MINERAL COMPOSITION

Soil Type	Mineral Composition
Texas	Smectite, Illite, Kaolinite, Gypsum, Quartz
California	Smectite, Illite, Kaolinite, Gypsum, Quartz
Colorado	Smectite, Illite, Kaolinite, Gypsum, Quartz

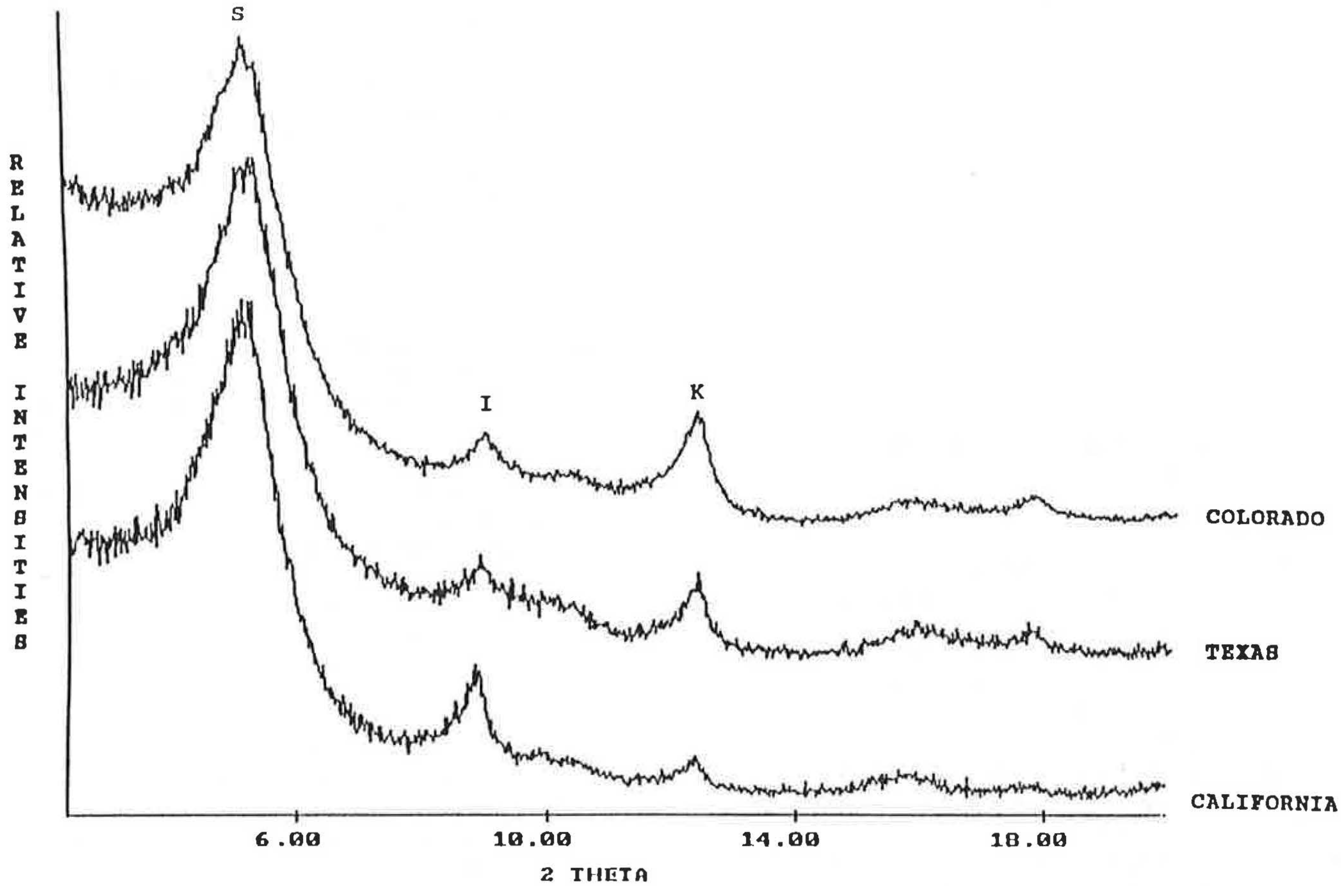


FIGURE 1 X-ray diffraction patterns showing clay mineral composition of soils. S = smectite, I = illite, and K = kaolinite.

TABLE 3 RESULTS OF CBR TESTS

Soil Type	Treatment	Length of Soak	CBR Value	% Swell
Texas	Untreated	4 Days	0.7	12
Texas	6% Ca(OH)2	4 Days	5.1	5.7
Texas	3% Ba(OH)2	14 Days	3.5	1.9
Texas	3% Ba(OH)2 + 6% Ca(OH)2	14 Days	21.2	3.2
Texas	3% Ba(OH)2 + 6% Ca(OH)2	40 Days	•	•
Texas	3% Ba(Cl)2	14 Days	•	•
Texas	3% Ba(Cl)2 + 6% Ca(OH)2	14 Days	4.3	11.6
Texas	3% Ba(Cl)2 + 6% Ca(OH)2	40 Days	•	•
Texas	Double Application of Lime	60 days	21.5	5
California	Untreated	4 Days	4.2	.7
California	6% Ca(OH)2	4 Days	10.4	0.02
California	3% Ba(OH)2	14 Days	5.1	0.24
California	3% Ba(OH)2 + 6% Ca(OH)2	14 Days	20.6	0.17
California	3% Ba(OH)2 + 6% Ca(OH)2	40 Days	48.7	-0.24
California	3% Ba(Cl)2	14 Days	3.2	1.5
California	3% Ba(Cl)2 + 6% Ca(OH)2	14 Days	24.8	0.08
California	3% Ba(Cl)2 + 6% Ca(OH)2	40 Days	36.9	-0.06
California	Double Application of Lime	60 days	45.7	0.65

• -- Data not available

PVC Testing

Testing of the Texas and Colorado soils using the PVC meter confirmed the CBR test results. For all soils, the mix of barium hydroxide plus lime significantly decreased in swell pressure compared with the lime treatment only (Table 4). The mix of barium chloride plus lime exhibited some improvement over the mix of lime only but not as significantly as that of the mix of barium hydroxide plus lime (Table 4).

Scanning Electron Microscope (SEM) Analysis

Samples were taken from the 14-day-soak soils pretreated with the two barium compounds and analyzed using SEM to determine if the formation of ettringite was being controlled. The California soil treated with the double application of lime had an abundance of ettringite, an elongated, needle-like mineral (*1*) (Figure 3a), as did the Texas soil. Ettringite was not detected in the California soil treated with either barium chloride or the mix of barium hydroxide plus lime (Figures 3b and 3c). By adding 15 percent barium hydroxide to the California soil, barium sulfate crystals were formed over a 2-

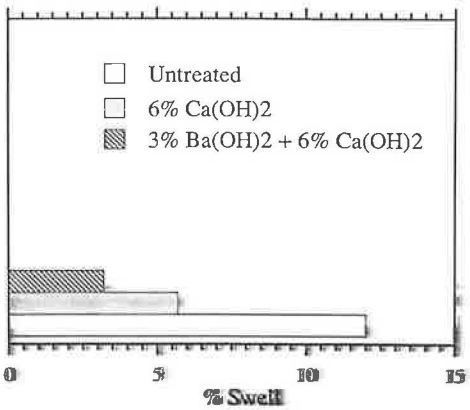


FIGURE 2 Percent swell from CBR tests on Texas soil.

month soaking period (Figure 3d). In the Texas soil, ettringite was found to be relatively abundant in the mix of barium chloride and lime (Figure 4a) and was present, but sparse, in the mix of barium hydroxide and lime (Figure 4b). Barium sulfates were formed in the Texas soil when treated with 15 percent barium hydroxide and 15 percent barium chloride (Figures 4c and 4d, respectively). Samples from the PVC tests also were analyzed with the SEM. Ettringite (Figure 5a) and barium sulfates (Figure 5b) were observed in the Texas soil treated with 3 percent barium hydroxide and 6 percent lime. Barium sulfate crystals (Figure 5c) were observed in the Colorado soil treated with 3 percent barium hydroxide and 6 percent lime. Although ettringite was observed in some treated samples by SEM analysis, it was not detected by XRD procedures.

DISCUSSION OF RESULTS

California Bearing Ratio Testing

The pretreatment of sulfate soils with barium compounds before lime application was most successful with the California soil. In these soils, the formation of ettringite was deterred and strength values were increased using both barium compounds. This may be because of the soils' relatively low soluble sulfate content, low clay content, and low plasticity. The Texas soil, which has a higher soluble sulfate content, greater clay content, and is more plastic, improved in strength and swell values under the barium hydroxide pretreatment but not under the barium chloride pretreatment. The higher content of sulfates in the Texas soil explained the formation of ettringite despite pretreatment methods. The barium ions are believed to be more available in the mix of barium hydroxide plus lime than in the mix of barium chloride plus lime. Pretreatment with larger amounts of barium hydroxide might be more effective in controlling ettringite formation in high-sulfate soils. However, these tests have not yet been conducted in the current study.

The increase in CBR values observed for the 40-day soaking period in the California soil is believed to be caused by cementitious effects of lime treatment forming calcium silicate hydrates and calcium aluminate hydrates through dissolution of Si and Al in the clay mineral structures. This can account for strength improvement over time, which has been demonstrated in other lime-treated soils (*15*). The barium compound are thought to react with the sulfates, forming less soluble barium sulfates, leaving a lesser amount of sulfates available to react with calcium hydroxide and aluminum to form ettringite. Reduction of ettringite formation leaves more free lime, keeping the pH above 12.4, allowing for more dissolution of the clay fraction to produce additional cementing materials during lime stabilization.

The double application of lime had improved strength values both for the Texas and California soils and a decrease in swell for the Texas soil over that of a single lime treatment. The soils in the double application study were soaked for longer periods of time, yet their CBR values increased and swell in the Texas soil was controlled to a degree. It is believed that the lime from the first application reacts with the sulfates to form gypsum and ettringite. The second application, which can be better mixed with the flocculated soil produced by the initial application, redistributes the sulfate minerals already formed. This second application of lime will furnish the calcium and high pH necessary to form calcium silicate hydrates and calcium aluminate hydrates in and around the pores, reducing the permeability and available water to the ettringite crystals.

TABLE 4 RESULTS OF PVC TESTS WITH 7-DAY SOAKING PERIODS

Soil Type	Treatment	Pressure exerted (lb./sq. ft.)
Texas	6% Ca(OH)2	7,600
Texas	3% Ba(Cl)2 + 6% Ca(OH)2	3,700
Texas	3% Ba(OH)2 + 6% Ca(OH)2	1,000
California	6% Ca(OH)2	5,400
California	3% Ba(Cl)2 + 6% Ca(OH)2	1,900
California	3% Ba(OH)2 + 6% Ca(OH)2	700
Colorado	6% Ca(OH)2	5,400
Colorado	3% Ba(OH)2 + 6% Ca(OH)2	700

PVC Testing

The lime-only treatment rated as a critical swell value for soil expansion. Lime added to soils treated with barium hydroxide reduced swell pressures in volume change tests, keeping the swell in the noncritical range. Lime added to soils treated with barium chloride controlled the swell to some degree and rated as marginal.

CONCLUSIONS

1. The amount and type of sulfates present, and the amount and type of clay material, are properties believed to play key roles in the poststabilization expansion of lime-treated sulfate soils.

2. The test results indicate that the swell resulting from lime treatment of sulfate soils may be controlled, and strength values increased, by pretreating them with barium compounds before lime application. Barium sulfates with low solubilities

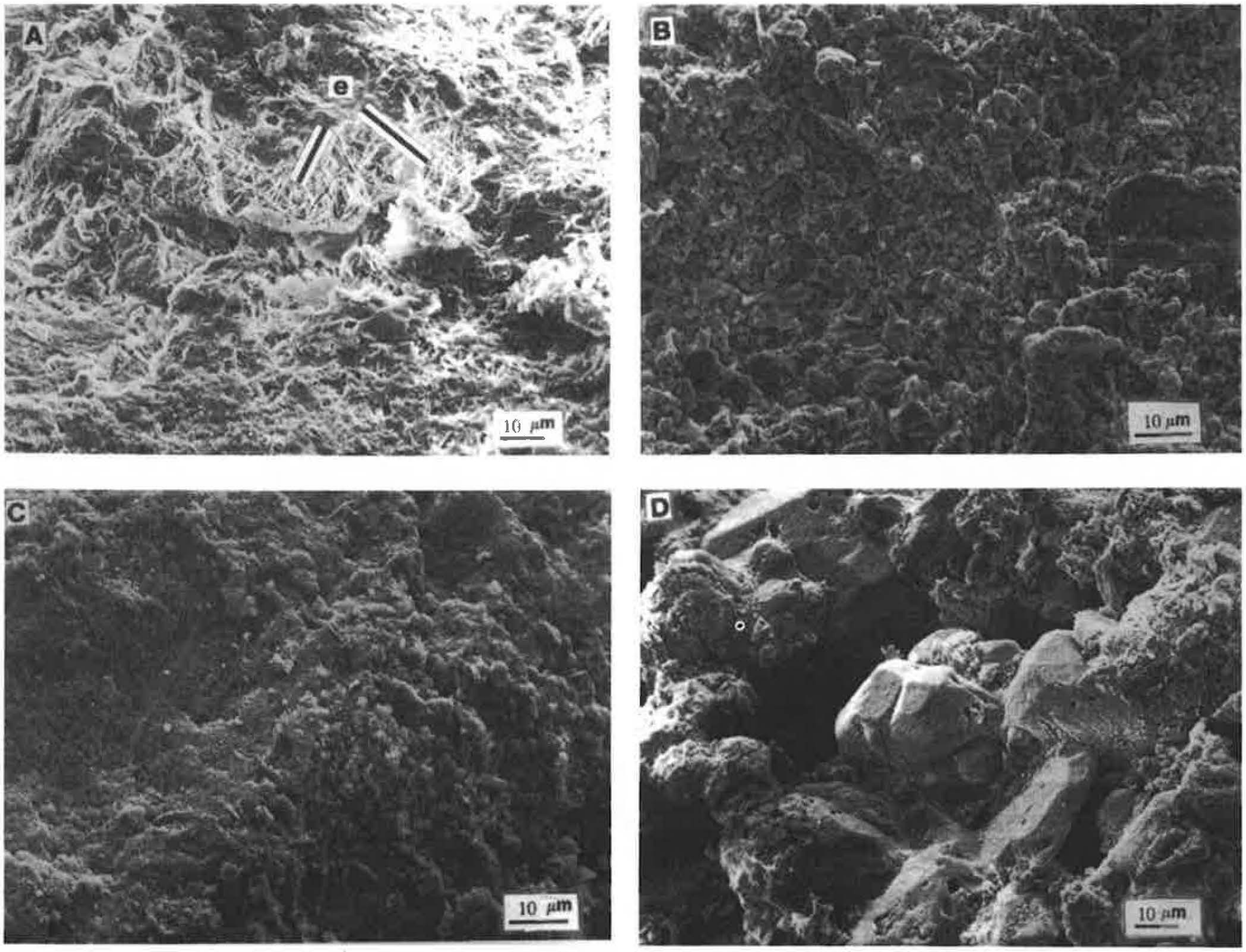


FIGURE 3 California soil: (A) double application of 3 percent lime + 3 percent lime CBR test with 60-day soak, showing ettringite, 720 ×; (B) 3 percent $Ba(OH)_2$ + 6 percent lime treatment, CBR test with 14-day soak, 1,050 ×; (C) 3 percent $Ba(Cl)_2$ + 6 percent lime treatment, CBR test with 14-day soak, 1,150 ×; (D) 15 percent $Ba(OH)_2$ treatment, 2-month soak, 730 ×.

are formed, removing the sulfate ions so that they are not free to react with the lime to form gypsum. If the sulfate availability is eliminated, the water-sensitive mineral ettringite cannot form. Barium hydroxide proved to be a more effective pretreatment compound than barium chloride.

3. Soils with low sulfate contents may be stabilized by applying the lime in two applications (double treatment). It is believed that the reactions forming gypsum and ettringite occur after the first application of lime and that the mixing of the second application breaks up the crystals and supplies more lime, which allows for the formation of cementing agents, increasing strength values and decreasing swell.

4. Because the use of barium compounds has never been applied to field studies, their impact on the surrounding environment, primarily groundwater, is unknown. Also, barium compounds are considerably more costly to use than lime. Therefore, future studies should concentrate on the method involving the double application of lime because it is more practical to use.

ACKNOWLEDGMENT

The studies presented here were part of a project funded through a research grant from Chemstar, Inc., whose support and cooperation during the study are gratefully acknowledged.

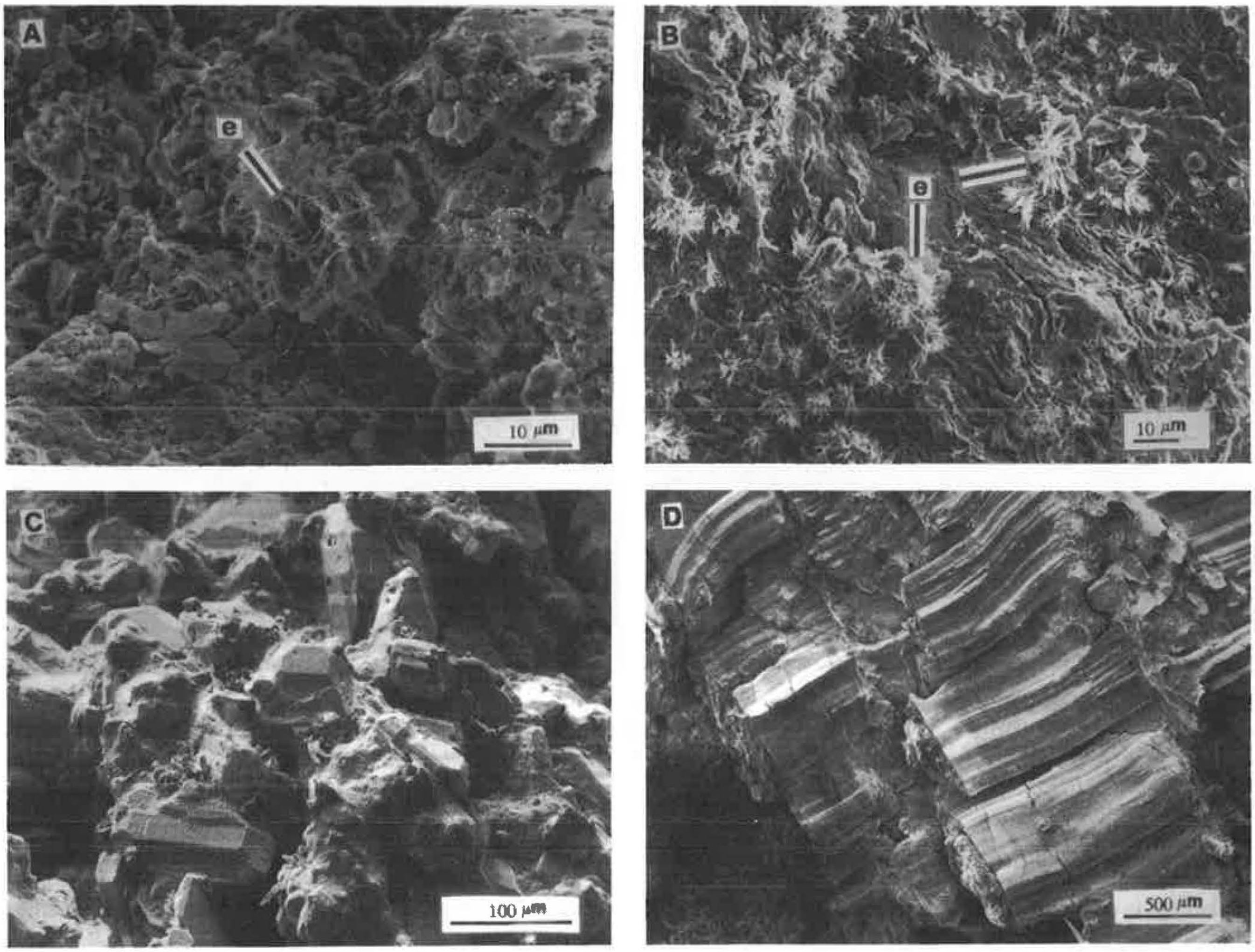
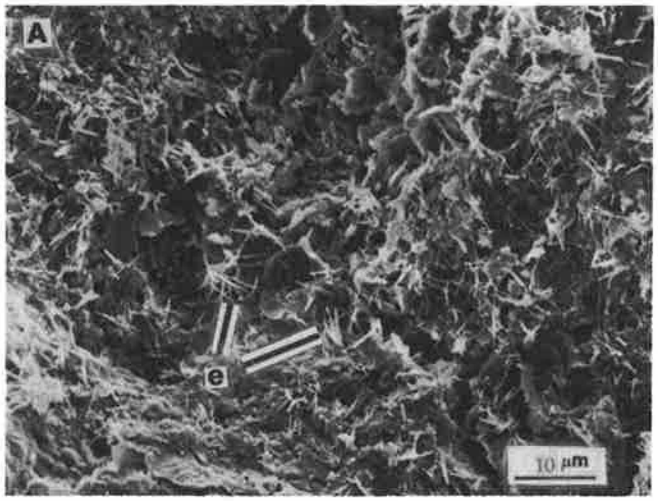
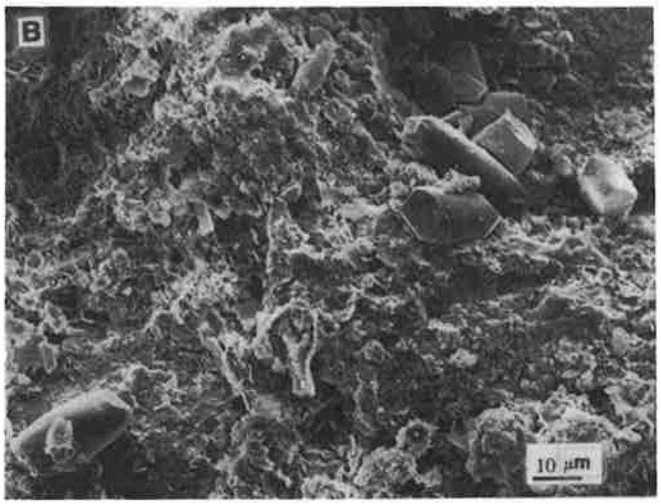


FIGURE 4 Texas soil: (A) 3 percent $Ba(OH)_2$ + 6 percent lime treatment, CBR test with 14-day soak, showing ettringite, 1,550 ×; (B) 3 percent $BA(Cl)_2$ + 6 percent lime treatment, CBR test with 14-day soak, showing ettringite, 790 ×; (C) 15 percent $Ba(OH)_2$ treatment, 2-month soak, 200 ×; (D) 15 percent $Ba(Cl)_2$ treatment, 2-month soak, 35.7 ×.





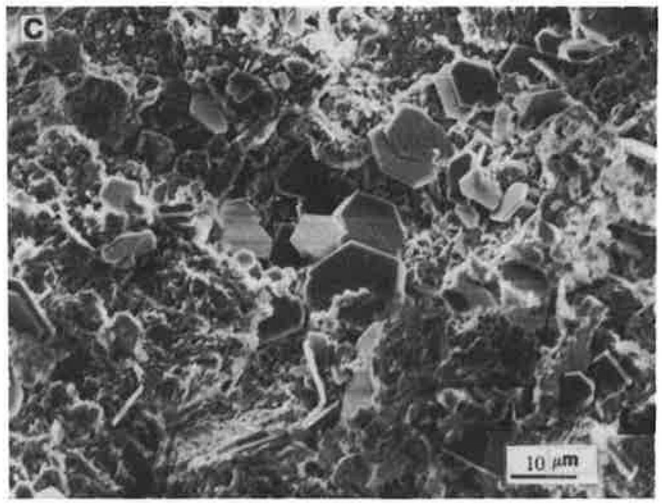


FIGURE 5 (A) Texas soil treated with 3 percent $Ba(OH)_2$ + 6 percent lime, PVC test with 7-day soak, showing ettringite, 1,350 ×; (B) Texas soil treated with 3 percent $Ba(OH)_2$ + 6 percent lime, PVC test with 7-day soak, 700 ×; (C) Colorado soil treated with 3 percent $Ba(OH)_2$ + 6 percent lime, PVC test with 7-day soak, 1,180 ×.

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Effect of Lime on Volume Change and Compressibility of Expansive Clays

ADNAN A. BASMA AND ERDIL R. TUNCER

Heave and settlement of clayey soils pose a difficult problem to civil engineers. Several methods are usually suggested to control this problem. The most common method is the addition of stabilizing agents, such as lime. An evaluation of the soil-lime system for two soils typical of the highly expansive soils existing in Irbid city in northern Jordan is presented. The lime was added to the soils at 0 to 9 or 12 percent. The soil-lime specimens were cured for 1 hr, 7 days, and 28 days, after which they were subjected to laboratory tests. The properties obtained were the grain size distribution, consistency limits, chemical composition, swell potential, swell pressure, compression and rebound indices, rate of swell and consolidation, immediate settlement, and primary consolidation as percent of total settlement. Generally, lime is found to be most effective in stabilizing heave and settlement of expansive clays.

Geotechnical engineers know that excessive heaving and settlement of clayey soils almost always cause serious damage to overlaying structures. In the past few decades, several investigators have conducted studies to evaluate the important factors that influence both heave and settlement of soils. In addition, various researchers have suggested different methods of stabilization to modify and improve soil properties (*1–3*). Stabilization techniques are usually mechanical or chemical, or both. Generally, the addition of chemical stabilizing agents, such as lime, cement, fly ash, salt, etc., are favored (*1,2,4–7*). Lime has been the most widely used chemical for clays. However, most researchers have concentrated on the effect of lime on the swelling of expansive clays with little attention given to the compressibility problems in such soils.

The effectiveness of lime in reducing the volume change of expansive clays and the possibility of lime stabilization to improve the compressibility characteristics of clays were investigated.

LIME-SOIL REACTION

When lime is added to clay soils in the presence of water, several reactions occur that alter some of the soil properties. These changes cause amelioration (*5,8*). Commonly, lime stabilization proceeds through a combination of (a) cation exchange, (b) flocculation and agglomeration, (c) carbonation reaction, and (d) pozzolanic reactions (*7–10*). Furthermore, lime addition to soils increases the pH of the soil-water system, reaching a maximum of about 12.3 when the soil is fully saturated with lime (*11*). In general, most researchers agree that fine-grained soils react favorably with lime, resulting in beneficial changes in soil plasticity, workability, and swell. Yet, in some cases lime is not the ideal solution to the volume change problem. There are several cases in the literature on lime-induced heave (*11,12*); however, this phenomenon is rare.

Hunter (*11*) stated that, under certain conditions, both the sulfate and clay minerals react with lime to form thaumasite and ettringite minerals, which cause the heave. Mitchell (*12*) reported a failure through ettringite growth in a parking lot in Wichita, Kansas. Additional cases of lime-induced heave were uncovered in Texas and Utah by Hunter (*11*).

SOILS OF IRBID CITY

Irbid city, located in the northern part of Jordan, has a semiarid climate. This climate is known to aggravate swelling problems, especially with the existing native soils. Generally, the soils in Irbid are mainly dark to reddish brown, weathered, firm, intensely fissured clay with an average clay content of about 65 percent. Most of these soils contain highly plastic and expansive clays (Figures 1 and 2) and hence have problems caused by swelling. This fact inevitably calls for stabilization. With the abundance of lime in Jordan, it seemed to be the most logical choice. For this study, two typical soils (termed Soils A and B) in Irbid were selected. The physical properties of the soils (Table 1 and Figures 1 and 2) and chemical analysis of the pore water (Table 2) are presented.

SAMPLE PREPARATION AND EXPERIMENTAL PROGRAM

The testing was conducted in two phases. Phase I entailed the determination of grain size distribution, consistency limits, and chemical analysis of the pore water in the soil that directly or indirectly affected both the volume change and compressibility. Phase II consisted of ascertaining the swelling and compressibility characteristics of the soils.

The soil, oven-dried for 4 days at 50°C, was then mixed with a calculated amount of hydrated lime to obtain a predetermined lime percentage, which varied from 0 to 9 or 12 percent. Water was added until it was equivalent to optimum water content plus 3 percent. The soil-lime-water was thoroughly mixed and kneaded by hand until the mixture became homogeneous. A specific weight of this mixture producing a wet unit weight as that in Table 1 for Soils A and B was then

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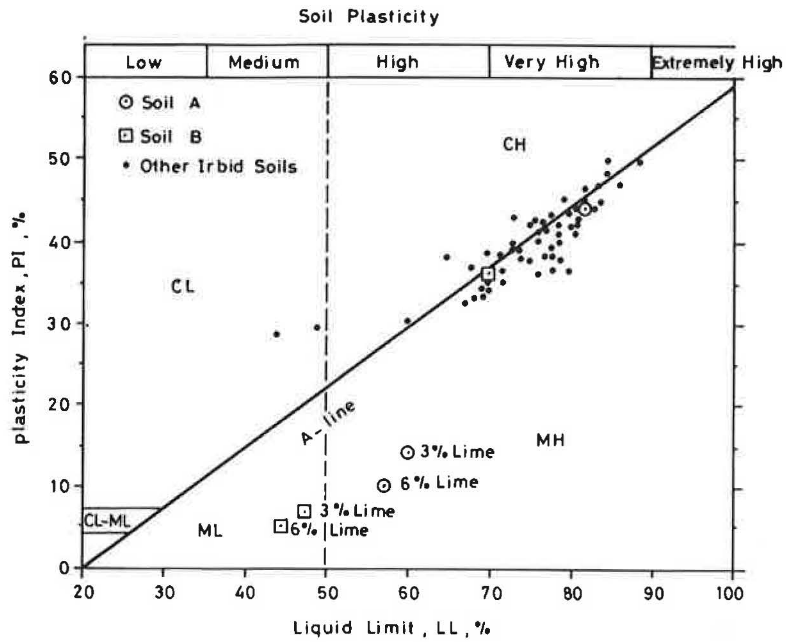

FIGURE 1 Plasticity chart of Irbid clay.

compacted at maximum dry unit weight in a standard Proctor compaction mold and a consolidation ring (of 76-mm diameter and 20-mm height) for tests in Phases I and II, respectively. The specimens were wrapped in plastic and tightly encased in a plastic bag to prevent moisture loss and were set to cure at 22°C and 70 percent relative humidity for 1 hr, (0 days), 7 days, and 28 days. After the prescribed curing period, about 1 kg of the soil in the compaction mold was thoroughly sieve-washed. The soil passing sieve #200 (<75 μm) was oven-dried (4 days at 50°C) for hydrometer analysis to determine the clay size distribution. The remaining soil in the mold was used for the consistency limits test.

Similarly, the consolidation ring containing the soil was placed in the loading apparatus after the curing period. An

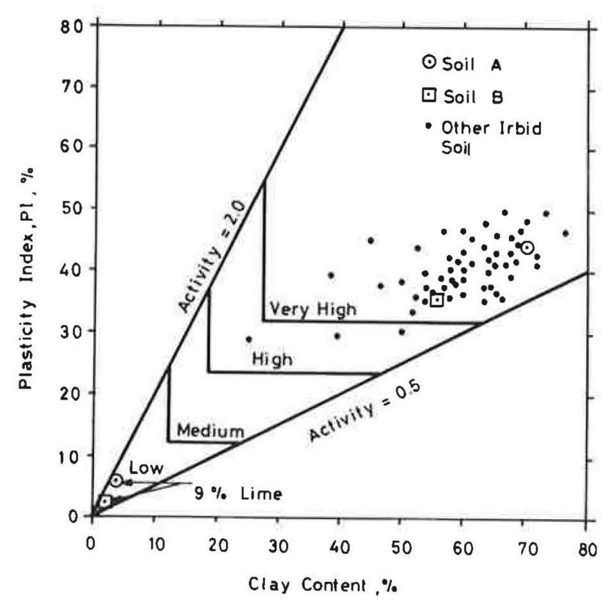

FIGURE 2 Potential expansiveness of Irbid clay.

initial seating pressure of 25 kN/m^2 was applied, which was assumed to be the surcharge load. Dial readings were recorded until deformation stopped, which occurred within the first 2 min of loading. The specimen was then submerged in water and dial readings were recorded at ¼, ½, 1, 2, 4, 8, 15, 30, 60, 120 min, and 24, 48, and 72 hr. In most cases, full expansion occurred after 24 hr, when the specimen was consolidated under incremental pressure levels. The swell pressure in this case is defined as the pressure required to consolidate a preswollen sample to its initial void ratio (or height).

PRESENTATION AND ANALYSIS OF RESULTS

Phase I

Effect of Lime on Grain Size Distribution

Variations in coarse grain fractions (>75 μm) and clay size fractions (<2 μm) of the soil for the various lime percentages and curing times are shown in Figures 3 and 4, respectively. Increasing lime percentage and curing time causes the clay particles to amass by cementation and to form silt and sand-like grains. This assertion is further substantiated by scanning electron micrographs (×200 magnification) of Soil A with 0, 3, 6, and 9 percent lime cured for 28 days (see Figure 5). Clearly, as the lime percentage increases, the soil becomes more granular.

Effect of Lime on Chemical Composition

Treated and untreated samples of Soils A and B were chemically analyzed for Na^+, K^+, Ca^{2+}, Mg^{2+}, Mn^{2+}, Al^{3+}, NO_3^-, HCO_3^-, Cl^-, SO_4^{2-}, and PO_4^{3-} by extracting the soluble ions in the soils at a water-soil ratio of 50:1. Figures 6 and 7 show the changes in the anions and cations with varying per-

TABLE 1 PROPERTIES OF TESTED SOILS

Physical Property	Soil A	Soil B
Depth of sampling (meters)	1.3	1.6
Natural water content, w_n (%)	33.8	29.2
Dry unit weight, γ_d (kN/m^3)	15.0	18.0
Optimum water content, w_{opt} (%)	30.5	26.8
Maximum dry unit weight, $\gamma_{d_{max}}$ (kN/m^3)	14.5	17.6
Specific Gravity, G_s	2.67	2.75
Initial void ratio, e_o	0.75	0.50
Liquid limit, LL (%)	81.5	70.0
Plasticity index, PI (%)	44.1	35.5
Shrinkage limit, SL (%)	2.9	14.1
Sand percent		
Course (2000 µm - 600 µm)	0.0	1.0
Medium (600 µm - 200 µm)	1.0	3.0
Fine (200 µm - 75 µm)	4.0	6.0
Silt percent (75 µm - 2 µm)	25.0	35.0
Clay percent (< 2 µm)	70.0	55.0
Liquid limit of particles < 2 µm (%)	112.0	98.0
Plastic limit of particles < 2 µm (%)	66.0	55.0
Activity, A	0.63	0.65
Free Swell (%)	120.0	90.0

TABLE 2 CHEMICAL ANALYSIS OF THE PORE WATER OF THE UNTREATED SOILS

Ions	Soil A		Soil B	
	mg/g of soil	% of total milliequivalent/liter	mg/g of soil	% of total milliequivalent/liter
Cations				
Na^+	1.41	51.26	1.36	66.92
K^+	0.15	5.45	0.10	4.35
Ca^{2+}	0.83	30.17	0.37	18.21
Mg^{2+}	0.19	6.91	0.13	6.40
$Mn^2{}_+$	0.00	0.00	0.00	0.00
Al^{3+}	0.17	6.18	0.08	3.95
Anions				
NO_3^-	0.02	0.40	0.03	0.93
HCO_3^-	1.51	39.81	1.45	45.13
Cl^-	1.50	39.55	1.22	37.97
SO_4^{2-}	0.75	19.77	0.50	15.56
PO_4^{3-}	0.02	0.47	0.01	0.40

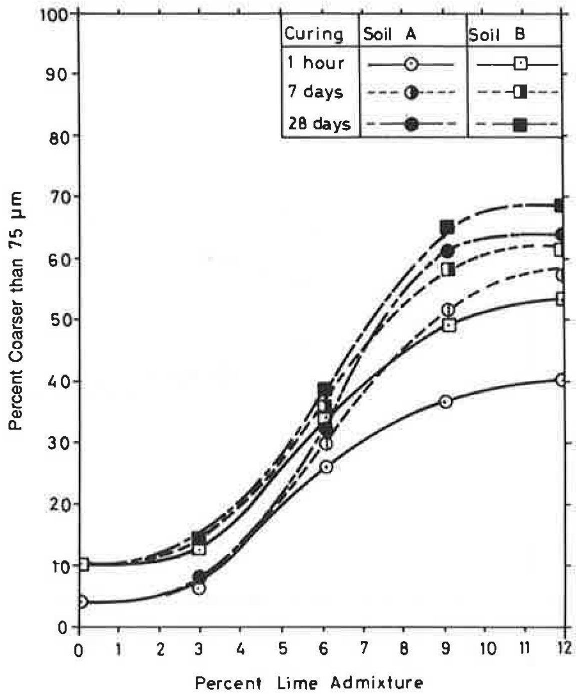

FIGURE 3 Effect of lime admixture and curing time on the coarser-grain fraction of Soils A and B.

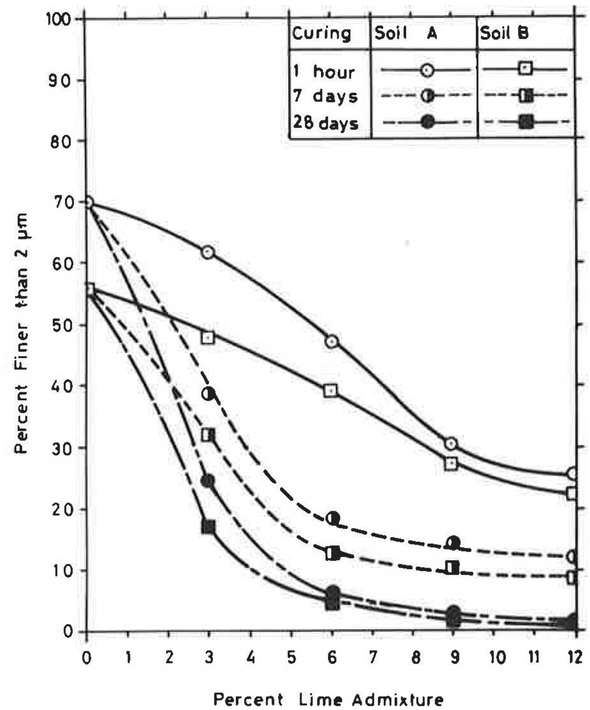

FIGURE 4 Effect of lime admixture and curing time on the clay size fraction of Soils A and B.

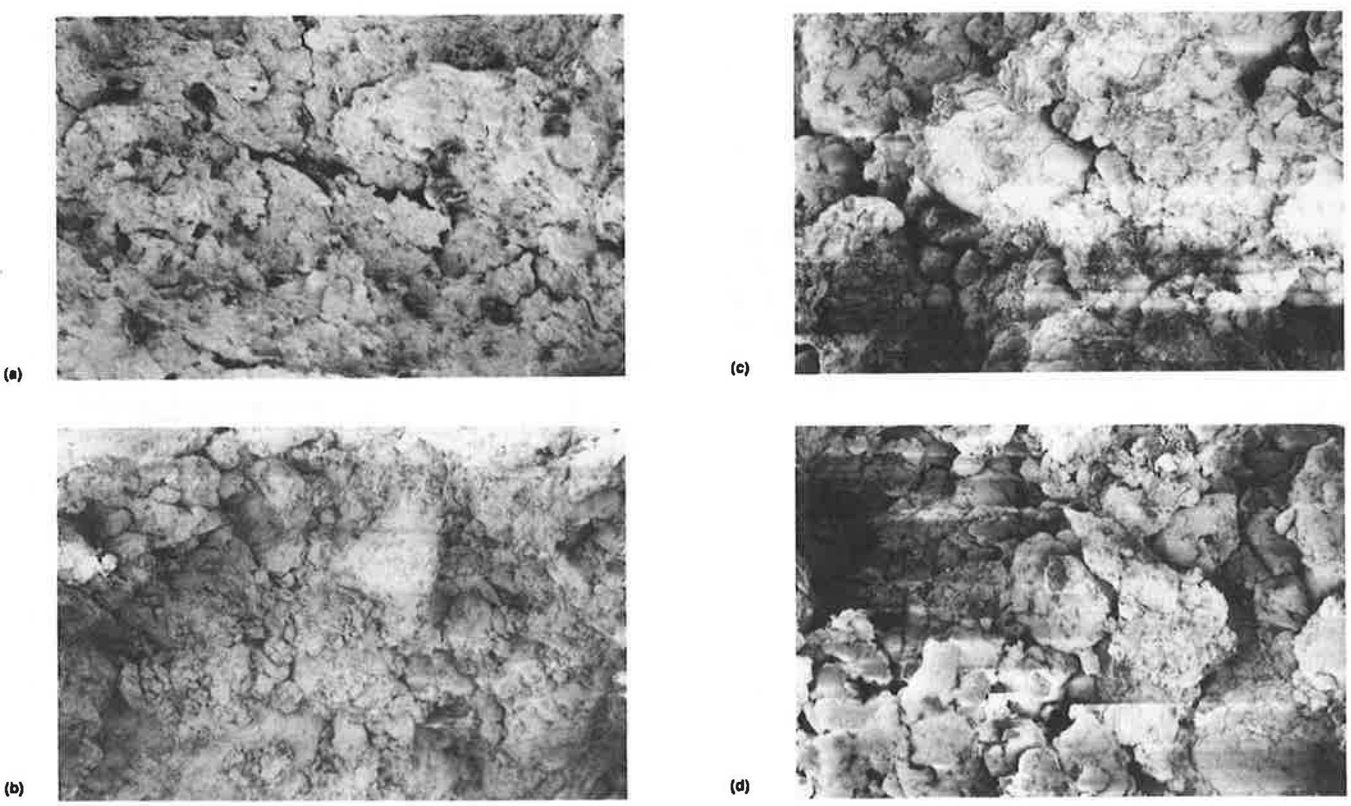

FIGURE 5 Scanning electron micrograph of Soil A with (a) 0 percent, (b) 3 percent, (c) 6 percent, and (d) 9 percent lime, cured for 28 days. (Magnification: ×96)

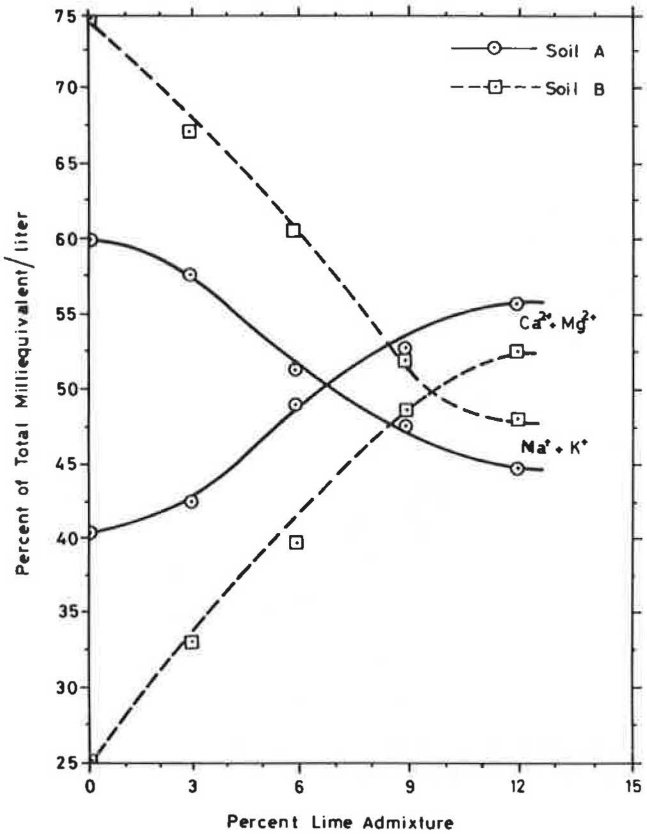

FIGURE 6 Variation of cations with lime for Soils A and B.

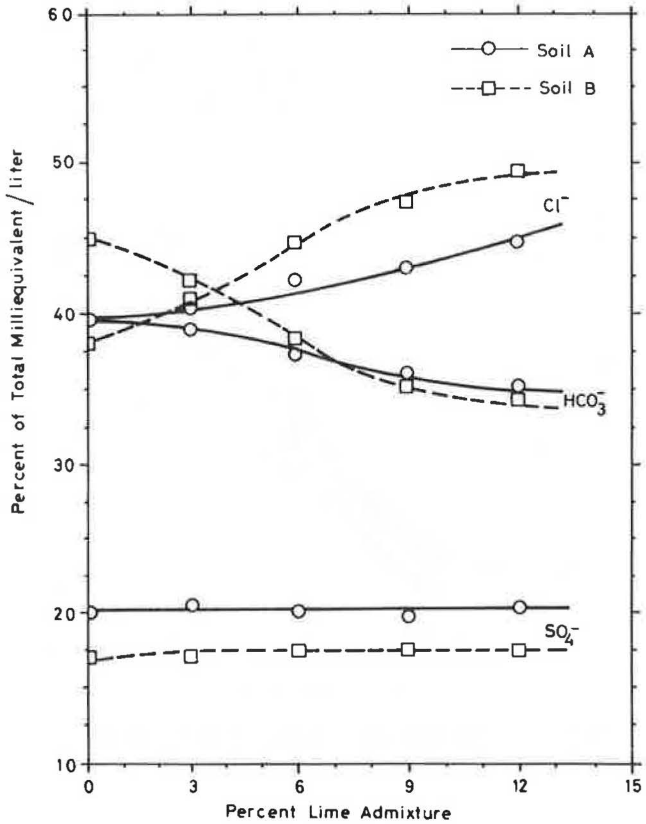

FIGURE 7 Variation of anions with lime for Soils A and B.

cents of lime. Flocculating agents Ca^{2+} and Mg^{2+} increase with increasing lime, whereas deflocculating agents Na^{+} and K^{+} decrease with increasing lime.

Effect of Lime on Consistency Limits

The results of the consistency limits test are shown in Figures 8 and 9 for Soils A and B, respectively. The plasticity index (PI) values of the soils are substantially decreased and the shrinkage limit is increased with increasing lime. However, no significant effect of curing time is noted. The reduction in plasticity can be ascribed to the increasingly granular nature of the soils with lime.

Phase II

Effect of Lime on Volume Change

Figures 10 and 11 show the effect of lime percentage and curing time on swell potential and swell pressure for Soils A and B. The swell pressure is obtained by consolidating the swollen sample to its initial height, i.e., the pressure in Figure 12 at zero expansion. Figure 12 shows a typical result. As can be seen from Figures 10 and 11, lime percentage and curing time profoundly reduce swell potential and swell pressure. The optimum water content plus 3 percent and the maximum unit weight were used as initial values. The decrease in swell

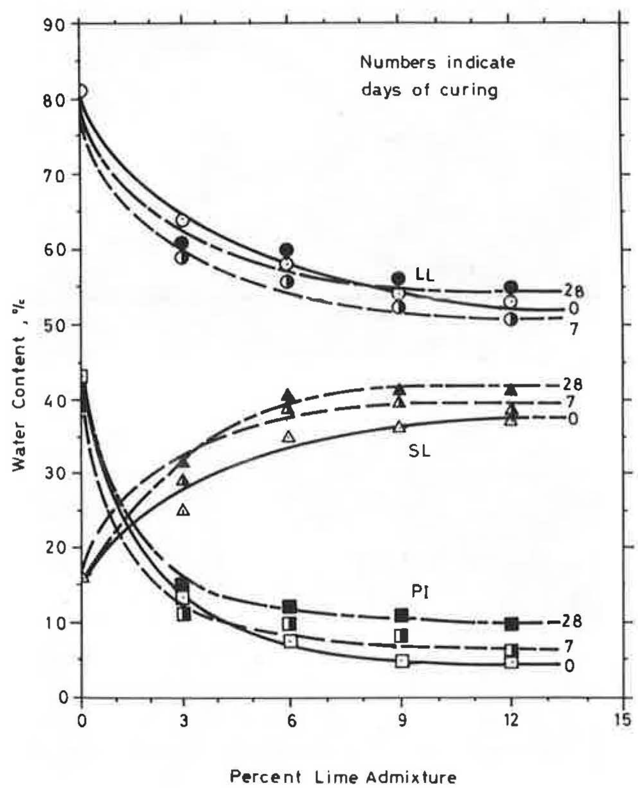

FIGURE 8 Effect of lime admixture and curing time on the consistency limits for Soil A.

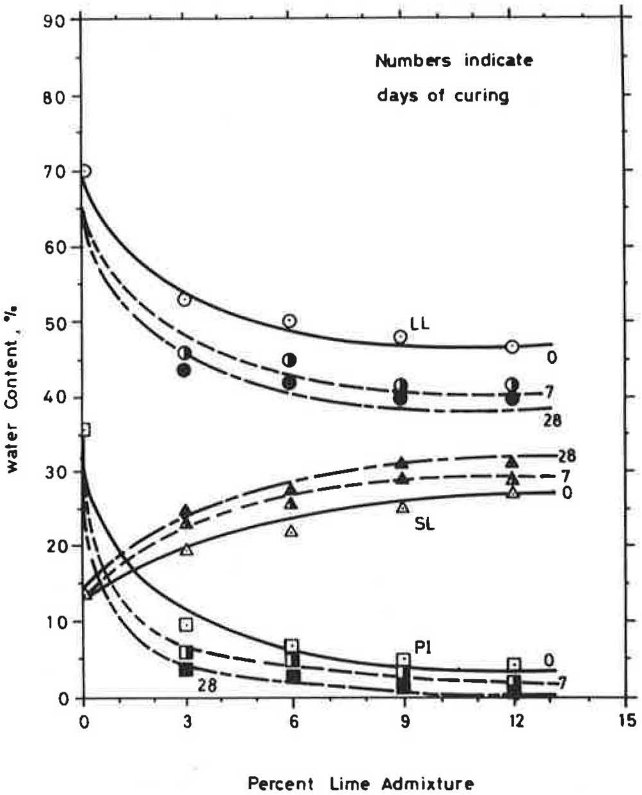

FIGURE 9 Effect of lime admixture and curing time on the consistency limits for Soil B.

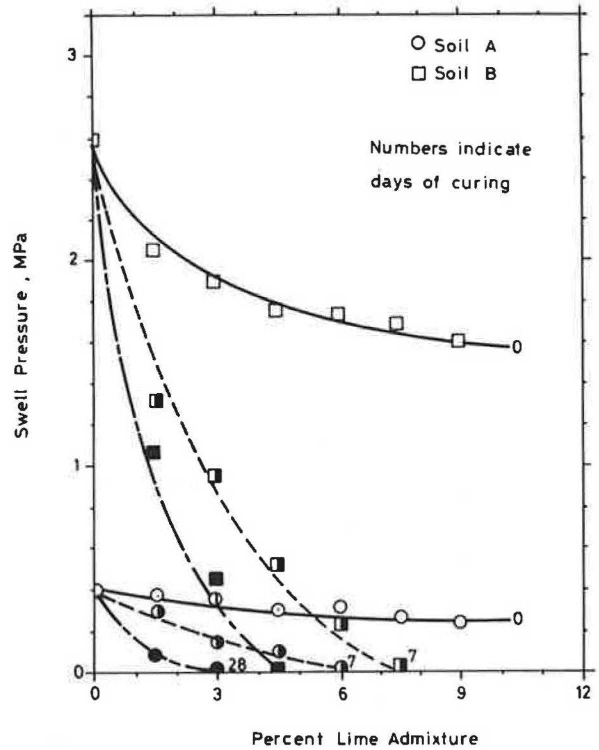

FIGURE 11 Effect of lime admixture and curing time on the swell pressure of Soils A and B.

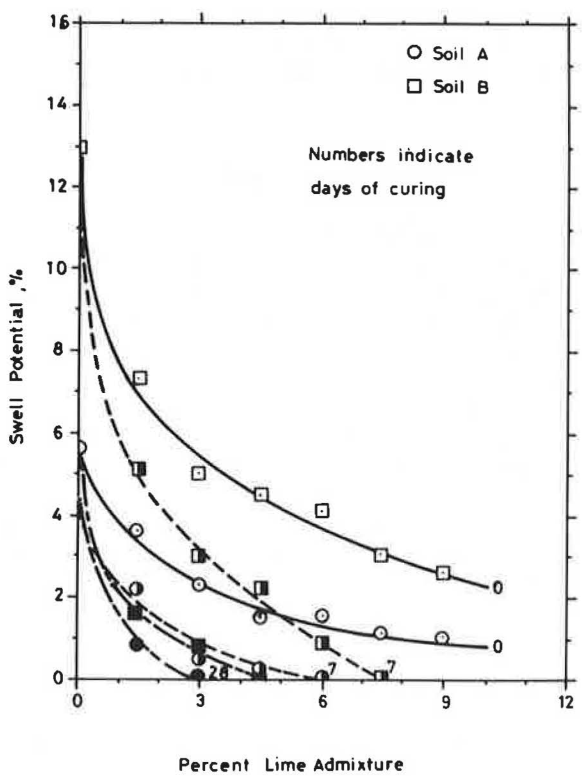

FIGURE 10 Effect of lime admixture and curing time on the swell potential of Soils A and B.

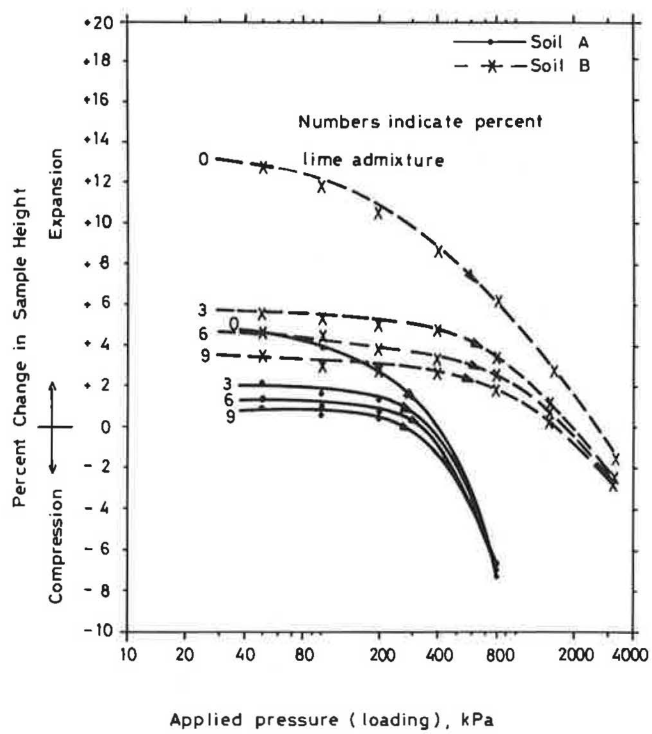

FIGURE 12 Percent change in sample height versus loading applied pressure for 1-hr curing time.

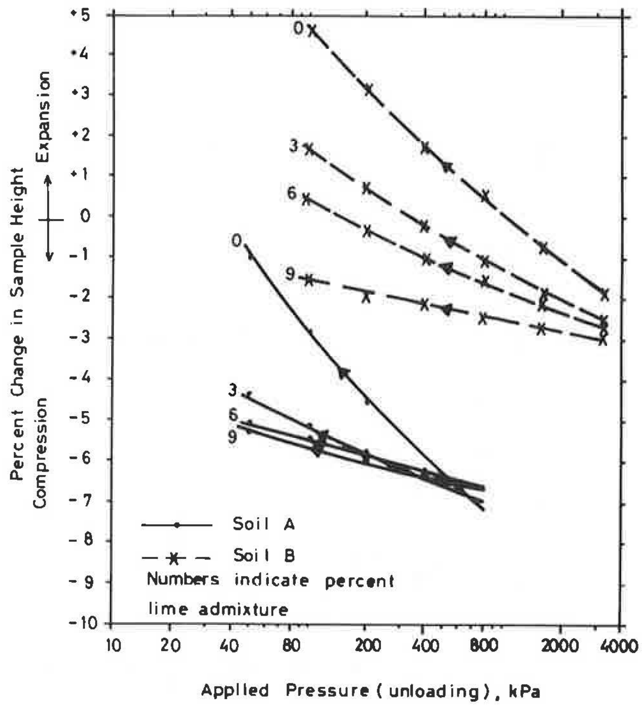

FIGURE 13 Percent change in sample height versus unloading applied pressure for 1-hr curing time.

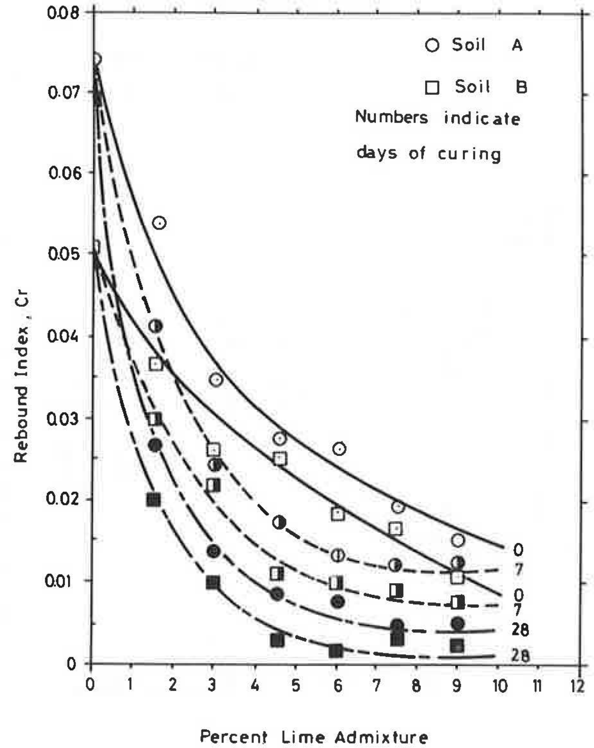

FIGURE 15 Effect of lime admixture and curing time on the rebound index for Soils A and B.

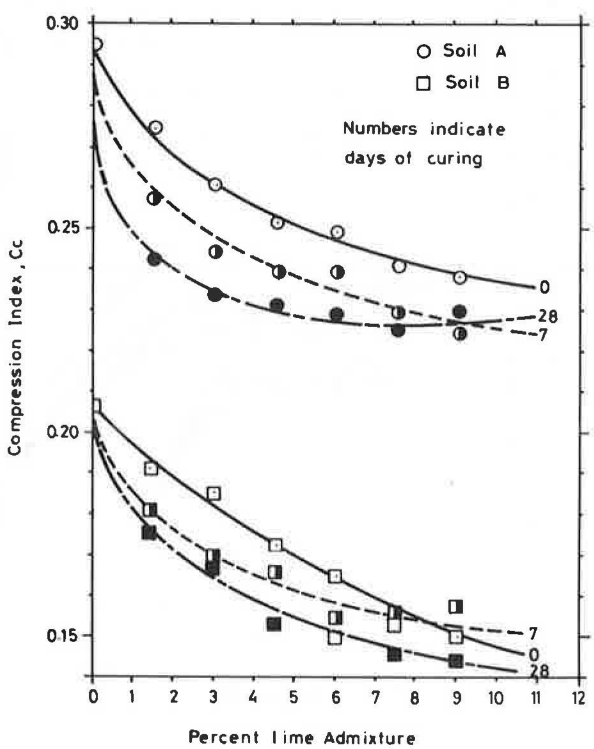

FIGURE 14 Effect of lime admixture and curing time on the compression index for Soils A and B.

characteristics can be attributed to the reduced water absorption tendency of the calcium-saturated clay and the development of a cementitious matrix that can resist expansion.

Effect of Lime on Compressibility and Rebound

Treated and untreated preswollen specimens with different curing periods were consolidated under incremental pressure levels. After the specimens were brought to their initial heights or beyond this point, the loads were gradually removed to study the rebound characteristics. Figures 12 and 13 show typical observed results. Using data from these figures, the compression and rebound indices are plotted in Figures 14 and 15, respectively. These figures suggest that both percent of lime and curing time have influence both on compression and rebound indices, but the effect is more pronounced on the rebound index. This increased tendency to resist compression and rebound can be accounted for by the cementing ability of lime.

Effect of Lime on the Type of Compression

Using the deformation versus time plots, the immediate settlement and primary consolidation were evaluated. Figures 16 and 17 show the variation of percent immediate settlement and primary consolidation of the total compression with per-

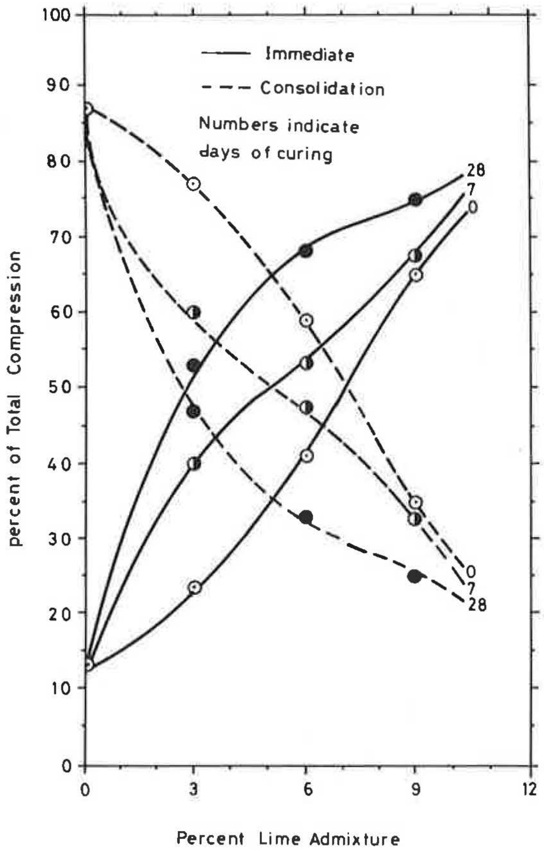

FIGURE 16 Effect of lime admixture and curing time on the type of settlement for Soil A.

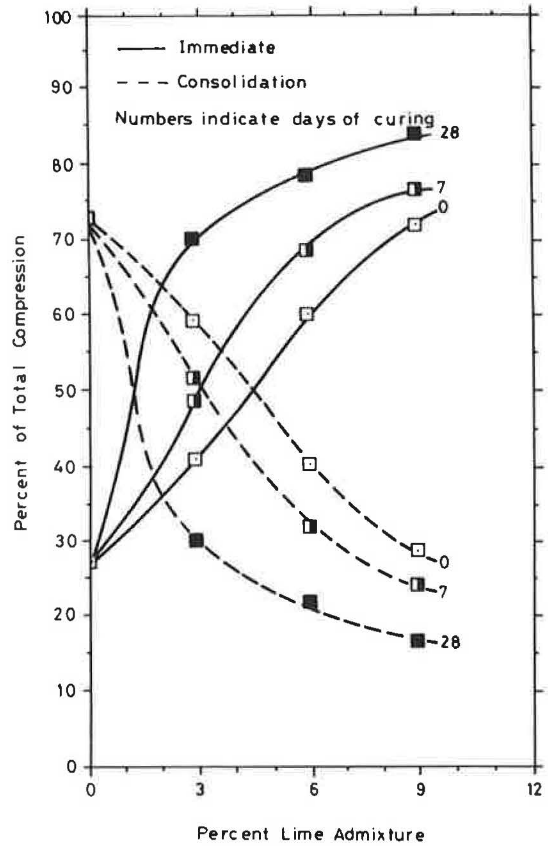

FIGURE 17 Effect of lime admixture and curing time on the type of settlement for Soil B.

cent lime and curing time at 800 kPa pressure level. These figures demonstrate an increase in immediate settlement and decrease in primary consolidation with increasing lime and curing time, indicating that the soils are approaching granular soil behavior.

Effect of Lime on Rate of Compression and Swell

The rate of compression, or consolidation in the case of saturated soils, is usually governed by the rate at which the pore water can escape from the soil. One parameter commonly used to define the rate of consolidation is c_v, which is defined as

$$c_v = \frac{k}{m_v \gamma_w} \tag{1}$$

where

k = coefficient of permeability,
m_v = coefficient of volume change, and
γ_w = unit weight of water.

Alternatively,

$$c_v = \frac{T_v d^2}{t_{\alpha c}} \tag{2}$$

where

T_v = time factor,
d = one-half the thickness of the specimen for two-way drainage, and
$t_{\alpha c}$ = time to α percent consolidation.

If α is taken as 50 percent, for instance, then $T_v = 0.196$; and with d being almost constant for a given pressure increment, then c_v will simply be a function of $t_{\alpha c}$. Thus, the rate of consolidation will be defined in terms of t_{50c}. For swelling, on the other hand, there is no readily available method for measuring the rate. Therefore, the rate of swell will be defined as the time to 50 percent swell, t_{50s}, i.e., the time to half the full swell. Figures 18 and 19 show t_{50c} and t_{50s}, respectively, as functions of percent lime and curing time for Soils A and B. Figure 18 was prepared for a pressure increment of 800 kPa. From these figures, both the rate of compression and swell increase, i.e., t_{50c} and t_{50s} decrease, with increasing lime. In explanation, as the percent lime and curing time increase, the soil becomes more granular, which implies that the permeability increases.

LIME TREATMENT COMPRESSION RATIO

One of the most important parameters usually used to define the compression of clays is the preconsolidation pressure, p_c.

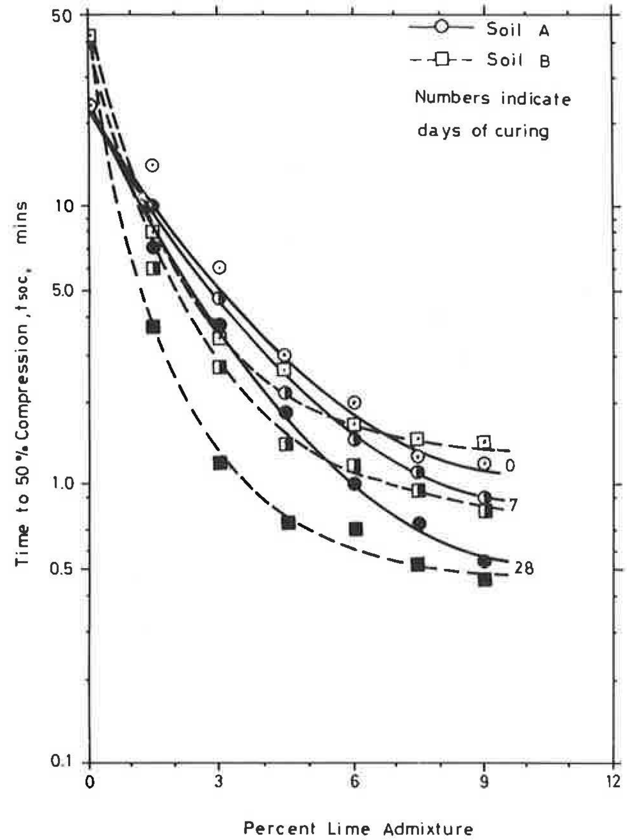

FIGURE 18 Effect of lime admixture and curing time on the rate of compression for pressure p = 800 kPa.

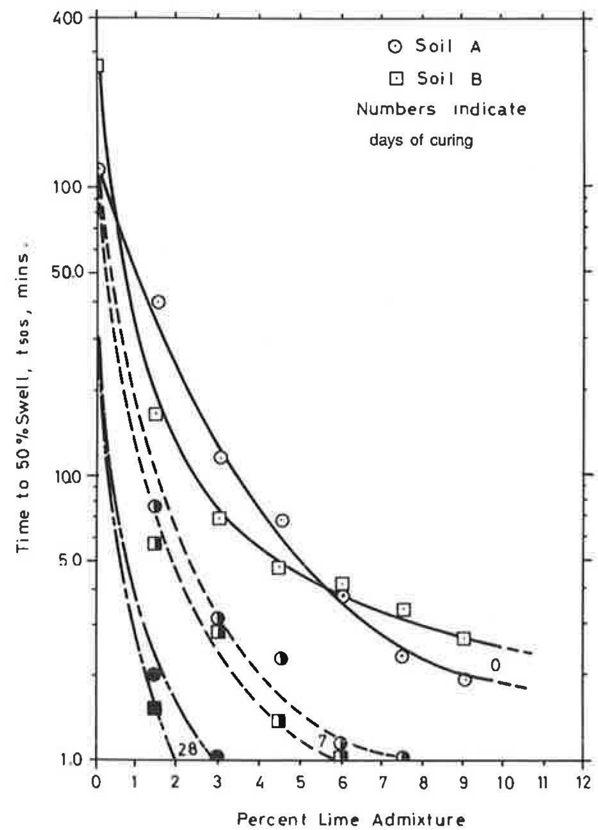

FIGURE 19 Effect of lime admixture and curing time on the rate of swell.

Higher p_c values indicate smaller compressibility of the clay as long as the applied pressure is less than p_c. Using this concept, the preconsolidation pressures of lime-treated specimens were estimated by Casagrande's method from the compression versus applied pressure plots. In order to determine how compressible a lime-treated specimen is in relation to an untreated specimen, the lime treatment compression ratio (LTCR) is defined as follows:

$$\mathrm{LTCR} = p_c(T)/p_c(U) \tag{3}$$

where $p_c(T)$ and $p_c(U)$ are the preconsolidation pressures of treated and untreated specimens, respectively. Thus, higher LTCR implies lesser compressibility. Figure 20 shows LTCR in relation to the percent lime admixture for Soils A and B. Observe that LTCR increases with percent lime and curing time; therefore, compressibility decreases. Also, LTCR is independent of the type of soils tested. This observation, however, may not be true for different soils. Furthermore, the preconsolidation pressure of lime-treated specimen bears no physical meaning, yet, it could still be considered as a good artificial measure of compressibility.

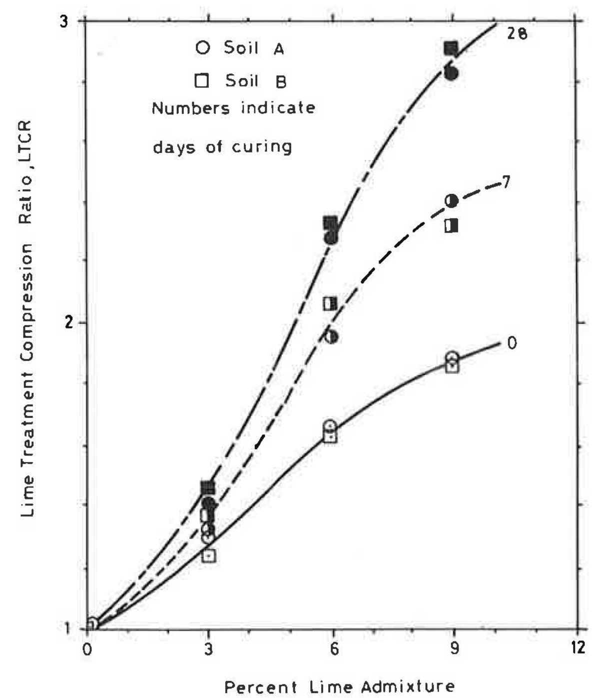

FIGURE 20 Variation of LTCR with lime and curing time.

CONCLUSIONS

The effect of lime treatment on volume change and compressibility of two soils from Irbid City was presented. Lime was introduced as an admixture up to a maximum of 12 percent by dry weight of the soil. From the results obtained and for the percentages of lime used, the following conclusions are warranted:

1. The grain size distributions of the soils tested are greatly altered by the addition of lime. The coarse-grain fractions increased whereas the clay fractions decreased with increasing lime and curing time.
2. Concentrations of calcium and magnesium ions in pore water increased, whereas those of sodium and potassium ions decreased with increasing lime.
3. Liquid limit and plasticity index decreased whereas shrinkage limit increased with percent lime. No significant effect of curing time was observed.
4. Addition of lime changed classification of treated soil from MH-CH for untreated to MH and ML at 3 and 6 percent lime for Soils A and B, respectively (see Figure 1). The classification at 9 percent lime was SM.
5. The changes in the physical properties caused by the addition of lime decreased the potential expansiveness of the soils from very high to low (see Figure 2). This is further reflected both in swell percent and swell pressure measurements, which decreased with increasing percent lime and curing time.
6. A reduction in compression and rebound indices is achieved with increasing percent lime. Curing time had no marked influence.
7. Increasing percent lime and curing time increased immediate settlement and decreased primary consolidation.
8. The rate of swell and consolidation increased with percent time.
9. The concept of LTCR was introduced and LTCR was found to increase both with percent lime and curing period. Higher LTCR signifies lower compressibility.

ACKNOWLEDGMENTS

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Overconsolidated Behavior of Phosphoric Acid and Lime-Stabilized Kaolin Clay

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Consolidation and shear strength properties of a kaolin clay stabilized with hydrated lime or phosphoric acid were investigated. Consolidation test results indicated that lime or phosphoric acid treatments cause a chemically induced preconsolidation effect of the normally consolidated kaolin clay. The overconsolidation behavior caused by chemical addition is influenced by the type and amount of chemicals and the applied original preconsolidation pressure. Lime or phosphoric acid also decreases the rebound compressibility index (C_r) of kaolin clay. Consolidated, undrained, shear box tests indicate that the addition of lime or phosphoric acid to clay increases its cohesion and its angle of internal friction and that the strength gain of chemically treated clay is improved and accelerated by consolidation with aging. These findings encourage consideration of chemical treatment as a possible alternative for the control of settlements of kaolin clayey soils.

The improvement of cohesive soils by chemical additives has been widely practiced to improve soil properties such as high plasticity, poor workability, high volume change potential, and low shear strength.

Efforts are made to select appropriate chemical stabilizers and to use them in the most efficient manner to improve poor soil conditions. Previous soil stabilization studies, for example, Radnor and McGee (*1*), Demirel et al. (*2*), Demirel and Davidson (*3*), Lyons and McEwan (*4*), Kelley and Kinter (*5*), Guinne (*6*), Thompson (*7*), Kezdi (*8*), Ghazali (*9*), Baghdadi (*10*), Balasubramaniam and Buensuceso (*11*), Fossberg (*12*), and Wissa et al. (*13*), indicated that hydrated lime and phosphoric acid improve many engineering properties of fine-grained soils, making them potentially competitive with other soil stabilizers, such as asphalt or portland cement. A review of the present state of the art of lime or phosphoric acid stabilization reveals that little work has been done on the consolidation behavior of chemically stabilized clays using lime or phosphoric acid and that there is not much detail on the compressibility behavior of stabilized-aged clay soil mixes.

Thompson (*7*) investigated plastic soils (with plasticity index values of 8 to 29) stabilized with lime and studied their triaxial shear and unconfined compressive strengths. He concluded that the shear strength of lime-treated soils increased primarily because of a greater increase in cohesion with small increase in the angle of shearing resistance. The shear strength of lime-soil mixtures increased with curing periods.

Fossberg (*12*) investigated some fundamental engineering properties of a lime-stabilized montmorillonitic clay. He concluded that lime-stabilized clays, in the saturated state, could be analyzed in terms of generally accepted concepts in regard to consolidation characteristics, suction, permeability, and shear strength. He reported that the clay behaved like a preconsolidated material on account of cementation of particles. This effect further increased with higher stabilizer content and curing time.

Wissa et al. (*13*) studied effective stress strength parameters of saturated soil-cement and soil-lime mixes. The study was conducted on Massachusetts clayey silt and Vicksburg Buckshot clay using consolidated undrained triaxial tests with pore pressure measurements. Mohr effective stress strength envelopes were essentially straight lines with slopes independent of curing time; but the cohesion intercept increased with increase in curing time, and the strength envelopes (both for lime and cement mixtures) were sensitive to the type and amount of stabilizer and type of soil.

Balasubramaniam and Buensuceso (*11*) investigated the strength and deformation characteristics of lime-treated soft Bangkok clays under undrained and drained triaxial conditions. They found that lime treatment causes a change in the strength and deformation characteristics of the soft clay from normally consolidated clay to that of an overconsolidated clay. They reported that this observation was valid for lime contents varying from 5 to 15 percent, and that the strength of treated clay increased with curing time.

Radnor and McGee (*1*) studied compressive strength of fine-grained soils stabilized with phosphoric acid. They performed series of unconfined and triaxial compression tests and found that the phosphoric acid caused an increase in the strength and that the strengths were higher for the soils with higher clay content. They also found that the confined strength was greater than the unconfined strength.

Demirel et al. (*2*) and Demirel and Davidson (*3*) investigated use of phosphoric acid for stabilizing clay soils and found that the strength and durability characteristics of compacted clays improved by phosphoric acid treatment. The improvement depends on the amount of phosphoric acid used and on the types and amounts of clay minerals in the soil. They concluded that there is an optimum amount of phosphoric acid content to produce the highest soaked and unsoaked unconfined compression strengths of stabilized soils. They also found that there appeared to be a curing time beyond which the soaked strengths do not increase further and this time is dependent on type and amount of clay minerals reacting and the amount of phosphoric acid available for the reactions. They reported that because phosphoric acid is a reactive chemical with the kaolin clay minerals (*3*), its addition to the fine cohesive soils may produce changes in the surface texture and increase angle of friction. But, with excess of acid, the lubrication of particles will tend to reduce angle of friction.

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Such behavior was reported to be dependent on the dry density and water content of the soils investigated.

Lyons and McEwan (*4*) did considerable experimentation with phosphoric acid and different types of clay soils with plasticity index values ranging between 13 and 44. They studied unconfined compression strength of each soil with different acid content and found that the unconfined compressive strengths of all types of clays used increased with the increasing amount of phosphoric acid.

Kelley and Kinter (*5*) studied the plasticity, moisture-density relations, volume change, and unconfined compressive strength of several fine-grained plastic soils stabilized by phosphoric acid. They concluded that phosphoric acid increased unconfined compressive strengths by various degrees. The addition of phosphoric acid brought the moisture absorption and volume changes to satisfactory levels. The moisture-density relationships changed slightly.

These studies indicate that there is a strong interaction between stabilization and consolidation in relation to strength and deformation characteristics. As compared to lime, the literature on phosphoric acid stabilization contains relatively less information on its compressibility characteristics.

An elaborate experimental investigation was carried out to explore the effects of chemical stabilization on compressibility of a clayey soil by choosing lime and phosphoric acid. Large amounts of data were collected over a 3-year period on the consolidation and shear strength properties of hydrated lime and kaolin clay mixes stabilized by phosphoric acid.

MATERIALS AND TESTING METHODOLOGY

The experimental investigation was carried out on pure kaolin clay. The clay was stabilized by adding varying percentages of lime and phosphoric acid. The properties of the tested materials are presented in Table 1. Mineral identification with X-ray diffraction methods for the pure kaolin clay is shown in Figure 1. The effect of stabilizers on the pH values of kaolin clay is shown in Figure 2.

Saturated specimens were prepared at the respective liquid limits of treated kaolin depending on the type and amount of added stabilizer. In case of lime-treated samples, dry pulverized kaolin was mixed with powdered lime and mixed thoroughly. To this, the required amount of moisture was added to mold test specimens. In case of phosphoric acid, dry pulverized kaolin and water were mixed together first, and to this mixture the required amount of liquid phosphoric acid was added and mixed. In order to compare the results with the pure kaolin condition, sets of data were obtained by running tests on pure kaolin at corresponding similar moisture contents.

In the various consolidation and shear strength tests, the amounts of hydrated lime or phosphoric acid were arbitrarily chosen to be 4, 8, and 12 percent by weight of dry kaolin.

TABLE 1 PROPERTIES OF MATERIALS USED IN TEST SERIES

1.	**Kaolin clay**			
	Source:	Saudi Ceramics Co., Riyadh, Saudi Arabia		
	Mineral:	Kaolin		
	Atterberg limits:	LL =	56 %	
		PL =	35 %	
		SL =	30 %	
2.	**Hydrated Lime**	CaO =	68.5 %	(Powder)
	(Composition)	SiO_2 =	3.0 %	
		Al_2O_3 =	1.0 %	
		Fe_2O_3 =	0.6 %	
		MgO =	1.0 %	
		SO_3 =	0.5 %	
		Ignition loss =	25.4 %	
		Total =	100 %	
3.	**Orthophosphoric acid**	H_3PO_4 =	98 %	(in liquid form)
	Source :	BDH Chemicals Ltd. Poole, England		
	Maximum limits of impurities:			
	Chloride (Cl)	=	0.001 %	
	Nitrate (NO_3)	=	0.002 %	
	Sulphate (SO_4)	=	0.01 %	
	Calcium and Magnesium	=	0.01 %	

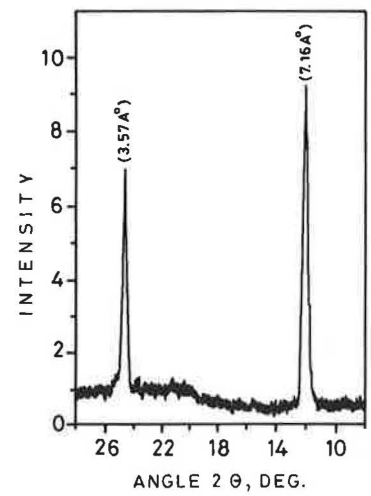

FIGURE 1 X-ray diffraction trace of pure kaolin clay.

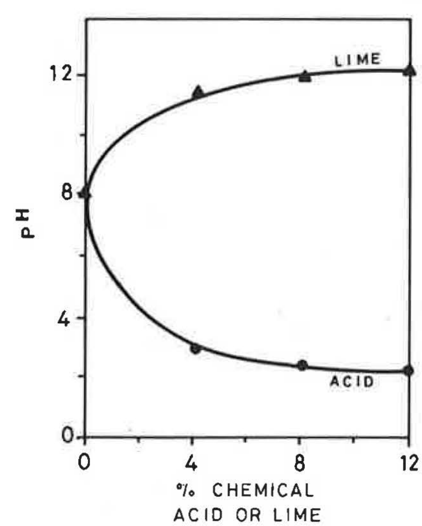

FIGURE 2 pH of phosphoric acid and lime-stabilized kaolin clay.

The preconsolidation pressures used in the investigation were 61, 122, and 245 kPa in all the test series.

In case of consolidation tests on untreated as well as treated soils, the ASTM D2435 procedure was followed. The specimens were loaded to the desired preconsolidation pressure, and then unloaded, and reloaded to 1 MPa. Taylor's square root fitting method was used to determine the coefficient of consolidation (C_v) values for various loading increments.

In the shear strength test series, the specimens were first preconsolidated to the desired pressure in the odometer rings, from which they were trimmed to give shear box specimens. The applied normal stresses were kept equal to the preconsolidation pressures. The specimens were sheared in undrained condition with a strain rate of 2 percent per minute.

The strength behavior of treated clay specimens with aging was studied under two series over varying periods of time, namely, 0, 7, 14, and 28 days both for lime and phosphoric acid treatments. These tests were carried out for only 8 and 12 percent chemical contents. In the first series (SR I), the unconsolidated, undrained, direct shear tests were conducted on stabilized-cured specimens at various ages. These specimens were molded, wrapped, and cured at room temperature (20°C). In the second test series (SR II), similar specimens were tested in the consolidated undrained conditions.

DISCUSSION OF TEST RESULTS

Atterberg Limits

Atterberg limits decreased because of the addition of phosphoric acid to the kaolin clayey soil. The decreases in plastic limit and shrinkage limit were slight and were more pronounced in the case of the liquid limit (Figure 3).

In case of lime treatment, the liquid limit decreased slightly, whereas the plastic limit increased and the shrinkage limit was almost the same as the value for untreated soil. The plasticity index values in both lime and phosphoric acid treatments decreased by the chemical addition, as shown in Figure 3.

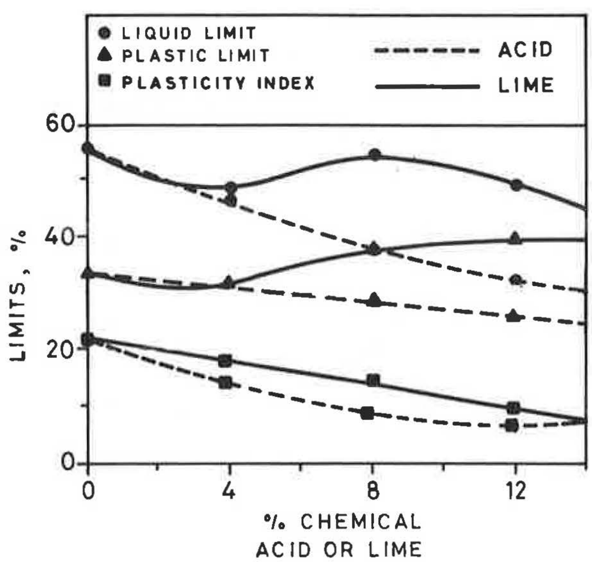

FIGURE 3 Atterberg limits of phosphoric acid and lime-stabilized kaolin clay.

Consolidation Parameters

From 40 consolidation tests, typical plots of settlement versus square root of time, and log P versus void ratio are shown in Figures 4 and 5, respectively, for pure kaolin, acid-treated, and lime-treated soils.

Chemical Preconsolidation Ratio Versus Percent Stabilizer

Figure 6 was obtained from the data presented in Figure 5, both for acid and lime treatments. Both acid and lime impart preconsolidation behavior on the soil. The term chemical preconsolidation ratio (CPR) has been used as a measure of the preconsolidation effects. CPR is defined as the ratio

$$CPR = \bar{p}_{cn}/\bar{p}_{co} \qquad (1)$$

where $\bar{p}_{cn}$ is the new preconsolidation pressure from e-log curve and $\bar{p}_{co}$ is the original applied preconsolidation pressure.

Phosphoric Acid Stabilization The effect of the phosphoric acid stabilizer at different levels of p_{co} is indicated in Figure 6. At low p_{co} (61 kPa), the addition of phosphoric acid increases the CPR for all amounts of acid. At high values of p_{co} (122 and 245 kPa), the addition of phosphoric acid increased the CPR of the 4 percent phosphoric acid. This increase was higher than at low p_{co} (61 kPa), but for 8 and 12 percent phosphoric acid there are decreases in the values of the CPR. This result indicates that the addition of high percentages of phosphoric acid combined with high values of p_{co} will not improve the CPR values.

At a low percentage of stabilizer, the higher the preconsolidation pressure, the higher the CPR. However, at higher percentages of stabilizer, the low preconsolidation pressure caused higher CPR. Thus, a low percentage of stabilizer and

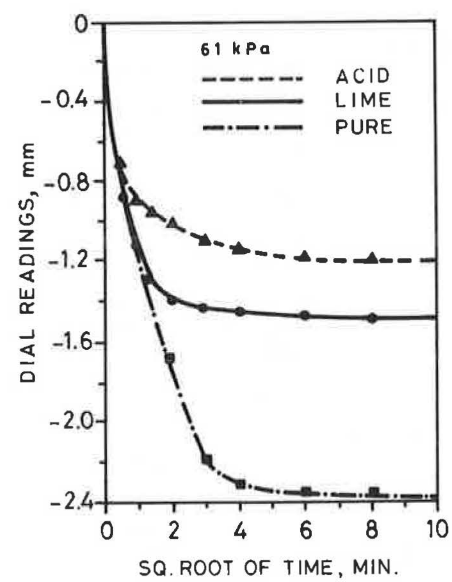

FIGURE 4 Typical square root of time versus settlement curves.

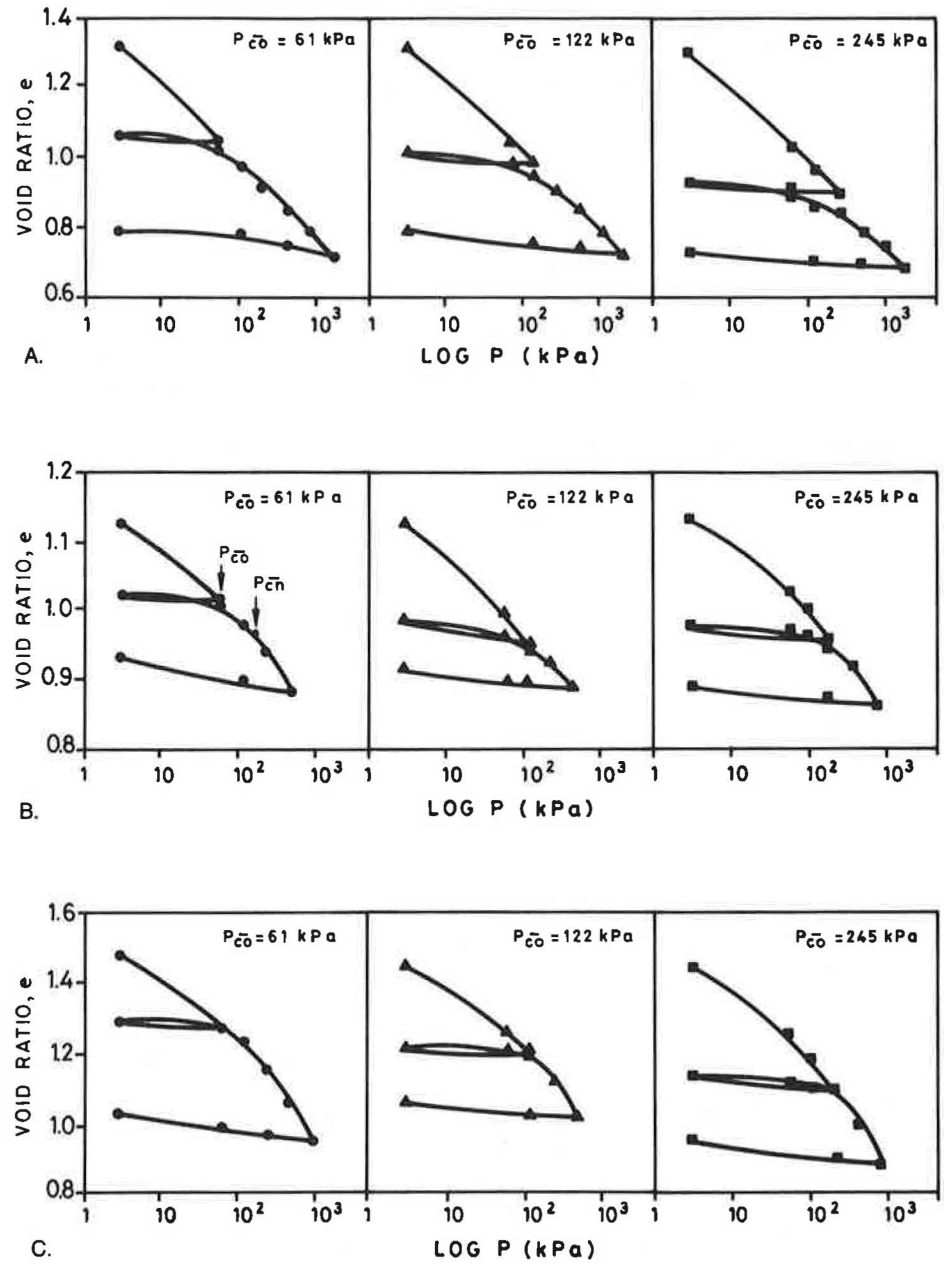

FIGURE 5 Typical void ratio versus log pressure curves: *a*, pure kaolin; *b*, acid-stabilized; and *c*, lime-stabilized.

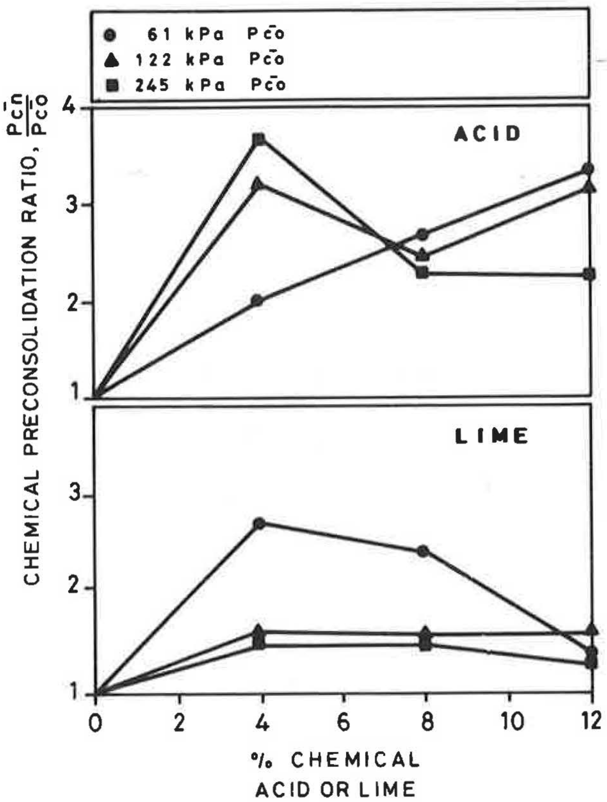

FIGURE 6 Chemical preconsolidation ratios of phosphoric acid and lime-stabilized kaolin clay.

a high preconsolidation pressure are more efficient in producing preconsolidation effects of kaolin clayey soil.

Lime Stabilization As indicated by Figure 6, at low to moderate percentages of lime, the lower the preconsolidation pressure, the higher the CPR value. Thus, a condition of low percentage of lime stabilizer and low preconsolidation pressure is more efficient in producing preconsolidation effects of kaolin clayey soil.

Rebound Compressibility Index (C_r)

The values of C_r were calculated from the consolidation data. The values of C_r (Figure 7) were chosen because of the overconsolidation effects produced as a result of the chemical treatment, and from a practical point of view, it is assumed that the loading will be within the overconsolidation range.

The C_r values were found to be higher in case of untreated soils at different levels of original preconsolidation pressure (p_{co}) for all molding water contents, (corresponding to liquid limits of treated soils), and various percentages of stabilizer. The values of C_r decreased by treating the soil either with phosphoric acid or lime.

The decrease in C_r was most often observed at moderate p_{co} values (122 kPa); at the higher and lower p_{co} values (245 kPa and 61 kPa), the change in C_r was slight. Furthermore, C_r values were generally higher for higher water contents, which indicates more compressibility.

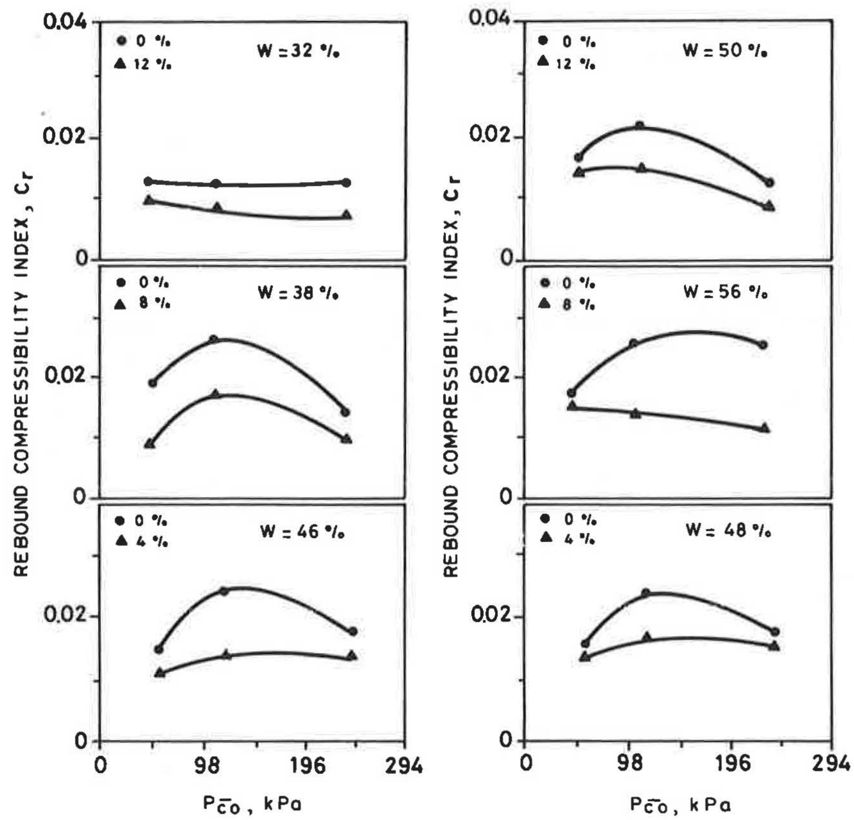

FIGURE 7 Rebound compressibility indices of (*left*) phosphoric acid and (*right*) lime-stabilized kaolin clay.

Coefficient of Consolidation (C_v)

Table 2 presents C_v values for pure and treated kaolin clay, as calculated from data shown typically in Figure 4. Each C_v value is the average of three values of the overconsolidated portion of the e–log p curve. The overconsolidated C_v values were chosen because of the practical application. Table 2 indicates that for all kaolin clay treated with phosphoric acid and lime, the C_v values are generally equal or less than the C_v values of the untreated clay at equal water contents. This result may be caused by cementation effects and lower permeability (*12*).

Shear Strength

Figure 8 shows that the addition of chemicals to kaolin clay increased the shear strength both for phosphoric acid and lime treatments at different levels of preconsolidation pressures. The increase in shear strength was higher in case of lime than for phosphoric acid. Also, both for cases of lime and phosphoric acid, the shear strength increased with increasing percentages of the chemicals and the applied preconsolidation pressures.

Values of the cohesion c and the angle of internal friction ϕ were obtained from the total failure envelopes of pure and stabilized kaolin clay by different percentages of phosphoric acid or lime (Figure 9). The plots of the untreated soil were at the same water contents as that of the treated soils, corresponding to saturated conditions of all samples. From the data for c and ϕ in Figure 9, lime treatment may be more effective than acid treatment in producing higher shear strength, characterized both by cohesion and angle of internal friction values. The figures also show that increase in shear strength is mainly caused by increase in cohesion, which is probably caused by the developed cementation.

Effect of Aging and Consolidation on Shear Strength

In order to study the effect of aging, the undrained shear strength (S_u) was obtained for a typical normal stress of 122

TABLE 2 COEFFICIENTS OF CONSOLIDATION (C_v) OF PURE AND TREATED KAOLIN CLAY AT EQUAL WATER CONTENTS

% Chemical	L.L. Acid	L.L. Lime	61 Acid	61 Lime	122 Acid	122 Lime	245 Acid	245 Lime
			Preconsolidation Pressures ($\bar{p}_{co}$) (kPa) — Coefficient of consolidation (C_v) cm^2/min					
0	46	48	0.30	0.30	0.31	0.28	0.33	0.29
4	46	48	0.16	0.30	0.30	0.31	0.18	0.33
0	38	56	0.25	0.33	0.34	0.35	0.42	0.40
8	38	56	0.25	0.32	0.24	0.32	0.35	0.32
0	32	50	0.23	0.36	0.45	0.47	0.41	0.47
12	32	50	0.20	0.36	0.26	0.35	0.15	0.30

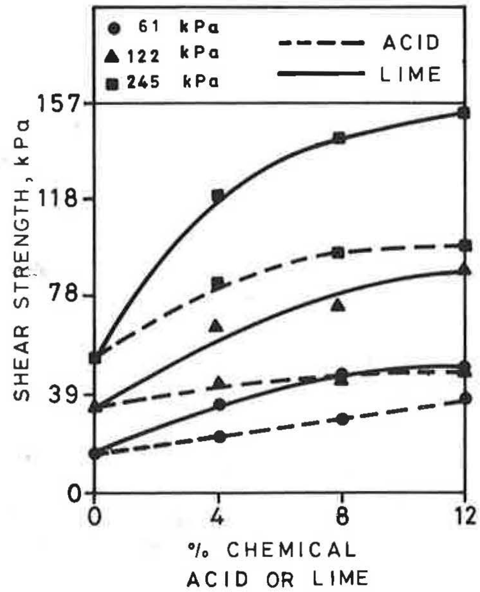

FIGURE 8 Shear strength of phosphoric acid and lime-stabilized kaolin clay.

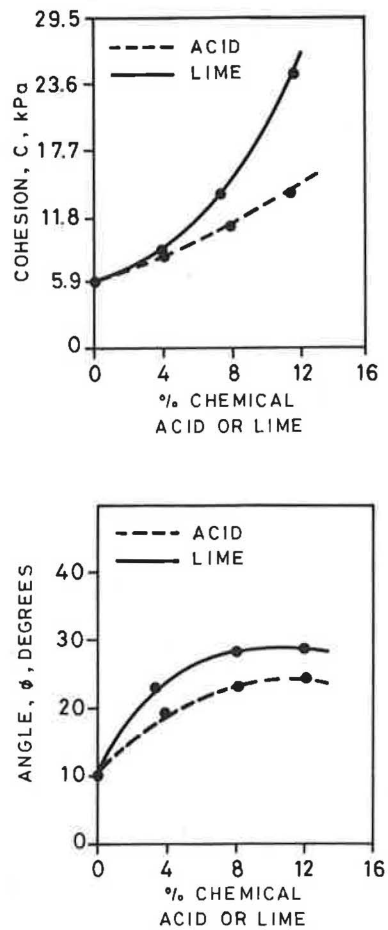

FIGURE 9 Shear strength parameters of acid and lime-stabilized kaolin clay: *top*, cohesion (c); *bottom*, angle (φ).

kPa of each series (SR I and SR II). Two values, 8 and 12 percent, of both stabilizers were used.

The data shown in Figure 10 indicate increase in S_u with increase in curing period and amount of stabilizer. However, the values of S_u were higher for the consolidated chemically treated clays (SR II) than for the unconsolidated chemically treated clays (SR I). The effect of consolidation is indicated by the hatched areas in Figure 10.

Another way of studying the data is shown in Figure 11, where the ratio of undrained shear strengths S_uII/S_uI are plotted against aging days. Figure 11 indicates that the ratio is high for aging periods up to 14 days, and it is higher for the 12 percent than for the 8 percent samples; after that, the ratio decreases, though still greater than 1; it becomes almost equal to 1 for 12 and 8 percent samples, for aging beyond 14 days. This result may lead to the recommendation of starting consolidation of chemically treated soils soon after placement or addition of chemicals. The results shown in Figures 10 and 11 indicate that consolidation improves and accelerates the strength gain of chemically treated kaolin clayey soils.

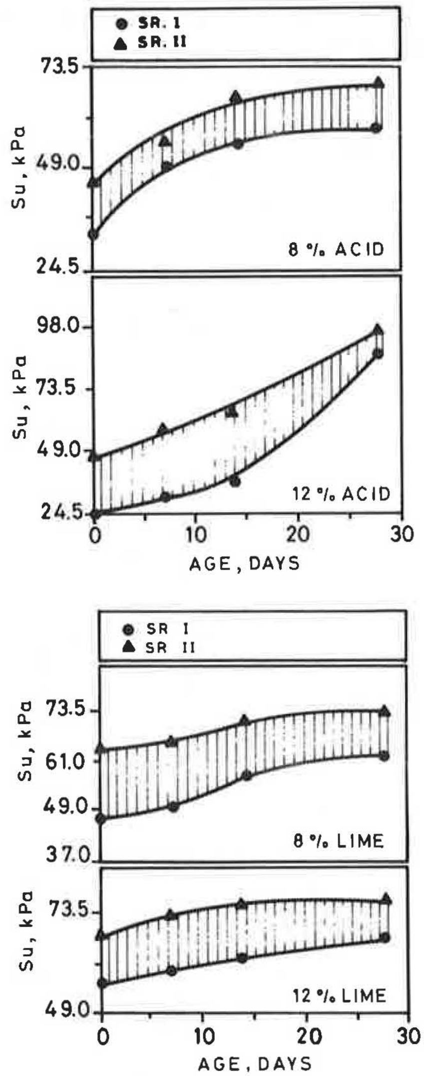

FIGURE 10 Effect of (*top*) phosphoric acid and (*bottom*) lime treatments on shear strength with aging of kaolin clay.

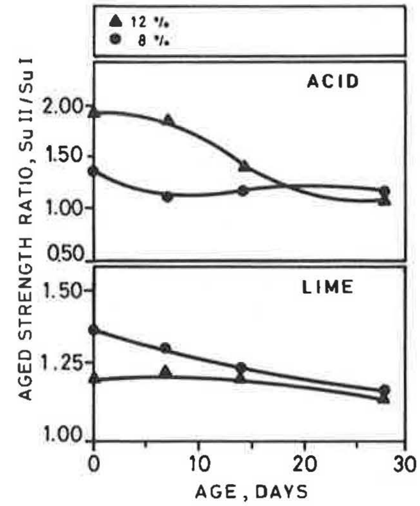

FIGURE 11 Aged strength ratios of phosphoric acid and lime-stabilized kaolin clay.

Hypothetical Example for Settlement Calculations

Problem

It is proposed to construct a 5.0-m-high embankment on top of a 2.0-m-thick pure kaolin clay layer underlain by dense sand. The properties of the pure kaolin and embankment material are shown in Figure 12.

Solutions

An analysis of the settlements of the kaolin clay both in treated (with 8 percent lime or phosphoric acid) and untreated conditions has been carried out using the following formulas (*14,15*):

$$\Delta H = [H_0/(1 + e_0)]C_r \log (P_0 + \Delta P/P_0) \qquad (2)$$

where

ΔH = settlement,
H_0 = layer thickness,
e_0 = initial voids ratio,
P_0 = original pressure, and
ΔP = incremental pressure.

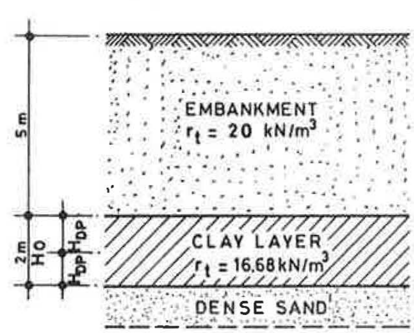

FIGURE 12 Example soil profile showing treated clay layer and embankment fill.

TABLE 3 SUMMARY OF EXAMPLE TEST DATA AND COMPUTED RESULTS

Material	Water content %	γ_t kN/m^3	e_o	C_r	C_v cm^2/min	ΔH cm	t_{90} min
Pure kaolin clay	38	16.68	1.1539	0.0264	0.34	2.015	24942
Pure kaolin clay	56	16.68	1.4670	0.0254	0.35	1.690	24229
Kaolin clay + 8% phosph. acid	38	16.68	1.1278	0.0178	0.24	1.375	35333
Kaolin clay + 8% lime	56	16.68	1.4640	0.0143	0.32	0.950	26500

$$C_v = T(H_{DP})^2/t \quad (3)$$

where

T = time factor,
H_{DP} = length of drainage path, and
t = time.

The data and calculated results are presented in Table 3.

The settlements (ΔH) of lime- and phosphoric acid-treated soils have been reduced by 44 and 32 percent of the untreated kaolin, respectively. This result means that there are considerable improvements in the settlement behavior of chemically treated kaolin clay, in addition to the improvements of shear strength. However, the settlements in case of treated clay take longer periods of time to achieve 90 percent consolidation, mainly because of reduced permeability.

On the basis of these results, chemical treatment is recommended as a possible alternative to reduce settlement, because of the chemically induced preconsolidation effects.

CONCLUSIONS

The study results warrant the following conclusions:

1. The addition of lime or phosphoric acid to pure kaolin clay decreases the Atterberg limits.
2. Low percentage of phosphoric acid stabilizer and high preconsolidation pressure are more efficient in producing preconsolidation effects of kaolin clay.
3. Low percentages of lime stabilizer and low preconsolidation pressure are more efficient in producing preconsolidation effects of kaolin clay.
4. C_r values decrease when treating the kaolin clay either with phosphoric acid or lime. The decreases in C_r values occur most often at moderate preconsolidation pressure.
5. At equal water contents, values for C_v of stabilized kaolin clay, when treated either with lime or phosphoric acid, are less than the C_v values of pure kaolin.
6. The addition of phosphoric acid or lime to pure kaolin clay increases its c and ϕ values at saturated conditions and equal water contents. These increases are greater for lime treatment than for phosphoric acid treatment.
7. The shear strength increases with the application of preconsolidation pressure both for lime- and phosphoric acid-stabilized kaolin clay mixes.
8. Consolidation with aging up to 14 days improves and accelerates the strength gain of chemically treated kaolin clay for either lime or phosphoric acid.
9. Consolidation of chemically treated kaolin clay is recommended to start soon after the addition of chemicals.
10. For the control of settlements of kaolin clayey soils, chemical treatment is recommended as a possible alternative.

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Strength and Stress-Strain Characteristics of a Lime-Treated Cohesive Soil

ERDIL R. TUNCER AND ADNAN A. BASMA

Strength and stress-strain characteristics of a cohesive soil in natural and lime-treated states were investigated and compared. For this purpose, a cohesive soil from a semiarid region in Jordan was selected and subjected to various laboratory tests. The experimental program involved three levels of treatment (3, 6, and 9 percent) with hydrated lime and a range of curing times (0, 7, 14, 21, and 28 days). The experimental results indicated that increasing the percent lime increases grain size, calcium ions, and the pH value, whereas it decreases the plasticity index, sodium ions, and dispersion. The compaction characteristics of the soil studied were not significantly affected by lime. Furthermore, the unconfined compressive strength and the undrained angle of internal friction increased because of the addition of lime and curing time. The undrained cohesion decreased with lime treatment up to 3 percent and increased for lime content >3 percent. The lime treatment strength ratio (LSR), defined as the ratio of the unconfined compressive strength of a treated specimen to an untreated one, was introduced. Greater values of LSR indicate that lime is more effective in stabilizing the soil as far as strength is concerned. For the soil studied, LSR increased both with lime percentage and curing time. In addition, the undrained modulus, E_u, increased significantly for values of LSR between 1.0 and 2.0. For LSR > 2.0, the increment in E_u was much smaller.

The stabilization of cohesive soils by lime has been of great interest for many years. In practice, lime has been used as an effective additive to improve various engineering properties of cohesive soils. Lime treatment in cohesive soils generally causes a decrease in plasticity, dispersion, compressibility, and volume change potential, and an increase in particle size, permeability, and strength (*1–3*). Such changes in engineering properties can be attributed basically to two types of chemical reactions that occur when lime is added to a wet soil. First, there are the reactions that take place immediately after mixing lime with soil—within a few minutes to an hour. These reactions are colloidal in nature and involve cation exchange and agglomeration-flocculation reactions that take place because of varying double-layer characteristics of individual clay particles. In this first stage, immediate changes occur in soil plasticity, workability, volume change potential, strength, and deformation properties. Second, the pozzolanic reaction is time dependent and takes place for a long period of time, several years in some instances. This reaction causes the formation of various types of cementation products that increase the strength and durability of the mixture much more than do the products from the reactions in the first stage (*4–6*).

The basic objective was to thoroughly study the stress-strain behavior in terms of the undrained modulus of elasticity and axial strain at failure as obtained from the unconfined compression test. This approach provides better insight to the general strength behavior and durability of cohesive soils treated with lime. Furthermore, the strength of lime-treated cohesive soils has been investigated by many researchers using the unconfined compressive strength more so than any other strength parameter. Therefore, as a second objective, the triaxial compression test was used to examine the variation in undrained cohesion and undrained internal friction angle caused by lime treatment. For this purpose, a cohesive soil located in a semiarid region in northern Jordan was selected. Also, a parameter for judging the effectiveness of lime treatment as far as strength is concerned was defined.

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LABORATORY INVESTIGATION

The selected cohesive soil was from the Irbid city area located in a semiarid region in northern Jordan. Samples were collected from a depth of 1 m below the ground surface. Previous studies performed on Irbid clay indicated that it is highly plastic, fissured, overconsolidated at shallow depths because of desiccation, and highly expansive in character (*7*).

The experimental program entailed three different levels of lime treatment: 3, 6, and 9 percent by dry weight of the soil. The specimens were cured at 22°C and 70 percent relative humidity for 1 hr (0 days), and 4, 7, 21, and 28 days. Hydrated lime, which is produced and commercially available in Jordan, was used throughout the study.

The soil characteristics studied here can be divided into two groups. The first group involved physical, compositional, and compaction characteristics that were measured with the intention of studying the effect of lime on them and to search for any systematic influence of these properties on the general strength behavior of the soil. (However, in determining these properties, curing was not taken into account except for consistency limits.) The second group comprised strength and stress-strain characteristics for which the effects of lime content and curing time were investigated in detail.

Physical, Compositional, and Compaction Characteristics

The soil both in natural and lime-treated states was subjected to various laboratory tests in accordance with ASTM standard procedures. These tests included grain size distribution, consistency limits, and standard Proctor compaction. A fourth level of treatment with 12 percent lime was considered only for consistency limits. The results are presented in Table 1, which also includes the plasticity index (I_p) values and the classification according to the Unified Soil Classification System. The effect of curing time on consistency limits was also studied in soils treated with lime and no significant variation was recorded.

In order to examine the effect of lime on the concentration of soluble ions existing in the soil, two extreme cases, natural and 9 percent lime-treated specimens of the soil, were analyzed for Ca, Mg, Na, K, Mn, Al, SO_4, NO_3, PO_4, and Cl ions. Pore water specimens extracted at a water-soil ratio of 50:1 were used for this purpose. Cations were analyzed using an atomic absorption spectrophotometric method. Anions of PO_4, SO_4, and NO_3 were analyzed using the same method, whereas Cl was studied using a titration procedure. The results are presented in Table 1.

The pH values of the natural soil and all the soil-lime mixtures were also measured. For this purpose, all specimens were extracted at a water-soil ratio of 1:1. An electronic pH meter with glass electrodes was used. The results are presented in Table 1. Marks and Haliburton (*5*) stated that pH values >10 increase the solubility of quartz and silica, and thus act as a catalyst to accelerate pozzolanic reaction in the soils. As presented in Table 1, the pH values obtained for 3, 6, and 9 percent lime treatment levels were all >10, ensuring the solubility of quartz and silica.

In order to study the effect of lime treatment on soil dispersion, the double hydrometer test was performed both on the natural soil and on all the soil-lime mixtures. In this experiment, the specimen was first subjected to the standard hydrometer test in which a dispersing agent was used; then the specimen was tested without using a dispersing agent. Oven-dried soil passing the No. 200 sieve was used in both cases. The percent dispersion is defined as the percent of particles smaller than 0.005 mm without using a dispersing agent divided by the percent of particles smaller than 0.005 mm using a dispersing agent. The percent dispersion values obtained for the natural soil and soil-lime mixtures are also presented in Table 1.

Strength and Stress-Strain Characteristics

Specimens, one in each set, were subjected to a standard unconfined compression test to obtain the unconfined compressive strength (q_u) and the stress-strain behavior of the natural soil and all the soil-lime mixtures. In addition, the effect of a range of curing times on these parameters was

TABLE 1 PROPERTIES OF NATURAL AND LIME-TREATED SOILS

Property	Percent Lime				
	0	3	6	9	12
Grain-size Distribution					
Sand, % (2 mm - 75 μm)	2.3	6.1	30.5	59.1	-
Silt, % (75 μm - 2 μm)	43.7	77.9	65.5	38.8	-
Clay, % (< 2 μm)	54.0	16.0	4.0	2.1	-
Consistency Limits					
Liquid limit, w_L, %	74.7	57.2	54.5	50.7	50.5
Plastic limit, w_P, %	35.7	46.7	47.2	45.3	46.8
Plasticity Index, I_P, %	39.0	10.5	7.3	5.4	3.7
Unified Soil Classification	CH-MH	MH	MH	SM	-
Compaction					
w_{opt}, %	31.0	29.0	30.0	29.5	-
$\gamma_{d_{max}}$, kN/m³	13.9	14.2	14.1	14.2	-
Ion concentration, mg/liter					
Na	8.16	-	-	6.06	-
K	0.44	-	-	0.46	-
Ca	7.47	-	-	15.49	-
Mg	1.11	-	-	0.03	-
Al	0.20	-	-	0.21	-
Mn	0.06	-	-	0.00	-
Cl	28.20	-	-	28.20	-
NO_3	1.90	-	-	3.20	-
SO_4	17.00	-	-	17.00	-
PO_4	0.68	-	-	0.68	-
pH value	7.9	10.5	11.2	11.5	-
Percent dispersion	66.7	60.4	32.7	32.7	-

"-" indicates that the specimen was not tested for that property studied. All specimens were compacted before testing by using a standard Proctor compaction effort at 30 percent water content, which was within a ± 1 percent range of the optimum water content values (see Table 1). Furthermore, specimens were tested in an unconsolidated-undrained (UU) triaxial test to determine the undrained cohesion c_u and angle of internal friction ϕ_u. In this test, three all-around pressures were used: 150, 300, and 450 kPa. The results of the strength tests are presented in Table 2, which contains the compressive strength (q_u), the undrained modulus of elasticity (E_u) (as obtained from the initial tangent of the stress-strain diagram), the axial strain at failure (ε_f), and the undrained shear strength parameters c_u and ϕ_u for various levels of lime treatment and curing times.

DISCUSSION OF EXPERIMENTAL RESULTS

Physical, Compositional, and Compaction Characteristics

The combined results of sieve and hydrometer analyses for the natural and lime-treated soils indicate that the percent of sand particles increases and the percent of clay size particles decreases as the lime content increases. Figure 1 shows the influence of lime treatment on particle size characteristics of the soils studied. The increase in particle size by the addition of lime can be attributed to the cation exchange phenomenon taking place at the surface of the clay particles by which the particles become electrically attracted to one another, causing flocculation and aggregation.

The results obtained from the double hydrometer test indicate that the percent dispersion decreases with increasing lime content up to 6 percent and remains constant beyond this point. Previous studies have also shown that the addition of lime converts a dispersive soil into nondispersive, erosion-resistant soil (*2,8*). Although the sodium ion is known to be the most effective factor causing dispersion in soils, the calcium ion is commonly accepted to be a flocculating agent (*9*). As indicated in Table 1, addition of lime causes the calcium ion concentration to increase and the sodium ion concentration to decrease. These effects in turn affect the double layer characteristics of clay particles and lead to an increase in attraction, thus causing flocculation and aggregation and consequently a decrease in dispersion.

TABLE 2 STRENGTH TEST RESULTS

Percent Lime	Curing Time, days	Unconfined Compression			Triaxial Compression	
		q_u kPa	E_u MPa	ϵ_f %	c_u kPa	ϕ_u degrees
0	0	290	16.7	3.25	212	2.8
	4	316	17.5	3.00	-	-
	7	359	19.3	3.25	-	-
	14	326	23.7	2.00	-	-
	21	342	27.9	1.50	-	-
	28	483	25.6	2.50	-	-
3	0	306	20.2	2.00	139	18.7
	4	370	29.6	1.50	-	-
	7	321	22.9	1.50	141	25.8
	14	415	27.6	1.75	-	-
	21	387	30.3	1.50	-	-
	28	445	37.9	1.25	146	27.4
6	0	385	28.7	1.25	219	20.5
	4	729	44.1	2.00	-	-
	7	949	47.3	2.25	313	28.8
	14	1267	53.7	2.50	-	-
	21	1383	54.5	2.50	-	-
	28	1540	50.6	3.00	368	37.2
9	0	390	29.1	1.50	243	22.3
	4	959	45.1	2.25	-	-
	7	993	52.7	2.00	356	32.6
	14	1640	57.8	3.00	-	-
	21	1737	56.6	3.00	-	-
	28	2049	56.6	3.50	418	38.3

"-" indicates that the specimen was not tested for that property

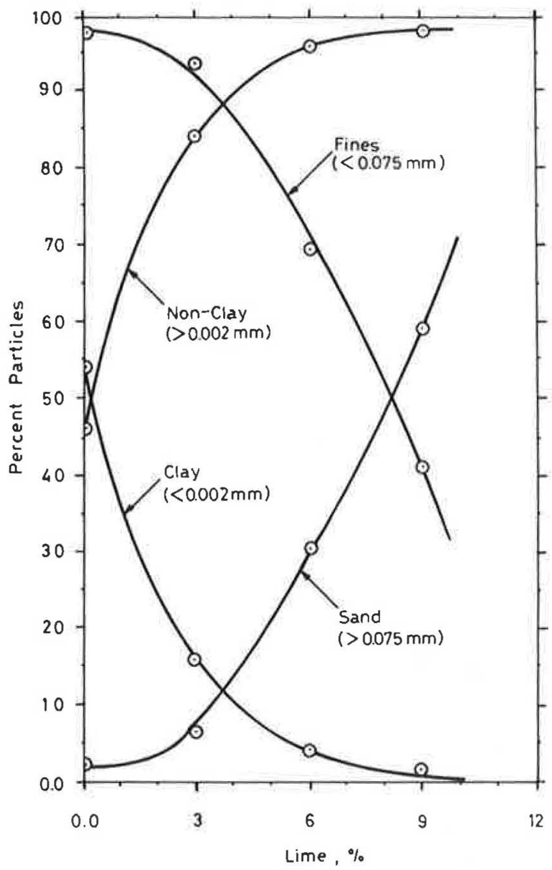

FIGURE 1 Effect of lime treatment on particle size characteristics.

The effect of lime treatment on consistency limits is indicated in Table 1. As lime content increases, causing a reduction in the plasticity index, the liquid limit decreases and the plastic limit increases. On the other hand, the curing time did not have any significant effect on the values of consistency limits. These results can be attributed to the flocculation of the clay particles by lime treatment, producing a soil with coarser particles and causing less plastic character.

The variations in particle size distribution and consistency limits also influence the classification of the soil. As shown in the plasticity chart of Figure 2, the natural soil is almost on the *A*-line in the high plastic range, but by the addition of lime at 3 and 6 percent levels, the soil becomes highly plastic silt, MH type of soil well below the *A*-line. At 9 percent lime treatment level, the soil is a coarse-grained soil with about 60 percent sand-size particles, and can be classified as silty-sand, SM type of soil.

The effect of lime treatment on the compaction characteristics is also presented in Table 1. From these results, it is clear that the maximum dry unit weight, $\gamma_{d_{max}}$, and optimum water content, w_{opt}, are not significantly affected by lime. This observation seems to contradict the general conception that lime reduces $\gamma_{d_{max}}$ and increases w_{opt}. However, El-Rawi and Awad (*1*) indicated that an increase in w_{opt} is attributed to pozzolanic reaction and a decrease is possibly caused by cation exchange. Thus, a balance between these two mechanisms will result in little or no change in w_{opt}, and consequently $\gamma_{d_{max}}$ is unchanged. This balance seems to be the case here.

Strength and Stress-Strain Characteristics

The variation of unconfined compressive strength with the level of lime treatment and curing time is shown in Figure 3. The natural soil exhibited a marginal gain in strength with time, indicating that it is slightly thixotropic in character. This observation is in agreement with the general thixotropic behavior of overconsolidated clays. As Figure 3 shows, the addition of 3 percent lime resulted in a relatively small increase in strength even after 28-day curing, indicating that this level of treatment is not sufficient to produce the required degree of pozzolanic reaction for the formation of adequate cementitious products. On the other hand, 6 and 9 percent lime treatment increased the unconfined compressive strength rather significantly. Thus, the unconfined compressive strength has

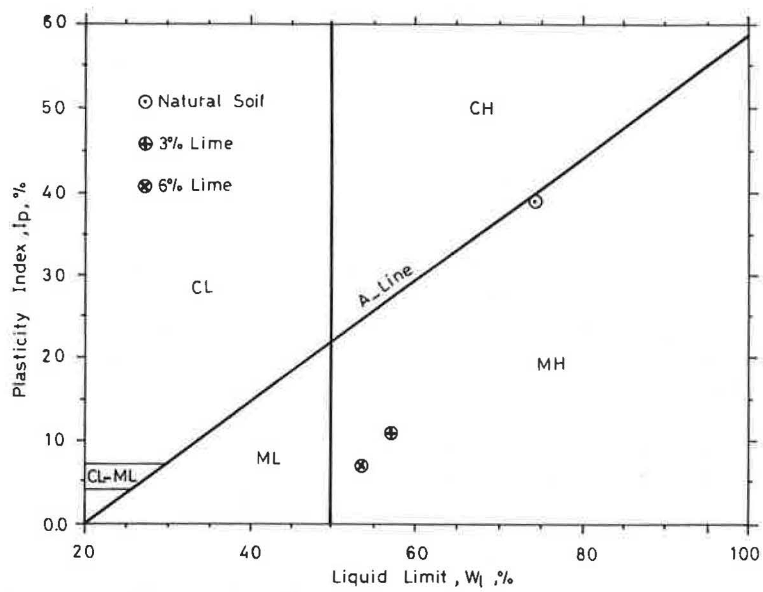

FIGURE 2 Plasticity chart.

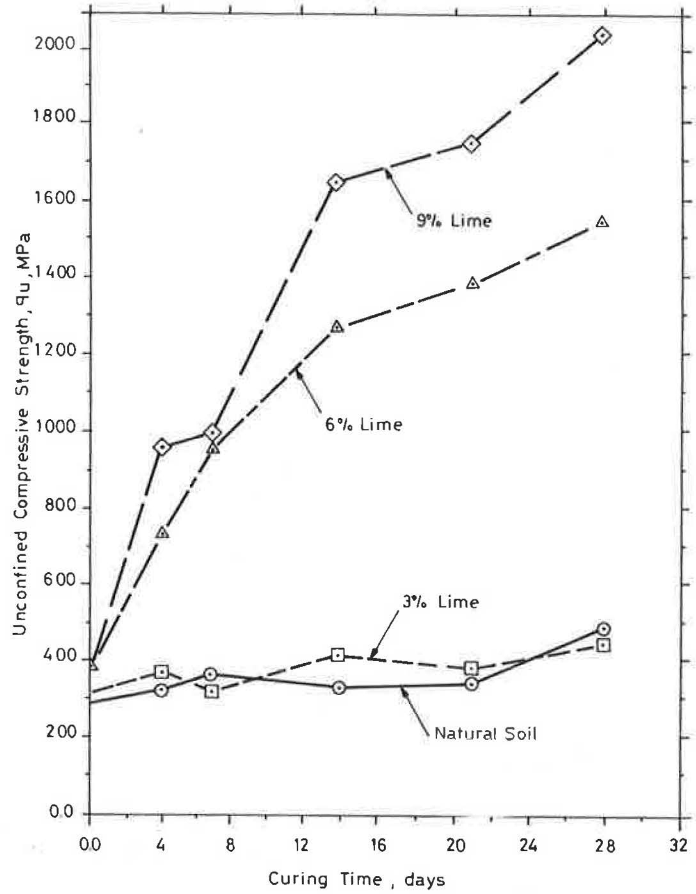

FIGURE 3 Effects of lime content and curing time on unconfined compressive strength.

a tendency to increase as the lime content and curing time increase. Because the increment in strength is small immediately after the addition of lime and larger increments require time, it is clear that the pozzolanic reaction is responsible for strength gain.

The specimens for the triaxial UU-test were prepared by compaction at optimum water content. They were not fully saturated for testing. This latter fact explains the nonzero measured values of ϕ_u for all tested specimens, as indicated in Table 2. The effects of lime content and curing time on c_u and ϕ_u are shown in Figures 4 and 5, respectively. These figures indicate that c_u decreases as ϕ_u increases at 3 percent lime treatment. This observation may be justified by the earlier assertion, in particular, that 3 percent lime merely causes cation exchange to take place with little pozzolanic reaction. Consequently, 3 percent lime content would be adequate enough to result in flocculation and aggregation but insufficient to form cementitious products. The basic changes, therefore, taking place with this amount of lime are in particle size and plasticity of the soil. Clay size content, for instance, drops drastically from 54 percent to 16 percent and plasticity index decreases from 39.0 to 10.5 percent, immediately after the soil is treated with 3 percent lime. These variations in physical properties suggest that the soil is approaching a granular nature, thus, causing a decrease in c_u and an increase in ϕ_u. With 6 and 9 percent lime, on the other hand, the pozzolanic reaction becomes more effective resulting in cementation and consequently in higher cohesion and internal friction angle. Furthermore, Figures 4 and 5 indicate that the curing time influences c_u and ϕ_u; however, its impact is more pronounced at higher levels of lime treatment. As expected, varying physical characteristics affect the internal friction angle more directly. Figures 6 and 7, respectively, show the effects of varying plasticity index and percent nonclay fraction on ϕ_u together with the influence of curing time.

Flocculation and cementation processes resulting from chemical reactions in lime-treated soils are expected to produce a brittle material. Such materials generally have steeper stress-strain diagrams associated with higher modulus of elasticity and are known to fail abruptly at the ultimate compressive load. Comparing various soils of similar strengths, the more brittle the soil is the smaller the axial strain at failure. The stress-strain diagrams presented in Figure 8 indicate that lime treatment leads to a more brittle behavior. This assertion can be justified by the fact that, although the increasing level of lime treatment does not cause any significant increase in strength at 0 days' curing, the axial strain at failure has a tendency to decrease whereas the undrained modulus of elasticity increases as the diagrams become steeper. At 28-day curing, both the natural and 3 percent lime-treated soils do not gain much strength, but they have an even smaller ε_f and greater E_u. The soils treated with 6 and 9 percent lime gain high strengths and they exhibit an abrupt type of failure at the ultimate compressive stress. Because of this high strength,

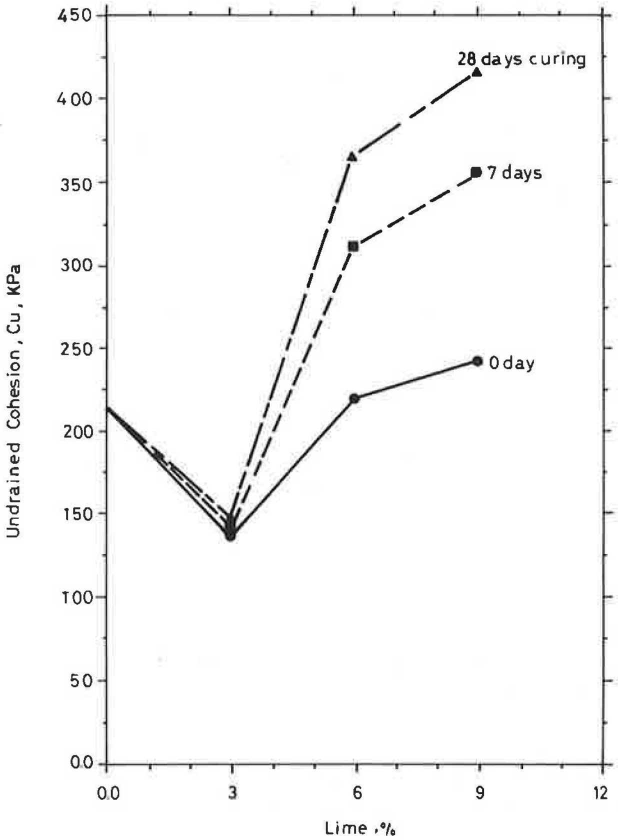

FIGURE 4 Effects of lime content and curing time on undrained cohesion.

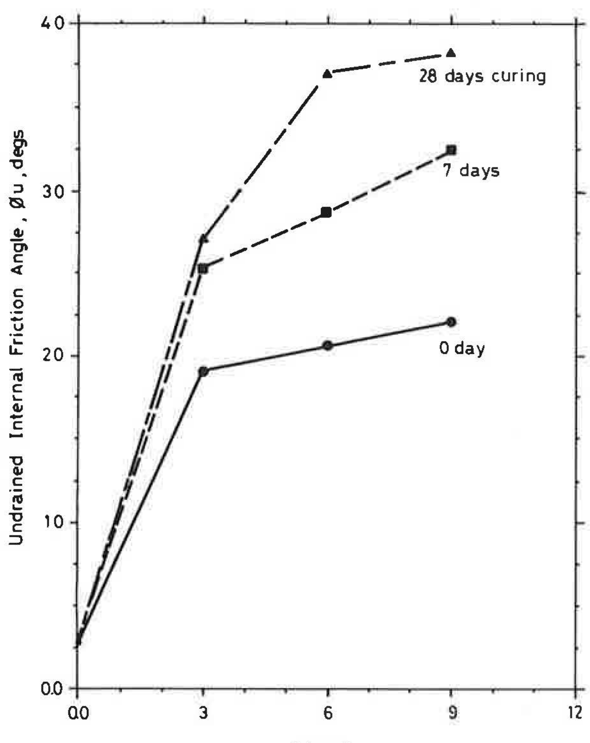

FIGURE 5 Effects of lime content and curing time on undrained angle of internal friction.

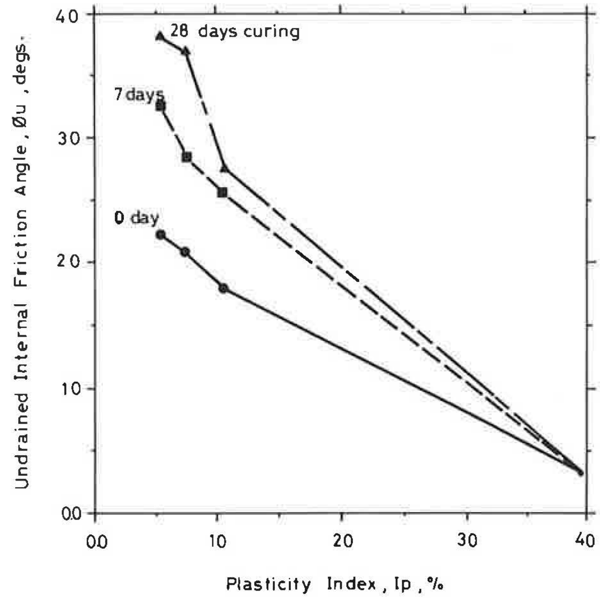

FIGURE 6 Effects of plasticity index and curing time on undrained angle of internal friction.

they resist failure until they undergo larger axial strains. The modulus values, on the other hand, become greater with steeper stress-strain relationships. Figures 9 and 10 show the variations in ε_f and E_u with varying lime content and curing time, respectively, in a more descriptive manner. Figure 11 shows the combined effects of lime content and curing time on the stress-strain characteristics, in particular, ε_f and E_u.

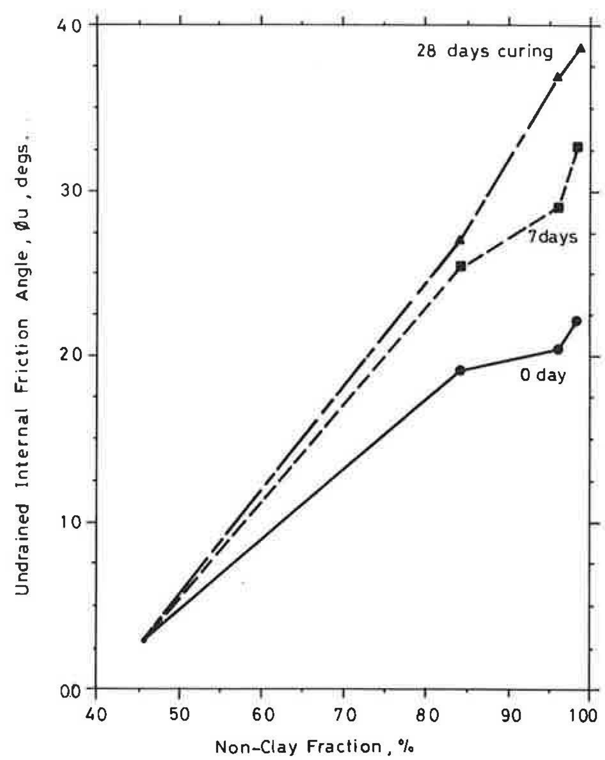

FIGURE 7 Effects of nonclay fraction and curing time on undrained angle of internal friction.

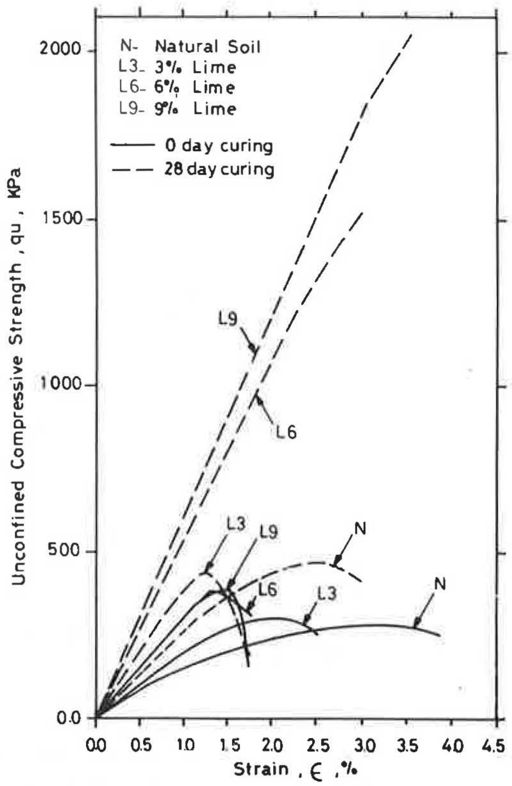

FIGURE 8 Stress-strain diagrams of the natural and lime-treated soils.

A parameter relating the effect of lime on strength characteristics of soils may be useful. With such a parameter, it should be possible to classify different types of soils in their response to lime treatment in terms of strength. Thus, the lime treatment strength ratio (LSR) is defined as the unconfined compressive strength of lime-treated soil compacted at optimum water content divided by the unconfined compressive strength of natural soil compacted at optimum water content. Figure 12 shows the variation of LSR with varying lime content and curing time. For up to 3 percent lime treatment, LSR remains at about the same value for any curing period. However, higher amounts of lime and curing time result in a significant increase in LSR. This suggests that up to the 3 percent level of lime treatment a gain in strength is not produced. Higher percentages of lime and longer periods of curing, however, will ensure a higher value of LSR, leading to a stronger soil caused by cementation, which is a direct result of pozzolanic reaction.

Figure 13 relates E_u with LSR. As this relation indicates, E_u increases in large amounts as LSR increases within the range of 1.0 to 2.0. For values of LSR >2.0, the increment in E_u becomes much less. This observation indicates that the stress-strain diagram steepens more at the initial stages of strength gain. After this stage, large increments in strength would not cause much change in E_u. This argument can be justified by studying Figure 10. In this figure, E_u reaches its maximum value almost within 7-day curing, especially with 6 and 9 percent lime-treated soils, whereas the strength keeps on increasing (Figure 3).

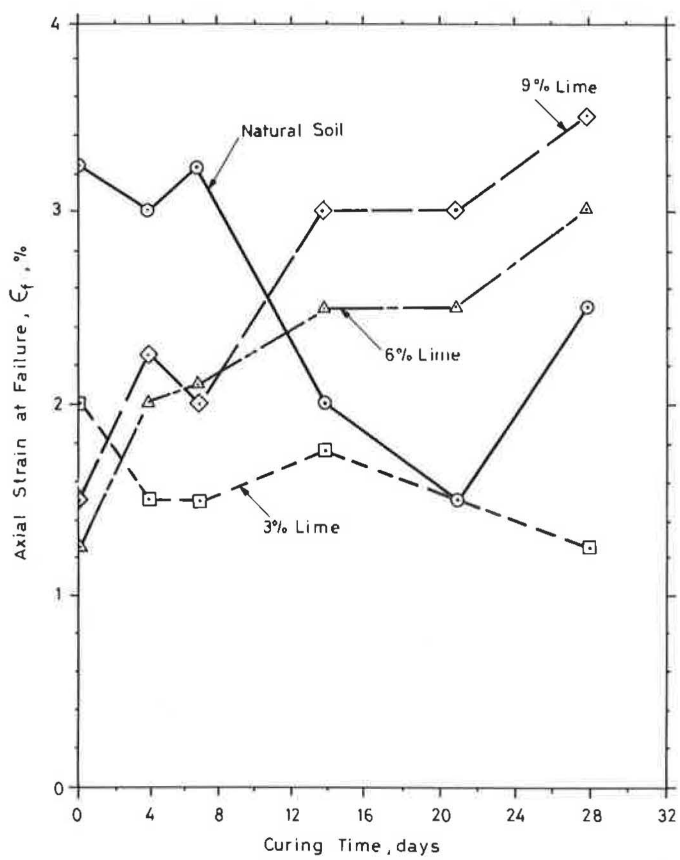

FIGURE 9 Effects of lime content and curing time on axial strain at failure.

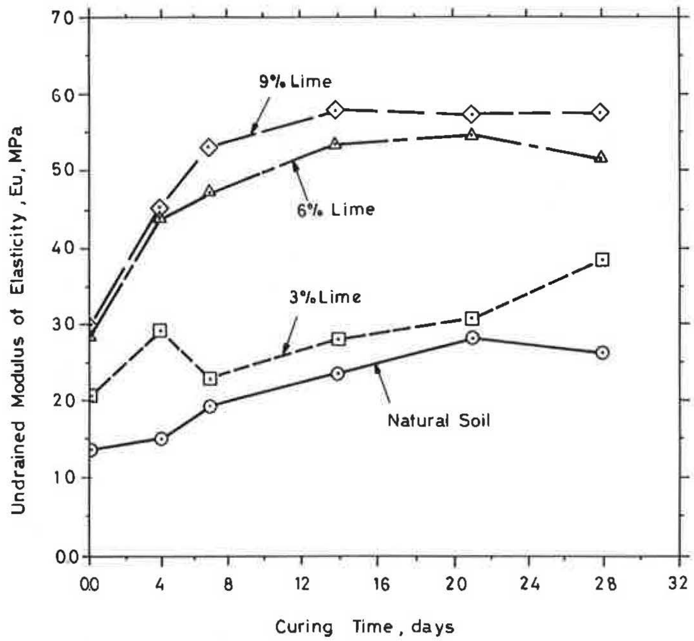

FIGURE 10 Effects of lime content and curing time on the undrained modulus of elasticity.

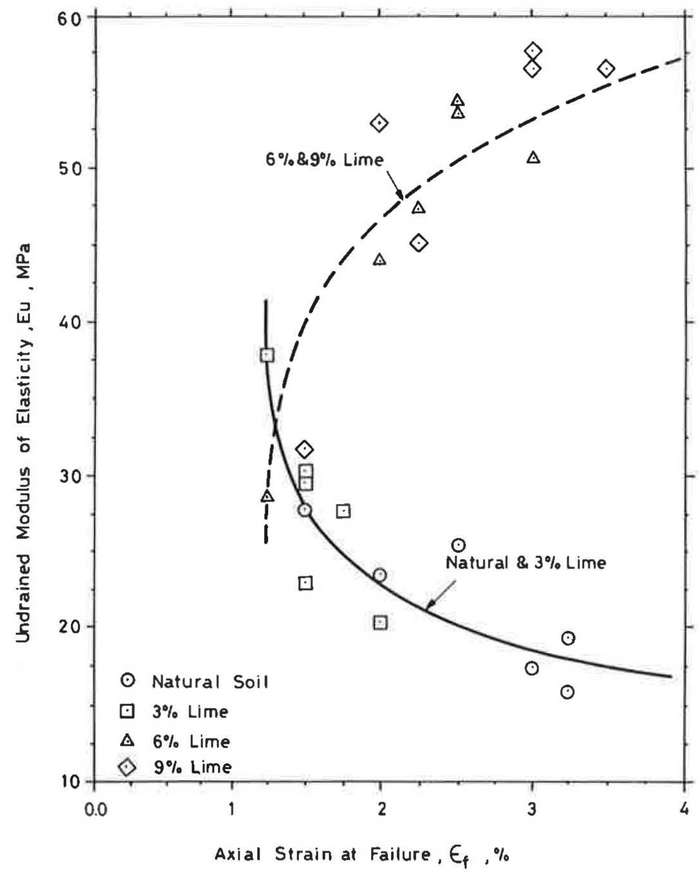

FIGURE 11 Undrained modulus of elasticity versus axial strain at failure for various lime contents.

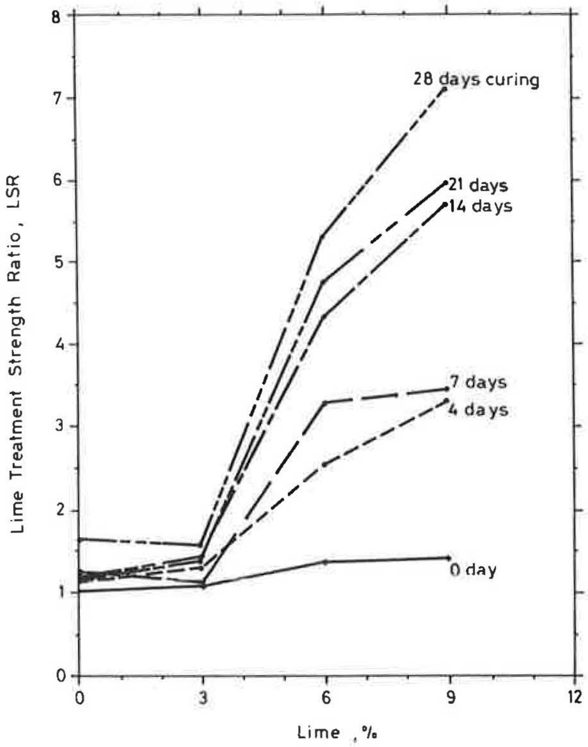

FIGURE 12 Effects of lime content and curing time on LSR.

CONCLUSIONS

On the basis of the findings of this study and the conditions evaluated, the following conclusions can be reached:

- Lime treatment influenced the physical, compositional, and compaction characteristics as follows:
  —Particle size increased;
  —Liquid limit decreased and plastic limit increased, consequently, plasticity index decreased;
  —Percent dispersion decreased;
  —Na ion concentration, which is responsible for dispersion, decreased, whereas flocculating Ca ion concentration increased;
  —The pH increased; and
  —Maximum dry density and optimum water content remained practically at the same level.
- The strength characteristics studied included unconfined compressive strength, q_u, undrained cohesion, c_u, and undrained internal friction angle, ϕ_u. The effects of lime treatment on these parameters are as follows:
  —q_u had a tendency to increase with increasing lime content and curing time, as long as the amount of lime was sufficient to cause the pozzolanic reaction required to form adequate cementitious products.
  —c_u decreased slightly during the initial stages of lime treatment (up to 3 percent lime) and then appreciably increased with increasing lime and curing time. This behavior indicates that to have an increase in c_u, a sufficient amount of lime must be added to the soil to cause the occurrence of pozzolanic reactions.
  —ϕ_u exhibited a significant increase even at low levels of lime treatment and curing time, indicating that the cation exchange phenomenon is effective in addition to pozzolanic reactions. Cation exchange causes variation in the physical properties that bring the soil to a more granular nature.
- Lime treatment made the soil more brittle and the stress-strain behavior of a lime-treated soil was similar to that of a brittle material. This point can be readily justified by examining the variations in strength, undrained modulus of elasticity, and axial strain at failure that resulted from lime treatment.
- A parameter termed "lime treatment strength ratio," LSR, is introduced. LSR is defined as the ratio of the unconfined compressive strength of the lime-treated soil to an untreated soil compacted at optimum condition. LSR was found to in-

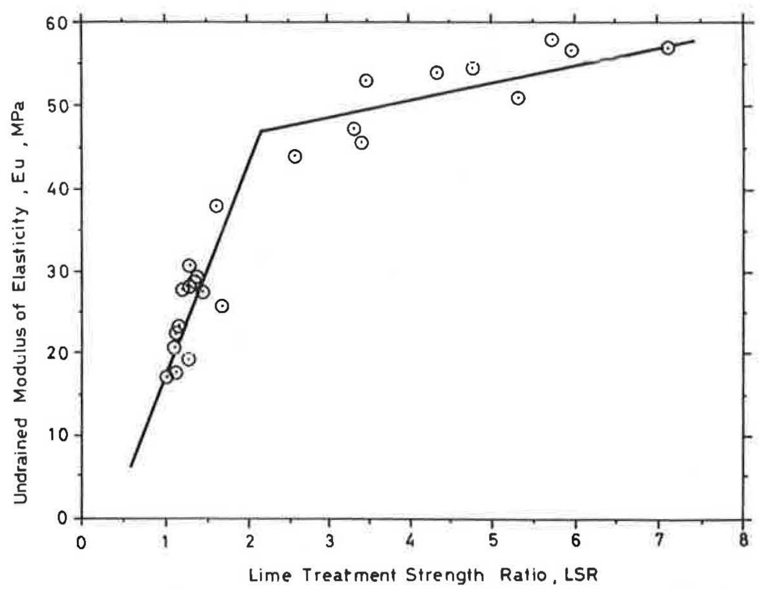

FIGURE 13 Relation between E_u and LSR.

crease both with lime and curing time. In addition, for the soil studied herein, the undrained modulus, E_u, increased significantly as LSR increased within the range of 1.0 to 2.0. However, for values of LSR > 2, the increase in E_u is less.

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California Bearing Ratio Improvement of Remolded Soils by the Addition of Polypropylene Fiber Reinforcement

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The California bearing ratio (CBR) of a micaceous silt, common to the Piedmont in the southeastern United States, was significantly enhanced by the addition of discrete polypropylene fiber reinforcement. Dosages of fiber ranging from 0.09 to 1.5 percent of the soil's dry unit weight were used in soil compacted to 100 percent of its standard Proctor maximum dry density. Fiber configurations consisted of monofilament fiber of 0.38- and 0.76-mm diameter as well as an equivalent fibrillated fiber of 0.38-mm diameter, a lattice-work comprising smaller-diameter webs and stems. Fiberlengths were 19 and 25 mm. The addition of fiber increased the CBR values 65 to 133 percent over unreinforced specimens, depending on fiber configuration and dosage. CBR values using 25-mm-long, 0.76-mm monofilament fiber reinforcement increased significantly up to a dosage of 1 percent, then began to decrease. The test results indicated that there is an optimum fiber dosage as well as an optimum configuration for improving a compacted soil's CBR value.

Limited data and research are available concerning improvement of the engineering properties of soils caused by the addition of random discrete fibers. Far more research has been performed on oriented soil-geosynthetic systems, including fabrics, geogrids, and fibers oriented perpendicular to a direct-shear failure plane. Of the research performed using random fibers, granular soils were typically used. Fibers used included fiberglass, polypropylene, steel, and cellulose (wood byproducts, reeds, etc.).

Compacted granular materials generally have excellent strength, incompressibility, and bearing ratios, and are not typically thought of as needing improvement. Thus, one of the primary objectives of this research was to identify if the addition of discrete, commercially available fibers could enhance the California bearing ratio (CBR) of soils with a significant cohesion strength component. Cohesive soils typically exhibit CBR values inferior to those of granular soils. The fibers themselves should be readily available, durable, and capable of being easily integrated into fill placement and compaction. Ease of placement implies that the fiber should be resistant to curling, bulking, clumping, etc. Furthermore, the testing associated with this approach should be routinely performed by the practicing geotechnical engineering community because the applicability and design values obtained from this technique must be verified in local practice.

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LITERATURE REVIEW

Virtually no published research is available concerning the effect on California bearing ratio from the addition of discrete fibers to compacted soil. Several papers have been published that discuss the effects of fiber reinforcement on compacted soil-cement. Craig et al. (*1*) performed testing on fiber-reinforced soil-cement test specimens. Fibers tested included straight steel, hooked steel, polypropylene, and fiberglass. Two fiber dosages were used (either 0.75 and 1.5 percent, or 1.0 and 2.0 percent), presumably added on the basis of percent dry weight. The soils tested consisted of a clean sand and a clayey sand. The tests performed included compressive strength, split tensile strength, direct shear strength, freeze-thaw, and wet-dry tests. Test results were variable, indicating that various fibers either enhanced or detracted from properties compared with unreinforced specimens, on the basis of fiber type, material tested, and the test performed.

Satyanarayana et al. (*2*) performed split tensile and compression tests of fiber-reinforced, soil-cement specimens where the tested soil consisted of a clay with a plasticity index (PI) of 33. Fibers tested consisted of asbestos and fiberglass, with dosages ranging from 1 to 3 percent by weight. Cement content values were 6, 8, and 10 percent. Both the tensile and compressive tests indicated a significant enhancement of strength at all cement content values and with all fiber dosages.

LeFlaive (*3*) and LeFlaive and Liausu (*4*) presented a patented process by which continuous strands of monofilament fiber were integrated into the subgrade. Triaxial strength testing of granular specimens reinforced in this manner indicate enhanced strength and modulus. Polypropylene fibers were typically used at dosages of 0.14 and 0.2 percent.

Gray and Ohashi (*5*) performed direct shear tests of beach sand reinforced with a variety of materials, including reeds, PVC plastic, or copper wire. The reinforcing was placed at varying angles to the shear plane both in dense and loose sand. Gray and Al-Refeai (*6*) performed triaxial tests using beach sand with reeds or fiberglass filament reinforcement oriented randomly throughout the specimens. This testing indicated that the shear strength typically increased with the addition of more fiber, and increased with an increase in fiber length. Their research also indicated that there was a critical confining stress above which failure envelopes for the fiber-reinforced material paralleled the failure envelope for the unreinforced material. Below a critical confining stress, the failure envelopes for the fiber-reinforced soils were steeper than the failure envelope for the unreinforced material, indicating a higher apparent angle of internal friction. The addition of fiber tended to increase the compression modulus over unreinforced sand.

Gray and Al-Refeai (*6*) also studied the effects of fiber dosage. For a given length-to-diameter ratio (aspect ratio), there seemed to be an asymptotic relationship between dosage and shear strength increase. Gray and Al-Refeai (*6*) also suggested that the critical confining stress could vary significantly with fiber smoothness, i.e., smoother fibers could exhibit a higher critical confining stress. Data also indicated that progressively higher aspect ratios decrease the critical confining stress.

Freitag (*7*) performed unconfined compressive strength testing on both reinforced and unreinforced sandy clay with a plasticity index of 22. Several polymer-based fibers were used, and the reinforced soil exhibited higher unconfined compressive strength values than the unreinforced soils. The percent strength gain was most apparent in specimens remolded at moisture contents wetter than optimum. Modulus values of all reinforced specimens were comparable to slightly inferior to the unreinforced specimens.

Noorany and Uzdavines (*8*) and Maher and Woods (*9*) have performed dynamic testing on randomly oriented fibers within sand. In both instances, there was a significant increase in the reinforced sand's shear modulus. Polypropylene fiber of various configurations was used. Dosages were 0.38 percent by weight (*8*) and from 1 to 5 percent by weight (*9*). Maher and Woods (*9*) also indicate that shear modulus is a function of aspect ratio, i.e., a higher aspect ratio yields a higher reinforced shear modulus. Furthermore, this research indicates an asymptotic relationship between fiber content and improvement in soil properties.

Setty and Chandrashekar (*10*) performed laboratory plate load tests on a clayey sand (PI = 10) reinforced with polypropylene fiber at dosages of 1, 2, and 3 percent, by weight. The 1 and 2 percent dosages showed an increase in ultimate bearing capacity over the unreinforced specimens, with the 2 percent dosage showing the most improvement. The 3 percent dosage had a decrease in ultimate bearing capacity compared to the unreinforced specimens. For a given load, the 1 and 2 percent reinforced specimens demonstrated less settlement than the unreinforced specimens. The 3 percent reinforced specimens had greater settlement than the unreinforced specimen.

Shewbridge and Sitar (*11*) describe a model for quantifying the effects of fiber reinforcement on the basis of shear zone width, fiber length, stiffness, and concentration. Their work was performed using large direct shear apparatus and a layered reinforcement-sand system. Reinforcement consisted of parachute cords, bungee cords, wood, aluminum, and steel rods.

A recent article published in the *Texas Contractor* (*12*) indicated the commercial feasibility of blending polypropylene fiber into soil for subgrade stabilization. Fibrillated polypropylene 25 mm long was blended into the soil at a rate of 109 g/m^2. A 7 percent cement stabilizer was also added. Fibers were spread with a specially modified former manure spreader and blended into the soil with a Bomag MPH100R. The fiber added an immediate load-carrying capacity to the processed subgrade, allowing quicker access by heavy construction equipment.

INITIAL TESTING

Purpose and Scope

The initial testing phase was conducted in 1985 and designed more as a qualitative "what will happen" to the CBR value of a cohesive material if discrete polypropylene fibers were blended. The soil selected was a residual silt derived from the in-place weathering of rock. The fiber selected was 0.76-mm monofilament polypropylene cut to 25-mm length (aspect ratio = 33). The fiber dosages were ½, 1, and 1½ percent, by weight, of the dry soil sample. These dosages were selected on the basis of perceived economics, as the greater cost of fiber at higher dosages was assumed to negate an increase in benefit. Polypropylene was chosen because of its availability, resistance to ultraviolet degradation, chemical stability, and reasonably high strength characteristics. The 25-mm length was deemed compatible with the sample size (152.5-mm diameter) and piston diameter (49.5 mm) and exhibited excellent resistance to bulking and curling. Bulking and curling were perceived as the primary impediments to easily blending the fiber during commercial placement. Table 1 presents the pertinent material properties and configuration of the initial test fiber.

The soil selected was a residual reddish brown fine sandy silt derived from the in-place weathering of metamorphic bedrock. The sample location was Simpson, South Carolina. Overstreet and Bell (*13*) found that the sample location is within the Southern Piedmont physiographic province. Likely parent material of the soil is a Precambrian granitoid gneiss within the Charlotte Group of rocks. The reason this soil was selected is that the Piedmont-derived silts typically exhibit poorer CBR characteristics than Coastal Plain material found within the same general area of practice. Table 2 presents the index properties of this material.

Test Procedures

All testing was performed in general accordance with the then current edition of the American Society for Testing and Ma-

TABLE 1 INITIAL STUDY FIBER PROPERTIES

Fiber	Diameter, mm	Length, mm	Specific Gravity	Tensile Strength, kPa x 10^5	Tensile Modulus, kPa x 10^5
Monofilament Polypropylene	0.76	25	0.91	9.9	7.5

TABLE 2 TEST SOIL INDEX PROPERTIES

Property	Test Results
Specific Gravity	2.79
Gravel, % (>4.75 mm)	0
Sand, % (>0.075 mm; <4.75 mm)	13
Silt, % (>0.005 mm; <0.075 mm)	30
Clay, % (<.005 mm)	57
Liquid Limit, %	52
Plasticity Index, %	17
Natural Moisture Content, %	24
Unified Soil Classification	MH

terials (ASTM), Volume 4.08 (*14*). Three standard Proctor compaction tests (ASTM D698) were performed on each of the soil-fiber mixtures as well as a control (nonfiber) specimen. All tests were performed by technicians working in the geotechnical laboratory of a geotechnical consulting firm, with the testing integrated into the everyday routine of the firm. The Proctor samples were first oven-dried, and each Proctor soil specimen was weighed to the nearest gram. The dosage of fiber was calculated, and the fiber was weighed on an electronic balance to ±0.01 g. The fiber was then added to the sample and blended by hand until a uniform mix was visually obtained. Water, measured to the nearest milliliter, was then added and the mixture again hand blended to achieve a uniform consistency. The samples were then allowed to cure for a period of at least 24 hr before molding. This blending procedure was used in all subsequent phases of testing.

The maximum dry density and optimum moisture content for each soil and soil-fiber group were taken as arithmetic averages of the three tests, and this information was used to mold the CBR specimens. The CBR tests were performed in general accordance with ASTM D1883 (*14*). The CBR specimens were molded to a density equal to approximately 100 percent of the soil's or soil-fiber's standard Proctor maximum dry density, approximately at its optimum moisture content. The samples were molded in six lifts using a manual tamp 50 mm in diameter with machined graduations to obtain approximately equal lift densities. Three specimens per dosage (including control specimens) were molded in this manner.

The CBR specimens were then placed in a water bath in a controlled temperature environment, and allowed to soak for a period of 96 hr. A surcharge stress of about 3.64 kPa was applied using steel weights. Volume change measurements were taken with a dial gage accurate to the nearest 0.03 mm (0.001 in.).

After the 96-hr soaking period, CBR tests were then performed. Deflection readings were taken with a dial gage accurate to 0.03 mm (0.001 in.) and load was obtained from an 8.9-kN proving ring. CBR was calculated according to ASTM D1883 (*14*), and the arithmetic average of the three tests was calculated per dosage.

Initial Test Results

Table 3 presents the average Proctor test results. As can be identified from this table, the addition of increasing volume of fiber generally caused a modest increase in maximum dry density as well as a slight decrease in optimum moisture content. Note that the moisture content was calculated as the weight of water divided by the weight of solids, including soil and fiber. This approach was deemed the most practical, as it was difficult to separate and remove the individual fibers from the soil. Although the maximum dry density at 1 percent fiber was the same as for the ½ percent fiber dosage, the maximum dry density generally increased with a higher fiber content. The no net change in maximum dry density from ½ to 1 percent dosage would tend to substantiate the general premise by Hoare (*15*) that the inclusion of fibers increased the resistance to densification. However, a dosage of fibers at 1½ percent of the soil's dry weight increased the maximum dry density of the soil-fiber mix.

The CBR test results are presented on Table 4. No corrections to CBR values were required because all plots of penetration versus stress were initially linear. The calculated CBR values at 5.08 mm were, in all instances, greater than those at 2.54 mm. Thus, the higher CBR values at 5.08 mm are presented in Table 4. An increase in fiber content actually tended to decrease the CBR value. Recall that the maximum dry density for the soil with 1½ percent fiber by weight was greater than the maximum dry density for both the ½ percent and 1 percent fiber dosage. An increase in density logically should yield a higher CBR value. However, more swell occurred in the 1 and 1½ percent dosages than in the unreinforced specimens. The swell in the ½ percent dosage was

TABLE 3 INITIAL STUDY PROCTOR TEST SUMMARY

Material	Average Maximum Dry Density, kg/m³	Average Optimum Moisture Content, %
Soil	1505.8	28.0
Soil Plus 0.5 % Fiber[1]	1531.5	26.7
Soil Plus 1.0 % Fiber[1]	1531.5	26.1
Soil Plus 1.5 % Fiber[1]	1541.1	25.5

[1]0.76 mm monofilament polypropylene, 25 mm long, weight of fiber based on dry weight of soil, i.e., fiber weight = (percent/100)(dry soil weight)

TABLE 4 INITIAL STUDY CBR TEST RESULTS

Material	Average Swell, %	CBR at 5.08 mm Penetration
Soil	0.14	5.4
Soil Plus 0.5 % Fiber[1]	0.13	11.7
Soil Plus 1.0 % Fiber[1]	0.28	12.6
Soil Plus 1.5 % Fiber[1]	0.17	11.7

[1]0.76 mm monofilament polypropylene, 25 mm long, weight of fiber based on dry weight of soil

comparable to slightly less than the swell obtained for the unreinforced specimens. The greater number of coarse fibers may have created more avenues for water to infiltrate the specimens, contributing to a higher swell. Greater swell could also have occurred because of elastic expansion of the randomly oriented fibers.

The data show a decrease in CBR values for the 1½ percent dosage, indicating there is an optimal fiber dosage beyond which CBR values decrease. It is possible that the larger volume of fibers in the 1½ percent dosage caused many of the fibers to be in contact with one another. The slick finish of the fibers would tend to decrease the punching shear resistance if there were considerable fiber-to-fiber contact.

The results of this initial testing were deemed favorable. These results formed the basis of subsequent laboratory testing performed in 1988.

SUPPLEMENTARY TESTING

Purpose and Scope

Crude calculations of likely in-place costs, even with only a ½ percent inclusion of 0.76-mm monofilament polypropylene fiber, indicated that the process may not warrant widespread use simply because the cost of the fiber was significant compared with the likely benefit obtained in a thinner pavement section. In order to reduce the cost, an equivalent number of 0.38-mm monofilament polypropylene fiber was substituted to determine if the number of fibers was a principal governing criterion rather than its diameter. Also, an equivalent 0.38-mm-diameter fibrillated polypropylene fiber, composed of a lattice-work array of webs and stems that could stretch laterally, was selected to identify if style of fiber could possibly influence the CBR value. The number of fibers for this new phase of testing was based on the previous ½ percent fiber dosage.

In a further attempt to minimize the weight of fiber and subsequent in-place costs, the fiber length was reduced from 25 to 19 mm. For example, the number of 0.76-mm fibers per cubic meter is approximately 744,150, based on a ½ percent by weight fiber dosage. The weight of 25-mm-long fibers in each cubic meter would then be 7.68 kg. If the same number of fibers were used, but the diameter reduced to 0.38 mm and the length reduced to 19 mm, the resulting weight of fiber per cubic meter would be reduced to 1.46 kg. Thus, the calculated dosage of the 0.38-mm fiber that would yield the same number of fibers as the ½ percent dosage of 0.76-mm fibers is 0.09 percent, by weight. The length reduction increased the aspect ratio of the new fibers to 50. Table 5 presents a summary of the fiber properties used for the supplemental testing.

The fibrillated fiber comprises webs and stems, and resembles a lattice-work when stretched. The fiber is also a flat, rectangular tape shape rather than the cylindrical shape of the monofilament fiber. The individual fibers that make up this lattice-work are of much smaller equivalent diameter (0.11-mm stems and 0.08-mm webs) than the composite diameter of 0.38 mm. The lattice-work would likely break apart to various degrees during blending, thus disseminating a larger number of smaller-diameter fibers implied by the previous calculations.

TABLE 5 SUPPLEMENTAL STUDY FIBER PROPERTIES

Fiber	Diameter, mm	Length, mm	Specific Gravity	Tensile Strength, kPa x 10^5	Tensile Modulus, kPa x 10^5
Monofilament Polypropylene	0.38	19	0.91	5.7	7.5
Fibrillated Polypropylene	0.38[1]	19	0.91	6.2	7.1

[1]Composite diameter comprised of 0.11 mm stems and 0.08 mm webs

Supplementary Test Procedures

The test procedures used for this phase of testing were the same as those procedures used for the initial phase of testing. The necessary Proctor tests both for the 0.38-mm monofilament and 0.38-mm fibrillated fibers were performed to identify the maximum dry density and optimum moisture content to which test specimens would be molded.

Supplementary Test Results

Proctor test results are presented in Table 6. The control specimen and ½ percent, 0.76-mm dosage test results are included for comparison. Addition both of the 0.38-mm monofilament and 0.38-mm fibrillated fiber caused an increase in the average maximum dry density beyond that of the control (nonfiber) and ½ percent 0.76-mm monofilament specimens. The optimum moisture contents for the new fiber types were less than the control and ½ percent 0.76-mm monofilament specimens. The 0.38-mm fibrillated fiber specimens exhibited the highest maximum dry density and lowest (or comparably lowest) optimum moisture content of all specimens tested, including the previously tested 1½ percent, 0.76-mm fiber-reinforced specimens.

CBR test results are presented in Table 7. Again, no corrections to CBR values were required because of the linearity of the initial portions of the plots of penetration versus stress. The calculated CBR values at 5.08 mm were again greater than those at 2.54 mm. In all instances, addition of fibers significantly increased the CBR value, compared with those of unreinforced specimens. The sample with the 0.38-mm fibrillated fiber came closest to duplicating results achieved with the larger-diameter, longer, 0.76-mm monofilament fiber. However, neither of the smaller-dosage, smaller-diameter fiber-reinforced specimens matched the CBR values of the previously tested ½ percent, 0.76-mm monofilament, fiber-reinforced specimens.

The smaller-dosage, smaller-diameter, fiber-reinforced specimens swelled approximately twice the magnitude of the control and larger-dosage, larger-diameter specimens. An increase in dry density of the smaller-diameter reinforced specimens is probably responsible for the majority of the swell increase.

CONCLUSIONS

The addition of polypropylene fibers significantly improved the CBR value of the soils tested. The improvement ranged from a 65 percent increase for the 0.09 percent, 19-mm-long, 0.38-mm-diameter monofilament fiber dosage to a 133 percent increase for the 1.0 percent, 25-mm-long, 0.76-mm-diameter, monofilament fiber dosage.

The initial research suggests that there is an optimal fiber dosage for improvement of the CBR value. Table 4 indicates that the dosage that yields the greatest improvement in CBR value is approximately 1 percent. Dosages greater than the optimal dosage decrease the CBR value. With increasing fiber content, there was a decrease in confining soil between the fibers, possibly to the extent that sliding occurred at fiber-to-fiber contact points.

Soil reinforcement with the same number of shorter, 19-mm-long, 0.38-mm-diameter monofilament fiber reinforcement did not produce the same CBR as specimens reinforced with the 25-mm-long, 0.76-mm-diameter monofilament fiber. The longer length and greater cross sectional area of the 0.76-mm-diameter fibers compared to the 0.38-mm monofilament fibers appear to be more important criteria than the number of fibers in enhancing the CBR. Gray and Ohashi (*5*) found that a decrease in fiber length should decrease shear resistance. Conversely, Gray and Al-Refeai (*6*) describe data that indicate there should be a shear strength increase with increasing aspect ratio. The aspect ratio for the 0.38-mm-diameter monofilament fiber was 50 compared with 33 for the 0.76-mm-diameter monofilament fiber. This difference suggests that the concept of increasing the fiber's aspect ratio to achieve a higher strength or CBR is probably only valid for one fiber type where only the length is varied, not comparing several fibers of similar configuration whose lengths and diameters vary.

Specimens reinforced with the same number of 19-mm-long, 0.38-mm equivalent diameter fibrillated fibers yielded a 16 percent higher CBR than the 0.38-mm-diameter mon-

TABLE 6 SUPPLEMENTAL STUDY PROCTOR TEST SUMMARY

Material	Average Maximum Dry Density, kg/m³	Average Optimum Moisture Content, %
Soil	1505.8[1]	28.0[1]
Soil Plus 0.5 % 0.76 mm Monofilament Fiber, 25 mm Long	1531.5[1]	26.7[1]
Soil Plus 0.09 % 0.38 mm Monofilament Fiber, 19 mm Long	1537.9	26.0
Soil Plus 0.09 % 0.38 mm Fibrillated Fiber, 19 mm Long	1558.7	25.6

[1]Test results from initial study

ofilament fiber-reinforced specimens, and came closest to duplicating the CBR values for the 25-mm-long, 0.76-mm-diameter monofilament fiber-reinforced specimens (see Table 7). These results suggest that fiber configuration or shape can significantly affect CBR values. The fibrillated fiber is a relatively flat and rectangular tape shape compared with the cylindrical configuration of the monofilament fiber. A small percentage of the fibrillated fiber did break apart into smaller segments of webs and stems, and thus the total number of discrete fibers disseminated throughout the soil mass was more than that calculated. Thus, the slightly greater number of fibrillated fibers could have contributed slightly to the greater CBR value of the fibrillated fiber-reinforced specimens.

Addition of fiber generally increased the standard Proctor maximum dry density. The increase in Proctor maximum dry density does not substantiate the premise by Hoare (*15*) that the inclusion of fibers increases the resistance to densification.

Table 6 indicates that 0.38-mm, fibrillated, fiber-reinforced specimens exhibited the highest maximum dry density and lowest optimum moisture content of all the specimens tested. A logical expectation would be that these specimens should exhibit the highest CBR value of all specimens tested. However, the highest CBR value was obtained with a specimen reinforced with a 1 percent dosage of 0.76-mm-diameter monofilament fiber 25 mm long that exhibited a dry density approximately 2 percent less and an optimum moisture content approximately 2 percent more than the 0.38-mm, fibrillated, fiber-reinforced specimen. Thus, assessment of traditional nonreinforced soil mechanics indices of maximum dry density and optimum moisture content to postulate the results of CBR tests on fiber-reinforced specimens is probably not valid.

Likewise, use of swell measurements as indicators to predict CBR results of fiber-reinforced soils does not appear to be valid. Table 4 indicates that the swell for the 1 percent dosage of 0.76-mm-diameter fiber is approximately double the swell of both the ½ and 1½ percent dosages. However, the CBR value for the 1 percent dosage is about 8 percent greater.

DISCUSSION OF RESULTS

The addition of polypropylene fiber significantly improved the CBR values of the soils tested. These results, coupled with the dynamic testing results obtained by Noorany and Uzdavines (*8*) and Maher and Woods (*9*), suggest a potentially significant approach to improving soil subgrade support characteristics.

The 1986 *AASHTO Guide for Design of Pavement Structures* (*16*) recommends that the soil's resilient modulus be used in the design of flexible pavements and that the soil's modulus of subgrade reaction be used for the design of rigid pavements. From the literature review, it is apparent that the addition of fiber significantly improves the dynamic shear modulus of the materials tested, and that the bearing capacity and incompressibility of a fiber-reinforced soil can be superior to an unreinforced soil. It is, therefore, likely that the addition of fiber could improve the resilient modulus as well as modulus of subgrade reaction used for these designs. Detailed research should be performed to quantify the degree of improvement, taking into account fiber finish, length, shape, and dosage. The research should focus on commercially available fibers that can easily be blended into the soil.

The subgrade stabilization project in Texas (*12*) demonstrated that discrete fiber can be easily mixed and compacted into subgrade soils by equipment commonly used in subgrade stabilization. The fibrillated fiber used in this application is also commercially available, and is identical to the fibrillated fiber used in the previously discussed research. The testing associated with this research could have been performed by numerous public agencies and commercial firms, not just university environment research facilities. Although continuing research must still be performed to properly quantify the mechanisms of CBR enhancement, there is sufficient evidence in the literature and this current research to indicate that the addition of fiber to improve soil subgrade support is a practical, quantifiable, and biddable process.

The research indicates that there could possibly be "designer fibers" that could have application for different types of soil. Fiber is manufactured in many shapes and finishes, and it is possible that these different manufactured products could provide an optimal CBR increase for different soils. For example, the tape shape may be more beneficial for CBR enhancement in cohesive soils, and the monofilament shape may be more beneficial in granular soils. Further research should be performed to study the effects of fiber finish, length,

TABLE 7 SUPPLEMENTAL STUDY CBR TEST RESULTS

Material	Average Swell, %	CBR at 5.08 mm Penetration
Soil	0.14[1]	5.4[1]
Soil Plus 0.5 % 0.76 mm Monofilament Fiber, 25 mm Long	0.13[1]	11.7[1]
Soil Plus 0.09 % 0.38 mm Monofilament Fiber, 19 mm Long	0.26	8.9
Soil Plus 0.09 % 0.38 mm Fibrillated Fiber, 19 mm Long	0.30	10.3

[1]Test results from initial study

shape, and dosage on CBR values. Again, the research should focus on fibers that are commercially available and can easily be blended into the soil.

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Full-Depth Reclamation with Calcium Chloride

JAMES M. SHEPARD, JAMES PICKETT, AND MICHAEL KIENZLE

Low-volume secondary roads requiring rehabilitation can be restored using the full-depth reclamation process with calcium chloride to achieve increased bearing capacity, minimize frost heave damage, and reduce highway maintenance expense. Full-depth reclamation uses a pulverizer to grind the asphalt surface, blending it with the gravel base to a depth of 8 in. The road is then reshaped and approximately three-quarters of the required calcium chloride is added. Additional pulverization is performed to ensure a uniform mixture of road material and calcium chloride. Following this, the road is graded, rolled, and final application of calcium chloride is made. Testing of full-depth reclamation with and without calcium chloride addition indicates that use of the reclamation process achieves a dense, stable, granular layer, improving overall pavement strength compared to original pavement condition. The addition of calcium chloride enhances this stabilization of the granular layer 10 percent beyond strength measured in the untreated reclaimed road section. A 50 to 60 percent reduction in frost heave can be expected in reclaimed sections of road using calcium chloride.

In a full-depth reclamation process, existing bituminous surfaces are pulverized and blended with a predetermined amount of granular base material in place, resulting in a more uniform and denser base. Calcium chloride aids in this densification process and provides the added benefit of reducing frost heave. Full-depth reclamation may be used to upgrade almost any secondary roadway. Full-depth reclamation with calcium chloride is of particular benefit to roads with structural deficiencies in the surface or base course. This process will not correct subgrade deficiencies. City and suburban streets, county and secondary roads, parking lots and storage areas, as well as airport runways and highway shoulders have all benefited from the process of full-depth reclamation.

A case study of full-depth reclamation with and without calcium chloride is described to demonstrate the benefits achievable with calcium chloride addition in low-volume secondary road maintenance.

ACTIVE PROPERTIES OF CALCIUM

In order to choose projects where calcium chloride can be successfully used, a description of its active properties and their relationship to soil stabilization is necessary.

Calcium chloride is a hygroscopic and deliquescent chemical. Hygroscopicity is the property of absorbing moisture from the air; deliquescence is the property of dissolving in this moisture to form a liquid solution. When calcium chloride deliquesces, the resulting solution is hygroscopic and absorbs moisture until equilibrium is reached between the vapor pressure of the solution and that of the surrounding air. Figure 1 shows the atmospheric conditions under which calcium chloride will deliquesce. A project using calcium chloride must be located in an area where relative humidity is greater than 40 percent. Table 1 indicates the water absorption capability of calcium chloride solutions at 77°F and various relative humidity levels. Calcium chloride solutions will act to add or remove water from the environment, until equilibrium is achieved.

The vapor pressure of calcium chloride solutions is significantly less than that of water at the same temperature. Because evaporation is a direct function of vapor pressure, it takes place at a much slower rate from a calcium chloride solution versus water at any given temperature and relative humidity. Surface tension of calcium chloride solutions is higher than that of pure water—a property further inhibiting evaporation versus pure water.

The effect of hygroscopicity, deliquescence, lower vapor pressure and increased surface tension provided by a calcium chloride solution is a stabilization project that maintains an optimum moisture content longer than if calcium chloride were not present—an important factor in obtaining and retaining maximum density in a well-graded mixture.

In order to achieve soil stabilization with calcium chloride, clay soils similar to those shown by the AASHTO group A-2 classification are required. The positively charged calcium ion reduces the negative charge on the clay particle. This serves to reduce the repulsion between clay particles and the thickness of their insulating water film. The reduced repulsion coupled with a surface tension greater than plain water results in a tremendous magnification of the attractive forces between clay particles. The binding ability of the fines is thereby greatly increased as is road stability.

Calcium chloride solutions freeze at extremely low temperatures. Figure 2 shows this relationship. When a solution containing less than 29.8 percent $CaCl_2$ is gradually cooled, crystals of ice will form as the saturation curve is intersected. Further cooling will serve to increase the calcium chloride content of the remaining liquid solution with complete solidification only occurring at −67°F. Thus, calcium chloride depresses the freezing point of capillary water in soil that serves to minimize frost heave damage in road maintenance projects using calcium chloride.

PROCESS DESCRIPTION

Preliminary testing and inspection of the road must be performed before reconstruction, including the evaluation of ex-

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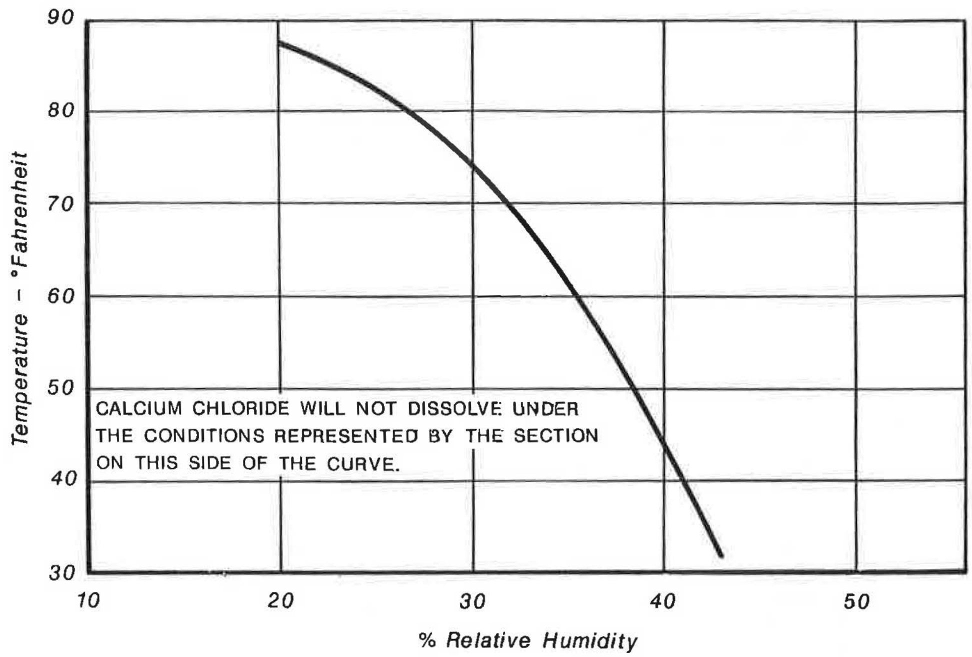

FIGURE 1 Conditions for deliquescence of calcium chloride.

TABLE 1 HYGROSCOPICITY OF CALCIUM CHLORIDE SOLUTIONS

RELATIVE HUMIDITY %	WATER ABSORBED BY ONE POUND OF CALCIUM CHLORIDE POUNDS	STRENGTH OF FINAL SOLUTION OF CALCIUM CHLORIDE %	VAPOR PRESSURE OF FINAL SOLUTION OF CALCIUM CHLORIDE mm OF Hg
95	8.2	8.5	22.2
90	4.5	14.5	21.0
85	3.3	18.3	20.3
80	2.5	22.0	18.9
75	2.1	25.0	17.5
70	1.8	27.5	16.6
65	1.6	29.8	15.5
60	1.4	32.0	14.0
55	1.3	34.0	12.7
50	1.2	36.0	11.8
45	1.1	37.9	10.5
40	1.0	39.9	9.5
35	0.9	42.1	8.5
30	0.8	44.5	

(Atmospheric humidities in equilibrium with Calcium Chloride solutions at 77°
NOTE: At 77°F the vapor pressure of pure water is 23.8 mm of Hg and it
tension is 71.9 dynes per cm^2.

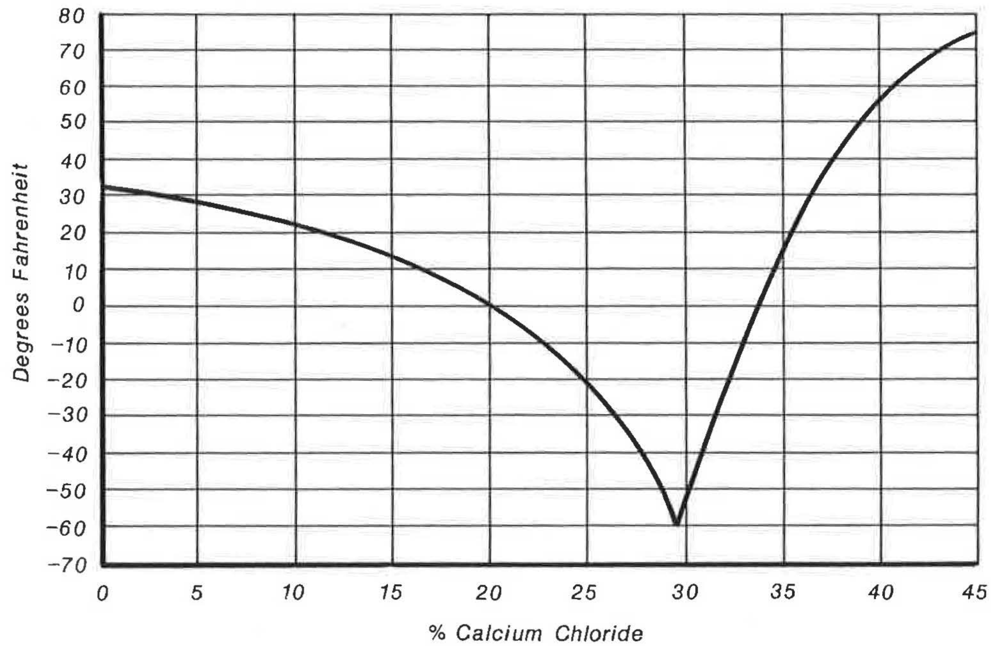

FIGURE 2 Ice saturation and apparent solidification of commercial calcium chloride solutions.

isting road conditions and determining the desired project requirements. Content and thickness of the existing asphalt structure and base material, in concrete with load requirements, will dictate the type and amount of any additives necessary to obtain the highest structural value for the investment. The depth of the bituminous surface must be checked to determine the design depth of pulverization. On the basis of these results, the engineer may choose the correct process for the given road conditions, thus allowing a contractor to select the proper equipment to accomplish the job.

The process of full-depth reclamation involves preparation of existing road material, including full-depth pulverization and mixing of asphalt pavement and granular base material. Pulverization is a mechanical process that physically breaks and crushes the asphalt pavement and a portion of the underlying base material to a suitable gradation. The pulverized asphalt cement acts as a binder in the base material, producing a stronger, homogeneous base that will support heavier loads. The design depth of pulverization is usually taken as twice the depth of the bituminous surface layer (approximately 8 in.). A pulverized bituminous layer blended with granular base material distinguishes full-depth reclamation from cold in-place recycling.

The pulverization process is performed by reclaiming machines, pulvimixers, stabilizers, or crushing units. These machines provide gradation and mixing requirements in one or two passes. These machines are self-propelled and usually consist of a rotating cutter mandrel equipped with carbide-tipped teeth. The unit breaks up existing asphalt pavement, while simultaneously pulverizing and blending it with the gravel base material. After pulverization is complete, excess material may be removed and redistributed or additional aggregate of proper gradation added depending on project requirements and design specifications.

With pulverization complete to proper gradation, additives such as liquid calcium chloride, asphalt emulsions, cement, or lime may be added to the reconstructed base to obtain longer-term performance. When calcium chloride is used as an additive, the aggregate mass is pulverized as required to thoroughly mix all materials and to meet the following gradation requirements:

U.S. Sieve Designation	*Openings (in.)*	*Percent by Weight Passing*
2 in.	2.00	100
1 in.	1.00	30–65
No. 200	0.0029	3–12

The total recommended amount of 35 percent liquid calcium chloride to be used is 1 gal/yd^2 (4.70 lb/yd^2) of road surface. The first application of 35 percent liquid calcium chloride is applied after the first pulverization at a rate of 0.75 gal/yd^2 (3.60 lb/yd^2) of road surface. The aggregate is then pulverized a second time to ensure proper gradation and mixing of the asphalt, gravel, and calcium chloride. A thorough mixing of calcium chloride is essential. After the materials are shaped, graded, and compacted, the existing base is sealed with 0.25 gal/yd^2 (1.10 lb/yd^2) of 35 percent liquid calcium chloride.

The stabilization effect of the calcium chloride is sufficiently immediate and the use of this process is for low-volume roads. The resultant surface may be opened to speed-controlled traffic, if required, without experiencing excessive wear or deterioration. The base course with calcium chloride should be allowed to cure for several weeks before construction of the final wearing surface. The length of time necessary for proper curing of the base course is dependent on climatic conditions of humidity and rainfall.

In order to achieve long-lasting results, a wearing course should be set on top of the base. Depending on traffic load, this wearing surface may consist of a single- or double-seal coat, cold mix, or varying thicknesses of asphalt cement.

FIELD TRIAL DESCRIPTION

Studies and tests of full-depth reclamation with and without calcium chloride base stabilization were conducted. The objectives of the case study (*1*) were to determine the effects of full-depth reclamation with calcium chloride stabilization on a low-volume road pavement structure. Data were acquired on both a stabilized test section and a control section.

The town of Caledon, Ontario, 10 mi north of Toronto, defined the study section of the Chinguacousy Road to be in need of base stabilization to remedy known structural deficiencies. The Chinguacousy Road is a rural two-lane roadway. Over the years, the study section had undergone surface treatment and hot-mix asphalt patching to maintain a dust-free trafficable surface. The study section on Chinguacousy Road was selected on the basis of the relative uniformity of the bituminous surfacing thickness and the granular material type.

The test area was divided into two 500-ft-long sections with an intermediate buffer area 65 ft in length dividing the two sections. Figure 3 shows this area.

The pavement conditions before reconstruction were determined by sampling through test pits. These conditions are described with respect to subgrade soil type, granular quality, and bituminous surfacing.

The subgrade soil, clayey silt to silty clay till, is part of the Halton Peel till stratigraphic unit. The liquid and plastic limits were determined to be 30 and 17 percent, respectively. The classification of the subgrade soil is A-2-6 in the AASHTO classification system and SC in the unified classification system.

The granular layer is a mixture of sand and gravel, composed primarily of hard carbonates. Gradation of the granular layer is presented in Table 2. Residual chloride levels (buildup from winter salting operations) were in the range of 850 to 910 ppm at depths from 3.20 to 6.0 in. in the pavement. Granular thickness of the base course varied between 8.0 and 12.0 in.; the granular thickness of the subbase course was irregular and not present in several areas.

The bituminous surfacing consisted primarily of a heterogeneous mixture of hot-mix asphalt and surface treatment. It

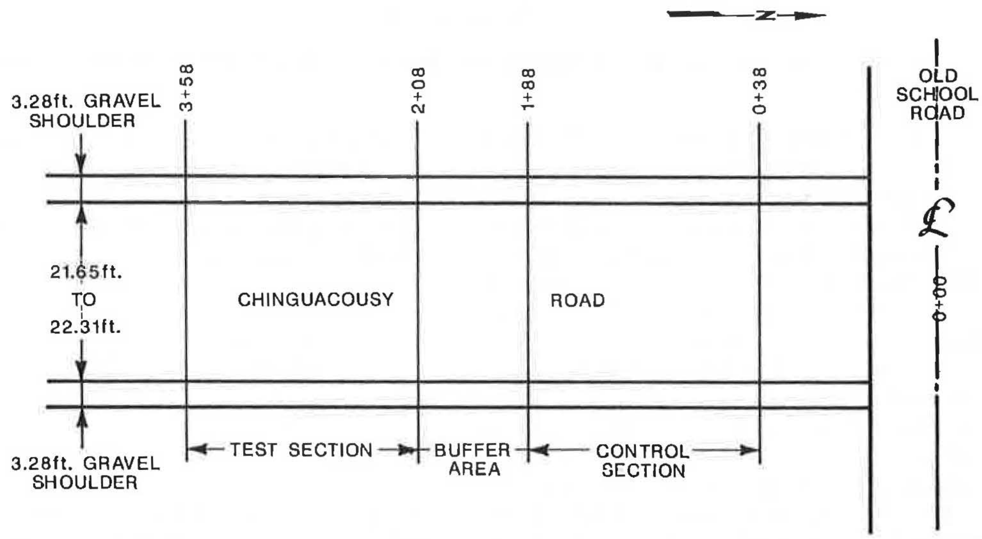

FIGURE 3 Test site diagram for Chinguacousy Road, Caledon, Ontario.

TABLE 2 BASE COURSE GRADATION ON CHINGUACOUSY ROAD, CALEDON, ONTARIO

		Percent By Weight Passing			
U.S. Sieve Designation	Opening (inches)	Granular Base Before Pulverization Test Area 1	Granular Base Before Pulverization Test Area 2	Granular Base Before Pulverization Test Area 3	Granular Base After Pulverization Test Area 4
1 Inch	1.00	100	100	100	100
3/4 Inch	0.75	97	100	99	100
Number 10	0.0787	43	20	34	30
Number 40	0.0165	21	12	17	13
Number 200	0.0029	7	7	7	6

had a fairly uniform thickness of 2.4 to 3.2 in. throughout. The average asphalt content was found to be 5.1 percent with an average asphalt penetration of 22.

The study section of the Chinguacousy Road was reconstructed by the process of full-depth reclamation. Both the control section and the test section were treated with the same equipment, materials, and procedures. The only difference in construction was application of calcium chloride to the granular base of the test section.

The existing bituminous surface layer was pulverized and blended with the underlying granular base to a design depth of approximately 6 in. using one pass of a reclaiming machine. The machine traveled at a speed of 55 ft/min and produced an average aggregate top size of 0.75 in. The exposed aggregate was then graded for full road width to restore crossfall. Additional imported sand and gravel granular base course was placed on the test and control sections using end-dump and belly-dump methods. Water was applied using a tanker truck with a spray bar attachment to achieve suitable moisture conditions. The total average additional granular thickness was approximately 2 in. after grading and shaping. Liquid calcium chloride (35 weight percent solution) was applied at a rate of 80 gal/yd^2 (3.80 lb/yd^2) to the graded granular base of the test section. This application represents approximately 1.0 percent calcium chloride by weight of dry aggregate for a 6-in. layer thickness. The loose granular was mixed with a reclaiming pulvimixer with one pass, and then graded and shaped. A single steel drum vibratory compactor compacted the loose granular. A final application of liquid calcium chloride was made at a rate of 0.20 gal/yd^2 (0.90 lb/yd^2). After curing from August 11 to August 29, 1989, the road surface was double surface treated with stone chips and emulsion.

A residual chloride test from October 1989 on granular samples obtained from test pits indicated that the effective depth of the calcium chloride was approximately 6 in. A visual inspection of the test pits under in situ granular conditions found a comparatively greater moisture content and darker color to a depth of approximately 6 in. Residual chloride data from October 1989 are presented in Table 3.

Nuclear compaction tests on the granular base material were taken both in the control and test section on August 29, 1989, just before the application of the surface treatment by the town of Caledon. Compaction results indicate that the average percent compaction on both sections was 98 percent standard Proctor maximum dry density (SPMDD) with a range of 94 to 100 percent SPMDD in the test section and 96 to 100 percent SPMDD in the control section. The results are based on an average SPMDD of the pulverized and blended granular of 138 lb/ft^3.

FIELD TRIAL RESULTS

Falling weight deflectometer (FWD) testing was carried out in August 1989, October 1989, April 1990, and August 1990. The FWD is a nondestructive testing device that imparts a dynamic load impulse to the pavement surface similar to a moving vehicle. The deflection response of the pavement to the applied loads is measured by seven seismic transducers. At Chinguacousy Road, FWD testing was undertaken in the outer wheelpath of the north- and southbound lanes at a spacing of 33 ft. The applied load magnitudes of the north- and southbound lanes were 6,000, 9,000, and 12,000 lb.

The FWD center deflections, normalized to 9,000 lb and 70°F, which are a measure of the overall composite strength of the pavement (including subgrade soil) immediately under the loading point, are presented in Table 4. The data indicate a relative increase in pavement strength of 3.4 to 15.1 percent in the test section as compared with the control section over 12 months. April 1990 FWD results indicate a decrease in pavement strength both in the test and control sections with the test section exhibiting a lower loss. Seasonal variability of road strength is a well-known phenomenon. As shown

TABLE 3 RESIDUAL CHLORIDE ON CHINGUACOUSY ROAD, CALEDON, ONTARIO

Sample Depth (inches)	Sample Location and Chloride Concentration in ppm: Borehole 6 Before Stabilization	Borehole 7 Before Stabilization	Saw-Cut #1 (Test Section) (1)	Saw-Cut #3 (Control Section) (1)
0 to 3	- (2)	-	2090	730
3 to 6	910 (3)	850 (3)	1160	460
6 to 8	-	-	620	590
8 to 12	-	-	460	440

(1) Test data from samples obtained approximately two months after Calcium Chloride stabilization.

(2) - Indicates Area Untested

(3) Test data represent benchmark chloride concentrations in original pavement structure before stabilization.

TABLE 4 SUMMARY OF FWD-NORMALIZED CENTER DEFLECTIONS

Section	Initial Mean Deflection August, 1989 (inches)	Mean Deflection October, 1989 (inches)	Mean Deflection April, 1990 (inches)	Mean Deflection August, 1990 (inches)	Percent Reduction In Deflection (12 Months)
Test Section					
Northbound Lane	0.052	0.035	0.059	.033	36.5%
Southbound Lane	0.049	0.035	0.052	.033	32.7%
					34.6%
Control Section					
Northbound Lane	0.042	0.033	0.050	.033	21.4%
Southbound Lane	0.041	0.031	0.047	.029	29.3%
					25.4%

in Figure 4, the strength loss in April 1990 is an expected result (2).

The calculated deflection basin areas, as defined by Hoffman and Thompson (3), are presented in Table 5. The basin area is a measure of the ability of the pavement to distribute an applied load. In general, the magnitudes of the basin areas tend to be higher as pavement load distribution improves. Table 5 indicates an overall increase of approximately 2 percent in the calcium chloride–stabilized test section versus the untreated control section over the 12-month period. Once again, the expected seasonal impact can be noted when comparing results from April 1990 to October 1989 or August 1990.

Resilient moduli tests were performed on both the calcium chloride–treated granular and untreated granular samples that were recovered from the on-site pulverization and mixing process. The resilient moduli testing was carried out using closed-loop electrohydraulic MTS equipment at McMaster University, Hamilton, Ontario. Testing was based on the interim SHRP protocol (4,p.46), with some modifications to the preparation of granular test specimens. The samples were prepared at 100 percent standard Proctor density and were frozen to facilitate handling and setup in the MTS triaxial cell unit. A load duration of 0.1 sec and a cycle duration of 1 sec were used with a haversine stressed pulse. The resilient modulus is a measure of the applied axial stress over the recover-

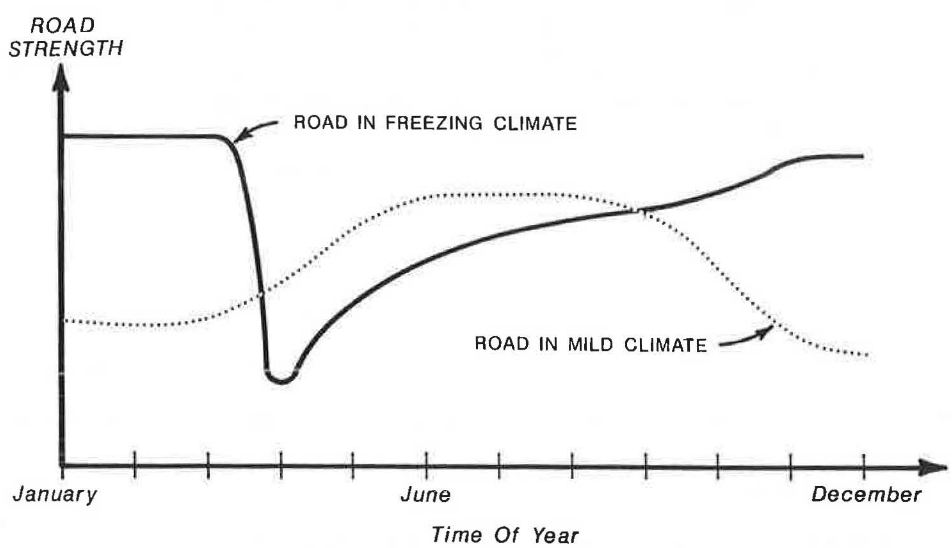

FIGURE 4 Seasonal variability of road strength.

TABLE 5 SUMMARY OF FWD BASIN AREAS

Section	Initial Mean Deflection August, 1989 (inches)	Mean Deflection October, 1989 (inches)	Mean Deflection April, 1990 (inches)	Mean Deflection August, 1990 (inches)	Percent Increase In Deflection (12 Months)
Test Section					
Northbound Lane	0.0165	0.0189	0.0177	0.0185	12.1%
Southbound Lane	0.0173	0.0193	0.0185	0.0185	6.9%
					9.5%
Control Section					
Northbound Lane	0.0173	0.0189	0.0177	0.0188	8.7%
Southbound Lane	0.0173	0.0189	0.0177	0.0185	6.9%
					7.8%

able strain for given magnitudes of load, load duration, and bulk stress conditions. Test results from Table 6 show that the calcium chloride–stabilized granular has a 24 to 36 percent higher modulus than untreated material for conditions of low bulk stress. The moduli of the stabilized and nonstabilized material are similar at higher bulk stress conditions.

Slate's work states that there is reduced freeze-thaw susceptibility of materials treated with calcium chloride and that there is satisfactory long-term retention of calcium chloride in the pavement structure. Samples treated with 1.0 percent calcium chloride by weight of dry aggregate may experience 50 to 60 percent reduced frost heaving, compared to untreated samples (*5*).

Residual chloride testing was conducted on the test section of the Chinguacousy Road in August 1990. Samples were analyzed from the edge and center of the road at depths

TABLE 6 SUMMARY OF RESILIENT MODULI TEST RESULTS

Test Load (Cell Pressure = 3 psi)	Resilient Modulus (psi)		
	Sample A (not stabilized)	Sample B (stabilized)	Sample C (stabilized)
Load = 55 pounds	59,600	81,400	74,300
Load = 90 pounds	35,000	34,600	40,600
Load = 128 pounds	24,700	22,100	26,100

NOTES:
1) Loading duration was 0.1 seconds with a cycle duration of 1 second and a haversine-shaped stress pulse
2) Stabilized samples treated with 1.0 percent $CaCl_2$ by dry weight of aggregate
3) All samples prepared at 100 percent Standard Proctor Density and approximately 7 percent moisture content

TABLE 7 RESIDUAL CHLORIDE ON TEST SECTION OF CHINGUACOUSY ROAD, CALEDON, ONTARIO, AUGUST 1990 (PARTS PER MILLION)

Sample Depth (inches)	Before Stabilization	October, 1989 Test Section	August, 1990 Test Section Center of Road	August, 1990 Test Section West Edge
0 - 3	- (1)	2090	3690	2250
3 - 6	910 (2)	1160	2620	2140
6 - 8	-	620	1260	1960
10 - 12	-	460	639	1820
12 - 18	-	-	496	1860
18 - 24	-	-	933	1540
24 - 30	-	-	541	1190
30 - 36	-	-	326	809
36 - 42	-	-	320	532

(1) Indicates area untested

(2) Test data represent benchmark chloride concentrations in original pavement structure before stabilization

varying from 0 to 42 in. Table 7 indicates that the calcium chloride has been retained in the road structure over the 12-month period.

Visual condition surveys conducted August 1989, October 1989, January 1990, April 1990, and August, 1990 had good cross-sectional properties and no potholes in both sections.

CONCLUSION

Calcium chloride addition to the full-depth reclamation process provides an optimized maintenance procedure for low-volume roads in geographic areas where humidity and soil clay content support calcium chloride use. Data presented consistently indicate road strength has been improved in the test section using calcium chloride addition versus the control section with no additive.

Resilient moduli testing indicates an increase in strength (modulus) in the calcium chloride–stabilized section over the untreated granular section in the range of 24 to 36 percent at conditions of low bulk stress. Normalized FWD deflections indicate an in-place overall pavement strength increase in the test section compared to the control section was 3.4 to 15.1 percent over a 12-month period. Both the test and control section incurred a strength loss in April 1990, with the test section exhibiting a lower percentage loss from original values. These results reflect moisture retention (hygroscopic) properties of calcium chloride.

Residual chloride testing 12 months after application indicates retention of calcium chloride in the test section has been excellent. Previous studies (*6*) indicate satisfactory long-term retention of calcium chloride in pavement structures can be expected. Reduced freeze-thaw susceptibility is also indicated.

The analysis of AASHTO design methods for low-volume aggregate-surfaced roads given in the *AASHTO Guide for Design of Pavement Structure* (*7*) indicates that for the Chinguacousy Road test section the strength improvement in the granular base layer because of calcium chloride stabilization should result in an increase in allowable standard-axle traffic loadings of 40 to 50 percent, or an increased pavement life of 3 to 5 years.

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Epoxy-Resin-Based Chemical Stabilization of a Fine, Poorly Graded Soil System

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Results are described of a research effort on the epoxy-based treatment of fine, poorly graded soil found at some localized low-duty airport sites and in the north slopes of Alaska. Statistical models are developed for the stabilization of clay-silt pavement systems at low-duty airports. A nontraditional method of soil stabilization that improves the subgrade strength properties of poorly graded clay-silt was identified. This soil system is considered one of the most difficult soil types to stabilize, in part because of its poor particle size distribution. Among several organic additives tested, the two-part epoxy system—bisphenol A/epichlorohydrin resin plus a polyamide hardener—gave the best result as measured by the dry California bearing ratio (CBR) test. The choice of the dry CBR test performed to ASTM specification was motivated by a need to capture optimum moisture content as an experimental variable. Within the limits of the laboratory test conditions, the statistical regression models developed support the hypothesis that the marginal increase in CBR values caused by a 1 percent increase in epoxy resin application is 11.1 and the marginal degradation of CBR caused by a 1 percent increase in moisture level is −5.6. Also, only additive level, moisture content, and temperature are significant variables influencing soil strength.

Materials engineers are frequently confronted with the problem of improving the bearing strength of unsuitable soils through soil stabilization. FAA provides guidance to airport owners, operators, and designers on methods to increase the load-bearing capabilities of the subgrade to support aircraft loads (*1,2*). Existing methods include the application of various combination of lime, cement, and fly ash to native materials and the replacement of unsuitable material with improved, higher-quality material. Commercial airport pavements that serve aircraft with high landing gear loads are required to be stabilized in a manner that ensures adequate strength of the supporting layers.

However, in small remote airports serviced by low-duty aircraft such as some on the north slopes of Alaska and in major portions of Florida where incompetent subgrade conditions exist, the difficulty of obtaining materials for standard methods of soil stabilization are evident. The need therefore exists to quantify the manner in which some nontraditional additives interact to increase the stability of native incompetent subgrade material.

The engineering properties of a clay-silt system were altered by use of a nontraditional chemical additive that when mixed uniformly into the soil system changed the surface molecular properties of the soil grains and, in most cases, cemented the grains together, resulting in increased strength. The clay used was kaolinite. The use of mechanical and traditional chemical stabilization techniques on the clay-silt system was precluded in this research.

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RESEARCH OBJECTIVES

An aggregate framework was developed for understanding the strength behavior of clay-silt systems and the subsequent formulation of a statistically based model for airport pavement subgrade stabilization through the combined use of an epoxy resin, bisphenol A/epichlorohydrin, and a polyamide hardener as a stabilization agent. The model presented enables the airport pavement designer to predict expected soil strength and effect design under a wide range of feasible combinations of these variables at a variety of potentially low-duty airport sites.

The research scope includes a state-of-the-art investigation, automated data collection, and analysis of full-scale laboratory data and formulation of a statistically based model for soil stabilization of airport pavement subgrades. This research is applicable to low-duty airport pavements. These pavements are defined as "landing facilities to accommodate personal aircraft or other small aircraft engaged in nonscheduled activities such as agricultural, industrial, executive, or industrial flying." Such pavements will not be required to handle aircraft load exceeding a gross weight of 30,000 lb. The total depth of pavements for low-duty airport pavements rarely exceeds 22 in., this being the limit for clay-silt soils with low CBR value, typically 3 to 4 (*3*).

LITERATURE REVIEW

Review of literature revealed that little work has been done on clay-silt soil stabilization using organic additives; however, much work has been documented in the literature on the use of traditional additives such as lime, cement, and fly ash.

A study by McLaughlin suggested that engineers will be more inclined to use stabilization techniques to strengthen pavement structures in the wake of such factors as increasing aircraft payloads, traffic frequency, scarcity of sites with good subgrade bearing values, and the dwindling supply of conventional aggregate (*4*). McLaughlin (*4*) revealed that soil mixtures with lime, cement, and fly ash (LCF) base courses have been used extensively, and complex, long-lasting chemical reactions that produce resultant materials with acceptable mechanical properties were obtained under the right conditions of temperature and moisture. However, hostile in situ pavement conditions, if allowed, could lead to a weak pavement system because of infiltration of sulfate- and carbonate-bearing moisture or a strong alkaline ground water condition.

Simpson et al. (*5*) report about the successful application of the Shell EPON® epoxy resin as an asphalt binder. The paving material, epoxy asphalt concrete (EAC), is a combination of graded mineral aggregate and an asphaltic binder containing epoxy resin that is converted into a polymer with unusual solvent and heat resistance (*5*). The EAC could be used as an overlay on existing pavements or as the major structural element in a pavement. Construction using the EAC required conventional hot-mix plants, conventional self-propelled paving machines or hand-raking, and normal rubber-tired rollers for compaction. In a series of comparative tests conducted, EAC out-performed asphaltic concrete in terms of added strength, load-carrying capacity with minimum thickness, resistance to chemical attacks, rutting, and high-temperature jet blasts. Specific areas of successful application of EAC reported included overhaul and maintenance areas used for jet planes at several military airbases.

Kinter (*6*) documents the results of 20 years of cooperative efforts between the FHWA and the chemical industry to investigate and develop specific compounds (or combinations thereof), industrial products, and wastes for the purpose of soil stabilization or compaction aids, or both. About 50 chemicals and proprietary products were tested (*7*). The study concluded that no single chemical or combination of chemicals has been found effective as a major soil stabilizer.

Carpenter and Lytton (*8*) report that clay-silt systems will experience a volumetric contraction on freezing in a condition of constant moisture content. The freezing process reorients the clay particles, leading to volumetric contraction. This research helps to quantify the magnitude of the freeze coefficient to be used in pavement design under frost conditions and damage assessment.

Edris and Lytton (*9*) characterized the performance-related characteristics of fine-grained subgrade soils containing 20 to 70 percent clay. They developed statistical models for resilient modulus, resilient strain relations, and permanent deformation per unit length or residual strain. The research demonstrated that dynamic properties of fine-grained soil such as modulus of resilience depend strongly on traffic volume (measured by load cycles), climatic factors (measured by soil suction and temperature), and soil composition.

Chou investigated the marginal effect of the variability of design parameters in the California bearing ratio (CBR) equation on pavement performance (*10*). The research indicated that pavement integrity is significantly affected by variations in pavement thickness, in wheel loads, and in subgrade CBR value. Variations in tire contact area had the least influence on pavement performance.

Brabston (*11*) investigated the FAA soil compaction criteria for airport pavement subgrades using laboratory compaction and triaxial tests to determine resilient and permanent axial strain. Three soil types compacted to densities at or below current FAA compaction criteria were subjected to repeated axial loading in a triaxial tests chamber. The FAA uses ASTM standards as compaction criteria (*12*).

RESEARCH METHODOLOGY

ASTM D1883–73 CBR test procedure was adopted for strength quantification (*12*). The CBR test is a generally accepted and reliable strength measurement approach and is applicable to airport pavement design methodology.

The choice between using a dry CBR test and a wet one is influenced by many factors, chiefly the condition of the soil system in the field on a short- and long-term basis. The dry CBR test was adopted because it allowed the specification of clay-silt soil optimum moisture content as an experimental variable for analysis. Also moisture control is more practical and implementable at localized low-duty airport sites.

In order to gain control over the various factors that influence field CBR, representative ranges of all the selected variables hypothesized as influencing CBR were tested and analyzed.

EXPERIMENTAL DESIGN

The variables hypothesized as influencing the resistance of a soil to deformation as measured by the CBR value were additive content (percent), moisture content (percent), clay-silt ratio, and curing temperature (°F).

Factorial Design

There are several advantages in studying the effects of several independent variables on a dependent variable, say CBR value, using factorial designs. First, and most significant, it is possible to determine whether the experimental independent variables interact in their effect on the dependent variable. Although an independent variable may affect a relatively small proportion of the variance of a dependent variable, its interactions with other independent variables may affect a large proportion of the variance. This phenomenon cannot be understood by the study of the independent variables in isolation.

Second, factorial designs afford the researcher greater statistical control, and therefore more discriminatory statistical tests than those tests typically associated with single variables. Factorial experiments allow the testing of the separate and combined effects of several variables using the same number or fewer experimental runs that would have been the case for several single-factor experiments.

The experimental design was fashioned to fit a factorial design matrix. Factorial designs facilitate the visualization and comprehension of similarities and simplifications in the experimental process and thus assist the task of model building and the estimation of main effects and interactions arising as a result of changes in the model experimental variables.

For the hypothesized experimental variables, a $4 \times 3 \times 3 \times 3$ (i.e., 4×3^3) factorial design in additive, moisture, clay-silt ratio, and temperature, requiring 108 CBR experimental runs, was used.

Levels of Variables

The levels of various variables hypothesized as affecting CBR were specified as follows:

Experimental Variables	*Factorial Design Levels*	*No. of Levels*
Additive percentage	0, ½, 1, 4	4
Moisture percentage	13, 17, 21	3
Clay-silt ratio	0.4, 0.5, 0.6	3
Temperature (°F)	40, 65, 90	3

An important consideration was the specification of the range of applicable moisture content for the experimentation. The dry density values obtained for a clay-silt ratio of 0.4 was about 114 lb/ft^3. For a clay-silt ratio of 0.6, it was about 107 lb/ft^3. Corresponding optimum moisture contents were 16 and 20 percent for clay-silt ratios of 0.4 and 0.6, respectively. The analysis indicated that maximum dry densities obtained for the three levels of clay-silt ratios corresponded to optimum moisture contents between 13 and 21 percent. The levels of moisture content were fixed at 13, 17, and 21 percent on the basis of earlier control test results obtained from the curves of maximum dry density versus moisture content under modified AASHTO compaction; also, because moisture content was desired at three equidistant levels in the factorial experimentation, 13, 17, and 21 percent were selected as representing practical values of optimum moisture content and optimum dry density variations for the clay-silt system.

Testing Procedure

The ASTM D1883–73 procedure for CBR testing was adopted for determining the strength of the clay-silt soil system (*12*). The physical properties of the clay and silt samples tested are presented in Table 1. The clay and silt samples were prepared in a manner closely following the ASTM D1557 method (*12*). The tests were carried out on unsoaked samples because the clay silt optimum moisture content was specified as an experimental variable.

The batching of various ratios of clay to silt by weight was done and resulted in a representative clay-silt sample weighing over 12 lb, to which was added the required amount of water. The sample was mixed to a uniform consistency and the epoxy-resin system was applied to the wet sample and mixed uniformly and manually to an even texture.

The samples treated with the epoxy-resin system were compacted in standard CBR molds specially lined with aluminum foil to preserve the molds and reduce demolding effects. The compacted specimens were trimmed to specification and covered with nylon wrappers for moisture preservation before being thermally soaked in a specially prepared curing chamber for 3 days, which was considered enough time for the attainment of steady state conditions.

TABLE 1 SUMMARY OF PHYSICAL PROPERTIES OF CLAY AND SILT TESTED

		SOIL TYPES		
	Properties	Clay	Silt	Clay-Silt Systems
1.	Liquid Limit %	60	22	37 - 45
2.	Plastic Limit %	32	19	24 - 27
3.	Plasticity Index %	28	3	13 - 18
4.	Optimum Moisture Content %			13 - 21
5.	Absorbed Moisture % (Hygroscopic)	1.5	0.5	1.0
6.	Soil Classification			
	a) FAA	E-8	E-6	E-7
	b) Unified System	CL	ML	CL/ML
	c) AASHO	A-7	A-4	A-5 to A-7
7.	Specific Gravity	2.63	1.84	2.33
8.	Percent Passing No. 200 Sieve	100%	100%	100%
9.	Hydrogen Ion Conc. pH, dry clay at 20% solids, airfloated	3.5-5.0	—	—
10.	Clay Fraction	100%	0%	40,50,60%

In order to facilitate the acquisition and reduction of the CBR test data, a microcomputer-based automatic data collection system was used (Figure 1). CBR testing of the thermally cured clay-silt stabilized samples was done using the motorized and automated SOILTEST CBR testing equipment.

The data collection system consisted of (a) the motorized SOILTEST CBR testing equipment complete with a loading platform, plunger, proving rings, force and displacement dial indicators, and other attached accessories; (b) displacement transducers; (c) linear variable displacement transformer; (d) signal conditioner; (e) digital display; (f) screw terminal boards (panels); and (g) personal computer (see Figures 2 and 3).

The automated data collection system fabricated in-house was used to record and analyze in real time the displacement and resistance to deformation of clay-silt samples prepared and compacted to standard specifications after thermal soaking in a temperature-controlled chamber. Occasional manual checks on the collected data were made for correlation and validation purposes. The data collected are presented in Table 2.

ADDITIVE SELECTION

In considering possible materials for the stabilization of the clay-silt system, the use of traditional methods was reviewed but not considered. Most of the traditional materials will, in small quantities less than 5 percent by weight, not impart sufficient strength to the clay-silt system under study to meet FAA requirements. This property is caused by the poor clay-silt system particle size distribution and accompanying loss of frictional strength component, coupled with the degradation of cohesive strength that could quickly result with the infiltration of moisture.

Traditional methods of clay-silt stabilization currently in use are given by Yoder and Witczak (*3*).

The results of CBR tests on the untreated clay-silt system support the position that effort should not be placed solely on the effects of increased compaction and reduced plasticity enhancement as a means of soil stabilization of clay-silt systems. Equally vital to the task of clay-silt soil stabilization are the combined effects of additive application, effective construction practices, provision of adequate roadway drainage and ditches, in addition to good compaction and plasticity enhancement techniques.

The search for effective additives that could adequately meet flexible pavement design requirements for low-duty airport pavements was therefore focused on organic materials and polymers. Candidate materials were screened through a survey of chemical companies, materials testing laboratories, in-house materials testing, and personal contact.

EPOXY-RESIN HARDENER SELECTION

The final stabilization additive selected consisted of a two-part epoxy resin system. The first part is a bisphenol A/epichlorohydrin resin of the epoxy resin family. This resin has negligible solubility in water, is a very viscous liquid, very light yellow in color, with a specific gravity of 1.17. Though a stable material, in the presence of a strong mineral or Lewis acid or a strong oxidizing agent, the epoxy resin can react vigorously to release considerable heat, but hazardous polymerization will not occur. The heat release during the use of the resin in this research was minimal and generally unnoticeable at the ¼ to 4 percent threshold of additive application. Preliminary tests at higher concentrations of epoxy resin, say 10 percent and more, released a noticeable amount of heat.

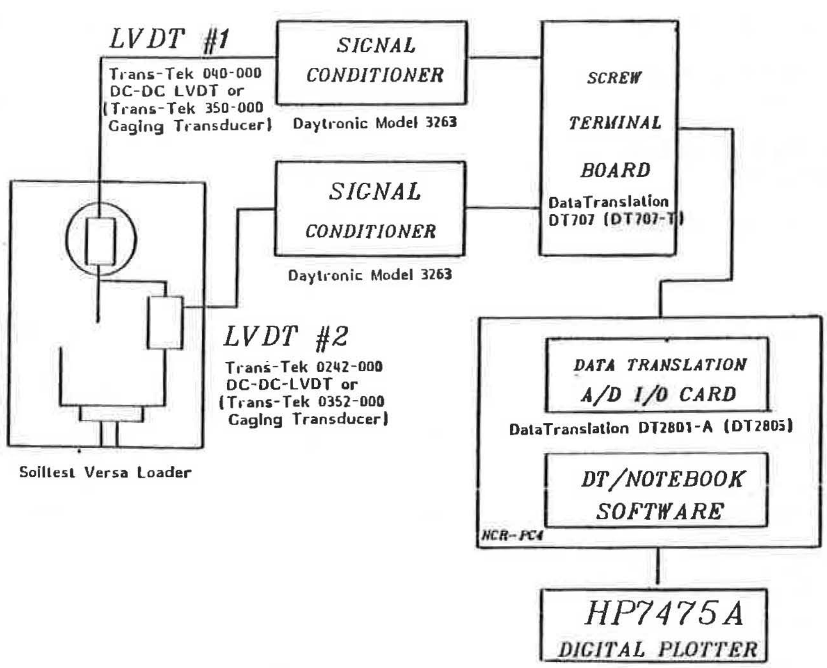

FIGURE 1 Schematic of automated data collection and display system.

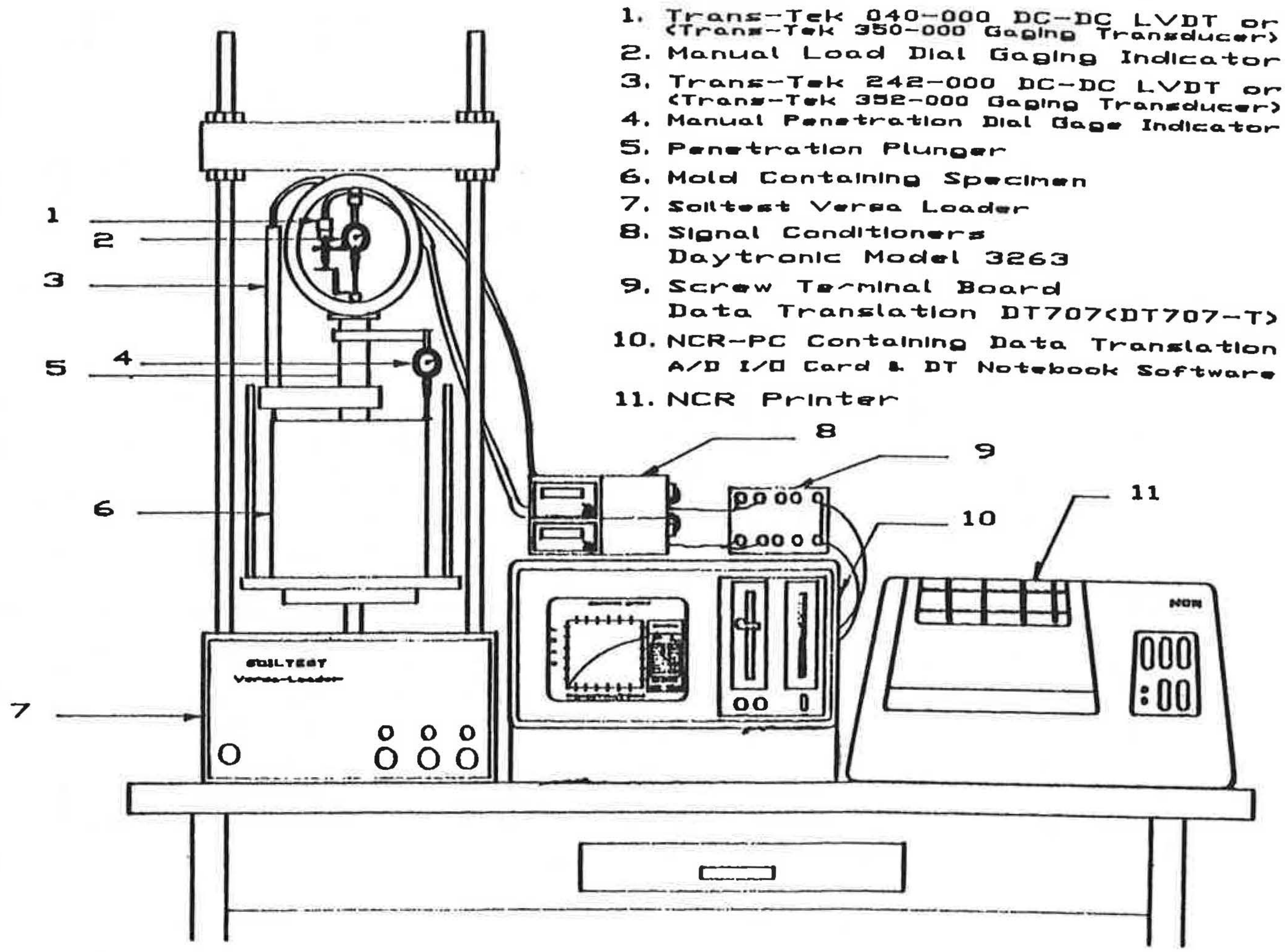

FIGURE 2 Automated data collection and display system.

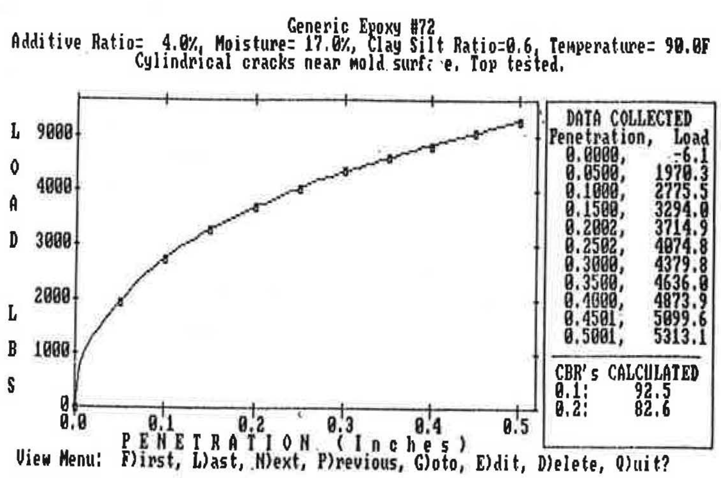

FIGURE 3 Screen output of automated data collection system.

TABLE 2 CBR VALUES FOR STABILIZED CLAY-SILT SYSTEM, FULL FACTORIAL EXPERIMENTAL DESIGN DATA MATRIX (EPOXY RESIN ADDITIVE, 3-DAY CURE PERIOD)

	C/S = 0.4				C/S = 0.5				C/S = 0.6				
%M	%A				%A				%A				T (°F)
	0	.25	1	4	0	.25	1	4	0	.25	1	4	
13	(2)* 61.2	(2) 55.8	(1) 71.0	(2) 86.2	(1)* 68.3	(2) 50.15	(2) 62.9	(1) 84.2	(3)* 37.7	(2) 44.9	(2) 48.8	(1) 89.6	40
17	(1) 3.9	(1) 16.2	(1) 19.1	(1) 42.5	(2) 19.9	(1) 37.4	(2) 31.45	(1) 44.3	(2) 35.0	(1) 60.6	(2) 53.86	(1) 66.4	40
21	(1) 0.8	(1) 4.2	(3) 14.5	(3) 47.0	(1) 1.5	(1) 8.2	(1) 9.3	(3) 23.2	(1) 3.9	(2) 11.3	(1) 18.6	(2) 25.0	40
13	(1) 39.6	(1) 44.6	(1) 68.3	(1) 103.7	(1) 24.6	(1) 41.2	(1) 71.9	(1) 100.6	(1) 22.7	(1) 38.0	(1) 52.0	(1) 96.2	65
17	(1) 4.4	(1) 10.0	(1) 24.8	(1) 47.0	(1) 21.7	(1) 36.0	(1) 42.6	(2) 60.5	(1) 34.2	(1) 55.7	(1) 62.0	(1) 93.4	65
21	(1) 1.2	(1) 4.4	(1) 20.9	(1) 64.0	(1) 2.4	(1) 7.1	(1) 14.7	(1) 40.0	(1) 5.0	(1) 17.8	(1) 31.7	(1) 39.3	65
13	(2) 82.0	(2) 89.5	(1) 45.6	(1) 135.4	(2) 67.2	(2) 52.3	(1) 86.4	(2) 134.2	(2) 26.9	(1) 33.9	(1) 51.3	(2) 115.9	90
17	(2) 3.5	(2) 6.8	(1) 28.5	(1) 47.38	(2) 13.7	(2) 26.6	(2) 36.6	(1) 70.05	(1) 45.7	(1) 59.0	(1) 64.15	(1) 93.52	90
21	(1) 0.7	(1) 9.5	(1) 27.2	(5) 87.1	(1) 1.4	(1) 14.1	(3) 20.0	(3) 48.7	(1) 6.1	(1) 6.1	(1) 44.1	(1) 33.8	90

* Denotes number of CBR values averaged to obtain indicated CBR

The second part of the two-part epoxy resin system, a curing agent, is a viscous, water-insoluble polyamide, tan in color, with a specific gravity of 0.97. In the presence of a strong oxidizing agent, it could react to form nitrogen oxides and carbon monoxide or to release free polyamides that are also deleterious to health. The likelihood of this reaction occurring in the field is low because of protective surface treatment. The polyamide, however, could be replaced without loss of strength with amidoamines (*13*).

Epoxy resin is an organic chemical group composed of polymerized molecules consisting of split oxygen molecules bonded with two carbon atoms already united in some way. Epoxy resin is an amorphous, natural organic substance that could be of plant extraction or synthesized by, for example, the dehydrohalogenation of the chlorohydrin prepared by the reaction of epichlorohydrin with a suitable di- or polyhydroxyl substance or other molecule containing active hydrogen (*13*). Numerous (over 20) types of epoxy resin are possible and resin formulation to suit a particular application is almost always necessary. For the achievement of high strength, the use of a hardener or setting or curing agent is always essential for cross-linking between the epoxy resin and hardener molecule. The mix of epoxy resin and curing agent used in all the tests was 1:1 by weight.

Though epoxy resins harden through exothermic reactions, the quantities needed for stabilization, generally 4 percent or less, would not generate heat and stress that could lead to cracks and other imperfections. The addition of clay and silt as fillers for the epoxy resin system provides heat sinks, and the release of the heat generated can be controlled through choice of hardeners or variation of hardener concentration to control the duration the hardening requires. Though epoxy resins can set in as short a time as 20 min, they can be designed to set in 3 hr, thereby easing and making flexible the time required for construction procedures preparatory to stabilization and subsequent construction equipment cleanup.

An important strength delivery factor in epoxy resin application is the mixing quality. Thorough mixing of the two-part system for a definite length of time until mixing resistance drops off noticeably is imperative for effective cross-linking and strength development.

EPOXY RESIN COST VERSUS EFFECTIVENESS TRADE-OFF

The cost of epoxy (bisphenol A/epichlorohydrin) resin and polyamide hardener varies depending on the type and application. At an average cost of \$1.76/lb, the cost of the epoxy system is much higher than those associated with traditionally used additives such as lime, cement, or fly ash, which cost about \$0.04/lb. However, with an effective dispensing mechanism, the fractional weight application level of an epoxy system for achievement of the same effectiveness as judged by CBR strength delivery may range between 1/6 and 1/25, or less, of that required using stabilization agents such as cement or fly ash.

With the advent of high-technology epoxy application techniques since the 1970s, the gap in the total cost advantages of using conventional materials such as lime, cement, or even fly ash rather than epoxy resins in engineering construction is beginning to narrow when such factors as high strength delivery, lower unit weight of epoxy system required, and total cost, including labor, materials, and other indirect cost factors are all considered (*13*). The advantages of epoxy resin include

1. Reduction in direct cost of repairs by substituting high-technology application techniques for costly manual labor.
2. Introduction of improved consistency and quality leading to savings in terms of reduced deterioration because of the strength and durability of epoxy-treated materials.
3. Accelerated strength attainment within a few hours because of the formation of a solid, dense material capable of withstanding inclement weather, heavy traffic, and chemical attack. The strength of epoxies is borne out by the fact that broken chunks of concrete bonded together by epoxy resin become stronger than before disintegration (*13*).
4. Reduction in indirect cost of repairs, such as delayed air traffic, can be substantial in addition to the reduction in the frustration of aircraft and vehicular operators caused by these unwelcome delays.

Through use of the rapid setting properties of epoxy resins, runway closure resulting in delay and disruption of air traffic operations could be reduced from a time span of months to hours because of the significant engineering properties of epoxy resins.

Large-scale applications based on epoxy resin have been gaining considerable acceptance in the construction industry. In Philadelphia, for example, a successful use of over 10,000 gal of epoxy resin was reported in the construction and structural repair of the city's Schuylkill Expressway (*14*).

SOIL STABILIZATION COST ANALYSIS

An analysis of the cost of an actual laboratory soil stabilization validation experiment conducted at Wright-Patterson Air Force Base in Dayton, Ohio, is presented. The analysis allows a comparison of the cost of epoxy resin application with that of conventional stabilization materials such as cement, as presented in Table 3 using the following data:

Item	*Cost ($ per indicated unit)*
Additives	
Epoxy resin	1.62/lb
Polyamide hardener (V-40)	1.89/lb
Reference additive (cement)	2.78 per 80-lb bag
Aggregates	
Clay soil	6.00/ton
Silt soil	6.00/ton
Labor	15.00/hr-person

In the validation study, a time base of 1 hr was assumed for stabilizing 6.25 ft^3 of material using the productive capacity of one worker.

Overhead costs are specified at 50 percent of the total labor and material costs.

Clay-silt stabilization requires 20 to 30 percent of cement by weight for effectiveness (*3*). In the foregoing analysis, 25 percent was used. Clay-silt specific gravity was 2.67; and water specific gravity was 1.00.

Total weight of 6.25 ft^3 of stabilized material used in the validation experiment is approximately 1,043 lb.

REGRESSION ANALYSIS

The results of the $3^3 \times 4$ factorial design experiments in temperature (TEMP), clay-silt ratio (CS), moisture percentage (PM), and additive percentage (PA), respectively, were subjected to descriptive and inferential statistical analyses. These analyses yielded estimates of the effects of the various independent variables on CBR, the dependent variable. These regression analyses were performed to determine the form of the statistical models suitable for predicting the CBR of a clay-silt system when stabilized at various levels of the independent variables. The comprehensive second-order, stepwise, multiple linear regression analysis was performed. The key feature of this procedure is that a number of intermediate regression models are obtained adding one variable at a time. The variable added at each step is the one that makes the greatest improvement in the goodness-of-fit. The significance level for staying in the model was set at 0.05, and that for exiting at 0.10. These values correspond to confidence levels of 0.95 and 0.90, respectively.

A graphical display of the CBR data in Table 2 is provided in Figures 4–6. The figures all show a striking and similar CBR response of the clay-silt system to additive treatment at the temperatures of 40°F, 65°F, and 90°F. The CBR-degrading influence of moisture in the clay-silt soil system is generally emphasized by all the plots and particularly by the evenly split clay-silt mixture (C/S = 0.5).

The main regression model postulated for the prediction of clay-silt system soil strength was obtained by pooling all of the experimental data. The result obtained using the stepwise regression procedure involving first-order terms only is

$$\text{CBR} = 91.69 + 11.07(\text{PA}) - 5.62(\text{PM}) + 44.97(\text{CS}) + 0.14(\text{TEMP})$$

$$(R^2 = 0.76) \qquad (1)$$

where

PA = epoxy resin additive level (percent),
PM = moisture content level (percent),
CS = clay-silt ratio (decimal), and
TEMP = temperature of curing (°F).

All the regression coefficients were significant at the 3 percent level. The model therefore supports the hypothesis that moisture content increase leads to a degradation of CBR values by its reduction of the cohesive and frictional strength of the soil particles. The CBR test measures the shear resistance of a soil to deformation under applied loads. Because a clay-silt system is being tested, increases in the clay content of a

TABLE 3 SOIL STABILIZATION COST ANALYSIS [IN DOLLARS PER TOTAL WEIGHT OF STABILIZATION (1,043 lb)]

COST ELEMENTS	CONTROL CASE		EPOXY ADDITIVE CASE		CEMENT CASE	
	Weight	Cost*	Weight	Cost*	Weight	Cost*
MATERIAL COST ANALYSIS						
CLAY*	530	0.0	519	0.0	519	0.0
SILT*	357	0.0	346	0.0	346	0.0
WATER (MOISTURE)	132	0.0	143	0.0	143	0.0
ADDITIVE	0	0.0	33	58.1	216	7.5
TOTAL MATERIAL COST PER 6.25 CU. FT.		0		58.10		7.50
LABOR COST ANALYSIS						
1 Pers. @ $15.0/Hr. for 1 Hr .		15.00		15.00		15.00
OVERHEAD COST						
50% OF LABOR + MATL COST		7.50		36.50		11.25
TOTAL COST		22.50		109.60		33.75
COST RATIOS		1.00		4.87		1.50

(Labor Rate : 1 person @ $15.00/hr. for 1 hr.)

*Material is not Imported

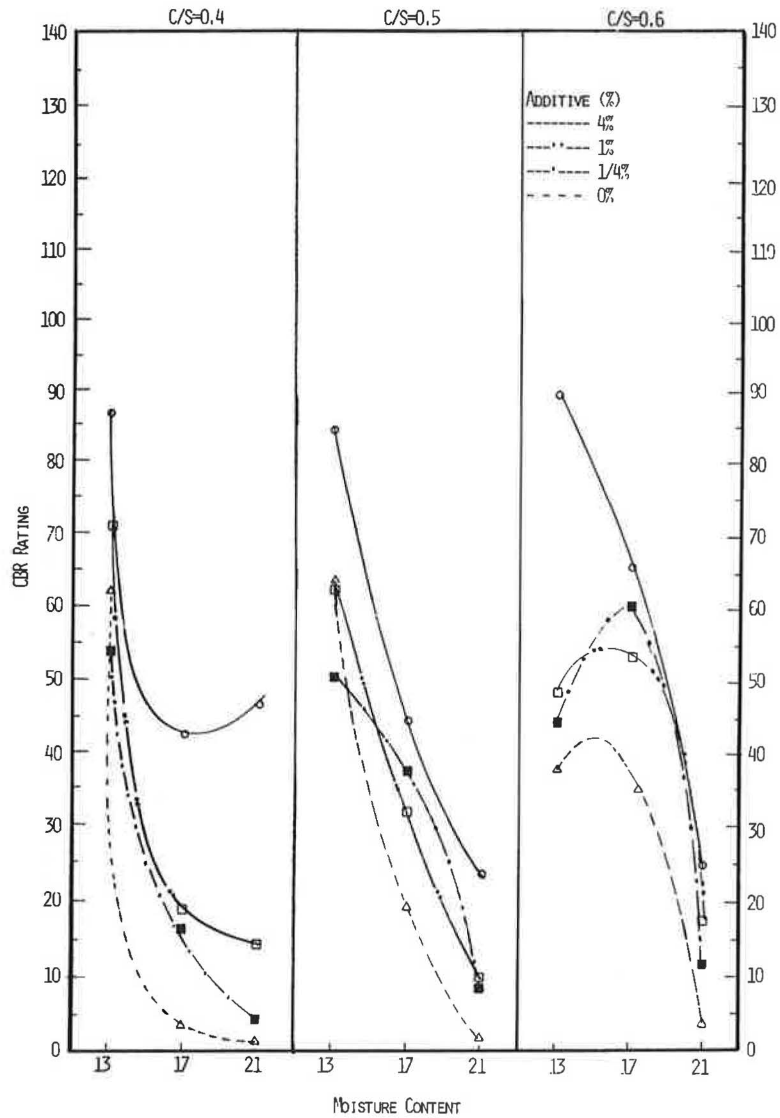

FIGURE 4 CBR versus moisture content by additive at three levels of clay-silt ratio (curing: 3 days at 40°F).

clay-silt soil lead to increased cohesive strength and an overall increase in the shear strength and consequently the CBR. This result is consistent with Coulomb's law of shear resistance.

The marginal increase in the clay-silt CBR values caused by a 1 percent increase in additive content was 11.07 units. The temperature component of the CBR model has a positive coefficient that supports the hypothesis that increased temperature leads to increased strength formation with the use of the epoxy resin.

A nomograph for clay-silt soil system CBR prediction using epoxy resin as the stabilizing agent is shown in Figure 7. The chart is used by first connecting the molding temperature and clay-silt ratio scale values with a straight line that cuts the auxiliary line F1. The intersection with F1 is next connected by a straight line to the molding (optimum) moisture content scale values to yield an intersection of F2 that is finally connected to the additive percentage scale value to yield the predicted CBR.

An alternative model was specified using a two-step regression method as suggested by Edris and Lytton (*9*). In the first step, a linear multiple regression analysis of the logarithms of the dependent and independent variables is obtained. Taking the antilog of the coefficients specifies the best unbiased linear estimator of the powers of the independent variables. For the second step, a linear multiple regression of the independent variables raised to the powers determined in the first step is performed. The appeal of this approach is that there is no predetermined polynomial form, rational function, or power law expression for the model, and it is reported that this method produces a consistently higher coefficient of determination (R^2) (*7*).

The results obtained from the two-step procedure follow.

Step 1:

$$\text{Ln CBR} = 16.102 + 0.040 \text{ Ln PA} - 4.257 \text{ Ln PM} + 1.511 \text{ Ln CS} + 0.139 \text{ Ln TEMP}$$

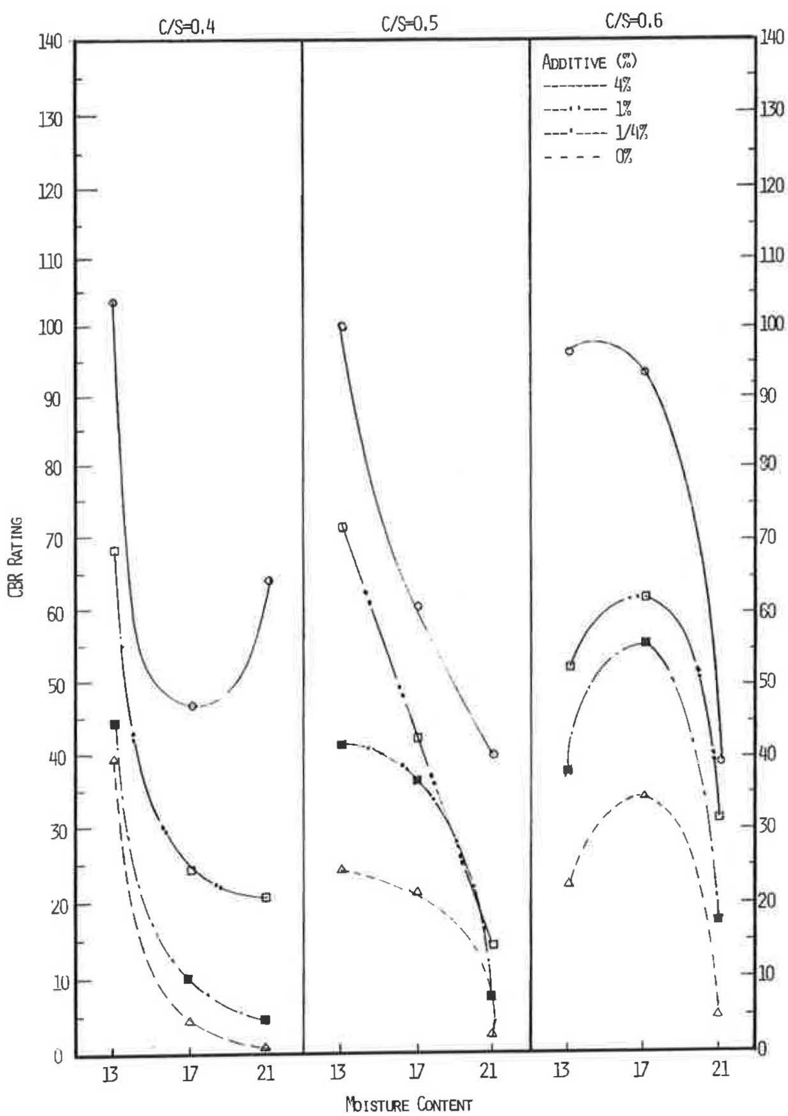

FIGURE 5 CBR versus moisture content by additive at three levels of clay-silt ratio (curing: 3 days at 65°F).

$(R^2 = 0.71)$ (2)

Step 2:

$$\text{CBR} = 6708.25 + 10.58(\text{PA})^{1.041} - 6431.74(\text{PM})^{0.0142} + 106.80(\text{CS})^{4.532} + 0.06(\text{TEMP})^{1.149}$$

$(R^2 = 0.77)$ (3)

A casewise multiple linear regression analysis of the data at fixed levels of selected independent variables was performed. The results for fixed levels of additive, clay-silt ratio, and temperature are as follows:

Fixed additive level:

- Additive level = 4 percent:

$$\text{CBR} = 160.87 - 7.47(\text{PM}) + 0.57(\text{TEMP})$$

$(R^2 = 0.75)$ (4)

- Additive level = 1 percent:

$\text{CBR} = 125.78 - 4.95(\text{PM})$ $(R^2 = 0.61)$ (5)

- Additive level = ¼ percent:

$\text{CBR} = 117.85 - 5.10(\text{PM})$ $(R^2 = 0.56)$ (6)

- Additive level = 0 percent:

$$\text{CBR} = 90.50 - 5.43(\text{PM}) + 56.00(\text{CS})$$

$(R^2 = 0.70)$ (7)

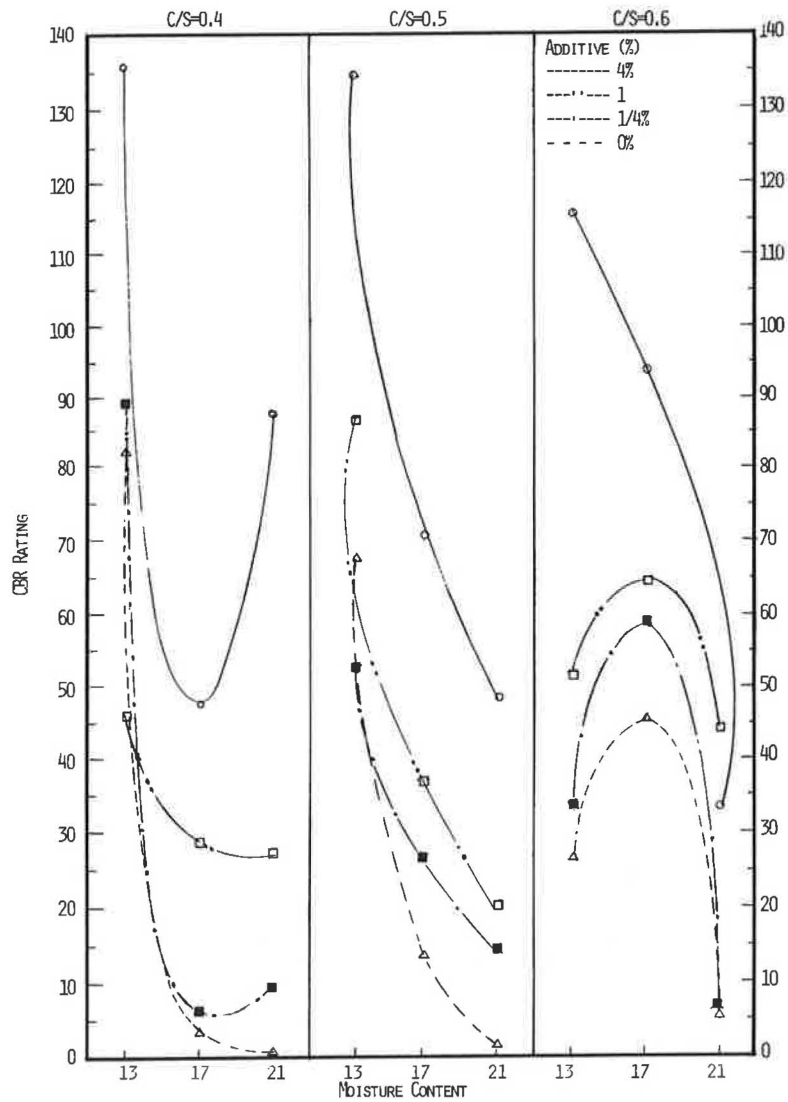

FIGURE 6 CBR versus moisture content by additive at three levels of clay-silt ratio (curing: 3 days at 90°F).

Fixed clay-silt ratio:

- Clay-silt ratio = 40 percent:

$$CBR = -423.20 + 13.38(PA) + 516.62(PM) + 0.39(PM)^2 + 0.19(TEMP)$$

$(R^2 = 0.88)$ (8)

- Clay-silt ratio = 50 percent:

$$CBR = -75.7 + 10.10(PA) + 1693.44(PM)$$

$(R^2 = 0.85)$ (9)

- Clay-silt ratio = 60 percent:

$$CBR = 233.09 + 10.16(PA) - 1785.80(PM) - 0.30(PM)^2$$

$(R^2 = 0.79)$ (10)

Fixed temperature:

- Temperature = 40°F:

$$CBR = -101.37 + 7.10(PA) + 1684.45(PM) + 0.54(CS)$$

$(R^2 = 0.83)$ (11)

- Temperature = 65°F:

$$CBR = -81.33 + 12.33(PA) + 1330.43(PM) + 0.48(CS)$$

$(R^2 = 0.81)$ (12)

- Temperature = 90°F:

$$CBR = -74.60 + 14.37(PA) + 1683.76(PM)$$

$(R^2 = 0.76)$ (13)

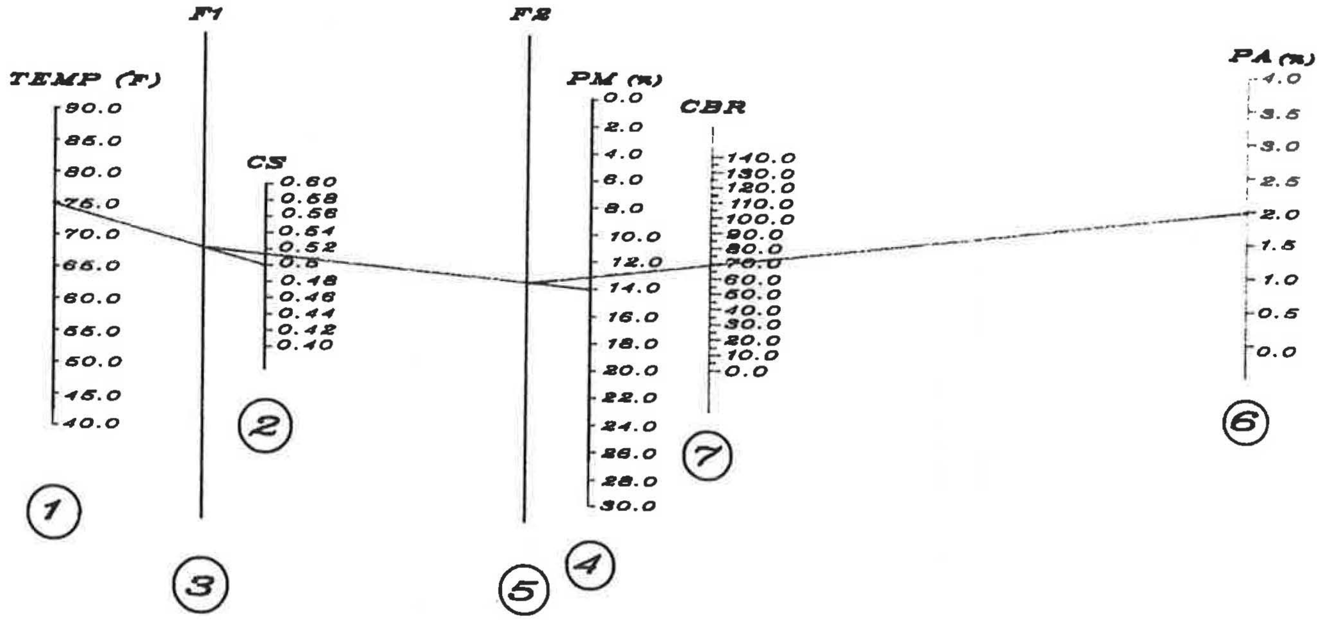

FIGURE 7 Nomograph for clay-silt system CBR prediction using epoxy-resin stabilizing agent (see Equation 1).

DISCUSSION OF RESULTS

In problem airport sites where an exclusive preponderance of clay and silt exists, the use of epoxy resin as a stabilizing agent has been demonstrated as a viable method of improving the bearing strength of clay-silt mixtures. This assertion is based on the analysis of experimental data obtained through the stabilization of a clay and silt mixture using a two-part epoxy system. The first part is a bisphenol A/epichlorohydrin resin and the other a polyamide hardener.

The CBR values obtained from the experimental test runs were either at 1 or 3 days after thermal soaking in a test chamber regulated to 40°F, 65°F, or 90°F.

STABILIZED CLAY-SILT SYSTEM RESULTS

The design of airport pavements depends on several important inputs, one of which is the strength of the subgrade. This parameter, in turn, depends both on the native moisture content and soil density, and, in cases involving chemical stabilization, the additive properties. Usually, the soaked CBR test is used in determining the subgrade strength. This represhents the worst-possible field condition; however, in cases where factors such as ambient moisture content and compaction characteristics are known with certainty, use of the unsoaked CBR strength is justified and was used in this study.

Plots of CBR value versus moisture content by clay-silt ratio stratification are shown in Figures 4–6 for all tested temperatures. The general similarity of the plots attests to the internal validity of the generated data. The variations in the graphs indicate the effects on the CBR of the variables whose changes are being stdied. In particular, the plots in Figures 4–6 indicate that in the presence of epoxy resin as additive for the clay-silt system the CBR value increases with temperature from 40°F to 90°F.

The increase in CBR value with temperature is explained partly by the increased physical bonding between the clay-silt particles made possible by the epoxy resin. The strength gain is also explained by the accelerated curing of the stabilized clay-silt mixture in response to increased curing temperatures. The implication is that temperature loss will cause a decrease in CBR value.

Figures 4–6 indicate that for a 4 percent epoxy resin application at 40°F the range of CBR improvement over the unstabilized clay-silt soil case varies approximately from 10 to 60 units. At 90°F, the strength increase obtained over the unstabilized cause from application of 4 percent epoxy varies from approximately 27 to 90 CBR units. At lower percentages of epoxy application, proportionately lower CBR increases are obtained. The plots reveal that at lower clay-silt ratios, the greater relative improvements in CBR are generally possible at higher moisture contents, whereas at higher clay-silt ratios, greater relative improvements occur at lower moisture content ranges.

Superimposing the effect of pore water pressure caused by increased moisture content on the preceding hypothesis explains the effectiveness of the epoxy resin both at higher moisture contents for low clay-silt ratios and at lower moisture contents for higher clay-silt ratios. The adsorption of water by the flat, clay particles of high surface area is smaller for samples with lower clay-silt ratios; consequently, the effect of pore water pressures on the molded samples is smaller, and relatively greater CBR improvements are possible. For samples with higher clay-silt ratios (0.6), the pronounced effect of pore water pressure leads to a further degradation of CBR over and above that resulting from the deemphasizing of stronger resin-silt particle bonds caused by an increased proportion of clay particles. The clay-silt ratio of 0.5 represents roughly the onset of the threshold of CBR improvement caused by varying levels of clay particles in the stabilized clay-silt mixture.

Explanation for the relatively better performance of CBR values at a lower clay-silt ratio (0.4) is that, because increases in clay content increases the clay skin around the larger silt particles leading to increased specific surface of the mixture, avenues for effective bonding between the silt particles and epoxy resin additive become increasingly limited in favor of the clay-silt physical bond enhanced by compaction. The relatively weaker clay-silt physical bond fails quickly under shearing forces imposed by the CBR plunger, leading to lower CBR test results compared to the situation in which a lower clay-silt ratio allows for the formation of only a lower degree of clay-silt bonds in favor of stronger bonds that lead to higher CBR values.

The use of multivariate analysis of variance technique (MANOVA) on the full factorial data in Table 2 supports the hypothesis that the increases in CBR values caused by epoxy-based treatment compared to the untreated soil CBR values are significant. These increases are statistically explained by the variables, additive percentage, and temperature and their interactions.

CONCLUSIONS

1. The effectiveness of a two-part epoxy system, bisphenol A/epichlorohydrin resin plus a polyamide hardener, in enhancing the soil bearing strength of clay-silt systems has been demonstrated against a baseline of an unstabilized clay-silt system (Table 2). The dry CBR test served as an evaluation tool for appraising the effectiveness of additive application. The dry test was chosen to enable the estimation of the effects of moisture content on the relative soil strength of a stabilized clay-silt system.

2. Within the ranges of the experimental variables and the data obtained in this research, the two-part epoxy system studied is effective for additive applications between ¼ and 4 percent at the least. The largest unsoaked CBR value obtained was 135 at the 4 percent level of epoxy application.

3. The nomograph for dry CBR value prediction developed from this research effort allows a quick estimate of the expected CBR for specified levels of soil stabilization parameters incorporated into the model. The marginal increases in CBR values caused by a 1 percent increase in epoxy resin application level, clay-silt ratio, temperature (°F), and moisture level are 11.1, 0.45, 0.14, and −5.6, respectively. The degradation of dry CBR value with increasing moisture content is consistent with engineering experience.

4. In this study, the influence of moisture content in degrading the CBR value has been found to depend on the curing condition, the ratio of clay to silt, and temperature. At the 4 percent level of resin application, a waterproofed stabilized soil system is not obtained; therefore, a limited number of wet CBR tests have been performed. These wet CBR tests confirmed that soaked CBR values ranging from 27 to 63 can be obtained even under 3 or 7 days of soaked test conditions. Above the 10 percent level of epoxy resin application, a waterproof system of stabilized clay-silt soil was obtained. Under unsoaked testing conditions, the onset of CBR degradation occurs at moisture levels of about 14, 17, and 18 percent for clay-silt ratos of 0.4, 0.5, and 0.6, respectively, on the basis of a cutoff dry CBR value of 40. In addition, the combined effects of clay-silt ratio and temperature were found to be significant in influencing dry density, and consequently CBR values. This result is confirmed by the work of Carpenter and Lytton (*8*) that explains the internal forces at work in clay-silt systems and the contraction and heave behavior of such samples under different temperature conditions.

5. On the basis of the cost of a laboratory validation experiment successfully conducted to verify the effectiveness of the chemical stabilization method outlined in this study, the cost of epoxy resin chemical stabilization may vary between one and one-half to about four times the cost (of material, labor, and overhead) per cubic yard of conventional stabilization (using materials such as cement). This cost does not take into account the pavement failure costs associated with construction on an unstabilized terrain. The higher estimate of relative cost factor, specified as four-fold, would probably be lower as the labor cost of construction increases, other factors remaining constant. Other factors that could conceivably affect the parameter include the prevailing clay-silt soil condition, equipment sophistication, labor efficiency, volume or scale of epoxy resin stabilization, epoxy resin demand and production economics, and other market forces.

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