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RESEARCH RECORD 839

Soil Stabilization 1982

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Authors of the Papers in This Record

Anderson, R. B., GAI Consultants, Inc., 570 Beatty Road, Monroeville, PA 15146
Armstrong, J. Clyde, Department of Civil Engineering, University of Texas at Arlington, Arlington, TX 76019
Barenberg, Ernest J., Department of Civil Engineering, 1110 Newmark CE Lab., 208 North Romine, University of Illinois at Urbana-Champaign, Urbana, IL 61801
Chang, Ta-Teh (Dave), School of Engineering, Oklahoma State University, Stillwater, OK 74078
Ciolko, A. T., Construction Technology Laboratories, Portland Cement Association, 5420 Old Orchard Road, Skokie, IL 60077
Ford, Charles M., U.S. Army Corps of Engineers, 12652 Castil Court, Woodbridge, VA 22192; formerly with Auburn University
Hajek, Benjamin F., Department of Agronomy and Soils, Auburn University, Auburn, AL 36849
Head, W. J., Department of Civil Engineering, West Virginia University, Morgantown, WV 26506
Litton, L. L., Engineering Research Institute, Iowa State University, Ames, IA 50011
Lockett, Larry W., State of Alabama Highway Department, 11 South Union Street, Montgomery, AL 36130; formerly with the Department of Civil Engineering, University of Alabama
Lohnes, R. A., Department of Civil Engineering, Iowa State University, Ames, IA 50011
McQuade, P. V., GAI Consultants, Inc., 570 Beatty Road, Monroeville, PA 15146
Moore, Raymond K., Department of Civil Engineering, Auburn University, Auburn, AL 36849
Nussbaum, P. J., Construction Technology Laboratories, Portland Cement Association, 5420 Old Orchard Road, Skokie, IL 60077
Petry, Thomas M., Department of Civil Engineering, University of Texas at Arlington, Arlington, TX 76019
Tayabji, Shiraz D., Construction Technology Laboratories, Portland Cement Association, 5420 Old Orchard Road, Skokie, IL 60077
Thompson, Marshall R., Department of Civil Engineering, University of Illinois at Urbana-Champaign, Urbana IL 61801

Design, Construction, and Performance of Lime, Fly Ash, and Slag Pavement

ERNEST J. BARENBERG AND MARSHALL R. THOMPSON

A pavement (approximately 54 000 yd^2) that consists of a 10-in thick lime and fly ash aggregate base and a 3-in thick asphalt concrete surface was constructed in 1976. The pavement serves heavy coal trucks that haul to a power plant. Ten thousand tons of fly ash and 30 000 tons of slag were used in the construction of the project. A brief description of the materials and mixture design process, a discussion of the thickness design approaches used, a brief description of the construction operations and quality control procedures, a summary of results of postconstruction testing, and a description of the performance trends over the past five years are presented. The structural capacity of the pavement has not decreased since construction. Although some transverse and longitudinal cracking and very limited fatigue cracking have occurred, pavement performance as of the summer 1981 has been good.

A pavement (approximately 54 000 yd^2) that consists of a 10-in thick base of lime and fly ash aggregate and a 3-in thick asphalt concrete surface was constructed in 1976. The pavement serves the Central Illinois Public Service Company (CIPS) electric generating station, approximately 3 miles south of Coffeen, Illinois. The principal traffic on this pavement is multiple-unit trucks that carry coal to the plant or remove the bottom ash and fly ash from the plant to a dump site.

This paper presents a brief description of the materials and mixture design process, a discussion of the thickness design approaches used, a brief description of the construction operations and quality control procedures, a summary of results of postconstruction testing, and a description of the performance trends over the past five years.

MATERIALS

The pavement consisted of an asphalt concrete surface and a lime, fly ash, and slab base layer placed directly on the prepared subgrade.

Asphalt Concrete

The asphalt concrete surface layer met Illinois Department of Transportation (IDOT) specifications for a class 1 surface. A class 1 surface is a high-type asphalt concrete with a minimum Marshall stability of 1700 lb.

Base Material

The base material was a lime and fly ash aggregate (LFA) mixture blended in proportions to provide the densest possible mix for the materials used. The lime used in the LFA was a monohydrated, high-calcium lime supplied by the Mississippi Lime Company, St. Genevieve, Missouri. The fly ash was obtained from the Commonwealth Edison electric generating plant near Kincaid, Illinois. The plant burns crushed coal, and the fly ash is collected by using cyclone-type collectors. The fly ash was conditioned with approximately 20 percent moisture prior to stockpiling. The fly ash is of average quality with moderate pozzolanic reactivity.

The aggregate used in the mixture was a boiler bottom slag produced by quenching the utility plant bottom ash with water. The slag was produced by the CIPS electric generating plant near Coffeen, Illinois, and was taken randomly from the slag disposal area. A typical gradation of the slab is given in the table below.

Sieve	Percentage Passing
1/2 in	99.5
3/8 in	98.4
No. 4	91.1
No. 10	51.0
No. 20	9.1
No. 50	4.5

Subgrade

The subgrade soils are medium-to-heavy clays derived from weathered thin loess deposits over Illinoian till. The soils generally classify as CL or CH in the unified classification system. Typical soaked California bearing ratio (CBR) values are 3-5, and moduli of subgrade reaction values vary between approximately 75 and 100 lb/in^3.

Mixture Design

Criteria for LFA mix design are based on material durability and strength. To ensure good performance the material must be (a) durable, (b) placed with adequate thickness for its strength and anticipated traffic loads, and (c) constructed by using proper placement, compaction, and curing techniques.

LFA mixture design involves four major steps:

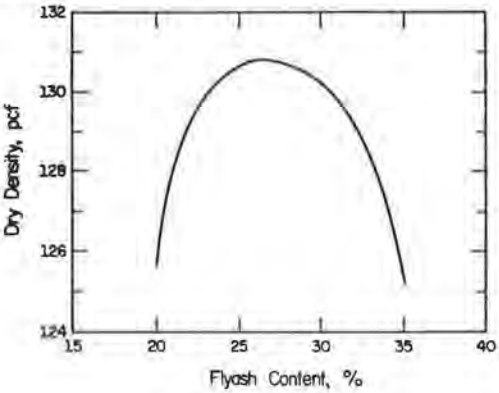
1. Ensure that the lime and fly ash will react to form the necessary cementitious bonds,
2. Ensure that the aggregate is sound and will bond to the lime fly-ash matrix produced by the reactions of the lime and fly ash,
3. Provide adequate fly ash to completely fill all voids in the aggregate, and
4. Provide an adequate amount of lime to produce the desired chemical reactions.

Experience with the Kincaid fly ash and the lime used indicated that these components were mutually reactive. Also, experience with slags produced from other wet-bottom-boiler installations indicated that the aggregate had the potential to produce a high-quality paving material. Thus, the mix design process was reduced to choosing the proper fly ash and lime contents.

The amount of fly ash required to produce a quality paving material is a function of the voids in the aggregate. To produce a quality LFA mixture, sufficient lime and fly ash matrix material must be in the mix to slightly overfill the voids in the aggregate.

The quantity of fly ash required to fill the voids is determined from the density of the compacted mix. Figure 1 shows that, as the quantity of fly ash in the mix is increased, the density of the mix first increases and then decreases. The fly ash content that produces the maximum density in the mix is the optimum fly ash content. This optimum corresponds to the point where all the voids in the aggregate are filled. Additional fly ash tends to separate the aggregate particles, thus reducing the overall density of the mixture. The optimum fly ash content (dry fly ash from the stack) was 27.5 percent.

Figure 1. Mixture-density and fly ash-content relations under AASHTO T-99 compaction.



Dry fly ash from the stack is very finely divided (more than 95 percent passing the No. 200 sieve). When conditioned with water, some fly ashes tend to set up and produce fly ash balls or lumps in the stockpile. It is the fine fraction (No. 200 sieve) of the fly ash that primarily reacts with the lime. Many of the fly ash balls are larger than some aggregate particles, thus the large fly ash agglomerations cannot serve either as filler for the voids or to react with the lime. Thus, it is necessary to either break up these agglomerations or to screen the fly ash and discard the coarse fraction.

Experience shows that quality LFA mixes can be produced if 100 percent of the fly ash passes the 3/8-in sieve and approximately 85 percent passes the No. 4 sieve. With this gradation control, however, some adjustments must be made in the fly ash quantities to compensate for the coarse fly ash agglomerations in the mix.

For this particular project, a screen was placed on the mixing plant fly ash hopper to eliminate the coarse fly ash agglomerations from the mix. Also the amount of fly ash in the mix was increased from the optimum content of approximately 27.5 percent to 32.5 percent to compensate for the nonuniform distribution of the fly ash throughout the mix, for changes in coarseness in the fly ash, and for nonuniform moisture distribution in the fly ash. The resulting mix was slightly fat with fly ash, which is the ideal design for achieving high mix quality.

Lime content in LFA mixes is based on the amount required to produce a mix of adequate strength and durability. In addition, experience shows that, for high-caliber mixing plants, there will be a variation in lime feed of about 0.25 percent, although less-efficient plants may have a variation in lime feed of up to 0.5 percent. Since a portable plant was to be set up just for this job (as contrasted with a permanent and continuously operating plant), it was determined that the minimum lime content for the mix should be around 3 percent by weight. A check of the mix by using the 3 percent lime content indicated that, after curing at 100°F for 7 days, the mix developed a strength of approximately 1800 lb•f/in². This strength is adequate for structural considerations and durability requirements. Comparable flexural strengths for this mix were between 325 and 350 lb•f/in² after 7 days at 100°F curing. Thus, the lime content for the mix was set at 3 percent by dry weight.

PAVEMENT DESIGN

Thickness design of pavements with LFA mixes can be accomplished by using the American Association of State Highway and Transportation Officials (AASHTO) procedures and assigning an equivalency value (A_2) to the LFA mix. The LFA pavement can also be considered as a slab and the Westergaard elastic slab theory or the Meyerhoff ultimate load theory used for design. For this project, the results from all three approaches are presented.

Traffic Projections

The principal traffic on this pavement was the construction traffic associated with the modification and expansion of the CIPS electric generating station, trucks that haul coal to the completed plant, and trucks that haul slag and fly ash from the plant to the disposal area. In addition there was to be some local farm-to-market traffic.

Projected average daily traffic over the life of this pavement was as follows:

Passenger cars	600
Single-unit trucks	350
Multiple-unit trucks	250

The terms single- and multiple-unit trucks are consistent with the IDOT method for computing a traffic factor (TF) [total equivalent 18-kip single-axle loads (ESAL)] based on axle load equivalencies. For class 3 roads in Illinois, assuming a 20-year analysis period, the IDOT procedure for calculating the traffic factor by using the above traffic data yields the following:

For rigid pavements,

$$TF = 20[(0.73 \times 600) + (22.445 \times 350) + (206.955 \times 250)] \times 10^{-6}$$
$$= 1.19 = 1.19 \times 10^6 \text{ 18-kip ESAL} \quad (1)$$

and for flexible pavements,

$$TF = 20(0.073 \times 600) + (17.885 \times 350) + (144.905 \times 250) \times 10^{-6}$$
$$= 0.8506 \text{ for flexible pavements} = 850\,600 \text{ 18-kip ESAL} \quad (2)$$

Typical Designs

With a TF of 0.85 for flexible pavements and an Illinois Bearing Ratio (IBR value is similar to CBR) of 5.0 (AASHTO soil support equal to 4.0-4.5), the required structural number value (from IDOT design nomograph) is 3.6. This value was used for developing comparative designs.

According to the IDOT design procedure for flexible pavements, the structural value for the pavement is given by the equation:

$$D_t = A_1D_1 + A_2D_2 + A_3D_3 \quad (3)$$

where D_1, D_2, and D_3 are the thicknesses in inches of the surface, base, and subbase layers, respectively, and A_1, A_2, and A_3 are coefficients that are a function of material type and properties.

Based on the data reported earlier (compressive strengths well in excess of 1000 lb•f/in² after 7 days of curing at 100°F), the proposed LFA mix has the potential to be above average in quality. The material coefficient value (A_2) assigned by IDOT for LFA mixes of this quality is 0.28. Assuming the high-quality class 1 asphalt concrete has A_1 coefficient value of 0.40, the structural number for the typical pavement section selected is as follows:

3-in AC surface at 0.40 = 1.20
10-in LFA base at 0.28 = 2.80
SN for entire section = 4.00

The SN value of 4.0 compares with a required SN

value of 3.6 for a subgrade with an IBR of 5.0, and a required SN value of 3.8 for a subgrade IBR of 4.0. Thus, even if the subgrade was weaker than expected, the design section with a SN of 4.0 should be adequate to carry the design traffic.

Based on the laboratory test results and assuming a reduction factor of 0.6 for going from the laboratory to the field, the LFA material used for the design can be assumed to have a compressive strength in excess of 1000 lb•f/in² (1800 x 0.6 = 1080), and a corresponding flexural strength greater than 200 lb•f/in². Preliminary mix design data indicated that 28-day compressive strengths of 1200-1500 lb•f/in² under summer field conditions are not unrealistic with corresponding flexural strengths in the range of 250-300 lb•f/in².

The pavement was analyzed by using the Westergaard model for flexural stresses and fatigue failure in the LFA base and for ultimate load capacity by using the Meyerhof theory. Ahlberg and Barenberg previously recommended the use of a Meyerhof-based procedure (1).

The ultimate load capacity for the section with a 3-in surface and 10-in LFA base was used on a flexural strength of 200 lb•f/in² for the LFA material and was calculated for both interior and edge-loading conditions. Assuming a k for the subgrade of 100 lb/in³ and discounting the contribution of the 3-in asphalt concrete surface, the ultimate load capacities were 27 kips for edge loading and 40 kips for interior loading. Relating these values to 10-kip wheel loads (20 000-lb single-axle loads) gives factors of safety against failures of 2.7 for edge loading conditions and 4.0 for interior loading conditions.

Analysis of the 10-in thick section by using the Westergaard slab theory and a 10-kip wheel load, again assuming a subgrade k of 100 lb/in³, and ignoring the contribution of the 3-in surface layer, gave flexural stresses in the LFA base of 110 lb•f/in² for interior loading and 160 lb•f/in² for edge loading. The maximum stress ratio for interior load conditions is 110/220 = 0.55 and for edge loading conditions 160/200 = 0.80. Field experience with LFA materials indicates that, as a result of the normal strength gain characteristics of these materials with time, a pavement that has a stress ratio of less than 1.0 will generally not fail in fatigue.

The pavement is considered adequate by using all three thickness design criteria.

CONSTRUCTION AND QUALITY CONTROL

The LFA mix was blended in a mixing plant capable of producing up to 600 tons/h. The plant consisted of one feed hopper each for the slag and the fly-ash components, a major storage silo, and a smaller secondary feed silo for the lime, a storage tank for water, a continuous mixing, twin-shaft pugmill, and the necessary belts, gates, and controls to feed and control the amount of components that go into the pugmill.

The mixing plant was set up in the slag-disposal area for the CIPS generating station and the slag loaded directly from the disposal pile into the hopper with a front-end loader. No attempt was made to control or adjust the slag.

Conditioned fly ash from the Kincaid ash-disposal area was delivered and stockpiled at the mixing plant site. The fly ash was scalped on a 3/8-in screen and fed directly into the fly ash feed hopper. Moisture content of the fly ash varied from 9 to 12 percent.

Lime was delivered in pneumatic trucks on a daily basis as needed. The lime was transferred from the delivery trucks to the lime storage silo by using compressed air. The storage silo was large enough to hold one 20-ton truckload of lime.

The lime, fly ash, and aggregate components were fed from their respective hoppers through calibrated gates onto a main feeder belt and delivered into a continuous-flow pugmill. Enough water was added during pugging to bring the mix to its optimum moisture content (approximately 7.5 percent). The mix flowed through the pug into a surge hopper and was then loaded into open dump trucks.

The mix was delivered to the road site, spread with a dozer-mounted spreader box, and compacted with vibrating steel wheel rollers. The entire 10-in thickness of LFA mix was spread and compacted in a single lift. Curing was accomplished by using a bituminous prime coat or a sealer.

The asphalt concrete surface course was placed by using conventional paving procedures.

Quality control procedures included the collection and testing of grab samples of the mix as delivered to the road site and the conducting of in situ density tests on the compacted materials. Specific tests on the grab samples included compacting Proctor-sized specimens and measuring the strength of the mix after 7 days of curing at 100°F and conducting titration tests on fresh samples of the mix to determine its lime content.

The number of Proctor samples and titration tests per day varied from two to four depending on the amount of material produced. An attempt was made to get at least three samples each day regardless of production. Because of some problems with the mechanical reliability of the plant, this was not always possible.

Results from the quality control tests are presented in Figures 2 and 3. Figure 2 shows the daily average and range for lime contents in the mix determined from the titration tests. Note that, for the first three weeks, the lime contents were highly variable and somewhat below the design value of three percent. These data reflect the inexperience of the contractor with this type of construction, the poor mechanical condition of the plant, and improper calibration techniques. After September 1 the lime content control was considerably improved.

Figure 3 shows the daily ranges and averages for LFA compressive strengths. These data also reflect the poor quality and lack of uniformity during the first few weeks of construction. As with the lime content, the compressive strength data were much improved after September 1.

A regression analysis was made by relating compressive strength to mixture lime content. The regression equation for the relation is as follows:

$$S = 408 + 156L$$

S = Compressive strength for 7-day cure at 100°F (lb•f/in²)
L = Lime content (%)
R = 0.271, significant at $\alpha = 0.025$

From these data, the importance of good control on lime content is apparent.

POSTCONSTRUCTION TESTING

After construction was completed (October 1976), 4-in diameter cores were taken in late November 1976. Full-depth cores were recovered in all of the sample sites. Data from the core samples are summarized in the table below.

Resilient modulus for asphalt concrete at 76°F was 360 000 lb•f/in².

For the LFA mixture,

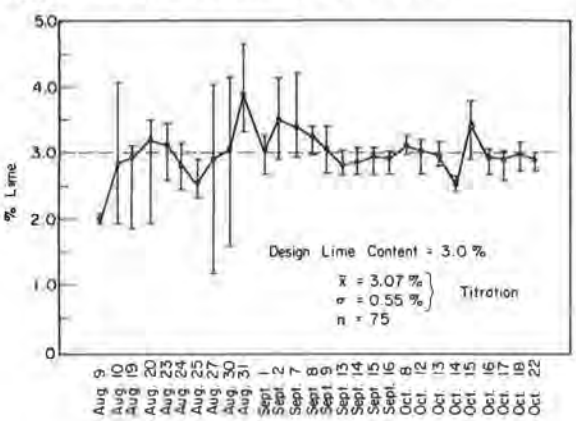
Item	Field Samples from Sept. 12, 1978 (lb·f/in²)
Unconfined compressive strength	1130
Split tensile strength	127
Resilient modulus	2×10^6

Postconstruction field cores in November 1976 were as follows:

Item	Age at Coring (days)	Compressive Strength (lb·f/in²)
Excessive	110	570
lumps in	110	740
fly ash	110	830
	110	1210
Avg		838
Fly ash	100	875
screened	96	3470
on 3/8-in	90	1230
sieve	48	1090
	40	738
Avg		1481

The pavement section described was subsequently selected for inclusion in the IDOT-University of Illinois IHR-508 flexible pavement research project (2). IHR-508 activities included Benkelman beam testing, IDOT road rater (8-kip peak-peak capacity) testing, and a soil and material sampling program. IHR-508 data were concentrated in a 100-ft long representative test section. A general full project length road rater (NDT) evaluation was conducted in April 1981.

Figure 2. Daily results from titration test for lime content.



Benkelman beam and road rater data are summarized in Table 1.

Soils and materials data were established for samples (bulk samples, cores, thin-walled tube subgrade samples) collected during September 1978 evaluation activities. Summaries of the data are presented in the preceding text table and below. A moisture-density-CBR plot for the subgrade is shown in Figure 4. Details of the testing procedures are presented elsewhere (2).

Gradation	Percentage
Sand, 2-0.05 mm	14.4
Silt, 0.05-0.002 mm	65.1
Clay, <0.002 mm	20.5

Liquid limit = 28.0
Plastic limit = 18.0
Plasticity index = 10.0
Unified classification = CL
Compaction characteristics (AASHTO T-99)
Maximum dry density = 102.2 lb/ft³
Optimum water content = 15.3 percent

PAVEMENT PERFORMANCE

The pavement described has been in service for nearly five years. As of the summer 1981, the pavement has carried nearly 1.75×10^5 tons of coal, more than 100 000 tons of top and bottom ash, and considerable local traffic. The only maintenance thus far has been the placement of a double surface treatment (slag cover aggregate) to improve the skid

Figure 3. Field compressive strength data.

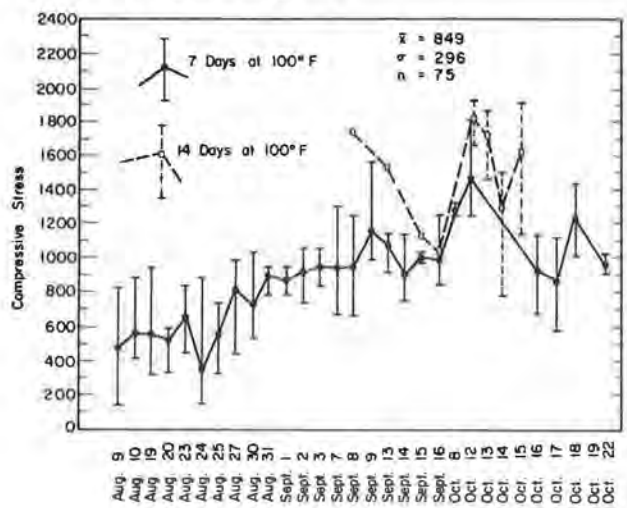


Table 1. Summary of deflection data.

Test Date	Asphalt Concrete Temperature (°F)	Benkelman Beam Δ (mils) Avg	Benkelman Beam Δ (mils) SD	Road Rater DO[a] Avg	Road Rater DO[a] SD	Road Rater Area[b] (in) Avg	Road Rater Area[b] (in) SD
5/23/78	93	12.1	2.2	10.7	0.98	29.4	0.96
9/12/78	85	14.5	3.2	9.5	0.70	28.8	0.88
10/26/78	50			7.2	1.33		
4/18/79	68			10.5	1.58	30.8	1.55
8/8/79	120			11.7	1.57	28.6	0.47
4/23/81				10.5	1.30	29.4	0.82
4/23/81				9.8	4.75	28.4	3.00

Note: Data are for the 100-ft long test section with the exception of the last data entry, which is for the entire project length.

[a]DO is the center of 12 in-diameter load plate deflection (8 kip peak-peak vibratory loading; 15 Hz).
[b]Area is a measure of the pavement surface deflection basin (2).

Figure 4. Moisture-density CBR relations for Coffeen subgrade soil.

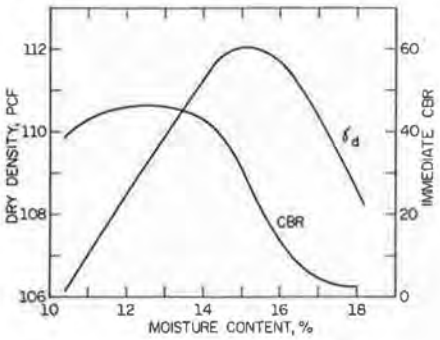


Figure 5. Typical transverse crack found at 30- to 40-ft intervals throughout the project.

resistance of the pavement.

The road rater data indicate that the structural capacity (maximum deflection and area) has not decreased over the 5-year life of the pavement. The pavement behavior is relatively insensitive to seasonal effects, as evidenced by the road rater data (Table 1) for several different periods of the year. Such behavior is typical of pavement sections that contain base courses that have a high rigidity.

The field coring data (September 12, 1978 information) indicate that the strength of the LFA mixture was greater than the project average construction compressive strength (Figure 3) of 849 $lb \cdot f/in^2$. There are no indications of any freeze-thaw durability distress in the LFA mixture as of that date.

Figure 6. Isolated longitudinal edge crack.

A close inspection of the project was conducted in the summer 1981. At that time transverse cracking had occurred; crack spacing ranged from approximately 50 to 150 ft. A typical transverse crack is shown in Figure 5. Most of the crack spacing intervals are greater than 75 ft. Limited longitudinal cracking has appeared in both the inner and outer wheel paths. A typical longitudinal crack is shown in Figure 6.

In isolated locations some fatigue (alligator cracking patterns) distress (shown in Figures 7 and 8) has occurred. Some of these areas have required surface patching and others are unmaintained. Studies to determine the factors that contribute to this distress are being planned. A likely cause for these disturbed areas is the significant variation observed in lime and fly ash mixture quality used (e.g., proportions and compaction). These variations were due in large part to inexperienced personnel of the contractor blending and placing the mix.

Asphalt concrete rut depths in the outer wheel path are approximately 0.45 in in the heavily loaded lane (entrance road to power plant) and 0.20 in in the less heavily loaded lane (exit from the plant area). Longitudinal roughness in the wheel paths is low and the overall ride quality is high.

Overall, the pavement has provided good performance. The county engineer has recently purchased a plant to set up in the slag disposal area to provide a constant supply of LFA mix to be used in future road construction in the county and adjacent counties.

SUMMARY

The design, construction, and performance of an

Figure 7. Areas of isolated alligator cracking.

Figure 8. Closeup of alligator cracking in isolated areas.

extensive (54 000 yd^2) lime and fly ash pavement project (10-in thick LFA base, a 3-in thick asphalt concrete surface) is documented. The structural capacity of the pavement has not decreased since construction. The pavement performance as of summer 1981 has been good. A major attribute of the LFA base course pavement was the extensive use of by-product materials (10 000 tons of fly ash, 30 000 tons of slag) in the project.

ACKNOWLEDGMENT

We wish to thank Terry Wells of CIPS and Anthony Georgeff, Superintendent of Highways for Montgomery County, for their assistance and support of this project.

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Evaluation of Heavily Loaded Cement-Stabilized Bases

S.D. TAYABJI, P. J. NUSSBAUM, AND A.T. CIOLKO

A field evaluation was carried out to determine performance of heavily loaded cement-stabilized bases. Ten projects, located in Oregon, Idaho, and British Columbia, were surveyed. Bases at six projects were used as log-sorting yards and at the other four projects as container-port storage areas. Stabilized base thickness ranged from 6 to 18 in. Cement content of the stabilized base generally varied from 5 to 8 percent. Log-sorting yards carried wheel loads that exceeded 80 kips. Wheel loads at container ports ranged from 10 to 25 kips. Performance was evaluated visually. Properties of base and subgrade materials were determined in the laboratory from samples obtained at each project site. Pavement analysis was conducted to determine stresses in the base. Also, required base thickness was computed for each site. Thickness was chosen to just sustain the estimated number of wheel loads up to the time of survey. It was found that base thickness computed from existing design procedures was generally more than as-constructed thickness. Since bases at all project sites are performing well, it is concluded that present design procedures for conventionally stabilized materials are conservative for heavily loaded high-quality cement-stabilized bases investigated in this study.

Since 1935 thousands of miles of cement-stabilized bases have been constructed. Extensive laboratory and field testing has been done on stabilized bases that meet criteria for soil-cement. These bases ranged in thicknesses from 5 to 9 in (1). Compressive strength was generally 400-500 $lb \cdot f/in^2$ (2-4).

Very little information has been reported for stabilized bases that have thicknesses of 12 in or greater and have compressive strength in excess of 1000 $lb \cdot f/in^2$. Information is available on limited full-scale traffic tests conducted by the U.S. Army Corps of Engineers on soil-cement pavements 21 and 25 in thick (5).

Use of high-quality thick-cement-stabilized bases has been increasing for heavily loaded facilities such as log-sorting yards, container ports, and log-haul roads. However, present methods for design of such pavements are based on extrapolations of results from laboratory testing and field evaluations of 5-9 in thick soil-cement pavements. To improve design of future heavily loaded high-quality cement-stabilized bases, a field evaluation of several such facilities was conducted. Information obtained included data on materials, design, construction, performance, and maintenance. This information was analyzed to determine the feasibility of using high-quality cement-stabilized bases for very heavily loaded roadways. The adequacy of existing thickness design procedures for such bases was also evaluated.

Table 1. Project features.

Location	Use	Thickness (in)		Base Cement Content (% by Weight)	Subgrade	
		Stabilized Base	Asphalt Surface		Type	CBR
Lynterm[a]	Container port	15.0	2.0	7	Gravelly sand	15
Vanterm[a]	Container port	18.0	1.5	8	Gravelly sand	15
Seaboard[a]	Container port	8.0	2.0	7	Gravelly sand	15
Frazer[a]	Container port	8.0	2.0	6	Sand	9
Caycuse[b]	Log-sorting yard	14.0	None	8-13	Clayey silt	6
Sweethome[c]	Log-sorting yard	12.0	3.0	5	Silty clay	2
Tomco[c]	Log-sorting yard	12.0	3.0	5	Sandy gravel	40
Bauman[c]	Log-sorting yard	12.0	3.5	5	Silty clay	5
Foster[c]	Log-sorting yard	12.0	5.0	5	Silty clay	4
Cascade[d]	Log-sorting yard	18.0	3.0	8	Gravelly sand	8

[a]Vancouver.
[b]Vancouver Island.
[c]Oregon.
[d]Idaho.

Figure 1. Representative stabilized base materials.

Idaho Oregon Vancouver

Table 2. Base material test data.

Location	Density (lb/ft^3)	Compressive Strength (lb·f/in^2)	Modulus of Rupture (lb·f/in^2)	Modulus of Elasticity (000 000s lb·f/in^2)
Lynterm	152	4690	630	4.2
Vanterm	148	2520	420	3.9
Seaboard	148	2120	380	2.0
Frazer	150	2900	450	2.5
Caycuse	149	4210	585	2.5
Sweethome	136	1600	320	1.3
Tomco	144	2420	484	0.7
Bauman	144	1690	338	0.6
Foster	144	2090	418	0.5
Cascade	130	1340	268	0.4

OBJECTIVE AND SCOPE

The objective of the project was to determine the feasibility of using high-quality cement-stabilized bases for facilities where very heavy loads would be applied for a limited number of applications. Objectives were accomplished by conducting field surveys of 10 facilities to obtain information on design, materials, cost, construction, loading, performance, and maintenance. Compressive strength and modulus of elasticity were determined from tests on cores taken at project sites. California bearing ratios (CBRs) were determined from tests on subgrade soils taken at project sites. An analysis was conducted to determine application of existing thickness design procedures to heavily loaded stabilized roadways.

FIELD INSPECTIONS

Field inspections were made at 10 facilities that have stabilized bases that carry heavy wheel loads.

Project Features

Project location, use, and design features are shown in Table 1. Projects were located in British Columbia, Oregon, and Idaho. Stabilized bases at these locations were used as storage areas for container ports or for log-sorting yards. The facilities were constructed between 1971 and 1980.

Lean concrete bases were used at projects in Oregon and Idaho. At Caycuse on Vancouver Island a zero-slump roller-compacted lean concrete was used. This high-quality base was selected to minimize operational surface abrasion. Excessive abrasion was anticipated at Caycuse due to operation of fork lifts that have prongs lowered for pushing logs across the yard.

Design features of thickness and cement content are listed in Table 1. Base thickness varied from 8 to 18 in. Where base thickness exceeded 12 in, base materials were placed in two or three lifts of about 6 in each. Asphalt-wearing surface thickness varied from 0 to 5 in. No asphalt was placed on the roller-compacted concrete used at Caycuse. Cement content generally varied from 5 to 8 percent. However, at Caycuse percentages as high as 13 percent were used.

Materials

Materials used at the surveyed facilities can be classified into three categories. At facilities located in the Vancouver area, roller-compacted concrete was used. At facilities in Oregon and also at some facilities in the Vancouver area, lean concrete made with crushed aggregate was used. Soil-cement bases made with gravelly sand were used in Idaho. Core specimens shown in Figure 1 illustrate texture and particle-size distribution of stabilized materials representative of those used at projects in Idaho, Oregon, and Vancouver. These specimens are from 4-in diameter cores taken at each project.

Cores were tested in the laboratory to determine compressive strength, modulus of elasticity, and density. Tests were made in accordance with ASTM C 42-77 and ASTM C 469-65. Test results are listed in Table 2. There was general correspondence between compressive strength, density, and modulus of elasticity. Largest values were obtained from Vancouver cores. The strength values are substantially higher than those usually obtained for conventional cement-stabilized materials.

Lengths of cores obtained from the Bauman project site were insufficient for determining modulus of elasticity. A modulus of elasticity value for Bauman bases was estimated based on the relation between compressive strength and modulus of elasticity obtained for cores from the other three Oregon projects.

Modulus of rupture values listed in Table 2 were computed from compressive strength. For the high-strength Vancouver bases, the relation between flexural and compressive strength for concrete shown in Neville's Figure 5.4 (6) was used to determine modulus of rupture values. Remaining values were computed by using the relation between compressive and flexural strength for soil-cement presented in Figure 14 of Felt and Abrams (7).

Subgrade samples obtained from each project were tested at Construction Technology Laboratories to determine CBRs. Tests were conducted in accordance with ASTM D 1883-73. Values obtained and soil identification are listed in Table 1.

Construction Cost Data

Costs associated with construction of stabilized bases were obtained during interviews conducted at each project site. Cost information is reported in the table below.

Location	Base Cost (cents/yd²/in of thickness)	Year Constructed
Lynterm	52	1977
Vanterm	40	1975
Seaboard	NA	1971
Frazer	NA	1972
Caycuse	58	1976
	67	1979
Sweethome	49	1976
Tomco	39	1976
Bauman	65	1980
Foster	54	1978
Cascade	38	1976

Cost varied from \$0.38 to \$0.67/yd²/in of thickness. No clear price trend is apparent because many interacting factors varied at each site. Some factors that influence cost are project size, project location, material availability, year constructed, cement content, site conditions, construction procedures, and equipment availability.

Construction

Construction equipment and procedures varied at each project. Procedures used with roller-compacted base at Caycuse are typical of projects in Vancouver. At Caycuse, a Barber-Greene continuous-flow plant was established at the aggregate stockpile about 2 miles from the log-sorting area. The mixture was produced at a rate of about 300 tons/h.

The mix was transported to the job site in dump trucks. There it was placed with a Barber-Greene SA-190 asphalt paver equipped with electronic grade control. The newly placed mix was then compacted by a Dynapac 25- to 30-ton self-propelled vibratory roller. About three passes were required to achieve specified density. A Hyster rubber-tired roller was used to tighten the surface.

At Tomco in Oregon, a central mix two-shaft pugmill plant with a vane-type cement feed was used. Belt scales measured and controlled aggregate and cement feed. Production rate of the plant was about 500 tons/h. The plant is shown in Figure 2.

Laydown equipment at Tomco was a Blaw Knox spreader on a 380 Michigan dozer. A vibratory single-drum roller with a load of 200 lb/linear in was used for compaction of 6-in-thick base lifts. About four coverages were required to achieve specified compaction. The blade of a CAT 14-E controlled from a string line was used for trimming.

Curing generally consisted of keeping exposed surfaces wet for seven days or until covered with additional lifts or surfacing. Sprinklers or water trucks were used for wetting.

Loads

Six of the projects surveyed carried wheel loads that exceeded 80 kip. These six were all log-sorting yards. A tire of the type used on log-handling equipment is shown in Figure 3. This tire is capable of carrying a load of 100 kip. At container ports, wheel loads ranged from 10 to 25 kip.

Loading details for all projects are given in Table 3. The number of axle applications listed in Table 3 was estimated from average daily operations, working days per year, and pavement age.

Performance

A subjective rating of performance was made based on visual observations of cracking, rutting, and repairs. Data obtained from performance observations are listed in Table 4. This table shows that only limited cracking, rutting, or patching were observed. All projects, with the exception of Caycuse, had an asphalt surface. Therefore, extent of base cracking is not known. However, extensive cracking that would have resulted in loss of base support would have been apparent at the asphalt surface.

Figure 2. Batch and mix plant for cement-stabilized base materials.

Figure 3. Tire for log-handling equipment.

Table 3. Loads.

Location	Axle Load (kip)	Wheel Load (kip)	Load Configuration (in)			Tire		Estimated Axle Applications (000s)
			Between Axles	Wheel Base	Between Duals	Width (in)	Area (in^2)	
Lynterm	100	25	120	100	20	15	300	45
Vanterm	56	28	66	144		13	310	350
Seaboard	40	10	100	100	13	8	130	90
Frazer	40	10	100	100	13	8	130	24
Caycuse	200	100	360	183		30	888	90
Sweethome	162	81	276	127		16	742	30
Tomco	162	81	276	127		30	742	67
Bauman	162	81	276	127		30	742	10
Foster	162	81	276	127		30	742	36
Cascade	170	85	312	127		30	850	48

Table 4. Performance indicators.

Location	Pavement Rating	Cracking Extent (ft/100 ft^2)	Crack		Rutting (in)	Patching (% of Area)
			Width (in)	Spacing (ft)		
Lynterm	5.0	0.01	3/16	250	None	0.01
Vanterm	5.0	0.8	1/8-3/16	100	None	None
Seaboard	3.5	0.2	1/8-3/16	Random	3/4	0.3
Frazer	5.0	0.01	1/8-1/4	500	None	None
Caycuse	4.0	1.0	1/2	120	1/4	0.3
Sweethome	4.3		None		None	1.0
Tomco	3.8		None		None	0.25
Bauman	4.5	None	None		None	None
Foster	5.0	None	None		None	None
Cascade	5.0	1.7	3/16-1/2	60	None	None

Table 5. Maintenance.

Location	Age (years)	Repair Activity	Cost ($)	Comments
Lynterm	3.5	Patching	None	Patching at time of construction
Vanterm	5.0	None	None	
Seaboard	12.0	Additional asphalt layers near dock	3500/year	To correct settlement due to subgrade subsidence
Frazer	8.0	None	None	
Caycuse	4.0		1700 First year 5200 Second year	To replace areas damaged by fork lift prongs
Sweethome	4.0	Asphalt resurfacing	2000 Total	
Tomco	4.5	Asphalt patching	1000 Total	
Bauman	0.5	None	None	1980 construction
Foster	2.0	None	None	
Cascade	4.0	None	None	

Performance was rated on a scale of 0-5, where 4-5 is very good, 3-4 is good, and 2-3 is fair. Eight of the 10 projects were rated very good. The lowest rating of 3.5 was assigned to the pavement at Seaboard, where 0.75-in-deep rutting was observed in a localized area of subgrade subsidence.

Maintenance

Maintenance activities and costs for each project are listed in Table 5. A regular maintenance budget is established at Seaboard. Other operators indicated there has been no need to allocate maintenance funds in yearly budgets.

ANALYSIS

A thickness design procedure was used to determine required base thickness for each project surveyed. Rated wheel loads obtained from manufacturers' literature and estimated number of load applications to the time of the field survey were used for computation. Thus, theoretical base thickness is the thickness that would just sustain the estimated number of load applications. Theoretically, any more applications would result in base failure.

The design procedure used was based on concepts given by the Portland Cement Association (1). In the procedure that was used, maximum flexural stress for a given base thickness is computed for placement of interior wheel loads. A fatigue curve is used in conjunction with modulus of rupture values to determine allowable load applications. The thickness that results in allowable load applications equal to the estimated load applications is the required pavement thickness. At projects where an asphalt surface exists, base thickness was obtained by subtracting an equivalent asphalt surface thickness from the required pavement thickness. It was assumed that 1 in of asphalt surface is equivalent to 0.5 in of stabilized base.

Fatigue models are not available for good-quality stabilized base material subjected to a limited number of heavy loads. Data are available for concrete or for soil-cement material subjected to large numbers of low-magnitude load applications. Because of the high compressive-strength values obtained from cores at the sites investigated, the fatigue curve for concrete was used for design computations.

Theoretical required base thicknesses are plotted versus as-built thicknesses in Figure 4. For 8 of

Figure 4. Theoretical versus actual base thickness.

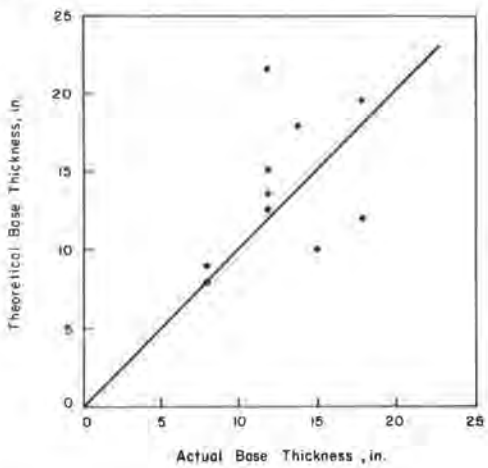

Figure 5. Thickness versus modulus of elasticity.

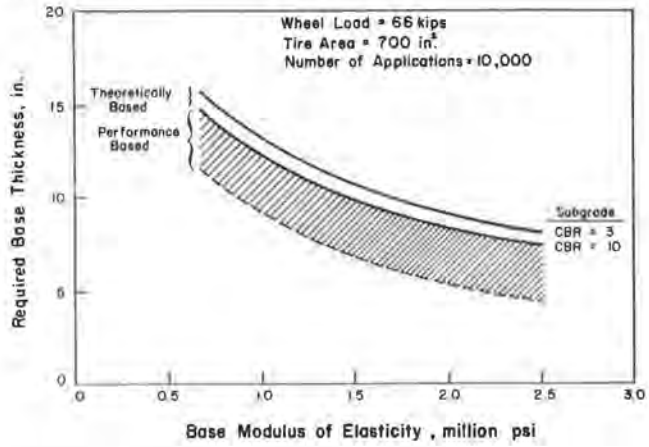

Table 6. Analysis results.

Location	Wheel Load (kip)	Base Thickness (in)	Subgrade Modulus (lb/in³)	Maximum Deflection (in)	Maximum Stress (lb·f/in²)
Lynterm	25	15	220	0.015	237
Vanterm	28	18	220	0.007	103
Seaboard	10	8	220	0.020	359
Frazer	10	8	180	0.020	387
Caycuse	100	14	160	0.037	450
Sweethome	81	12	80	0.076	505
Tomco	81	12	420	0.037	330
Bauman	81	12	140	0.073	397
Foster	81	12	130	0.082	384
Cascade	85	18	160	0.051	205

the 10 projects, as-built base thickness was equal to or less than the required computed thickness. However, stabilized bases at all surveyed sites were performing well. Computed required thickness values obtained by using a fatigue relation for soil-cement were even larger than those obtained by using a fatigue curve for concrete. Therefore, present thickness design procedures can be considered very conservative for the type of conditions encountered at the sites investigated. These conditions included heavy loadings, generally less than 100 000 load applications, and good-quality stabilized base materials.

Analysis was also conducted to determine maximum flexural stress and deflection at each project site. A finite element computer program for analysis of slab on elastic foundation was used. Pavement details, material properties, and rated single axle load placed at an interior location were used as input. Results are presented in Table 6. Maximum computed pavement deflections range from 0.007 to 0.082 in and maximum flexural stress in the base ranged from 103 to 505 lb·f/in².

CONCLUSIONS

Survey and analysis results show that high-quality cement-stabilized bases perform well under very heavy loading. Results also indicate that present design procedures for high-quality cement-stabilized bases are very conservative. For example, theoretical design considerations would indicate that 8 of the 10 surveyed pavements should have failed. However, even though 8 pavements are thinner than required by theory, their performance was rated good to very good.

Optimized thickness design procedures for high-quality cement-stabilized materials can be developed if realistic performance criteria and fatigue models are available. For example, for pavements, use of initial cracking as failure criteria for fatigue loading may be too severe. Pavements continue to perform satisfactorily even after cracks have appeared. Also, there is a need to investigate the fatigue behavior of high-quality cement-stabilized base material.

Development of an optimized design procedure could result in significant cost savings. An example is shown in Figure 5. The solid lines represent design thicknesses as a function of stabilized base modulus of elasticity. The two solid lines are for subgrades that have CBR values 3 and 10. Performance-based design thicknesses are within the shaded area below the solid lines. Thus, thickness reductions of up to 3 in may provide satisfactory performance.

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Coal Refuse and Fly Ash Compositions: Potential Highway Base-Course Materials

W.J. HEAD, P.V. McQUADE, AND R.B. ANDERSON

The necessity of using waste products in construction is becoming both evident and crucial as waste disposal continues to have a negative impact on the environment, disposal costs escalate, and traditional materials become scarce and expensive. Two by-products of the coal industry—coal refuse and fly ash—show promise for use in highway base-course applications. The already abundant supplies of these materials are expected to increase. Summarized in this paper are results of studies of the physical and engineering properties of both unstabilized and stabilized mixtures of coal refuse and fly ash. In addition, comparisons of performances of several hypothetical pavement systems are presented. The base courses of the pavements were either a crushed stone or coal refuse and fly ash mixtures. Findings indicate that stabilized coal refuse and fly ash mixtures are technically feasible base-course materials. In-service feasibility of the mixtures should be established by appropriate field testing. Unstabilized mixtures appear unsuitable for base-course applications because of questionable wet-dry and freeze-thaw durability. Both traditional substances and waste products should be considered as candidate construction materials. Technology for assessing competitive materials is available for many applications.

Waste use continues to concern engineers and others responsible for construction, environmental protection, and energy conservation. Waste products may be feasible alternatives to expensive or scarce conventional construction materials. Waste use obviates disposal problems. In addition, proper use of waste is an energy-conservation practice.

Two waste products that can be combined with appropriate stabilizing agents to yield potentially useful construction materials are coal refuse and fly ash. This paper deals with laboratory development and characterization of mixtures of these substances. Included are assessments of potential use of selected mixtures as highway base-course materials.

According to McQuade and others, (1, pp. 8-12):

> Coal mine refuse is usually comprised of clays, claystone, and/or shales which occur immediately above and below the coal or are interbedded in the coal seam itself. The exact nature of the refuse is a function of the geologic development of the coal seam....The automated [mining] equipment may extract portions of the mine floor and roof, in addition to interbedded impurities, with the coal. This results in the production of larger volumes of refuse material which are rejected in the [coal] cleaning process....Fly ash is a by-product of the coal combustion process. It is a very fine, light dust which is collected from stack gases....It is primarily comprised of rock detritus which collects in fissures of coal seams. The chemical composition of fly ash is highly variable....

Production of both coal refuse and fly ash exceeds use by huge margins. Refuse production is estimated to approach 200 000 000 tons annually (1, pp. 8-12). Most of the refuse is deposited in disposal sites. Annual production of fly ash approaches 50 000 000 tons. Approximately 8 400 000 tons were used in 1978 (2); excess ash remains in disposal sites.

The nature of both coal refuse and fly ash is complex; thus, general material characterization for design purposes is not possible. In addition, supply of these materials far exceeds demand. Consequently, significant use must at least accompany or indeed supplant disposal of the materials, given the finite extent of disposal sites and the general negative environmental impact associated with such sites. In light of these principles, the research reported here was undertaken for the following purposes: (a) to determine strength and durability characteristics of selected mixtures of coal refuse and fly ash, (b) to demonstrate that coal refuse and fly ash can be combined to yield potentially useful construction materials, and (c) to assess the feasibility of using mixtures of coal refuse and fly ash in pavement construction. The scope of the research effort was restricted to assessments of potential utility of selected mixtures as highway pavement base courses through the use of a computer-based pavement-performance-simulation program. Matters that deal with availability of refuse and ash and economic feasibility related to conventional construction materials were addressed in the research program and reported elsewhere (1, p. 8-12; 3); they are not considered here.

This paper is divided into four parts. A summary of the engineering characteristics of coal refuse and fly ash samples is presented followed by strength, durability, and environmental quality assessments of selected blends of coal refuse and fly ash. Next, the effects of various stabilizing agents on the strength and durability of the blends is presented. Finally, performances of four hypothetical highway pavements are compared with the aid of a pavement-performance-simulation program. The base courses of the pavements were a crushed stone aggregate, an unstabilized coal refuse and fly ash mixture, a lime-stabilized refuse and ash mixture, and a portland-cement-stabilized refuse and ash mixture. Base-course thicknesses, ambient temperatures, and subgrade support conditions were variables in the simulation program.

CHARACTERISTICS

Coal Refuse and Fly Ash Samples

Samples of coal refuse were obtained from 18 loca-

Table 1. Characteristics of coal refuse and fly ash samples.

Sample Identification	Classification	As-Received Moisture Content (%)	Atterberg Limits (%)			Specific Gravity	Max. Dry Unit Weight[a] (lb/ft^3)	Optimum Moisture Content (%)	Percentage Finer Than Sieve	
			w_l	w_p	PI				No. 4	No. 200
I-1a	GP	5.8	35.3	NP	NP	2.03	84.3	13.6	7	0
I-1b	GW	6.7	34.8	33.1	1.7	2.26	88.5	10.0	11	2
I-2	GP	8.1		NP	NP	2.27	97.0	15.0	49	3
II-1	GW	10.2		NP	NP	2.06	105.5	8.0	19	1
II-2	SC	8.5	34.9	24.7	10.2	2.58	112.1	12.0	52	27
II-3	GW	14.5	21.9	NP	NP	1.98	100.6	11.5	36	5
II-4	SP-SM	23.6	38.8	NP	NP	1.47	81.0	13.0	62	11
II-5	GW	2.8	25.5	NP	NP	1.73	99.3	7.6	14	3
II-6	GP-GM	7.7	25.2	20.9	4.3	2.34	120.0	8.8	32	8
II-7	GW	3.3	26.7	23.8	2.9	2.08	100.7	10.0	31	4
IV-1	GP-GM	8.8	20.8	NP	NP	2.15	114.6	8.8	33	7
V-1	SM	8.1	19.1	17.0	2.1	2.15	105.2	9.0	57	15
V-3	SM	13.4	27.5	21.6	5.9	2.07	97.8	13.4	88	19
VI-1	GW	2.5	26.8	21.5	5.3	2.46	112.4	10.3	25	3
VI-2	GW	7.7		NP	NP	1.72	86.5	8.2	32	2
VII-1	GP					2.47	100.9	4.3	6	0
IX-1	SM	14.5	49.2	33.1	16.1	2.60	108.2	14.7	52	32
IX-2	GC	7.6	32.3	22.7	9.6	2.51	113.4	13.0	37	12
FI-1	ML	0.2			NP	2.27	87	21.0	100	95
FII-1	ML	0.3			NP	2.34	85.0	23.5	100	85
FII-3	ML				NP	2.32	83.2	23.3	100	79
FII-6	ML	0.2			NP	2.23	69.3	29.0	100	60
FII-7	ML	22.6			NP	2.73	90.2	25.5	100	72
FIV-1	ML				NP	2.35	64.0	40.5	100	84
FV-1	ML	0.3			NP	2.25	87.0	20.0	100	84
FV-3	ML	4.3			NP	2.30	92.7	14.5	100	90
FVI-1	ML	0.3			NP	2.24	83.8	22.0	100	85
FVI-2	ML	0.1			NP	2.24	86.4	21.6	100	84
FVII-1	ML	0.1			NP	2.41	86.3	24.6	100	93
FIX-1	ML	4.9			NP	2.52	84.9	21.7	100	83
FIX-2	ML	0.6			NP	2.44	72.8	33.0	100	92

Note: NP = nonplastic.
[a]Standard Proctor compactive effort.

tions in the east, south, and midwestern portions of the United States. Both anthracite and bituminous coal refuse supplies were represented. Sample age ranged from several hours to approximately 80 years. Thirteen fly ash samples were obtained from power plants near the coal refuse sources. The bases on which the samples were selected and the sample identification scheme are presented elsewhere (1, pp. 8-12).

Classification and compaction tests were performed on each coal refuse and fly ash sample. The results are summarized in Table 1. These results and results of other constituent identification and classification tests indicated and confirmed the following:

1. Characteristics of both by-products were highly variable, particularly those of the coal refuse samples where texture and plasticity ranged from coarse to fine-grained and from nonplastic to plastic, respectively; and
2. Characteristics of all samples were generally representative of those reported in the literature for both by-products.

Results of the initial characterization tests served as aids in developing blends of coal refuse and fly ash and in interpreting results of tests performed on the mixtures in latter stages of the laboratory testing program.

Coal Refuse and Fly Ash Mixtures

Three types of laboratory tests were employed in the characterization and evaluation of mixtures of coal refuse and fly ash; these were gradation-compaction-strength tests, durability tests, and environmental quality tests. Selected test results appear in Tables 2 and 3. Complete results appear elsewhere (3). The following observations appear warranted.

Gradation

The mixtures tested did not experience extensive particle degradation after compaction; consequently, preand post-compaction gradations were essentially identical. Only minor particle breakdown was observed as result of increased compactive effort. Weak, frangible particles do not necessarily constitute coal refuse.

Compaction

Changes in fly ash content affect both maximum dry unit weights and optimum water contents of the mixtures. Increasing the fly ash content generally tends to increase the maximum dry unit weight, and excessive amounts of fly ash tend to decrease unit weight. Exceptions to these trends were noted.

Strength

Mixture strength, as reflected by California bearing ratio (CBR) and Hveem stabiliometer values, varies as a function of fly ash content; an optimum ash content was apparent for most blends. Maximum 0.1-in (2.5-mm) penetration CBR values varied considerably from about 11 to about 68. Significant changes in CBR accompanied small changes in ash content for most of the mixtures.

Durability

Wet and dry durability was assessed by submerging compacted samples in water at room temperature for 24 h followed by oven drying at 225°F for 24 h. The wetting-drying cycle was repeated and the specimens

Table 2. Characteristics of coal refuse and fly ash mixtures.

Sample Identification	Mixture Composition (% of dry weight)		Maximum Dry Unit Weight[a] (lb/ft³)	Optimum Water Content[a] (%)	CBR, 4-day soak[a]			Fineness Modulus[b]		Permeability[c] (cm/s)	Hveem Stability Resistance Value[d]
	Coal Refuse	Fly Ash			0.1-in Penetration	0.2-in Penetration	Swell (%)	Before Compaction	After Compaction		
I-2	100	0	97.0	15.0							
	90	10	108.8	9.7	2	3	2.8	4.3	4.1	3.8×10^{-6}	
	80	20	112.4	9.8	17	26	0.2	4.0	3.6	1.5×10^{-6}	
	70	30	113.2	11.1	7	9	1.1	3.0	3.1	2.5×10^{-6}	
	60	40	112.6	10.6	10	13	1.6	2.7	2.3		
	0	100	87.0	21.0							
II-1	100	0	105.5	8.0							
	90	10	103.2	9.2	42	48	0.5	4.8	4.0		63
	80	20	106.0	9.2	29	28	1.0	4.2	3.7		
	70	30	108.0	9.2	24	26	1.4	2.7	3.5		
	60	40	107.8	10.0	17	18	1.8	2.9	3.1		
	0	100	85.0	23.5							
II-5	100	0	99.3	7.6							
	90	10	110.3	8.5	50	63	0.5	5.4	5.1		
	80	20	109.0	10.0	49	61	0.4	4.7	4.8		
	70	30	108.1	9.4	51	60	0.7	4.5	4.4		
	60	40	104.5	10.5	29	37	0.3	3.6	4.0		
	0	100	83.2	23.3							
II-6	100	0	120.0	8.8							
	90	10	115.4	9.1	10	14	1.2	5.1	4.3		
	80	20	111.6	9.8	40	44	1.7	4.4	3.6		
	80	20	111.6	9.8	11[e]	14[e]	0.7[e]				
	80	20	118.6[f]	8.0[f]	60	52	0.5	3.8	3.8		
	80	20	118.6[f]	8.0[f]	37[e]	42[e]	0.5[e]				
	70	30	106.4	10.7	43	42	1.8	3.7	3.5		
	60	40	103.0	12.0	31	27	3.5	3.4	2.9		
	0	100	69.3	29.0							
II-7	100	0	100.7	10.9							
	90	10	100.0	10.8	31	35	1.5	4.9	4.7		
	80	20	100.7	12.5	18	39	0.6	4.3	4.0		
	70	30	101.2	12.0	31	36	1.3	4.2	4.4		
	60	40	100.4	13.2	40	40	0.7	3.6	3.6		
	0	100	90.2	25.5							
IV-1	100	0	114.6	8.8							
	90	10	110.1	9.2	40	53	0.4	5.1	4.2		
	80	20	105.3	11.9	54	56	0.3	4.2	3.8		78
	70	30	100.6	15.0	29	29	0.5	3.4	3.1		
	60	40	93.5	18.1	26	32	0.6	3.2	2.8		
	0	100	64.0	40.5							
V-1	100	0	105.2	9.0							
	90	10	110.4	8.5	13	18	0.05	4.4	3.8		
	80	20	110.2	8.6	10	15	0.04	3.9	3.7		
	70	30	108.5	8.5	15	21	0.06	3.5	3.1		
	60	40	105.8	9.9	25	26	0.7	2.8	2.9		65
	0	100									
V-3	100	0	87.8	13.4							
	90	10	110.0	9.3	5	7	1.3	4.7	4.1		
	80	20	110.4	9.4	7	11	1.7	4.2	3.7		
	70	30	109.8	9.7	11	12	1.5	4.0	3.2		
	60	40	109.6	9.4	6	8	1.4	3.2	2.5		
	0	100	92.7	14.5							
VI-1	100	0	112.4	10.3							
	90	10	101.1	8.1	22	27	0.3	5.1	4.7		
	80	20	106.8	7.9	46	41	0.2	4.6	4.4		
	70	30	105.9	9.1	29	34	0.5	3.8	4.1		
	60	40	102.0	10.1	24	28	0.7	3.3	3.7		
	0	100	83.8	22.0							
VII-1	100	0	100.9	4.3							
	90	10	117.4	6.1	18	24	0.1	5.8	5.4		
	80	20	115.6	9.3	43	52	0.2	5.4	4.8		
	70	30	119.9	10.6	42	53	0.1	4.8	4.5		
	60	40	126.4	7.5	68	62	0.5	3.9	3.8		
	0	100	86.3	24.6							
IX-1	100	0	108.2	14.7							
	90	10	105.0	17.5	6	5	2.5	4.8	4.9		
	80	20	107.8	13.2	9	10	4.3	4.5	4.7		
	70	30	107.0	12.8	12	15	4.7	4.0	4.2	1.4×10^{-6}	
	60	40	106.4	12.5	21	20	4.7	3.8	4.2		66
	60	40	118.6[f]	11.0[f]	40	32	2.2	3.8	3.8		
	60	40	118.6[f]	11.0[f]	16[e]	12[e]					
	0	100	84.9	21.7							

[a]Standard Proctor compactive effort.
[b]Cumulative percentage retained on standard sieve series.
[c]Falling head test.
[d]Exudation basis.
[e]Two wet-dry cycles followed by 4-day soak.
[f]Modified Proctor compactive effort.

Table 3. Results of leachate quality tests.

Sample Identification	Mixture Composition (% of dry weight)		pH			Element	Measured Concentration (ppm)	Max. Permissible Concentration[d] (ppm)
	Coal Refuse	Fly Ash	Initial[a]	Final[b]	Permissible Range[c]			
II-6	70	30	9.2	3.9	12.5-2.0	Al	0.352	
						As	0.007	5
						Ba	0.078	100
						Cd	<0.001	1
						Cr	0.042	5
						Cu	0.084	
						Fe	4.223	
						Mn	0.389	
						Mo	0.112	
						Ni	0.540	
						Pb	0.160	5
						Se	0.001	1
						Zn	0.161	
IV-1	80	20	5.9	3.9	12.5-2.0	Al	0.163	
						As	<0.001	5
						Ba	0.136	100
						Cd	0.012	1
						Cr	0.041	5
						Cu	0.067	
						Fe	16.466	
						Mn	0.717	
						Mo	1.258	
						Ni	1.640	
						Pb	0.160	5
						Se	<0.001	1
						Zn	0.283	

[a]After mixing with deionized water. [b]After 24-h extraction period. [c]EPA corrosivity criteria. [d]EPA hazardous waste toxicity criteria (100 times drinking water standards).

were then soaked in water for four days prior to CBR testing. Most of the specimens experienced large decreases in soaked CBR. These results bring the durability of unstabilized coal refuse-fly ash mixtures into question. Attempts were made to determine mixture freeze-thaw durability. Test results were inclusive.

Environmental Quality

Two blends were employed to evaluate the quality of leachate from coal refuse and fly ash mixtures. Leachate samples were obtained by means of the U.S. Environmental Protection Agency (EPA) toxicity test extraction procedure (4). Results of leachate quality tests indicated that both samples were environmentally acceptable construction materials with respect to current EPA corrosivity and toxicity criteria. The samples tested were unstabilized and in a loose state. We anticipated that concentrations of elements in leachate from compacted mixtures would be significantly reduced over those obtained by the EPA extraction procedure and that leachate volume would be small. The very low permeabilities measured in the laboratory and reported in Table 2 are noteworthy in this regard. The addition of stabilizing agents to the mixtures should also decrease both concentration levels and leachate volume. However, generalizations apropos of all refuse-ash mixtures are not possible based on the limited data.

STABILIZED COAL REFUSE-FLY ASH MIXTURES

The effects of various stabilizing agents on selected refuse-ash mixtures were investigated. Stabilizing agents included portland cement, hydrated lime, asphalt cement, and emulsified asphalt. Only the cement and lime stabilization efforts are summarized here; detailed results are reported elsewhere (3).

Ten mixtures were chosen for testing with cement and lime. Initially, refuse and ash proportions were selected that yielded the highest CBR value in the unstabilized mixture. Subsequently, the coal refuse content was held constant and the fly ash content was reduced as the amount of stabilizer increased. This procedure was followed to maintain a constant fines content and to maximize the refuse content.

Portland Cement Stabilization

Type 1 portland cement was added to the mixtures in amounts that ranged from 8 to 14 percent by dry weight of the mixture. Cement contents that exceeded 14 percent were judged to be uneconomical. The stabilized specimens were prepared and cured according to ASTM 192 C, in 4-in diameter compaction molds. The unconfined compression test was employed in evaluating the strength of the mixtures. An adequately stabilized specimen was assumed to be one whose 7-day strength was 400 $lb \cdot f/in^2$ (1, pp. 8-12). Additional strength tests were conducted at 14 and 28 days for those mixtures that satisfy the 7-day criterion. Vacuum saturation tests, conducted according to ASTM C593, were also performed on several of the specimens to aid assessments of mixture durability. Test results are summarized in Table 4.

Lime Stabilization

High-calcium-hydrated lime contents of 4, 6, 8, and 10 percent by dry weight of mixture were employed in evaluating the effects of lime on coal refuse and fly ash mixtures. Lime contents that exceeded 10 percent were not considered because they were judged to be uneconomical. The stabilized samples were prepared according to ASTM D698 C in 4-in diameter compaction molds and cured in sealed containers at 70°F for 28 days. Strength testing procedures were the same as those employed in the investigation of cement-stabilized mixtures. The strength criteria assumed for lime-stabilized blends was 400 $lb \cdot f/in^2$ after a 7-day accelerated or 28-day

standard cure and based on modified Proctor sample preparation (5). Samples tested in the program reported here were not prepared according to modified Proctor specifications; consequently, assessments of the efficacy of lime-stabilization based on the 400 lb•f/in² criteria are not possible. However, on the basis of additional tests not reported here, it is believed that several of the mixtures would satisfy the strength criteria if specimens were prepared with modified Proctor com-

Table 4. Characteristics of portland-cement-stabilized mixtures.

Sample Identification	Mixture Composition (% of dry weight)			Molding Water Content (%)	Dry Unit Weight (lb/ft³)	Specimen Age (days)	Unconfined Compressive Strength (lb/in²)
	Coal Refuse	Fly Ash	Portland Cement				
I-2	80	20	0	12.4	108.8	7	21
	80	12	8	13.4	110.0	7	404
	80	9	11	13.3	110.8	7	672
	80	6	14	13.1	108.2	7	809
	80	12	8	9.9	109.5	14	771
	80	12	8	8.9	108.5	28	900
I-2, vacuum saturated	80	12	8	11.2	107.4	7	421
	80	12	8	11.5	107.8	14	552
	80	12	8	10.9	106.9	28	611
II-5	70	30	0	9.5	10.38	7	27
	70	22	8	9.0	99.1	7	171
	70	19	11	8.2	99.9	7	267
	70	16	14	9.4	101.2	7	262
II-6	70	30	0	10.9	102.5	10	16
	70	22	8	10.8	105.4	10	342
	70	19	11	11.2	107.4	10	369
	70	16	14	10.5	108.6	10	515
	70	18	12	10.7	112.8	7	530
	70	18	12	10.2	111.6	14	599
	70	18	12	11.6	110.8	28	979
II-6, vacuum saturated	70	18	12	10.8	113.0	7	356
	70	18	12	8.4	115.2	14	493
	70	18	12	11.7	111.6	28	728
II-7	60	40	0	13.3	101.2	6	47
	60	32	8	12.0	100.2	6	351
	60	29	11	14.3	96.3	6	393
	60	26	14	14.2	99.9	6	592
	60	29	11		91.8	7	157
	60	29	11			14	196
	60	29	11		91.9	28	258
	60	28	12	8.7	96.5	7	230
	60	28	12	8.3	97.2	14	193
	60	28	12	8.7	96.5	28	275
II-7, vacuum saturated	60	29	11		91.2	7	157
	60	29	11		89.4	14	231
	60	29	11		90.5	28	241
	60	28	12	9.1	95.8	7	111
	60	28	12	8.2	96.2	14	142
	60	28	12	8.6	97.2	28	132
IV-1	80	20	0	16.7	98.4	7	10
	80	12	8	10.4	108.3	7	64
	80	9	11	12.0	105.1	7	215
	80	6	14	12.3	106.2	7	292
V-1	60	40	0	9.8	104.2	15	19
	60	32	8	9.7	105.3	7	429
	60	29	11	9.5	106.2	7	846
	60	32	8	9.6	105.8	15	624
	60	29	11	9.3	106.8	15	1044
	60	26	14	10.4	106.8	15	1053
	60	32	8	9.9	103.2	14	565
	60	32	8	10.2	103.8	28	789
V-1, vacuum saturated	60	32	8	10.7	103.3	7	448
	60	32	8	10.1	103.3	14	424
	60	32	8	10.6	104.4	28	540
V-3	70	30	0	10.5	100.2	7	41
	70	22	8	10.5	96.9	7	209
	70	19	11	10.3	107.8	7	258
	70	16	14	10.2	98.6	7	354
VI-1	80	20	0	9.6	102.6	7	23
	80	12	8	8.3	103.7	7	320
	80	9	11	7.0	107.4	7	374
	80	6	14	8.2	107.2	7	380
VII-1	60	40	0	7.3	111.4	7	15
	60	32	8	7.4	110.8	7	279
	60	29	11	8.9	110.2	7	294
	60	26	14	7.4	112.0	7	534
	60	28	12	7.9	113.4	7	599
	60	28	12	7.8	113.0	14	662
	60	28	12	5.7	116.2	28	898
VII-1, vacuum saturated	60	28	12	7.0	115.6	7	425
	60	28	12	6.6	114.0	14	478
	60	28	12	7.6	113.1	28	500
IX-1	60	40	0	13.4	96.1	7	28
	60	32	8	11.8	94.3	7	55
	60	29	11	12.2	94.9	7	78

pactive effort. Mixture strengths and other test results appear in Table 5.

Observations

The following observations appear warranted:

1. Unconfined compressive strengths of stabilized mixtures are functions of both the type and amount of stabilizing agent present.
2. Mixtures that respond favorably to stabilization with cement may not respond favorably to stabilization with lime; the converse is also true.
3. Results of vacuum saturation tests indicate that the durability of cement-stabilized mixtures I-2, II-6, V-1, and VII-1 may be satisfactory.

Results of tests not reported here where other stabilizing agents were employed indicate that certain coal refuse and fly ash mixtures are amenable to stabilization with asphalt cement or emulsified asphalt.

Table 5. Characteristics of lime-stabilized mixtures.

Sample Identification	Mixture Composition (% of dry weight)			Molding Water Content (%)	Dry Unit Weight (lb/ft³)	Unconfined Compressive Strength[a] (lb/in²)
	Coal Refuse	Fly Ash	Lime			
I-2	80	16	4	14.0	102.6	142
	80	14	6	13.2	101.8	117
	80	12	8	14.5	98.2	111
	80	10	10	13.5	96.7	112
II-5	70	26	4	10.9	96.6	62
	70	24	6	10.2	95.6	112
	70	22	8	12.3	92.7	73
	70	20	10	11.0	89.0	28
II-6	70	26	4	13.3	103.7	129
	70	24	6	11.0	104.4	100
	70	22	8	12.6	101.9	88
	70	20	10	9.4	104.2	95
II-7	60	34	4	15.8	91.7	104
	60	34	6	15.3	90.9	105
	60	32	8	15.0	88.2	85
	60	60	10	16.2	87.1	83
IV-1	80	16	4	10.4	96.8	141
	80	14	6	9.4	98.5	146
	80	12	8	8.7	96.5	111
	80	10	10	9.4	96.6	88
V-1	60	36	4	11.9	99.0	223
	60	34	6	12.4	98.9	205
	60	32	8	10.7	99.8	156
	60	30	10	11.1	97.8	149
V-3	70	26	4	11.2	93.2	161
	70	24	6	10.5	92.5	205
	70	22	8	10.8	90.0	209
	70	20	10	10.6	90.3	196
VI-1	80	16	4	7.9	103.3	104
	80	14	6	7.3	102.6	94
	80	12	8	8.2	95.9	71
	80	10	10	9.0	94.7	68
VII-1	60	36	4	6.9	107.3	168
	60	34	6	8.2	106.7	165
	60	32	8	7.0	104.6	119
	60	30	10	6.7	105.8	168
IX-1	60	36	4	13.8	114.6	222
	60	34	6	13.7	114.4	276
	60	32	8	14.8	113.2	225
	60	30	10	14.6	113.3	163

[a]28-day-old specimens.

Table 6. Temperature arrays in VESYS comparisons.

Month	Avg. Monthly Temperature		Month	Avg. Monthly Temperature	
	Vum	Mgt		Vum	Mgt
January	49.7	28.3	July	84.6	71.7
February	53.3	30.7	August	84.7	70.8
March	59.5	41.4	September	78.0	65.4
April	68.6	51.2	October	70.1	52.8
May	75.2	54.9	November	59.1	44.2
June	81.6	68.0	December	52.3	35.2

PAVEMENT PERFORMANCE

The theoretical performances of 160 highway pavements were compared. Coal refuse and fly ash mixtures, both unstabilized and stabilized, were the base-course materials for 92 of the pavements; a crushed stone material constituted the base courses for the remaining pavements. The comparisons were accomplished with the aid of the VESYS II M pavement-performance-simulation program. The VESYS II M program predicts the behavior of a three-layer flexible pavement as a function of time in terms of rutting depth, slope variance, cracked area, and serviceability index. Material characteristics of the combined asphalt surface course and binder, the base course, and the subgrade are required input information. Additional variables are average monthly temperatures of the environment, the traffic loading, and the pavement serviceability limit.

Procedures for obtaining predicted performance of the pavements and material characteristics are presented in detail elsewhere (3). In sum,

1. Material characteristics of the asphalt layer, the crushed stone base course, and the clay subgrade were taken from the VESYS user's manual (6).
2. Two subgrade conditions were considered: The dry condition refers to a stiff and essentially elastic subgrade and the wet condition refers to a weak, viscoelastic subgrade.
3. Two temperature arrays were considered: The first array, designated Vum, represented a relatively warm climate. Vum was taken from the user's manual (6). The second array, Mgt, represented average monthly temperatures for a cooler climate.
4. Both the pavement performance limits and the traffic conditions adopted in the comparisons were the same as those in the manual (6).
5. The coal refuse and fly ash mixture selected for laboratory characterization in unstabilized, cement-stabilized, and lime-stabilized forms was the 60-40 blend of material V-1. This blend represents a silty sand-sandy silt material that exhibited fairly low CBR and Hveem R values and responded reasonably well to both lime and cement stabilization. Material V-1 was judged neither the best nor the poorest of the blends tested.

Details of the temperature arrays appear in Table 6. Characteristics of the base-course materials that served as input for the VESYS II M program appear in Table 7. The definitions and instructions for calculating creep compliance and permanent deformation characteristics in Table 7 can be found in Kenis (6). Results of the pavement performance simulations appear in Table 8.

Limiting performance criteria were adopted as aids in comparing predicted pavement performance. The criteria were the same as those employed in the design example in the manual (6):

1. Maximum rut depth of 0.5 in in 20 years,
2. Maximum slope variance of 10^{-3} radians in 20 years,
3. Maximum cracked area of 500 yd²/1000 yd² of pavement surface in 20 years, and
4. Minimum present serviceability index of 2.5 after 20 years.

Pavement systems that have minimum layer thicknesses that satisfy the criteria appear in Table 9.

Table 7. Base-course characteristics for VESYS program.

Base-Course Material	Creep Compliance [(10^{-6} in/in)/(lb·f/in^2)]											Permanent Deformation Characteristics	
	Time When Creep Compliance Was Determined												
	0.001 s	0.003 s	0.010 s	0.030 s	0.10 s	0.30 s	1 s	3 s	10 s	30 s	100 s	Gnu	Alpha
Crushed stone[a]	17	17		17	17	17	17	17	17	17	17	0.055	0.730
Unstabilized CR-FA at indicated water content[b]													
w = 7 percent[c]	74	80.5	87.5	94	96	100	100	100	100	100	100	0.012	0.875
w = 10 percent[d]	67.5	72	75	77	79	81	83	83	83	83	83	0.006	0.668
w = 13 percent[e]	200	205	212	217	229	233	240	250	250	250	260	0.619	0.185
Portland-cement stabilized CR-FA[f]	2	2	2	2	2	2	2	2	2	2	2	0	1.0
Lime-stabilized CR-FA[g]	2	3	5	6	8	12.5	12.5	12.5	12.5	16	16	0.003	0.554

[a]Characteristics taken from example design problem in Kenis (6).
[b]Sample V-1, 60-40 percent mixture coal refuse-fly ash, percentage of dry weight.
[c]Mixture water content less than standard Proctor optimum.
[d]Mixture water content equal to standard Proctor optimum.
[e]Mixture water content greater than standard Proctor optimum.
[f]Sample V-1, 60-32-8 percent mixture coal refuse, fly ash, and cement, percentage of dry weight, 28-day cure time.
[g]Sample V-1, 60-36-4 percent mixture coal refuse, fly ash, and lime, percentage of dry weight, 28-day cure time.

Table 8. Selected values from the VESYS program simulation of pavement performance.

Base-Course Material	Temperature Array[a]	Subgrade Condition	Thickness of Layer (in)		Rut Depth (in)	Slope Variance (10^{-6} radians)	Cracking (yd^2/1000 yd^2 of surface)	Present Serviceability Index After 20 Years	Expected Pavement Life[b] (years)
			Upper	Base Course					
Crushed stone	Vum	Dry[c]	4	6	0.18	1.19	1000	3.84	>20
			4	8	0.19	1.18	1000	3.83	>20
			4	10	0.20	1.09	1000	3.86	>20
			4	14	0.21	0.98	1000	3.89	>20
			6	6	0.24	2.08	0	3.86	>20
			6	8	0.25	2.08	0	3.85	>20
			6	10	0.25	2.00	0	3.87	>20
			6	14	0.26	1.80	0	3.91	>20
			8	8	0.25	2.17	0	3.83	>20
			8	10	0.26	2.14	0	3.84	>20
			8	14	0.26	2.00	0	3.86	>20
	Mgt	Dry	4	10	0.15	0.64	969	4.07	>20
			4	14	0.16	0.58	949	4.09	>20
			6	6	0.17	0.98	0	4.24	>20
			6	8	0.17	0.99	0	4.23	>20
			6	10	0.17	0.96	0	4.24	>20
			6	14	0.18	0.87	0	4.27	>20
			8	8	0.16	0.90	0	4.27	>20
			8	10	0.16	0.89	0	4.27	>20
			8	14	0.17	0.84	0	4.29	>20
	Vum	Wet[d]	4	10	1.08	27.47	1000	<1	3.7
			4	14	0.84	15.50	1000	1.2	8
			6	10	1.04	29.48	1000	<1	3.5
			6	14	0.86	17.49	833	1.09	7.4
			8	10	0.95	26.71	0	<1	4.6
			8	14	0.82	17.77	0	1.47	7.7
			10	10	0.90	25.57	0	<1	5.1
			10	14	0.81	18.90	0	1.46	7.4
			10	24	0.63	9.78	0	2.34	17.1
			10	30	0.56	6.79	0	2.73	>20
			12	14	0.78	18.76	0	1.51	7.6
			12	24	0.64	10.58	0	2.27	15.7
	Mgt	Wet	4	8	0.87	19.67	1000	<1	6.6
			4	10	0.77	14.05	1000	1.46	9.4
			4	14	0.63	8.62	1000	2.13	14.7
			6	8	0.73	15.84	0	1.77	10.1
			6	10	0.68	12.60	0	2.06	13.1
			6	14	0.59	8.25	0	2.55	>20
			6	20	0.49	5.13	0	3.04	>20
			8	8	0.62	11.98	0	2.23	15.2
			8	10	0.58	10.11	0	2.42	18.4
			8	14	0.53	7.44	0	2.73	>20
			8	16	0.50	6.38	0	2.88	>20
Unstabilized coal refuse and fly ash at 7 percent water content[e]	Vum	Dry	8	12	>10	>10^5	0	<0	0.3
			8	18	>10	>10^5	0	<0	0.3
			8	36	>10	>10^5	0	<0	0.3
			12	12	>10	>10^5	0	<0	0.3
	Mgt	Dry	8	12	>10	>10^5	0	<0	0.3
			8	18	>10	>10^5	0	<0	0.3
			8	36	>10	>10^5	0	<0	0.3

Table 8. Continued.

Base-Course Material	Temperature Array[a]	Subgrade Condition	Thickness of Layer (in) Upper	Thickness of Layer (in) Base Course	Rut Depth (in)	Slope Variance (10^{-6} radians)	Cracking (yd^2/1000 yd^2 of surface)	Present Serviceability Index After 20 Years	Expected Pavement Life[b] (years)
Unstabilized coal refuse and fly ash at 7 percent water content[e]	Vum	Wet	8	24	>10	$>10^5$	745	<0	0.3
			8	36	>10	$>10^5$	287	<0	0.3
			12	12	>10	$>10^5$	0	<0	0.3
	Mgt	Wet	8	24	>10	$>10^5$	0	<0	0.3
			8	36	>10	$>10^5$	0	<0	0.3
Portland-cement-stabilized coal refuse and fly ash[e]	Vum	Dry	4	4	0.11	0.45	0	4.49	>20
			4	6	0.11	0.37	0	4.53	>20
			4	8	0.11	0.34	0	4.55	>20
			4	10	0.11	0.29	0	4.58	>20
			4	12	0.11	0.25	0	4.60	>20
	Mgt	Dry	4	4	0.10	0.34	0	4.56	>20
			4	6	0.09	0.28	0	4.59	>20
			4	8	0.09	0.21	0	4.64	>20
	Vum	Wet	6	6	0.77	15.34	0	1.72	9.6
			6	8	0.60	9.46	0	2.43	18.6
			6	10	0.50	6.69	0	3.29	>20
	Mgt	Wet	4	4	0.66	11.56	0	2.17	14.5
			4	6	0.48	6.24	0	2.93	>20
			4	8	0.38	4.44	0	3.29	>20
Lime-stabilized coal refuse and fly ash[e]	Vum	Dry	4	4	0.11	0.45	0	4.49	>20
			4	10	0.12	0.34	0	4.55	>20
			6	8	0.19	1.05	0	4.20	>20
			6	10	0.18	0.90	0	4.26	>20
			8	8	0.20	1.29	0	4.11	>20
			8	10	0.20	1.17	0	4.15	>20
	Mgt	Dry	4	4	0.09	0.25	0	4.61	>20
			4	10	0.07	0.12	0	4.70	>20
			6	8	0.11	0.37	0	4.53	>20
			6	10	0.11	0.30	0	4.58	>20
			8	8	0.12	0.42	0	4.50	>20
			8	10	0.11	0.36	0	4.54	>20
	Vum	Wet	4	8	0.92	25.47	0	<1	5.4
			4	10	0.75	19.09	0	1.58	8.4
			4	14	0.57	13.09	0	2.25	15.5
			4	20	0.48	10.36	0	2.59	>20
			6	8	0.90	21.15	0	1.12	6.3
			6	10	0.78	16.63	0	1.62	8.9
			6	14	0.62	11.97	0	2.23	15.2
			8	8	0.84	18.26	0	1.40	7.5
			8	10	0.75	14.52	0	1.79	10.4
			8	14	0.62	10.54	0	2.31	16.6
			8	20	0.49	7.53	0	2.79	>20
	Mgt	Wet	4	8	0.48	7.07	0	2.84	>20
			4	10	0.41	5.55	0	3.11	>20
			4	14	0.33	4.29	0	3.38	>20
			6	8	0.46	5.44	0	3.05	>20
			6	10	0.40	4.27	0	3.29	>20
			6	14	0.32	3.18	0	3.56	>20
			8	8	0.42	4.62	0	3.21	>20
			8	10	0.38	3.62	0	3.42	>20
			8	14	0.31	2.61	0	3.68	>20

[a]Arrays are found in Table 6. [b]Approximate time for serviceability to reach 2.5. [c]Clay subgrade, 16 percent water content. [d]Clay subgrade, 23 percent water content. [e]Defined in Table 7.

Unstabilized blends are absent from Table 9 because none of the analyzed pavement systems exhibited satisfactory performance when an unstabilized mixture was the base-course material, regardless of the temperature or subgrade conditions. Analyses of unstabilized blends at optimum and wet of optimum water contents were not conducted because those mixtures were even weaker than the unsatisfactory dry of optimum mixture.

The following observations appear warranted based on assessments of results of the VESYS pavement performance simulation:

1. Effects of changes in subgrade conditions and temperature array on predicted performance were as anticipated. Pavements that incorporate the dry (stiffer) subgrade consistently yielded lower rut depths, slope variances, and cracked areas and higher serviceability indices than did identical sections that incorporated the wet subgrade. In addition, the colder climate array resulted in more favorable performance than the warmer array for identical sections.

2. Layer thicknesses of minimum, satisfactory, stabilized-mixture base-course pavement systems are less than the thicknesses of corresponding layers of minimum systems where crushed stone material was the base course.

3. For systems of equal corresponding layer thicknesses, the performance of pavements with stabilized coal refuse-fly ash mixtures exceeds the performance of the crushed-stone base-course pavements.

4. The unstabilized mixtures analyzed herein are structurally unsuitable for highway base-course applications.

CONCLUSIONS

Major conclusions that emerged from this study were as follows:

1. Several of the unstabilized coal refuse and

Table 9. Minimum pavement systems that satisfy performance criteria.

Base-Course Material	Temperature Array[a]	Subgrade Condition[b]	Thickness of Layer (in)		Present Serviceability Index after 20 Years	Controlling Performance Criterion
			Upper	Base Course		
Crushed stone	Vum	Dry	6	6	3.86	Cracking
		Wet	10[c]	36[c]	2.88[c]	Rut depth
	Mgt	Dry	6	6	4.24	Cracking
		Wet	8	16	2.88	Rut depth
Portland-cement-stabilized coal refuse and fly ash[d]	Vum	Dry	4	4	4.49	Minimum thickness[e]
		Wet	6	10	2.85	Rut depth
	Mgt	Dry	4	4	4.56	Minumum thickness[e]
		Wet	4	6	2.93	Rut depth
Lime-stabilized coal refuse and fly ash[d]	Vum	Dry	4	4	4.49	Minimum thickness[e]
		Wet	8	20	2.79	Rut depth
	Mgt	Dry	4	4	4.61	Minimum thickness[e]
		Wet	4	8	2.84	Rut depth

[a]Arrays found in Table 6. [b]Defined in Table 8. [c]Values estimated from trends established in Table 8. [d]Defined in Table 7. [e]Minimum layer thickness considered was 4 in.

fly ash mixtures appeared to be feasible base-course candidate materials based on the results of CBR and Hveem stabilometer tests. However, results of laboratory durability tests suggest that the long-term durability of unstabilized mixtures is questionable.

2. Coal refuse and fly ash mixtures may be responsive to stabilization with one or more agents (e.g., portland cement, lime, asphalt cement, and emulsified asphalt). Consequently, use of stabilized mixtures in base-course applications appears technically feasible.

3. The likelihood of a serious negative environmental impact arising from stabilized mixtures in base-course applications is remote.

4. Hypothetical pavement systems that have cement and lime-stabilized coal refuse and fly ash mixture base courses yielded thinner surface and base course layers than pavements that have a crushed stone base for the same loading, temperature, and subgrade conditions. Conversely, for systems with equal thicknesses of corresponding layers, the systems that incorporated stabilized mixture base course exhibited better hypothetical performance than systems that incorporated the crushed stone base courses.

5. The VESYS II M pavement performance simulation program provides a rapid procedure for comparing large numbers of pavement systems.

6. Use of unstabilized coal refuse and fly ash mixtures as highway base-course materials is highly questionable.

7. In-service field testing of cement and lime-stabilized compositions should be accomplished to evaluate performance and long-term durability of the mixtures.

8. Waste products should be considered as candidate construction materials along with traditional materials. Current technology makes comparisons and assessments of competing materials possible.

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Lime-Soil Mixture Design Considerations for Soils of Southeastern United States

LARRY W. LOCKETT AND RAYMOND K. MOORE

The Thompson procedure for lime-soil mixture design should be modified when soils of the southeastern United States are evaluated. The dominance of montmorillonite in the clay fraction of some Southeastern clays, especially those of the Alabama and Mississippi blackbelt, creates the need for careful consideration of compaction moisture contents for the lime-treated specimens. The effect of lime modification on these clays causes the optimum moisture content to increase by as much as 20 points (based on the increase in plastic limit). Therefore, the lime-treated soil must be compacted at a higher moisture content than the untreated soil. Although the different moisture contents confound the comparisons of unconfined compressive strength, the potential for moisture deficiency in the lime-treated material must be eliminated. A comparison of plastic limits for the untreated and lime-treated soil will provide an indication that the lime-soil mixture design will require this modification. The use of a modified accelerated curing procedure is recommended for soils of the southeastern United States. Data developed in this research program indicate that the Thompson-accelerated curing criteria of 48 h at 120°F (49°C) overestimate the 28-day, 75°F (24°C) unconfined compressive strengths of lime-treated blackbelt soils by an average of 22 percent. A 65-h, 105°F (41°C) accelerated-curing sequence underestimates the 28-day, 75°F (24°C) unconfined compressive strengths by approximately 25 percent. We, therefore, recommend that a 72-h (3 days is more convenient for laboratory scheduling than 65 h) accelerated-curing sequence at 105°F (41°C) be employed when the Thompson procedure is used for Southeastern soils.

Two major considerations for lime-soil laboratory mixture design procedures involve selection of the specimen preparation procedure and the curing temperature and curing time regime to simulate field curing. The Thompson procedure of lime-soil mixture design (1,2) has been evaluated for its use with soils of the southeastern United States. The results of this study indicate that modifications of the original procedure are desirable when the clay soils of the Southeast are being evaluated for their lime reactivity.

SPECIMEN PREPARATION

The Thompson procedure (1,2) uses a comparison of unconfined compressive strengths by using untreated and lime-treated compacted soil specimens. The selection of moisture contents for the compacted specimens is critical for the proper densification and for the lime-soil-water modification and stabilization reactions. Ideally, the moisture contents for the untreated and lime-treated compaction specimens would be identical to eliminate confounding of moisture content with the comparison of unconfined compressive strength, which is directly affected by moisture content.

Research reported by Moore and Brown (3) and Rosser and Moore (4) concerning the lime treatment of Alabama blackbelt soils has indicated the likely possibility that lime reactivity of heavily montmorillonitic Southeastern soils is underestimated by using moisture contents for untreated and lime-treated soils based on the approximate optimum of the untreated material. The changes in the engineering properties caused by the lime-modification mechanisms appear to increase the required moisture for the lime-treated specimens. Therefore, different moisture contents would be necessary to obtain an accurate measurement of lime reactivity.

ACCELERATED CURING OF LIME-TREATED SOILS

Both curing time and curing temperature have a dramatic effect on the increase in strength of lime-treated soils (5-9). As early as 1961 Herrin and Mitchell found that the increase in the rate of strength was directly proportional to curing temperature (9). When Thompson originally defined lime reactivity as the difference between the maximum compressive strength of the lime-soil mixtures and the compressive strength of the natural soil, he selected a 28-day curing period at 25°C (73°F) because (a) field conditions may not allow longer curing periods; (b) if the treated soil is lime reactive, the pozzolanic compounds will develop to a significant degree within this time period; and (c) curing temperatures in excess of 60°C (140°F) are unrealistic when compared with field conditions (10).

Various researchers have used the effect of short-term elevated temperature to accelerate curing periods in the laboratory to predict 28-day strengths (10). Anday (11) analyzed both fieldand accelerated-curing data for Virginia soils on a strength versus maturity basis. Maturity was defined as the product of curing temperature and its duration; therefore, the concept of degree-days as a measure of maturity was introduced. Anday arbitrarily selected 0°F (-18°C) as the datum temperature. The round figure of 3000 Fahrenheit degree-days (40 days 75°F or 24°C) for field curing was selected for comparative analyses. This research indicated that short-term laboratory curing at both 120° and 140°F (40° and 60°C) could be used to reasonably predict 40- to 45-day field strengths. However, Anday recommended two days at 120°F (49°C) to simulate 40- to 45-day field strengths because (a) when compared with field conditions the temperature is more realistic, (b) less moisture loss, (c) convenience in 48-h curing time, and (d) better accuracy.

Laguros, Davidson, Handy, and Chu (5,12) reported that strengths induced by temperatures in excess of 140°F (60°C) may very well never be obtained through normal curing. Their work with a Wisconsin lime-treated loess cured at 140°F for 7 days generated strengths that could not be matched by curing at 70°F (21°C) for as long as 160 days. On the other hand, strengths obtained after 7 days of curing at 110°F (43°C) were indicative of those produced by normal curing at 70°F for 80 days.

Davidson, Mateos, and Barnes (5,10,13) reported that the strength of a Kansan till stabilized with lime and a small percentage of sodium hydroxide and cured for 28 days at 70°F (21°C) could be approximated by samples cured for 2 days at 100°F (38°C).

Howard (5,10) investigated several accelerated-curing schemes for lime-treated kaolinitic clays in South Carolina. He reported that accelerated curing for 24, 40, 48, and 72 h at 120°F (49°C) approximated 20, 40, 60, and 90 days of laboratory curing at 72°F (22°C). However, he also reported that different lime percentages resulted in different curing periods to predict 28-day normally cured strengths. To approximate 28-day normal strengths for 4.5, 6.5, and 8.5 percent lime in the mixtures required 29, 26, and 34 h at 120°F, respectively.

Drake and Haliburton's work (5,10,14) with two Oklahoma lime-stabilized soils indicated that the most appropriate accelerated-curing temperature was

105°F (41°C). The time-strength curve generated for Permian red clay cured at 120°F (49°C) has neither the general shape nor slope of the curve produced by samples cured at 80°F (27°C) in a moist atmosphere. The researchers proved through differential thermal analysis that pozzolanic products generated at 80°F were mineralogically identical to those created at 105°F. Therefore, since temperatures in excess of 105°F may produce completely different pozzolanic reaction products, as opposed to simply accelerating their formation, Drake and Haliburton (1,6,10) recommended that accelerated curing temperatures be limited to 105°F.

Data developed by Ruff and Ho (7) and Townsend and Donaghe (9) suggest that the temperature at which a different type pozzolanic reaction product is generated lies between 73° and 104°F (23° and 40°C).

Biswas (5) evaluated the effects of several elevated curing temperatures for periods as long as 120 h on a variety of soils. Contrary to the results reported by Drake and Haliburton, Biswas's findings indicate that all three elevated temperatures (105°, 120°, and 140°F) produced pozzolanic products similar to those generated by normal curing. Biswas concluded that either 30 h at 120°F (49°C) or 65 h at 105°F (41°C) could be used to approximate normal curing at 75°F (24°C) for 28 days, but the lower curing temperature was recommended because it is more realistic, less sensitive to changes in the curing period, and creates less moisture loss.

As a result of their work with Vicksburg silty clay and Vicksburg buckshot clay, Townsend and Donaghe (9) concluded that a universal standard accelerated-curing period for predicting 28-day normal curing strengths is not tenable. They further concluded that the question of whether or not a soil is lime reactive depends on the evaluation criteria. In addition, their results indicate that any criteria that use curing temperatures in excess of 105°F (41°C) are misleading. They report that this is because all of the lime-treated soils in their study cured at 120°F (49°C) met Thompson's criteria (Δq_u > 50 lb•f/in²) but only the silty soils exhibited the necessary strength gains to be termed lime reactive when cured at 75°F (24°C) for 28 days. Also, only the clay soils passed Biswas's reactivity criteria (minimum q_u of 100 lb•f/in² for 30 h at 120°F or 65 h at 105°F), which was intended to forecast lime reactivity based on normal curing procedures (5,9).

Townsend and Donaghe (9) reviewed the existing maturity prediction models for concrete and concluded that none are suitable for use with lime-treated soils. Therefore, a method based on 7-day normal curing strengths and strengths accelerated by curing at 105°F (41°C) was devised to estimate 28-day normal curing strengths. They demonstrated with data developed by Biswas (5) that the method is reasonably accurate up to about 30 normal curing days where the predictions begin to diverge from actual strengths.

MATERIALS

The 11-soil series (Boswell, Demopolis, Eutaw, Houston, Kipling, Leeper, Oktibbeha, Sumter, Susquehanna, Vaiden, and Wilcox) evaluated in this study were typical of those investigated by Rosser and Moore (4). Table 1 (4) presents a summary of selected physical data for the soil series.

The lime used in this research was an air-floated high-calcium-hydrated lime [$Ca(OH)_2$] processed such that 86 percent is finer than a No. 325 sieve (0.045 mm). This lime, manufactured by the Longview Lime Company, was derived from the Newalla limestone (almost pure calcium carbonate) near Saginaw, Alabama.

LABORATORY TESTING PROCEDURES

The laboratory testing was divided into two phases. Phase 1 was designed to determine the soils' lime reactivity, as defined by Thompson. It was composed of 110 specimens, 5 with lime and 5 without lime for each soil series. The major objective was to determine whether different moisture contents for lime-treated and untreated blackbelt soil would be required to determine the lime reactivity as opposed to the more conventional approach, which uses the same moisture content for lime-treated and untreated unconfined compression strength specimens. The order of sample preparation was randomized to spread sample preparation variances homogeneously throughout the population of compacted soil specimens. Specimens prepared without lime were compacted at a moisture content 3 percentage points below the plastic limit of the natural soil. Specimens prepared with 6 percent lime by dry weight of soil were compacted at a moisture content 3 percentage points below the plastic limit of the soil treated with 6 percent lime. These compaction moisture contents are approximately the optimum moisture contents of the treated and untreated soils.

The specimens were compacted by using a Harvard miniature compaction mold and a spring-loaded kneading compaction device. The spring tension was 30 lbs and specimens were compacted in three layers by using 25 tamps per layer. Specimens were extruded, weighed, wrapped in Saran Wrap to minimize moisture loss, placed in prelabeled zip-lock bags, and cured for 48 h at 120°F (49°C). At the end of the curing period, each specimen was removed from the oven, allowed to cool to room temperature, unwrapped, reweighed to determine moisture loss during curing,

Table 1. Selected soil physical data.

Soil Series	Plastic Limit, Untreated	Plastic Limit, 6 Percent Lime	Plastic Limit Change	Sand at 2.0-0.05 mm (%)	Silt at 0.05-0.002 mm (%)	Clay at <0.002 mm (%)	Percentage Montmorillonite in Clay Fraction	Percentage Montmorillonite Based on Total Soil
Boswell	31	43	+12	14.6	26.1	59.3	69.0	40.9
Demopolis	28	34	+6	12.5	38.7	48.8	29.1	14.2
Eutaw	29	38	+9	10.1	45.1	44.8	55.8	25.0
Houston	28	36	+8	11.6	30.8	57.6	57.1	32.9
Kipling	25	35	+10	15.5	33.9	50.6	50.3	25.5
Leeper	34	38	+4	3.0	33.6	63.4	58.2	36.9
Oktibbeha	32	38	+6	4.8	35.8	59.4	51.4	30.5
Sumter	25	44	+19	14.9	36.1	49.0	41.9	20.5
Susquehanna	27	47	+20	10.3	35.6	54.1	62.8	34.0
Vaiden	35	37	+2	4.6	25.2	70.2	58.4	41.0
Wilcox	42	44	+2	13.3	29.7	57.0	77.7	44.3

Table 2. Data summary.

Soil Series	Mean UCS Untreated (lb·f/in²)	Mean UCS 6 Percent Lime (lb·f/in²)	Compaction with Untreated (%)	Compaction with 6 Percent Lime (%)	Rosser and Moore Data: Mean UCS Untreated (lb·f/in²)	Rosser and Moore Data: Mean UCS 6 Percent Lime (lb·f/in²)	Rosser and Moore Data: Compaction with Untreated and 6 Percent Lime (%)
Boswell	41.28	68.82	28	40	28.85	56.64	28
Demopolis	44.74	86.10	25	31	47.33	70.46	25
Eutaw	37.52	37.36	26	35	34.17	57.67	26
Houston	28.80	167.80	25	33	16.57	70.61	25
Kipling	36.50	104.52	22	32	22.26	47.89	22
Leeper	31.78	68.80	31	35	33.88	68.17	31
Oktibbeha	27.70	147.10	29	35	20.13	84.29	29
Sumter	31.84	94.22	22	41	34.40	66.25	22
Susquehanna	31.72	98.32	24	44	33.29	15.83	24
Vaiden	42.48	85.40	32	34	17.40	103.23	32
Wilcox	26.24	81.04	39	41	15.28	99.73	39

Table 3. Comparison of t-statistics with Rosser and Moore's data.

Soil Series	t-Value	Alpha Level	Rosser and Moore: t-Value	Rosser and Moore: Alpha Level
Boswell	-2.27	0.96	-3.39	0.98
Demopolis	-1.51	0.90	-4.92	0.995
Eutaw	-17.96	1.00	-3.23	0.98
Houston	5.72	0.004	0.45	0.34
Kipling	2.08	0.05	-3.07	0.98
Leeper	-1.39	0.88	-1.63	0.91
Oktibbeha	7.25	0.002	1.66	0.09
Sumter	1.90	0.07	-3.02	0.98
Susquehanna	2.93	0.02	-10.71	1.00
Vaiden	-1.23	0.86	5.97	0.003
Wilcox	1.23	0.14	4.31	0.007

and its unconfined compressive strength determined at a strain rate of 1 percent/min.

Phase 2 of this research was designed to determine the effects of different accelerated-curing laboratory procedures on the unconfined compressive strength of lime-treated blackbelt soils. Five specimens for each of the 11 soil series and for each curing scheme were prepared as outlined for phase 1. The curing schemes evaluated included 48 h at 120°F (49°C) (phase 1), 65 h at 105°F (41°C), and 75°F (24°C) for each of 7, 14, and 28 days. Since the lime-treated specimens used in phase 1 were cured for 48 h at 120°F and therefore could be evaluated as a curing scheme in phase 2, a total of 330 compacted specimens was required.

STATISTICAL CONSIDERATIONS

The statistical design for phase 1 of this research is essentially the same as that reported by Moore and Brown (3). In order to minimize random testing variations associated with repetitive strength testing of identical lime-soil specimens, a sample population composed of five lime-treated and five untreated (control) specimens was planned for each soil series. This information will allow adequate statistical significance tests to be conducted (i.e., to determine whether the means of the treated and untreated strength data sets are significantly different by more than 50 lb·f/in² with some level of confidence). Since the variances (σ_1^2 and σ_2^2) of the lime-treated and untreated specimens strength population are unknown, the modified t-test of hypothesis, which does not assume homogeneous population variances, is used.

The null and alternate hypotheses are as follows:

H_o: $(\mu_1 - \mu_2) \leq 49.99$ lb·f/in²
(soil is not lime reactive)

H_a: $(\mu_1 - \mu_2) > 49.99$ lb·f/in²
(soil is lime reactive)

The test statistic for each soil series is calculated by the formula:

$$t = [(\bar{x}_1 - \bar{x}_2) - (\mu_1 - \mu_2)] / \sqrt{(S_1^2 + S_2^2)/n} \quad (1)$$

where

n = sample size,
$\bar{x}_1$ = mean q_u of lime-treated soil,
$\bar{x}_2$ = mean q_u of untreated soil,
$\mu_1 - \mu_2$ = desired difference in means (50 lb·f/in²),
S_1^2 = sample variance in lime-treated soil, and
S_2^2 = sample variance in untreated soil.

After the t-statistic is calculated, the probability of rejecting a correct hypothesis (type 1 or alpha error) can be determined by consulting a table that presents the distribution of t with (n-1) degrees of freedom instead of (2n-2) to compensate for the effects of possible nonhomogeneous variances of the two sample populations (15). Therefore, the alpha level so determined is the probability of error associated with declaring a soil as lime-reactive based on the data presented (10 unconfined compressive strength tests).

LIME-REACTIVITY RESULTS

Ten unconfined compression tests were performed on each of the 11 soil series, for a total of 110 tests. Five of these tests were conducted on compacted specimens prepared with 6 percent lime and at a moisture content equal to the plastic limit of the lime-treated soil minus 3 percentage points. The other 5 tests were performed on compacted specimens prepared without lime and at a moisture content equal to the plastic limit of the natural soil minus three percentage points. The results of these unconfined compression tests are presented in Table 2. Also included in Table 2 are the data reported by Rosser and Moore (4). Table 3 presents a comparison of calculated t-values and corresponding alpha values for the test of hypothesis for this research effort as well as those reported by Rosser and Moore (4). The alpha levels were estimated by interpolation between 5 percent values as presented by Fisher and Yates (16). Note that, by the Thompson definition (minimum $\Delta q_u = 50$ lb·f/in²), only 4 of the soil series were judged to be lime reactive with an alpha error of 5 percent or less. However, the Houston, Kipling, Sumter, and Susque-

hanna series exhibit much lower alpha levels when the strength specimens are compacted at the moisture contents as outlined in this research than those reported by Rosser and Moore (4). The t-statistic and associated alpha level for six of the other seven series compare relatively well with Rosser and Moore (4).

The Vaiden series indicates a low probability of being lime reactive in this research but Rosser and Moore (4) reported a high probability that the soil is lime reactive. The reason for this contradiction in test results is not known.

The results obtained in this research indicate that a majority of the soils, which exhibited a substantial elevation in their plastic limits with the addition of lime, experience a higher probability of being lime reactive when compacted at approximately its new optimum moisture content. Therefore, the postulated moisture deficiency suggested by Rosser and Moore (4) appears to be the major reason for the absence of laboratory lime reactivity for Alabama blackbelt soil by using Thompson's accelerated-curing procedure. This finding will require that a lime-soil mixture design procedure for Alabama and Southeastern fine-grained soils based on compacted laboratory specimen characteristics specify that the compaction moisture content for the lime-treated specimens be based on an estimated optimum moisture content of the lime-treated material by using the plastic limit of the lime-treated material as a guide or on the optimum moisture content of the lime-treated material as determined by a compaction test. This requirement should be implemented for surficial soils, although the change in optimum moisture content produced during lime modification may not be significant for many soils. Note that 15 soil series were shown by Moore and Brown (3) to be lime reactive at an alpha level of 0.25 or lower by using optimum moisture contents on the basis of the plastic limits of the untreated soil for both treated and untreated compaction specimens. However, none of these soils were montmorillonitic in composition.

ACCELERATED-CURING RESULTS

Since the 55 treated samples in phase 1 of the research plan were cured for 48 h at 120°F (49°C), which is one of the accelerated-curing schemes being evaluated, only 220 additional specimens were required for phase 2. Five unconfined compression test samples for each of the 11 soil series were compacted with 6 percent lime and allowed to cure for either 65 h at 105°F (41°C), 7 days at 75°F (24°C), 14 days at 75°F, or 28 days at 75°F. An unconfined compression strength data summary is presented in Table 4.

The means of the unconfined compressive strengths of the two accelerated-curing schemes bracket the means of the 28-day ambient curing strengths as illustrated in Figures 1-4. The vertical lines on the figures represent the range of values measured. The curing period of 48 h at 120°F (49°C) consistently overestimates and the curing for 65 h at 105°F (41°C) consistently underestimates the 28-day, 75°F (24°C) cure strengths, which should be indicative of field strengths. Figures 1-4 also include data from previous research (4) that again illustrate the differences in mean unconfined compressive strengths as outlined in the preceding section and Table 3.

Also, the difference is substantial in unconfined compressive strengths for soil-lime mixtures cured at 120° and 105°F (49° and 41°C). As illustrated in Table 5, neither accelerated-curing scheme closely approximated the mixtures cured for 28 days at ambient temperature. Table 5 presents the mean unconfined compressive strengths of the lime-treated

Table 4. Unconfined compression strength data.

Soil Series	48-h Cure 49°C		65-h Cure 41°C, 6 Percent Lime (lb·f/in²)	7-Day Cure 24°C, 6 Percent Lime (lb·f/in²)	14-Day Cure 24°C, 6 Percent Lime (lb·f/in²)	28-Day Cure 24°C, 6 Percent Lime (lb·f/in²)
	No Lime (lb·f/in²)	6 Percent Lime (lb·f/in²)				
Boswell						
$\bar{X}$	41.28	68.82	53.00	44.02	54.48	66.38
S	9.95	19.75	11.57	9.45	14.10	6.16
Demopolis						
$\bar{X}$	44.74	86.10	51.37	34.38	46.14	62.38
S	9.94	7.97	3.89	3.96	4.18	5.92
Eutaw						
$\bar{X}$	37.52	37.36	25.94	19.40	25.46	35.24
S	5.35	3.22	4.42	1.91	5.08	5.04
Houston						
$\bar{X}$	28.80	167.80	125.74	79.82	117.08	155.20
S	11.14	32.96	24.59	15.44	14.58	7.58
Kipling						
$\bar{X}$	36.50	104.52	47.90	38.46	47.12	62.08
S	8.65	17.34	5.05	5.47	4.75	4.61
Leeper						
$\bar{X}$	31.78	68.80	32.94	30.34	41.16	55.72
S	7.23	19.57	6.41	7.60	12.53	8.57
Oktibbeha						
$\bar{X}$	27.70	147.10	97.64	54.84	86.80	130.20
S	6.53	20.38	6.23	10.61	7.70	23.81
Sumter						
$\bar{X}$	31.84	94.22	62.32	40.64	46.36	81.00
S	11.31	9.17	10.15	5.53	6.48	13.66
Susquehanna						
$\bar{X}$	31.72	98.32	52.18	37.74	46.14	70.58
S	7.64	10.12	6.82	3.80	9.36	4.88
Vaiden						
$\bar{X}$	42.48	85.40	48.44	41.00	38.88	65.34
S	9.51	8.58	9.15	4.17	8.46	10.02
Wilcox						
$\bar{X}$	26.24	81.04	70.30	62.04	78.16	86.10
S	1.75	8.55	17.53	4.96	24.31	12.36

Figure 1. Mean unconfined compressive strength versus time for lime-treated samples for Boswell, Demopolis, and Eutaw soil series.

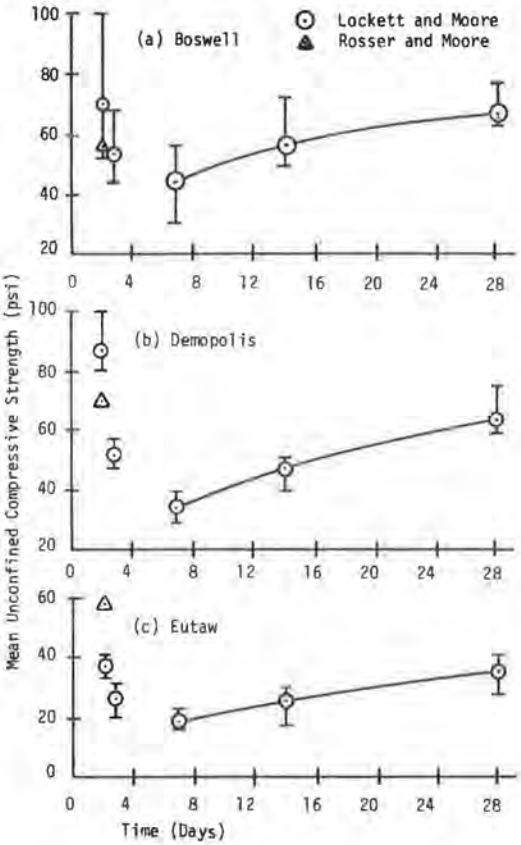

Figure 2. Mean unconfined compressive strength versus time for lime-treated samples for Houston, Kipling, and Leeper soil series.

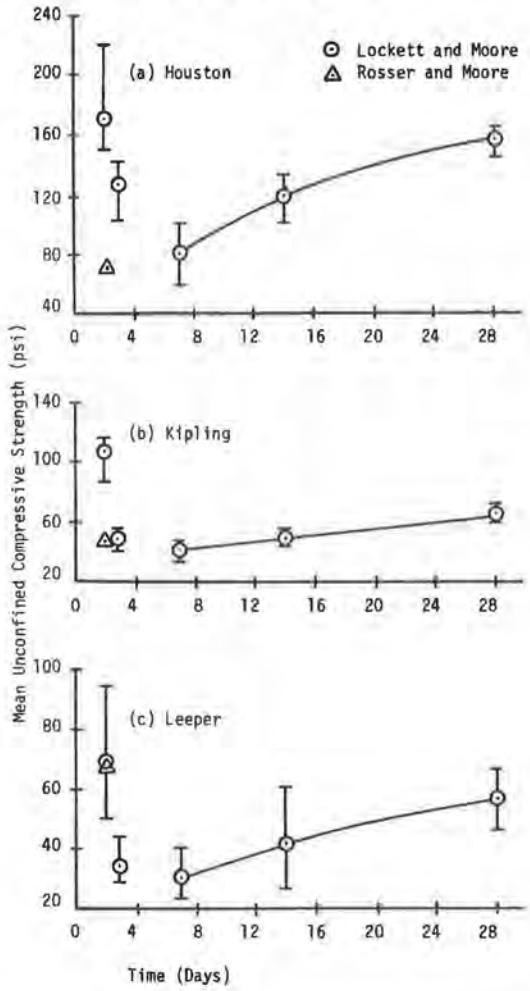

Figure 3. Mean unconfined compressive strength versus time for lime-treated samples for Oktibbeha, Sumter, and Susquehanna soil series.

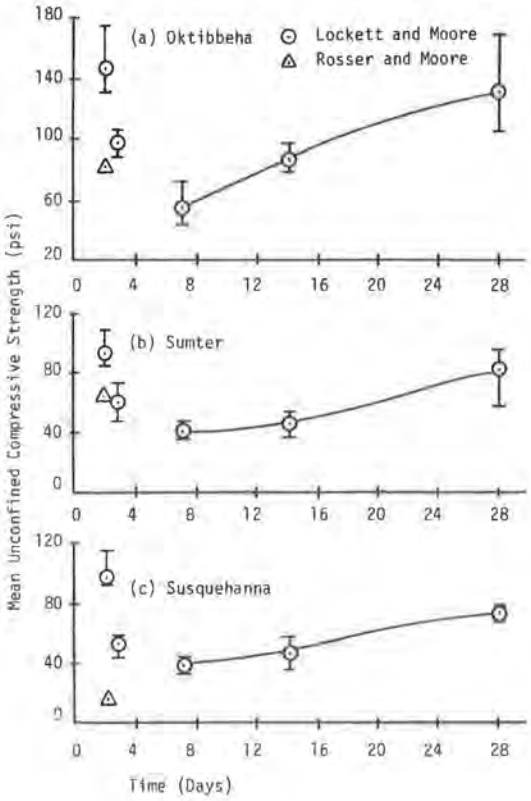

Figure 4. Mean unconfined compressive strength versus time for lime-treated samples for Vaiden and Wilcox soil series.

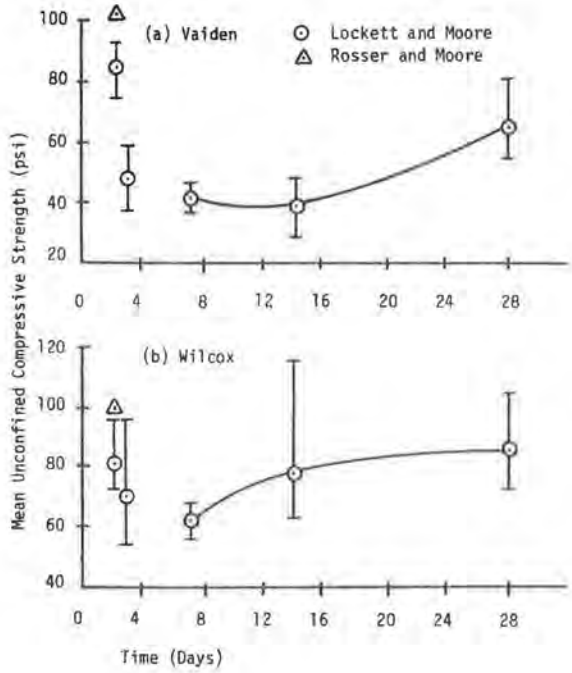

Table 5. Strength ratios by using 28-day strengths as base.

Soil Series	48 h q_u ÷ 28 day q_u	65 h q_u ÷ 28 day q_u	7 day q_u ÷ 28 day q_u	14 day q_u ÷ 28 day q_u
Boswell	1.04	0.80	0.66	0.82
Demopolis	1.38	0.82	0.55	0.74
Eutaw	1.06	0.74	0.55	0.72
Houston	1.08	0.81	0.51	0.75
Kipling	1.68	0.77	0.62	0.76
Leeper	1.24	0.59	0.54	0.74
Oktibbeha	1.13	0.75	0.42	0.67
Sumter	1.16	0.77	0.50	0.57
Susquehanna	1.39	0.74	0.53	0.65
Vaiden	1.31	0.74	0.63	0.60
Wilcox	0.94	0.82	0.72	0.91
$\bar{X}$	1.22	0.76	0.57	0.72
S	0.21	0.06	0.08	0.10

specimens cured for 48 h at 120°F, 65 h at 105°F, and 7 and 14 days at 75°F (24°C) as a proportion of the 28-day, 75°F cured strengths. The strength ratios created by the 65-h, 105°F curing period are the most consistent of the accelerated-curing schemes as indicated by the standard deviation (0.06) of the data. Note that the mean of the 65-h, 105°F accelerated-curing strengths approximate 75 percent of the 28-day, 75°F cured strengths. Also, the 65-h strengths are approximately equal to the 14-day ambient temperature strengths. Therefore, it may be possible to approximate 28-day field strengths simply by multiplying the 65-h, 105°F accelerated-curing strengths by 1.33.

CONCLUSIONS

The Thompson procedure for lime-soil mixture design should be modified when soils of the southeastern United States are evaluated. The dominance of montmorillonite in the clay fraction of some Southeastern clays, especially those of the Alabama and Mississippi blackbelt, creates the need for careful consideration of compaction moisture contents for the lime-treated specimens. The effect of lime-modification on these clays causes the optimum moisture content to increase by as much as 20 points (based on the increase in plastic limit). Therefore, the lime-treated soil must be compacted at a higher moisture content than the untreated soil. Although the different moisture contents confound the comparisons of unconfined compressive strength, the potential for moisture deficiency in the lime-treated material must be eliminated. A comparison of plastic limits for the untreated and lime-treated soil will provide an indication that the design of the lime-soil mixture will require this modification.

The use of a modified accelerated curing procedure is recommended for soils of the southeastern United States. Data developed in this research program indicate that the Thompson accelerated-curing criteria of 48 h at 120°F (49°C) overestimate the 28-day, 75°F (24°C) unconfined compressive strengths of lime-treated blackbelt soils by an average of 22 percent. A 65-h, 105°F (41°C) accelerated-curing sequence underestimates the 28-day, 75°F unconfined compressive strengths by approximately 25 percent. We therefore recommend that a 72-h (3 days is more convenient for laboratory scheduling than 65 h) accelerated-curing sequence at 105°F be employed when the Thompson procedure is used for Southeastern soils.

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Short-Term Active Soil Property Changes Caused by Injection of Lime and Fly Ash

THOMAS M. PETRY, J. CLYDE ARMSTRONG, AND TA-TEH (DAVE) CHANG

This paper describes research into the physical and chemical property changes that occur in an active clay soil during and shortly after injection with lime and lime and fly ash slurries. It reports on the changes measured for 20 properties by using 112 samples taken both before and after injection. It describes the design of the experiment, where four treatments were randomly applied in four replications. It describes the property changes noted that were significant and explains how the monitored ground surface elevations changed during the project. The paper describes the processes used for statistical analyses of property changes measured. This description includes two types of analysis of variance and comparison of means before and after treatment. Those properties concluded to have significantly been affected by injection stabilization included water contents, plastic indices, swelling potential, cation exchange capacities, calcium levels in pore water, and calcium levels in the exchange complex. The ground level monitoring analyses presented support the stabilizing effects of lime slurry pressure injection (LSPI). Conclusions include the relative ranking of the treatments applied where a single LSPI followed by a three-staged water injection proved most effective.

The damage to structures caused by change in the volume of active clay soil has been well documented and is estimated to exceed $2 billion annually. This is more than twice the damage caused by other natural disasters combined. Although these problem soils are abundant across the continental United States, they present the most crucial problem to transportation facilities in regions that have semiarid climates. This type of climate provides long periods of drying during which active soils may shrink significantly, followed by periods of intense rainfall when swelling of these soils causes substantial damage. One such area where these problems affect a great number of transportation facilities is the Dallas-Fort Worth metroplex of north-central Texas.

In order to alleviate or eliminate the problems associated with active clay soils, techniques for improvement to the soil site, such as excavation and replacement with inactive materials and stabilization of soils to limited depths, have been recommended by geotechnical engineers. The use of lime or fly ash as stabilizing agents has been popular for some time. Geotechnical engineers would like to understand better how these agents work in active clay soil subgrades, especially when injected under pressure to moderate depths.

Pressure injection of lime was introduced about 20 years ago. Studies have been conducted to determine the changes to the physical properties of soils and soil masses that occur as a result of this stabilization method (1-4). The results of these studies and experience with using lime slurry pressure injection (LSPI) have improved the understanding of how to apply this technique effectively; however, to date there have been no definitive studies that used statistically designed experiments to determine the changes that occur to the physical and chemical properties of active clay soils during and shortly after injection with lime slurries. In addition, no studies have been done to determine the effects of lime and fly ash injection on these soils.

The research reported here was undertaken to provide information about the changes to physical and chemical properties that occur in an active clay soil during and shortly after injection with lime and lime and fly ash slurries. The research site was the Dallas-Fort Worth Regional Airport and the laboratory tests were performed at the University of Texas at Arlington. The majority of material and financial support was provided by the Woodbine Corporation of Fort Worth through the College of Engineering's Construction Research Center. The soils involved in this study were highly active clay soils weathered from the Eagle Ford Shale geologic formation.

RESEARCH PROGRAM

This study uses a statistically designed experiment to report the changes that result in some 20 physical and chemical properties of an active soil subgrade when LSPI stabilization was performed. A site was provided on property of a large transportation facility where 12 areas were treated with four replications of three treatments, and 4 areas were used as untreated control. Prior to treatment, samples were taken to determine the natural properties of the subgrade and, subsequent to treatment, the areas were again sampled to determine property changes. In addition, the movement of the ground surface was monitored for these areas on a monthly basis throughout the duration of the project.

The project site was chosen because of the highly active nature of the clay soil subgrade and because of the highly fractured nature of this subgrade that was determined. This site and the soils profile are indicative of those encountered by transportation facilities in north-central Texas. The site was approximately 2 acres (7000 m^2) in size and was partitioned into 16 areas (called pads) that were squares, 35 ft (10.7 m) on each side. The locations of these pads and of the benchmark found 20 ft (6.1 m) deep are shown in Figure 1.

Design of Experiment

The three treatments and control areas were applied in four replications to the 16 pads. The treatments were selected to represent those currently used in injection-stabilization practice in order to provide relevant information to users of these techniques.

Treatment number one was a single injection, on 5-ft (1.5-m) centers to a depth of 7 ft (2.1 m), of a normally used lime slurry, followed by three similarly spaced and penetrating water injections. The lime slurry contained from 2.5 to 3 lb (1.1-1.4 kg) of hydrated lime and a surfactant at a rate of 1 part to 3500 parts by volume/gal (0.0038 m^3) of slurry. The water injected contained a similar quantity of surfactant. This treatment was labeled as a single LSPI plus three-staged water injection.

Treatment number two was a double-staged injection of the same lime slurry used in treatment number one. The resultant pattern of injections was located at approximate 2.5-ft (0.76-m) centers. There was an approximate one week delay between the stages of this treatment. This treatment was labeled as a double LSPI.

Treatment number three was a double-staged injection of a lime and fly ash slurry performed in the same manner as treatment number two. The fly ash used was obtained from an electric power generating plant by using lignite coal fuel. The contents of

Figure 1. General site layout.

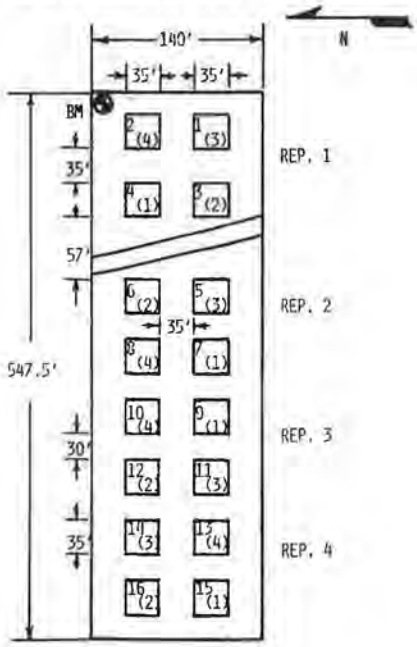
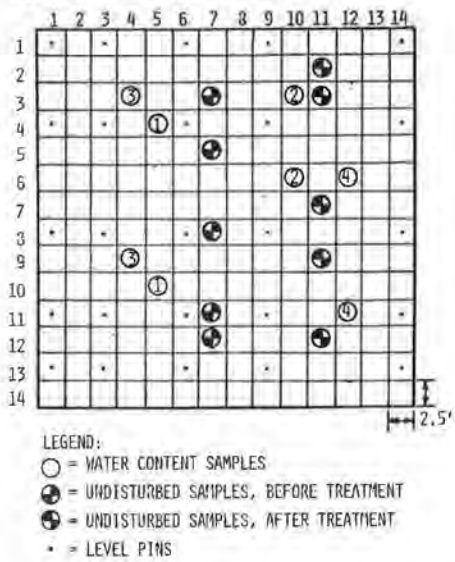


Figure 2. Sampling randomization.



this calcareous fly ash are shown in the table below:

Compound	Average (%)
SiO_2	47.8
Al_2O_3	20.0
Fe_2O_3	6.6
CaO	18.0
MgO	3.3
SO_3	1.5
K_2O	0.7

The lime and fly ash (LFA) slurry contained surfactants in the same quantities as the lime slurry described above, 1 lb (0.45 kg) of hydrated lime and 3 lb (1.4 kg) of fly ash/gal (0.0038 m^3) of slurry. This treatment was labeled as a double lime and fly ash slurry pressure injection (LFASPI).

The total quantities of hydrated lime used for single LSPI on four pads, double LSPI on four pads, and double LFASPI on four, were 59 tons (53 600 kg). The total quantity of fly ash used was 40 tons (36 600 kg).

The fourth treatment, applied to 4 of the 16 pads, was no stabilization. This was done to provide statistically significant information to compare with the stabilization treatments. After injection of stabilizing agents was complete, a 6-in-thick (15.24 cm) surface layer of the subgrade of all pads was mixed thoroughly and lightly compacted.

Twenty-five level pins were placed into the ground of each pad to monitor surface movements. This number was chosen to adequately measure movement but not interfere with sampling or injection procedures. The locations of these pins, shown in Figure 2, were chosen by a random process, which resulted in five columns with five rows of pins. The designation columns were chosen for east-west linear areas between injection points. Rows were designated as north-south linear areas between injection points.

A random selection process was used to apply each of the four treatments to the four pads in each replication. The design chosen was taken from some 20 sets generated to provide as wide a dispersal of treatments to positions within each replication. The pads to which treatments were assigned within the replications are shown in parentheses on each pad in Figure 1. Replications were chosen as groups of four pads in numerical order. This was done because the site topography included some variation in elevation longitudinally and a shallow erosion channel crossing between pads three and four and pads five and six (shown in Figure 1).

Sampling Program

In order to provide information on property changes, samples were taken from each pad before and after treatment. All samples were taken from positions between injection points that were chosen by using a random selection process. Because of expected sampling, 11 sets of positions were chosen so that the samples would be taken in each case within a single column and from selected rows within the column. The use of all samples taken from a single column facilitated sampling operations because a truck-mounted drill could be operated along single lines.

For initial property determination, samples were taken from five borings in each pad. A total of seven undisturbed samples was taken from these five borings. From each hole samples were obtained from 3 to 4 ft (0.91-1.21 m). In addition, to provide some information concerning property change with depth, samples were taken from 4 to 5 ft (1.21-1.52 m) and from 5 to 6 ft (1.52-1.83 m) in the center (number three) boring. The choice of this boring for the deeper samples was to accommodate the limitations of the project and to possibly provide representative information. The samples taken at this time were labeled and sealed in the field to provide identification of the 112 obtained and to preserve the natural properties of the soil. These samples were taken to the civil engineering laboratories of the University of Texas at Arlington for testing, as described later.

In addition to these samples, two sets of disturbed samples were taken during the period before injection for determination of water content only. As in the case of the first, undisturbed samples, these were taken at predetermined locations. In both cases, however, samples were obtained only at the 3- to 4-ft (0.91- to 1.22-m) level from each of

five borings. The second water content samples were taken immediately prior to the injection process. During each of the sampling periods before injection, all holes created in the pads were first filled with a bentonite slurry and then backfilled with soil from the site. This was done to prevent the presence of holes from affecting the injection process or resultant patterns of injected fluids.

The next samples obtained from the subgrade were used to determine the changes in water content that occurred prior to and during the three-staged water injections. On each of four occasions, undisturbed samples were taken from two previously determined borings within the five randomly chosen borings in the set. These samples were obtained by continuous push of shelby tubes to 6 ft (1.83 m). Samples were taken in the field from areas between injection seams and their water content determined in the laboratory. The resultant holes were backfilled with natural soil from the borings and on site.

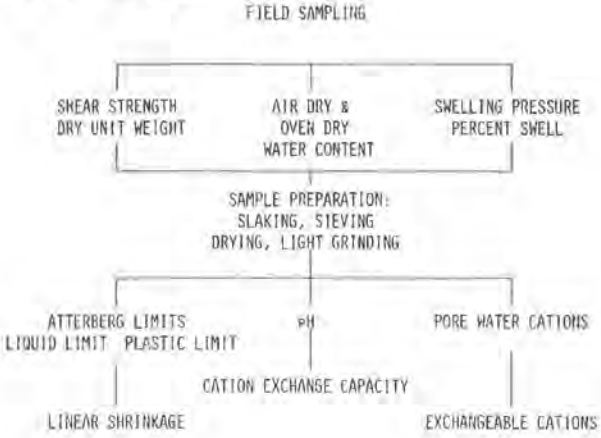
For the final property determination, samples were taken approximately one month after the injections were complete. Some 112 samples were obtained from previously determined random locations in the same manner as were the first property determination samples. These were identified, sealed, and transported to the testing laboratory as were the previously taken samples for property determinations. These samples were tested by using the process described below. The locations within the pads where samples were obtained for property determination before and after injection and for water contents during water injection are shown, along with level pin locations in Figure 2.

Testing Program

Samples taken for property determination before and after injection were tested for 20 properties. The flow chart of the testing program used is shown in Figure 3. The physical properties measured included shear strength, dry unit weight, swelling pressure, percentage swell, air-dry and oven-dry water contents, and selected Atterberg limits and related indices. Soil chemical properties determined included pH, cation exchange capacity (CEC), selected pore water cations, and selected exchange complex cations. Samples tested after injection were taken from between the seams of stabilizing materials in every case.

The testing procedures used for determination of physical property were those recommended by the American Society for Testing and Materials (ASTM).

Figure 3. Testing program.



Swelling tests were started at natural water content and with an initial surcharge load to apply the in situ overburden pressure. The air-dried water content tests were performed by using a temperature of 68°F (50°C) and a relative humidity of 68 percent. The process of sample preparation for Atterberg limits and chemical property testing included removal of seams of stabilizing agents and concretions, slaking in distilled water, wet sieving through a No. 40 series sieve, drying at 140°F (60°C), light crushing of clods to workable size, and reconstitution of moisture to a desirable level for testing. The Texas bar method was used for determination of linear shrinkage.

Determinations of chemical property were performed by using standard procedures specified by the Soil Conservation Service of the U.S. Department of Agriculture (5). The soil pH was determined by using a 1:1 mixture by weight of soil and distilled-demineralized water. Pore water cation extracts were made by using 1:1 mixtures as for the pH test. The CEC was determined by using the calcium replacement method. Exchange complex cation extractions were performed by using amonium acetate and soil mixtures. The concentrations of cation levels were determined by using an atomic absorption flame spectrophotometer.

RESULTS

The results determined during testing for 20 properties for 224 samples are too numerous to include in this report. The properties determined prior to injection have been reported previously, along with the analyses of variance and correlations of those properties (6). The results in this report, therefore, will be directed toward the ultimate purpose of the research project--to compare the changes of properties that occurred due to the injection stabilization treatments. The five samples taken at the same depth were used for all comparisons and analyses, since the other two did not provide sufficient or differing results for analysis.

The changes in the properties measured, the comparison of property means, and the changes that result in these means are of interest. This part of the comparison is normally used by geotechnical engineers when the number of samples is relatively small. A presentation of property means by treatments determined for before and after samples is offered in Table 1. The change to the means may be determined for each property. Note that any comparison of the mean change without statistical analyses, which is possible with the number of samples used in this study, is not complete. It is possible, however, to gain some insight into the basic soil property measurements and the variations present in soils across the site by using this information.

The clay soil at this site is very active, and changes to properties have occurred because of the injection processes applied. Some of the changes are possibly due to changes in personnel who performed the tests and some are due to variation of the soil from which the samples were taken. The analyses of variation, reported earlier (6), for the samples taken before injection showed considerable variation of all properties across the site and significant variances within the pads. Rather than discuss analyses of mean property changes at this point, it would be more significant and useful to proceed to the statistical analyses that were employed and their results. The only conclusion that can be reached after studying the mean property changes presented in Table 1 is that the injection stabilization processes applied significantly re-

Table 1. Mean property values.

Property	Property No.	Treatment 1		Treatment 2		Treatment 3		Treatment 4	
		Before	After	Before	After	Before	After	Before	After
Oven-dry water content (%)	2	24.4	26.8	27.5	27.1	26.9	29.2	27.9	24.7
Air-dry water content (%)	3	14.9	21.1	17.0	21.6	16.3	23.4	18.5	19.4
Plastic index (%)	6	51.0	33.0	50.0	36.0	51.3	40.3	52.8	39.6
Linear shrinkage (%)	7	24.7	21.7	23.9	21.9	24.0	22.6	24.2	21.8
Percentage swell	8	4.0	1.5	5.9	2.4	5.2	1.5	5.9	3.5
Swelling pressure (tons/ft^2)	9	1.4	0.1	1.9	0.4	2.0	0.2		
Shear strength (tons/ft^2)	10	3.9	1.4	3.9	1.9	2.3	1.8	3.4	2.7
pH	11	7.80	7.80	7.71	7.91	7.72	7.82	7.81	8.01
CEC-Calcium (milliequivalence/100g)	12	9.9	25.7	10.4	24.9	15.2	21.4	16.3	21.1
Pore water cations (milliequivalence/L)									
Na	13	14.5	5.5	10.9	7.6	15.5	6.3	13.4	7.1
K	14	0.19	0.49	0.34	0.33	0.67	0.47	0.22	0.36
Ca	15	1.5	2.6	1.4	3.2	1.7	2.5	3.6	2.6
Mg	16	1.77	0.77	1.54	0.84	1.65	0.55	2.63	1.13
Exchange complex cations (milliequivalence/100g)									
Na	17	6.0	5.6	6.4	5.4	9.1	10.8	7.9	8.5
K	18	0.94	1.34	1.12	1.42	0.95	1.35	1.63	1.53
Ca	19	41.4	41.1	58.2	42.2	25.5	42.5	45.7	37.7
Mg	20	4.0	3.6	4.3	3.6	4.0	4.6	4.9	5.8

duced the activity of the soil subgrade under study.

Statistical Analyses

Part of the originally stated purpose of the study was to design and conduct an experiment that would lend itself to statistical evidence of the changes in properties caused by injection stabilization. In fact, statistical significance of research findings is a necessity for such a study. As described above, this experiment was designed by using random selection processes and numbers of samples that provide the best statistical information possible. The selections were made to reduce bias in assignment of treatments, sample locations, and level pin locations and to minimize bias caused by topographic effects on the soil profile and drainage. The statistical analyses of results included analyses of variance, comparison of variances, and comparison of means (7).

The first step in the analysis was to investigate the variance of all property results, including those from both before and after injection samples. The analysis of variance, as it is called, for results from samples taken before injection was reported on previously (6), but two facts from that analysis are pertinent here. The first of these is that, for all properties measured, variance across the sites was significant and exceeded the variance within or between the pads. The second was that, for all properties determined, there was more variance within each treatment set of pads than within or between the pads of each treatment. These results are expected and even preferred from an analysis of variance so that the comparisons of property changes can be accomplished without problems that may arise from unique situations for one or two pads. In addition, these groupings of results for all properties used during studies on changes of properties were tested and found to be distributed essentially normally.

The results from testing of samples taken after injection were put through computer analysis to determine the F-statistics for an analysis of variance. The analysis included consideration of the same data groupings as for the samples obtained before injection. The F-test results were essentially the same for the samples taken after injection as for the samples obtained before injection. Once these analyses of variance were complete and the results as described above were determined, the process for finding the significance of property changes could proceed.

The first step of the property change comparisons and the second part of the statistical analyses was an analysis of variance between properties before and after injection. Each property was investigated for each treatment. The comparison and analysis of mean property changes may not be considered significant when the property values measured are for samples from different statistical populations. In other words, the soil would be a significantly different material.

The distribution of the F-statistic may be used to evaluate variances to indicate independence of populations. When used for this type of study, the computer program used determines probabilities of agreement between F-statistics. The significance level employed in this test is the 5 percent level and the results are shown in Table 2.

The estimations shown in Table 2 indicate that, when variances are significantly different, means may not actually represent average properties for different populations from the same site. Therefore, comparison of means that have significantly different variances may not prove to be a change of property caused by treatments. Ten out of the 19 properties under consideration had probabilities that indicate populations that were nonindependent for treatment one. Nine were nonindependent for treatment two. The results for treatment three property changes showed eight to be nonindependent, and for treatment four only five were found to be nonindependent. These are the possibly significant property changes for each treatment.

The third part of the statistical analyses was the comparison of changes to the mean values for each property and treatment. The first step in this process was to evaluate the significance of the differences in mean values before and after injection. The T-statistic test was used to estimate the difference of a set of mean values (μ_D). In this study, the null hypothesis ($\mu_D = 0$) indicates that there were no changes in properties between the mean values before injection (μ_A) and the mean values after injection (μ_B). The computerized analysis included a 5 percent significance level and P-values (probability) estimated by using a two-tailed test. The results are shown in Table 3.

The estimates shown in Table 3 indicate that about 90 percent of all physical property means were changed significantly. The smallest percentage change resulted in the pads of treatment four. The changes for samples from product treatments one,

Table 2. Analysis of variance results for before and after treatment.

Property	Probability			
	T1	T2	T3	T4
1				
2	0.810[a]	0.500[a]	0.573[a]	0.010[b]
3	0.445[a]	0.108[a]	0.835[a]	0.405[a]
4	0.591[a]	0.783[a]	0.230[a]	0.007[b]
5	0.000[b]	0.002[b]	0.001[b]	0.000[b]
6	0.824[a]	0.050[b]	0.009[b]	0.027[b]
7	0.298[a]	0.781[a]	0.063[a]	0.028[b]
8	0.008[b]	0.005[b]	0.000[b]	0.313[a]
9	0.009[b]	0.001[b]	0.000[b]	0.000[b]
10	0.000[b]	0.000[b]	0.965[a]	0.643[a]
11	0.095[a]	0.166[a]	0.005[b]	0.003[b]
12	0.474[a]	0.028[b]	0.606[a]	0.014[b]
13	0.005[b]	0.088[a]	0.080[a]	0.003[b]
14	0.000[b]	0.007[b]	0.000[b]	0.011[b]
15	0.055[a]	0.000[b]	0.000[b]	0.229[a]
16	0.002[b]	0.014[b]	0.000[b]	0.000[b]
17	0.002[b]	0.072[a]	0.000[b]	0.000[b]
18	0.406[a]	0.374[a]	0.000[b]	0.167[a]
19	0.000[b]	0.000[b]	0.334[a]	0.000[b]
20	0.965[a]	0.140[a]	0.004[b]	0.012[b]

[a]The variance of results before and after injection is not significantly different when P > 5 percent; therefore, nonindependence is shown.
[b]The variance of results before and after injection is significantly different when P < 5 percent; therefore, independence is shown.

Table 3. T-test results for difference of means.

Property	Probability			
	T1	T2	T3	T4
1				
2	0.186[a]	0.793[a]	0.027[b]	0.121[a]
3	0.000[b]	0.000[b]	0.000[b]	0.643[a]
4	0.043[b]	0.144[a]	0.007[b]	0.039[b]
5	0.013[b]	0.008[b]	0.033[b]	0.012[b]
6	0.000[b]	0.002[b]	0.000[b]	0.000[b]
7	0.010[b]	0.015[b]	0.044[b]	0.001[b]
8	0.002[b]	0.003[b]	0.010[b]	0.053[a]
9	0.000[b]	0.002[b]	0.073[a]	0.360[a]
10	0.003[b]	0.013[b]	0.208[a]	0.126[a]
11	0.779[a]	0.029[b]	0.096[a]	0.004[b]
12	0.002[b]	0.001[b]	0.053[a]	0.360[a]
13	0.000[b]	0.040[b]	0.000[b]	0.001[b]
14	0.011[b]	0.910[a]	0.171[a]	0.226[a]
15	0.009[b]	0.010[b]	0.032[b]	0.005[b]
16	0.029[b]	0.127[a]	0.003[b]	0.028[b]
17	0.689[a]	0.436[a]	0.276[a]	0.842[a]
18	0.114[a]	0.120[a]	0.600[a]	0.585[a]
19	0.980[a]	0.036[b]	0.000[b]	0.323[a]
20	0.465[a]	0.209[a]	0.186[a]	0.059[a]

[a]The null hypothesis ($\mu_D = 0$) is accepted when P > 5 percent. This indicates no statistically significant change in the mean values.
[b]The null hypothesis ($\mu_D = 0$) is rejected when P < 5 percent. This indicates that the mean value of samples before and after injection was significantly different ($\mu_B \neq \mu_A$).

two, and three are believed to be caused by the injections. Other changes noted for samples may have been caused by the effects of circumstances and the difference of samples and testing personnel.

The estimates of the differences in mean value in chemical properties are more varied. The most interesting of these are those that have to do with cation concentration changes in pore water and exchange complex extracts. These will be discussed in detail during the final step of the statistical analysis.

The concluding procedure used during the statistical analyses was a combined comparison of variance and change of means values by using the results shown in Tables 2 and 3. Criterion for complete acceptance of the significance of property change caused by injection were developed by using the Fand T-statistical tests together. Some comparisons and

Table 4. Statistical significance of treatment methods.

Property	Property No.	T1	T2	T3	T4
Oven-dry water content	2	PS	PS	SS	NS
Air-dry water content	3	SS	SS	SS	PS
Plastic index	6	SS	PS	PS	PS
Linear shrinkage	7	SS	SS	SS	PS
Percentage swell	8	PS	PS	PS	PS
Swelling pressure	9	PS	PS	NS	NS
Shear strength	10	PS	PS	PS	PS
pH	11	PS	SS	NS	PS
CEC	12	SS	PS	PS	NS
Pore water cations					
Na	13	PS	SS	SS	PS
K	14	PS	NS	NS	NS
Ca	15	SS	PS	PS	SS
Mg	16	PS	NS	PS	PS
Exchange complex cations					
Na	17	NS	PS	NS	NS
K	18	PS	PS	NS	PS
Ca	19	NS	PS	SS	NS
Mg	20	PS	PS	NS	NS

Note: SS = statistically significant, PS = partly significant, and NS = not significant.

analyses that previously resulted in changes that were thought accepted may now be rejected or may be decreased in degree of confidence depending on the combinations of Fand T-statistical combinations. The criterion and results from this combined analysis may be evaluated by using three cases:

1. The variance of the properties for samples taken before and after injection are not significantly different and the means if the properties are significantly different ($\mu_A \neq \mu_B$). For this case, the changes determined were found significant.
2. If the variances determined are not significantly different and $\mu_A = \mu_B$, the changes are not significant (same population, no change).
3. If both the variances and the mean values of properties for couples compared are significantly different, the changes determined may be considered partly significant and the results are not always completely conclusive.

The final determinations of significance for property mean value changes are shown by property and treatment in Table 4. The case that applies is shown for each comparison. The cases that are of most interest and importance are detailed below by property.

Water content changes:

Treatment one--Significant increase in air dried, partly significant increase in oven dried;

Treatment three--Significant increases in air dried and oven dried; and

Treatments two and four--No significant changes.

Atterberg limits and related indices--There were significant decreases in PI for all treatments, where samples from treatment one pads had the most significant change, followed by that for samples from treatment two and the least for samples from treatment three and four. There were no significant changes to measured linear shrinkage.

Swelling properties--Partly significant reduction of percentage swell for samples from treatments one, two, and three and almost no change in percentage swell for samples from treatment four. Partly significant reduction in swelling pressure for samples from treatments one and two, and almost no change in swelling pressure for samples from treatments three and four.

Figure 4. Monthly rainfall movement.

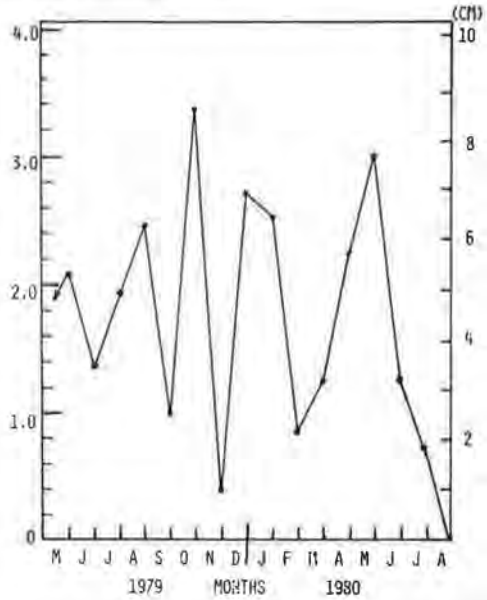
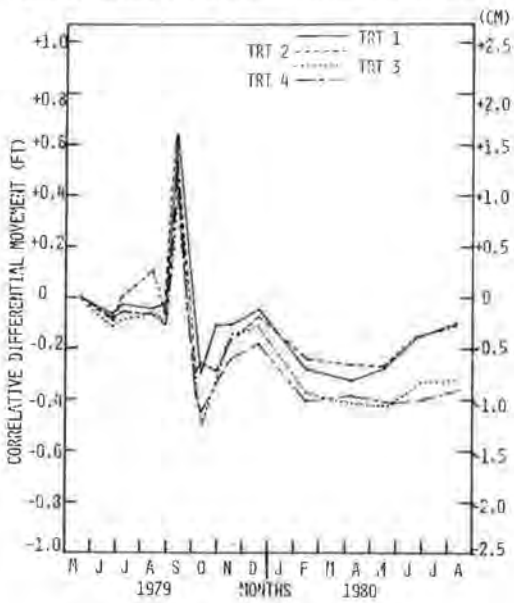


Figure 5. Monthly level differential movement in four treatments.



Shear strength--Partly significant reduction in shear strength for samples from treatment one, two, and three, with the least change for samples from treatment three. Essentially no changes in shear strength for samples from treatment four.

pH--Almost no change for samples from all treatments.

CEC--Significant increase for samples from treatment one, partly significant increase for samples from treatment two and three, and no changes for samples from treatment four.

Pore water cation concentrations:

Sodium--Significant decreases for samples from treatments two and three.

Potassium--Partly significant increase for samples from treatments one and four.

Calcium--Significant increase for samples from treatment one, partly significant increases for samples from treatments two and three, and significant decrease for samples from treatment four.

Magnesium--Partly significant decreases for samples from treatments one, three, and four; no changes noted in samples from treatment two.

Exchange complex cation concentrations:

Sodium--Partly significant decreases in samples from treatment two; no changes noted for samples from other treatments.

Potassium--Partly significant increases in samples from treatments one and two, partly significant decreases in samples from treatment four, and no changes noted in samples from treatment three.

Calcium--Significant increases in samples from treatments three, significant decreases in samples from treatment two, and no significant changes in samples from treatment one and four.

Magnesium--Partly significant decreases in samples from treatments one and two; no changes noted in samples from treatments three and four.

There were no analyses concerning the changes of dry unit weights because this property was not measured for after-injection samples.

Ground Level Movements Caused By Climatic Effects

The ground level movements that occurred before, during, and for approximately one year after injection were monitored by using the 25 level pins placed in each pad. Elevations measured were relative to a benchmark founded 20 ft (6.1 m) deep. Since there was no opportunity to cover the pads, the analyses related to change of elevations of the pad were related to change of elevation of the level pins. The changes in the elevations of the same 100 pins were measured and averaged for each treatment.

The ground surface movements monitored, therefore, were affected by rainfall, temperatures, and the injection processes. The movement should reflect general trends of rainfall and temperature and the specific event of injection. In addition, they should reflect the ability of the subgrade, injected or not, to resist general climatic cycles.

It is possible to analyze the ground surface movements determined by using Figure 4, which shows the monthly rainfall record, and Figure 5, which is a plot of average movement for pads from each treatment. In all cases the ground elevations fell during May and June 1979 because of low rainfall and increase of temperature. The elevations of all pads stabilized somewhat during July because of more rainfall. During July and August 1979 the lime slurry injections were carried out. A notable rise in the ground surface for pads in treatment one occurred when water injections were performed. Treatment one and two had the most affect of swelling the injected soil subgrade. The rainfall in September 1979, coupled with reduced temperatures, is believed responsible for the rise in the ground surface for all pads. The drastically lower rainfall of October 1979 is reflected in the elevation drop in the ground surface for the site. Note, however, that the ground surface movements for pads in treatments one and two were the least; that for pads of treatments three and four were greatest. During the rest of 1979 and until August 1980 the ground movements followed climatic events. In all cases, the movements for pads of treatments one and two were less than those for pads of treatments three and four. The trend supports the other results and substantiates the stabilizing effects of

treatments one and two versus treatments three and four.

CONCLUSIONS AND RECOMMENDATIONS

The objective of the investigation was to determine the property changes that occurred during and shortly after an active clay soil subgrade was injection-stabilized by using three different stabilization application procedures. The objective was realized by obtaining numerous samples before and after treatment. These were tested both physically and chemically. The results were then statistically analyzed for significance.

The results from the testing program, level pin elevation analyses, and statistical analyses indicate the following conclusions.

Treatment numbers one (single LSPI and three water injections) and three (double LFASPI) resulted in significant water change in content. Each caused an increase in mean water content of about 2.5 percent. Treatments two (double LSPI) and four (control) caused no significant change to water content.

Although all four treatments affected the plastic index of the soil subgrade, only treatment one had nonindependent variance and definite change in means. This change amounted to an 18 percent reduction. The results for samples from treatment two were much closer to proving nonindependence than those from treatment three or four, which means that treatment two was more effective in changing the measured plastic index than was either treatment three of four.

Although the statistical analyses resulted in only partial significance, there was significantly less potential of percentage swell for treatments one and three and less for treatment two than for treatment four. The changes in swelling pressure caused by treatments one and two were at least partly significant; the changes caused by treatments three and four were not statistically significant.

The changes in shear strength, although partly significant, occurred mainly where the soil moisture content increased. The stabilizing effects of the fly ash in treatment three probably account for the least loss in strength.

The significant change in CEC that occurred only for treatment one and the partly significant increase caused by treatment two is believed to occur because of the changes in cation concentrations that accompanied these. The less significant change to CEC caused by treatment three is believed to be from a lesser effect on these cation concentrations.

Although some of the stabilizing effects noted were caused by changes in moisture content, the cation concentrations in the pore water changes for each treatment provide insight to how these treatments chemically stabilize the soil:

1. Treatment one caused partly significant reductions in sodium, increases in potassium, and decreases in magnesium; however, it caused significant increases in calcium.
2. Treatment two caused significant reductions in sodium and partly significant increases in calcium.
3. Treatment three caused significant decreases in sodium, partly significant decreases in magnesium, and partly significant increases in calcium.
4. Treatment four caused partly significant decreases in sodium, partly significant decreases in magnesium, and significant decreases in calcium.

One may conclude, then, that treatment one significantly changed the pore water concentrations of calcium to stabilize, and treatments two and three did this to a lesser degree. The highly variant nature of soil pore water chemistry is further supported by these results.

The changes to exchange complex cation concentrations may be summarized by treatment:

1. Treatment one caused partly significant increases in potassium and decreases to magnesium;
2. Treatment two caused partly significant decreases to sodium, increases to potassium, decreases to magnesium, and significant decreases to calcium;
3. Treatment three caused significant increases to calcium; and
4. Treatment four caused partly significant decreases to potassium.

One may conclude that, other than the changes caused by highly variant exchange complex chemistry, the only stabilizing effect was noted in changes to calcium caused by treatment three.

Of the treatments applied during this study, treatment one (single LSPI followed by a three-staged water injection) performed best in stabilizing this soil. The next most effective treatment was number two (double LSPI). Treatment three (double LFASPI) had some stabilizing effects.

One may also state with confidence that LSPI affects the pore water calcium concentrations between the lime seams, especially when followed by water injections. In addition, LFASPI affects the calcium concentrations in the exchange complex of the soil between LFA seams.

The only soil mass effects studied were change in ground surface elevation. Results of these studies support the use of LSPI for reduction of ground surface elevation change caused by climate.

The recommendations offered as a result of the findings of this study are as follows:

1. In order to properly investigate comparisons of soil stabilizing agents and methods, statistical analyses, such as those used in this study, are a necessity.
2. Investigations into the stabilizing effects of chemically acting soil stabilizers should include studies of changes in the soil chemical property.
3. The research reported on in this paper should be extended to optimize injection-stabilization agents and techniques to include injection spacing, injection depths, and agent concentrations.

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Soil-Cement for Use in Stream Channel Grade-Stabilization Structures

L.L. LITTON AND R.A. LOHNES

Numerous streams in the loess hills of western Iowa are entrenching their channels, consequently there is a need for economical grade-stabilization structures to control this erosion. Soil-cement has been suggested as a possible low-cost construction material. A study was undertaken to determine the erosion resistance of cement-stabilized alluvium when subjected to water velocities equivalent to velocities over small drop structures in drainage basins that have areas less than 26 km² (10 mile²). A second objective was to compare erosion resistance of freeze-thaw specimens with durability as measured by the currently accepted brush test. Erosion and brush tests were conducted on alluvium-cement and alluvium-sand-cement mixtures. Laboratory erosion tests, at jet velocities less than 6.0 m/s (20 ft/s), result in lower weight losses than do brush tests of the same mixtures. The results of the two test methods, in terms of the selection of a cement content, are comparable when the erosion test is conducted at a velocity of 6 m/s (20 ft/s); however, the maximum weight losses are considerably higher for the erosion tests than for the brush test. As anticipated, increasing the sand and cement contents produces more durable soil-cement mixtures regardless of the test method. These laboratory results suggest that anticipated channel flows and velocities should be considered in the economical design of soil-cement for a grade-stabilization structure.

Stream channels in the loess hills of western Iowa have been entrenching as much as five times their original depth since the latter part of the last century. The degradation of the channels has been accompanied by widening as side slopes become unstable and mass movement occurs. For example, the Willow River drainage ditch as constructed in 1919 was 4.6 m (15 ft) deep and 6.7 m (22 ft) wide, but by 1958 the channel was 9.8 m (32 ft) deep and 21 m (70 ft) wide (1). The deepening and widening of these streams has jeopardized highway and railroad bridges by undercutting footings and pile caps, exposing considerable length of piling, and removing soil beneath and adjacent to abutments.

Various types of flume and drop structures have been used to stabilize these channels. Although a need has always existed for economical grade stabilization structures to protect bridges and culverts, the problem is especially critical at the present time because of rapidly increasing construction costs and decreased highway revenues. The cost of reinforced concrete drop structures constructed in western Iowa within the last two years has been as high as $66 000/m ($20 000/ft) of fall. Use of riprap is not feasible because of high cost and poor durability of locally available rock. Soil-cement has been suggested as an economical alternate construction material, especially in structures on smaller streams (2).

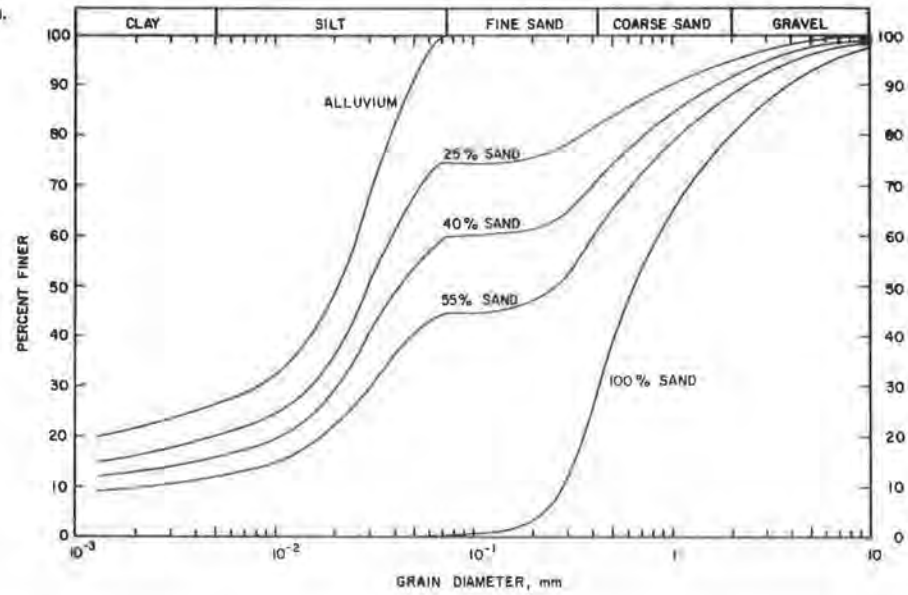
The use of soil-cement in water control structures dates back to 1951, when a test section was constructed as slope protection against wave erosion on the southeast shore of Bonney Reservoir in Colorado (3). The earliest application of soil-cement for protection against slope erosion in full-scale construction was at Merritt Dam, Nebraska, in 1961. Subsequent water-control applications of soil-cement include reservoir linings, small auxiliary spillways, highway embankment protection along rivers, dam diversion channels, and tailraces (4). The range of cement content used in these structures varies from less than 7 to more than 14 percent by weight of dry soil (3).

A major distinction between soil-cement design in water-control structures and in highways is that, for the former, durability is more important than strength. The durability of soil-cement is normally evaluated by wet-dry and freeze-thaw tests (ASTM D559-57 and D560-57 or AASHTO T135-57 and T136-57). The Portland Cement Association (PCA) recommendation for water-control structures is that the required cement content be 2 to 4 percent greater than the percentage necessary to meet the freeze-thaw and wet-dry criteria for brush loss used for highway applications (5). Research employing water jet and wave tank tests to simulate erosive forces indicates that, if portions of the structure are subjected to milder exposures, cement content may be reduced below the standard requirement (6). Other recommendations regarding soil-cement for water resources applications include central plant mixing, compaction to a minimum of 95 percent maximum density, and limiting the soils to material that contains not less than 55 percent passing the No. 4 sieve and not more than 35 percent or less than 5 percent passing the No. 200 sieve (7).

The need for economical construction material for grade-stabilization structures in western Iowa and the somewhat arbitrary nature of the standard brush test suggest that research on cement-stabilized, loess-derived alluvium is needed. The objective of this research is to determine the erosion resistance of cement-stabilized alluvium under water velocities that are the same as the velocities over small drop structures situated in the smaller watersheds of western Iowa. For drainage basins about 26 km² (10 mile²) in area and flood flows that have 10-50 year recurrence intervals, the velocities expected over 0.6- to 3-m (2- to 10-ft) drops range from 4.5 to 10.5 m/s (15-35 ft/s). Normal velocities in the stream channels would be lower so soil-cement specimens were tested at velocities that range from 1.5 to 7.5 m/s (5-25 ft/s).

The loess-derived alluvium selected for testing is a loam typical of a alluvium from western Iowa. None of this alluvium meets PCA gradation requirements. The erosion resistance of silty cement-stabilized soils can be increased by blending the soil with sand (6); therefore, mixtures of sand and alluvium were evaluated. The sand is typical of that available in the study area. If the sand were used in the grade-stabilization structures, it would almost meet the PCA specifications, so tests were run on the sand to provide a basis for comparison. Cement contents of the test specimens ranged from 5 to 13 percent.

Figure 1. Soil mixture gradation curves.



The stabilized specimens were subjected to both freeze-thaw and wet-dry testing. Hydraulic erosion tests on the wet-dry specimens resulted in negligible weight loss; therefore, only the results of the freeze-thaw tests will be discussed here. Details and results of the wet-dry tests on these mixtures can be found elsewhere (8).

TEST METHODS AND SPECIMEN PREPARATION

The alluvium, with an American Association of State Highway and Transportation Officials (AASHTO) classification of A-6, was obtained from the site of a future grade-stabilization structure. The gradation of the raw materials as well as the alluvium-sand blends used in the testing program are shown in Figure 1. Alluvium-sand mixtures that contained 25, 40, and 55 percent sand as well as sand were used to test the effects of sand content on the durability of soil-cement. Cement contents of 5, 7, and 9 percent, calculated as a percentage of the dry weight of the alluvium-sand mixture, were tested in all blends. The alluvium was tested with 5, 7, 9, 11, and 13 percent cement. The portland cement used in all specimens was type 1.

Rectangular soil-cement beams of the various alluvium-sand-cement ratios were used for the hydraulic load tests. The 76.5x76.5x200-mm (3x3x7-7/8-in) beams were compacted to 100 percent maximum density at optimum moisture content as determined in accordance with ASTM D558. Optimum moisture contents and maximum dry densities are given in the following table (note: 1 gm/cm³ = 0.004 lb/in³):

Soil Mixture (% by weight)			Optimum Moisture Content	Dry Density
Alluvium	Sand	Cement	(%)	(gm/cm³)
100	0	9	20.4	1.58
75	25	7	15.8	1.77
60	40	7	13.6	1.88
45	55	7	11.6	1.92
0	100	7	9.0	1.94

Soil-cement for the test beams was thoroughly mixed by hand before being placed as a single lift in a modified flexural beam mold. Compactive effort was applied from one side with a universal testing machine. After molding, the test beams were cured at 21°C (70°F) and 100 percent relative humidity for seven days.

One set of soil-cement specimens was tested in accordance with ASTM D560 (freezing-and-thawing tests of compacted soil-cement mixtures). Replicate soil-cement test beams were subjected to hydraulic erosion tests. The complete series of beams was subjected to 12 freeze-thaw cycles, each of which consisted of 24 h in a freezer at -20°C (-4°F) followed by 24 h in a humidity room at 21°C (70°F).

The erosion test was designed to simulate the velocities, hence the forces, anticipated from a free overfall of water. The velocities used were 1.5, 3.0, 4.5, 6.0, and 7.5 m/s (5, 10, 15, 20, and 25 ft/s). The relative durability of the test beams was expressed as a percentage weight loss. The beams were surface-dried and weighed after 60 min exposure to the water jet at a constant velocity. A normal testing sequence consisted of subjecting the beams to erosion by the lowest test velocity for a 60-min duration then increasing the velocity for each subsequent test. Long-term tests, up to 7 h, have indicated that 81-97 percent of the total loss occurs within the first hour of testing at a given velocity.

The apparatus used for applying the erosive force is shown in Figure 2. The curved plexiglass side panels funnel the water from the upper tank to the 12.7x406.4-mm (1/2x16-in) discharge slit. The test beams are located 152.4 mm (6 in) beneath the discharge slit in a removable sample box. Water is supplied to the test apparatus from a constant head tank to ensure consistent flow rates. The control valve was calibrated for various flow rates by discharging the flow into a weighing tank.

TEST RESULTS

The results of the brush tests on the freeze-thaw specimens (ASTM D560) are shown in Figure 3, where percentage weight loss is plotted versus percentage cement content. Because the PCA-recommended allowable weight loss for A-6 soil is 7 percent, none of the specimens of cement-stabilized alluvium is acceptable. The maximum weight loss in the brush test is 32 percent from the alluvium with a 5 percent cement content. Although the addition of sand to the alluvium changes its classification, the 7 percent maximum weight loss is used as the criterion

Figure 2. Erosion testing apparatus.

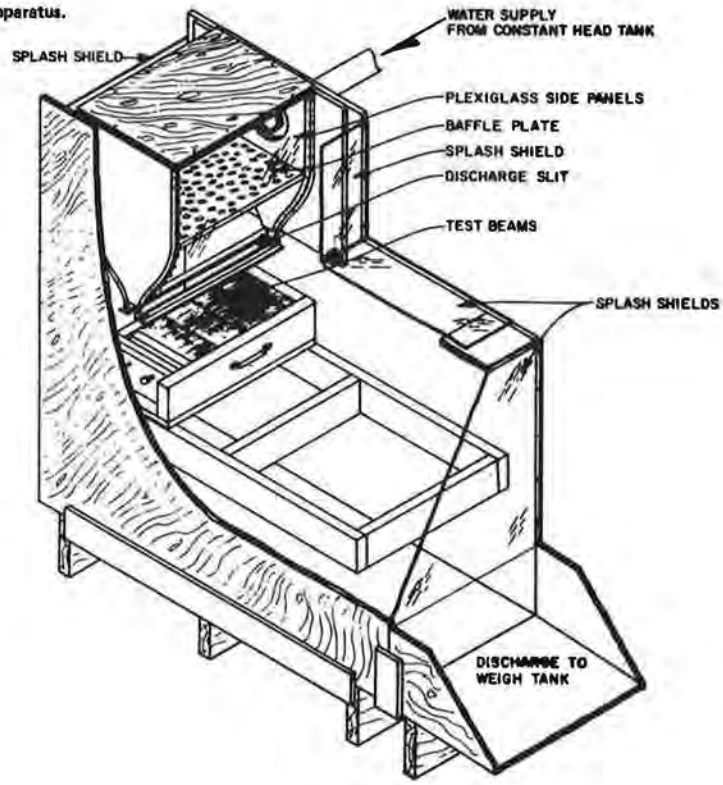
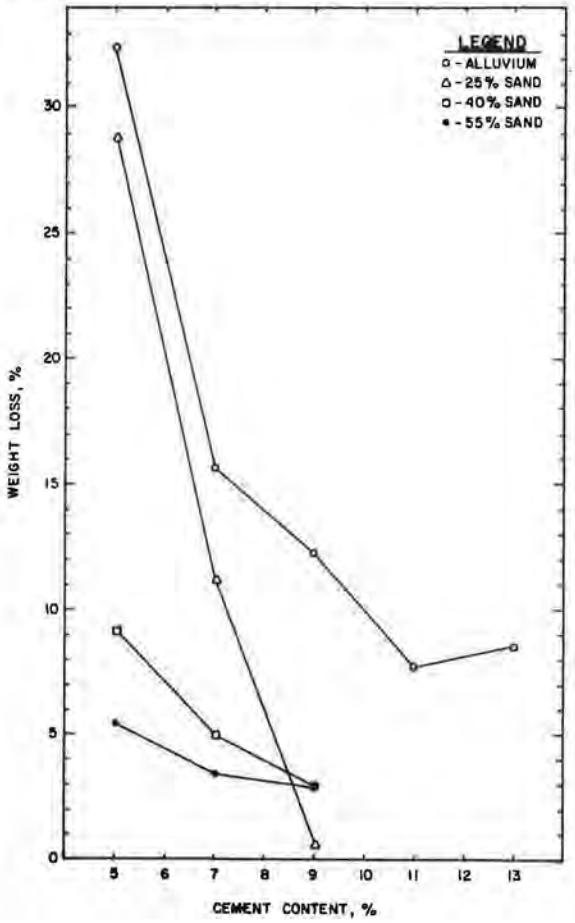


Figure 3. Weight loss versus cement content for brush tests.



for selecting minimum cement content of all the alluvium-sand mixtures. The alluvium-sand mixture that contains 25 percent sand is acceptable with a cement content about 8 percent, and the 40 percent sand mixtures is acceptable at about 6 percent. With 55 percent sand mixed with the alluvium, the mixture shows acceptable weight loss with cement contents as low as 5 percent. In this paper, the minimum cement contents indicated by the brush tests are compared with the results of the erosion tests.

Figures 4 and 5 are graphs of cement content versus weight loss for the stabilized mixtures at various erosion velocities. For a velocity of 6 m/s (20 ft/s), a weight loss of 100 percent occurred for alluvium stabilized with 7 percent cement and a weight loss of 65 percent occurred for alluvium that contains 9 percent cement. These losses are greater than the maximum lost in the brush tests. At a velocity of 4.5 m/s (15 ft/s), weight loss ranged from 28 to 8 percent for cement contents of 5-13 percent. For lower velocities, weight loss meets maximum acceptable levels at about 9 percent cement at 3 m/s (10 ft/s) and 7 percent cement at 1.5 m/s (5 ft/s), as shown in Figure 4.

Figure 4 shows that, at velocities below 3 m/s, all of the alluvium-sand-cement mixtures have weight losses below the allowable limit. At the 4.5 m/s velocity the 40 percent sand mixture has maximum allowable weight loss with about 5 percent cement, whereas the 55 percent sand content has allowable weight losses at all cement contents tested.

The 55 percent sand mixture has acceptable weight losses for all cement contents at 6 m/s velocity and for cement contents above 6 percent at 7.5 m/s (25 ft/s), as can be seen in Figure 5. The 40 percent sand content mixture has acceptable weight losses above 6 percent cement content at 6 m/s and above 9 percent at 7.5 m/s.

For the sand at all velocities and all cement contents, the weight loss was negligible; conse-

Figure 4. Weight loss versus cement content for erosion tests with velocities of 1.5, 3, and 4.5 m/s.

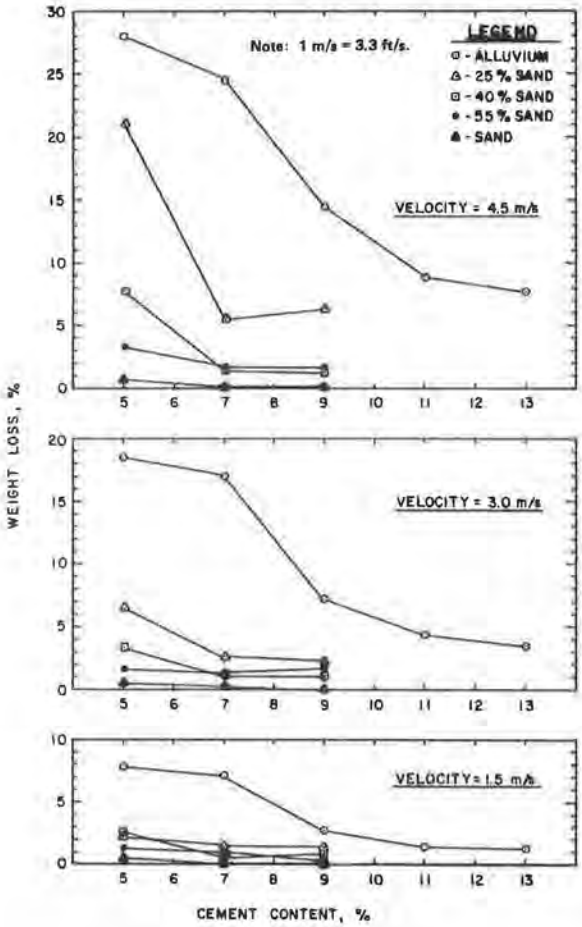
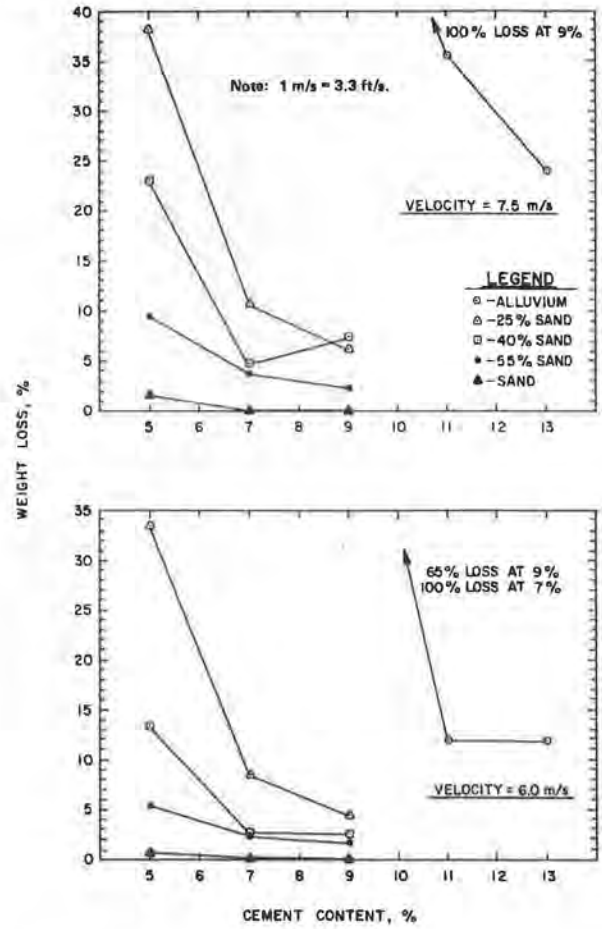


Figure 5. Weight loss versus cement content for erosion tests with velocities of 6 and 7.5 m/s.



quently, the minimum cement content is less than 5 percent. Table 1 summarizes the minimum cement contents as indicated by the brush and the erosion tests at the various velocities. Comparison of the results of the two methods of testing shows that, for velocities less than 6.0 m/s (20 ft/s), the brush test may be too conservative and result in uneconomical design mixes. At higher velocities, the brush method may be a reasonable criterion for selecting a minimum cement content; however, at velocities greater than 6 m/s, the maximum percentage of soil lost in the hydraulic tests far exceeds the maximum lost in the brush tests. This latter observation suggests that the brush test may not simulate the amount of soil lost under more severe channel erosion.

An alternate analysis of the data shows the relationship of erosion velocity to the durability of the soil-cement specimens. Figure 6 is a plot of percentage weight loss versus erosion velocity for the cement-stabilized alluvium at constant cement contents. For specimens stabilized with 5 and 7 percent cement, a nearly linear relation between weight loss and velocity exists up to a velocity of about 4.5 m/s (15 ft/s). Above that velocity the erosion loss increases almost exponentially. Similarly, for 11 and 13 percent cement the rate of loss is lower up to a velocity of 6 m/s, above which the percentage of loss per unit of velocity increases abruptly. A 9 percent cement content test was not run at velocities greater than 4.5 m/s. The relatively low rate of loss for lower velocities indicates the possibility of a threshold velocity below which losses may be tolerable and above which losses become excessive. This suggests the possibility of a more rational criterion for determining the allowable weight loss. Similar trends also appear in Figure 7 for the alluvium-sand mixtures with 5 percent cement. If the higher cement content mixtures have a threshold velocity, it is above the highest velocity of 7.5 m/s (25 ft/s) at which the specimens were tested.

Table 1. Minimum allowable cement contents for weight loss of 7 percent.

Soil Mixture	Minimum Allowable Cement Contents (%)					
	Brush Test	Erosion Test Velocities				
		1.5 m/s	3 m/s	4.5 m/s	6 m/s	7.5 m/s
Alluvium	>13	7	9	>13	>13	>13
Alluvium-25 percent sand	8	<5	<5	7	8	9
Alluvium-40 percent sand	6	<5	<5	5.5	6	9
Alluvium-55 percent sand	<5	<5	<5	<5	<5	6
Sand		<5	<5	<5	<5	<5

If the PCA criterion of a maximum of 7 percent loss is used as a limiting criterion, a maximum allowable velocity may be defined as the velocity at which the erosion loss for a given cement content equals 7 percent. The maximum allowable velocities are shown in Table 2. The maximum allowable velocity increases with increasing cement and sand content. The foregoing analysis suggests that

Figure 6. Weight loss versus erosion velocity for alluvium-cement mixtures.

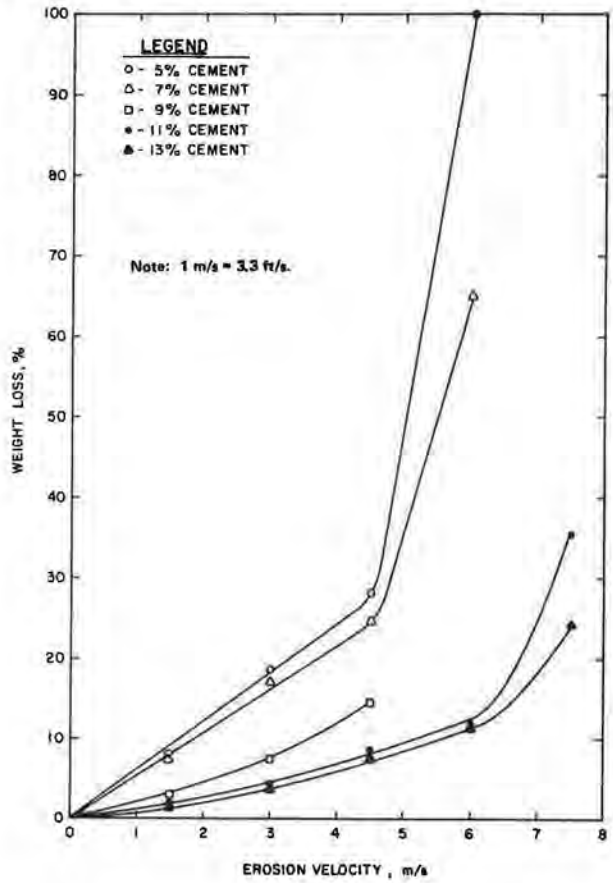
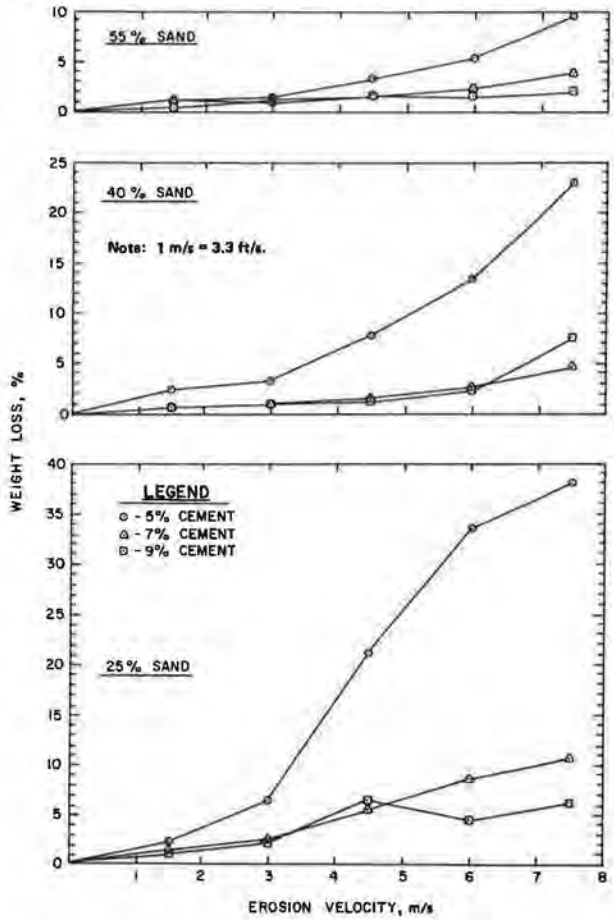


Figure 7. Weight loss versus erosion velocity for alluvium-sand-cement mixtures.



channel velocity is an important variable that should be taken into consideration when designing a mix for soil-cement in grade-stabilization structures.

DISCUSSIONS AND CONCLUSIONS

Current practice limits design of water control structures by using soils stabilized with portland cement to sandy soils. The procedure for selecting cement contents is a modification of the procedure used for highway design. In the case of the grade-stabilization structures designed to retard or stop channel degradation on small streams, current practice may be too conservative and lead to uneconomical structures. On the other hand, if high velocities are expected, current practice may not be conservative enough. A statistical probabilistic approach to design is unacceptable when applied to a dam or bridge in a populated area (9); however, the failure of a grade-stabilization structure on a small stream is unlikely to have immediate, devastating effects. Some consideration should be given to the use of materials that may be unacceptable in a large dam but may provide a reasonably long life and a realistic risk factor for a low-head grade-stabilization structure. The design of such a structure should consider the durability of the materials in terms of flow velocities and recurrence intervals in the channels into which they are placed.

Erosion tests at velocities less than 6 m/s (20 ft/s) on cement-stabilized alluvium and alluvium-sand mixtures from western Iowa result in lower weight losses than do brush tests on the same mixtures. At hydraulic velocity of 6 m/s, the cement contents that produce less than 7 percent weight loss are comparable to cement contents determined from the brush tests; however, at this and higher velocities, the hydraulic tests result in greater maximum losses than the losses produced by brushing. As expected, the addition of sand to the alluvium results in greater durability and less erosion at equivalent cement contents. Current design practice for water-control structure precludes the use of cement for stabilizing western Iowa loess-derived alluvium. However, this study indicates that, at low erosional velocities, cement-stabilized alluvium may be an economical and reliable construction material for grade-stabilization structures in small watersheds.

Table 2. Maximum allowable erosion velocity for weight loss of 7 percent.

	Maximum Allowable Velocity (m/s)				
	Cement Content				
Soil Mixture	5 Percent	7 Percent	9 Percent	11 Percent	13 Percent
Alluvium	1.2	1.5	2.7	4.3	4.6
Alluvium-25 percent sand	3.0	5.2	>7.5		
Alluvium-40 percent sand	4.1	>7.5	>7.5		
Alluvium-55 percent sand	6.5	>7.5	>7.5		
Sand	>7.5	>7.5	>7.5		

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Abridgment

Reaction Products of Lime-Treated Southeastern Soils

CHARLES M. FORD, RAYMOND K. MOORE, AND BENJAMIN F. HAJEK

Six soils series (Cecil, Chewacla, Eutaw, Sumter, Tatum, and Wilcox) of the southeastern United States were investigated by using x-ray diffraction analysis, thermogravimetric analysis, and scanning electron microscope. The study compared the natural soil with lime-treated soil (by using 6 percent high-calcium-hydrated lime) after a 48-h accelerated curing period at 49° C (120° F). The compaction specimens were prepared in a Harvard miniature compaction mold by using impact compaction and were sealed in plastic wrap during the curing phase to prevent moisture loss. Lime-soil reaction products of calcium oxide-alumina oxide-hydrate (C_4AH_3), C_3AH_6, calcium-silicate-hydrate (CSH) (gel) and CSH II were identified, although a different mixture of products was associated with each soil. Unknown products were also noted on the thermogravimetric analysis data at 440°, 450°, and 460° C. Both absorbed-solution and through-solution mechanisms appear to be involved in the formation of cementitious material. When compared with lime reactivity (i.e., unconfined compressive strength gain following the accelerated curing), the results for the montmorillonite dominated soils (Eutaw and Wilcox) suggest that excessive specific surface is a detriment to the development of significant cured unconfined compressive strength gains.

Six fine-grained soil series characteristic of those found in the southeastern United States were investigated by using x-ray diffraction, thermogravimetric analysis (TGA), and the scanning electron microscope (SEM) to determine the nature of lime-soil reaction products. Selected soil morphology, engineering physical property, and lime-reactivity data are presented in Table 1 (1-3). The soils exhibit a wide range of lime reactivity with the lowest strength gains noted for the montmorillonitic soils (Eutaw and Wilcox).

PREPARATION OF SOIL-LIME SPECIMENS

Specimens were prepared for SEM, x-ray diffraction, and TGA after an accelerated curing sequence (4). The soils were air dried, then dry mixed by hand with 0, 2, 4, and 6 percent high-calcium-hydrated lime by dry weight of the soil. Distilled water was added to each soil to achieve moisture contents approximately that of optimum for the lime-treated soil. Samples were compacted in a Harvard miniature mold with a 0.53-lb impact compaction hammer in 3 layers by using 25 blows/layer. Immediately after removal from the mold, the compacted specimens were sealed with plastic wrap to prevent moisture losses and then cured at 49°C (120°F) for 48 h.

SOIL TESTING PROCEDURES

X-Ray Diffraction

X-ray patterns for soils before and after treatment with 6 percent high-calcium-chemical lime were obtained by using a Norelco x-ray diffraction unit with a copper tube. For a detailed discussion of x-ray diffraction theory, see Jackson (5).

TGA

A Dupont 951 thermogravimetric analyzer and a Dupont 990 thermal analyzer and record console were used for TGA. About 10 mg of the entire soil sample were used. The sample was heated from 25° to 800°C at a constant rate of 20°C/min. Weight loss was a result of the release of surface water and structural hydroxyls. Minerals loose these hydroxyls within specific ranges of temperatures and at constant percentages of weight. Therefore, some minerals (gibbsite and kaolinite) may be identified quantitatively (5).

SEM

An AMR-100 SEM was used on the soils at various magnifications.

DISCUSSION OF RESULTS

Cecil

A sharp reduction in kaolinite and illite x-ray diffraction peaks indicated some degradation of those minerals. This was supported by TGA results where kaolinite peaks at 500°C were reduced in size. The new peak (reaction products of lime treatment) was identified by Glenn (6) at 140°C to be calcium-alumina oxide-hydrate (C_4AH_{13}), calcium-silicate-hydrate (CSH) (gel), and CSH I. Data by Ruff and Ho (7) preclude the existence of the latter. The peak at 230°C indicates that C_4AH_{13} makes up at least part of the 140°C weight loss. The new mineral found at 320°C was identified as C_3AH_6 (6). The 670°C peak could

Table 1. Soil properties.

Soil Series	Family	Natural pH	Liquid Limit (%)	Plastic Limit (%)	Percentage Passing No. 200	Unconfined Compressive Strength (lb·f/in²) 0 Percent Lime	6 Percent Lime	Lime Reactivity (lb·f/in²)
Cecil	Clayey, kaolinitic	4.9	52	33	85	20	184	164
Chewacla	Fine-loamy, mixed, thermic	8.2	24	17	43	48	201	153
Eutaw	Very fine, montmorillonitic, thermic	5.4	70	37	99	57	60	3
Sumter	Fine-silty, carbonic, thermic	8.4	51	24	88	59	90	31
Tatum	Clayey, mixed, thermic	4.3	33	24	83	36	77	41
Wilcox	Fine, montmorillonitic	4.3	72	42	90	12	14	2

Note: 1 lb·f/in² = 6.894 kPa.

correspond to the 700°C mineral described by Glenn as CSH II.

SEM photographs of the natural soil indicated a matrix of angular plates that grade from silt to clay particles of about 0.1 μm in diameter. The plates were in a card house or face-to-edge arrangement. The addition of 2 percent lime rearranged the particles into stacks. When Cecil was treated with 6 percent lime, the larger particles remained stacked, but the smaller clay plates were attached to these with their edges. Cementing products were not visible in the photographs. The edge-to-face structure of the soil supports the adsorbed state mechanism of stabilization described by Diamond and Kinter (8).

Chewacla

The x-ray diffraction pattern exhibited a large reduction of the 4.2 Å quartz peak and almost total destruction of the kaolinite peak. The reduction of the kaolinite was not so evident in TGA data, which indicated a breakdown in the crystal structure as well as a breakdown of the mineral itself. New minerals included CSH (gel) at 160°C and CSH II at 670°C. Another product was found at 440°C but was not identified in the literature.

The SEM photograph of natural Chewacla showed that clay platelets covered larger particles in a face-to-face arrangement. The addition of 2 percent lime produced an amorphous material that coated and bridged all particles. A cementing agent, which was apparently amorphous in nature since x-rays did not detect it, was clearly visible. This supports the presence of a through solution mechanism of stabilization (8).

Eutaw

The x-ray peaks in this soil were reduced, and the creation of a reaction product was shown at 18 Å. New peaks in the TGA data indicated the presence of C_3AH_6 and CSH II at 310° and 670°C, respectively.

By using SEM photographs, Eutaw in its natural state had a tissue-like structure (montmorillonite) throughout its matrix, and the clay particles dominated in the soil in a more or less random arrangement. The addition of 2 percent lime reduced the amount of montmorillonite but did not destroy it all. Clay-sized particles still had a random arrangement. Montmorillonite disappeared after the addition of 6 percent lime and clay platelets had a more oriented face-to-face pattern. There is no visual evidence of a cementing agent.

Sumter

Sumter reacted to the addition of lime by the reduction of x-ray kaolinite peaks. The TGA data indicated the creation of the 460°C mineral after a 2 percent lime treatment.

A distinguishing feature in the SEM photograph was the presence of skeletal remains of microscopic animals that are found in Selma Chalk, Sumter's parent material. These remains measured approximately 4 μm in diameter. Clay particles were in a face-to-face array. After 2 percent lime was added, the soil matrix became loose and random, and the shallow sea skeletons were still intact and abundant. A lime content of 6 percent oriented the clay plates into a face-to-face arrangement, and the skeletons disappeared from the sample.

Tatum

The kaolinite x-ray peak deteriorated with the addition of lime and a peak at 8.0 Å disappeared. Other peaks were reduced, but not as drastically. The 280°C TGA peak (gibbsite) was reduced while CSH (gel) and CSH II appeared at 150° and 670°C after lime treatment. The unknown 450°C was also present.

The SEM photographs indicated that natural Tatum consisted of irregularly shaped silt-sized and clay-sized plates. The small clay platelets exhibited a card house pattern. Two percent lime oriented the plates into stacks.

After the addition of 6 percent lime, the plates returned to the card house structure. Again, no cementing agent was observed. The card house structure and the moderate strength gains after the addition of lime could be caused by the adsorbed state process.

Wilcox

Both the two-to-one expandable minerals and the illite x-ray peaks almost entirely disappeared as a result of lime treatment. The quartz peak was reduced drastically. Gibbsite's TGA peak was reduced and a slight CSH II peak was the only observable new mineral.

The soil particles in natural Wilcox appear to be coated with montmorillonite in the SEM photographs. Most of the montmorillonite disappears after lime was added (2 percent), and the plates were in a face-to-face structure. The soil was rearranged into a card house structure after 6 percent lime was added. A trace of montmorillonite was still noted.

As with Eutaw, Wilcox has a high specific surface. Much of the lime was apparently used to break down the montmorillonite. The lime treatment created an edge-to-face arrangement, although the soil gained little strength after treatment and accelerated curing with 6 percent lime. The edge-to-face structure may be caused by the reduction of repulsive forces because of increased cation concentration. If actual stabilizing material had been

Table 2. Summary of results from TGA patterns.

Soil	Mineral	
	Reduced Peak[a]	New Peak[b]
Cecil	Gibbsite, kaolinite	C_4AH_{13}, CSH (gel), C_3AH_6, CSH II
Chewacla	Gibbsite, kaolinite	CSH (gel), 440°C (unknown), CSH II
Eutaw		C_3AH_6, CSH II
Sumter		460°C (unknown)
Tatum	Gibbsite	CSH (gel), 450°C (unknown), CSH II
Wilcox	Gibbsite	CSH II

[a]These peaks were found to decrease in intensity after the addition of 2 percent lime. The reduction continued with each increase in lime percentage.
[b]These peaks appeared in the TGA curves after the addition of 2 percent lime. They increase in intensity after each addition of lime.

produced through the adsorbed-state mechanism, strength gain would have been greater. Cementing materials could have been produced by the through solution process; however, the surface area was too large for the cementitious material to affect its unconfined compressive strength.

CONCLUSIONS

A summary of the lime-soil reaction product identifications is presented in Table 2. As noted, new TGA peaks were produced by reaction products at 440°C in Chewacla, 450°C in Tatum, and 460°C in Sumter that have not been identified.

The Eutaw and Wilcox soil series have clay fractions that are dominated by montmorillonite. The specific surface of the clay fraction is, therefore, quite high. Much of the lime was apparently used to break down or degrade the montmorillonite. The lime treatment created an edge-to-face arrangement, which may be created by the reduction of repulsive forces associated with double-layer compression in response to increased cation concentration. The net result in terms of unconfined compressive strength increase of lime reactivity, as defined by Thompson (4), was practically nil. If actual stabilization material had been produced through the adsorbed-solution mechanism, larger gains in strength would have been expected. Furthermore, the reaction products produced by the through solution process would also be ineffective because the surface area is too large for the cementitious material to create significant increases in unconfined compressive strengths, although C_3AH_6 and CSH II were detected following accelerated curing. Thus, it appears that too much fine clay will be detrimental to the development of lime reactivity even though soil-lime reaction products may develop.

Thompson (9) has stated that montmorillonite soils generally react well (in terms of lime reactivity) on the basis of a study that included 39 soils, but only three of the selected soils had montmorillonite percentages in excess of 20 percent of the total soil composition by weight. The Eutaw soil composition had 42 percent expanding 2:1 minerals (predominantly montmorillonite) and the Wilcox had 43 percent.

Cecil and Chewacla demonstrated significant gains in strength after lime treatment and accelerated curing. The SEM photographs suggest that adsorbed-solution mechanisms created the cementitious material for the Cecil, but Chewacla may have benefited from through-solution derived compounds. Although Tatum did not have as large an increase in strength, the SEM photographs suggest a through-solution cementitious material.

Subsequent research by Rosser and Moore (10) and Lockett (11) has determined that the moisture content used for the compaction of Sumter and Eutaw soil specimens as reported in this paper created a moisture deficiency for the lime-treated specimens. The effect of lime modification increased the plastic limit of Sumter by 20 points and of Eutaw by 10 points. Since the optimum moisture content of the lime-treated material would increase by approximately the same amount, the shift would mean that dry-side compaction was used for the lime-treated specimens for the Eutaw and Sumter soils. The use of a higher optimum moisture content (35 percent) for the Eutaw still produced no lime reactivity in terms of a gain in strength of unconfined compression. However, the Sumter lime reactivity was increased to 62 $lb \cdot f/in^2$ when a compaction moisture content of 41 percent was used. It appears that the high percentage of montmorillonite in the Eutaw still is accountable for the absence of strength gains. The response of the Sumter soils affirms the suspected moisture deficiency effect for that particular soil series.

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