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Transportation Research Record 501 was edited for Transportation Research Board by Joan B. Silberman.

CONTENTS

FOREWORD

Improvement in the structural properties of soil layers is generally considered to be stabilization whether or not additives are used. Compaction of soil satisfies this objective, and the paper by Majidzadeh, Guirguis, and Ilves presents a rapid method for evaluating the structural benefits of compaction.

Teng and Fulton report on a field evaluation program for cement-treated bases presenting as one of the preliminary conclusions a recommendation that the 7-day undisturbed curing period be altered to allow necessary local and construction traffic.

Duval and Alexander offer a procedure for soil-cement mix design that will be of special interest to secondary roads engineers and engineers for developing countries.

In their study of cold weather lime stabilization, Rosen and Marks found that the construction season could be extended in the fall providing the delay in strength gain is properly taken into account. This is based in part on the promise of autogenous healing of the soil-lime mixture during the next summer's curing season. Van Ganse describes immediate amelioration of wet cohesive soils by quicklime. One and one-half percent or less of quicklime by weight is usually sufficient to produce the desired effects. As an added benefit, a long-term strength gain is realized in most cases.

Although Shen and Akky deal with a cement-stabilized loam, the recommended test for erodibility would probably be appropriate for any stabilized soil.

This RECORD should be of interest to construction engineers and pavement evaluators as well as soils engineers.

RAPID METHOD OF SUBGRADE COMPACTION AND PERFORMANCE EVALUATION

Kamran Majidzadeh, Hani R. Guirguis, and George J. Ilves, Ohio State University

The dynamic modulus and its variations with time are predicted from dynamic deflections by using elastic layered theory. Field moisture and density variations are measured by using deep probe nuclear equipment. Undisturbed field samples were obtained just after construction and a year or two later. Laboratory testing showed agreement with field measurements, and it is concluded that the subgrade gains moisture after construction and suffers loss of density and modulus. Simulation of seasonal variations (saturation and freeze-thaw in an open system) was conducted in the laboratory, and the trends agreed closely with those obtained from the field.

IN the rational design of pavement structures, subgrade soil is considered as an important component of the system, and its performance or serviceability under repeated loading and weathering needs to be evaluated on a sound theoretical basis. For a rational design system, the performance characteristics of compacted subgrade soils should be known in terms of parameters that realistically describe the dynamic state of stresses, environmental variations, and related material properties throughout the life of pavement. These performance parameters include moisture-density relation, dynamic modulus or modulus of resilience, permanent deformation characteristics, and resistance to environmental factors. Of equal importance is the methodology of performance prediction and determination of the engineering properties associated with pavement serviceability. The development of a rapid means of field soil compaction quality control to ensure that desired engineering properties are achieved is also important.

The results of laboratory and field study of soil compaction and the relative significance of various performance parameters have been presented in detail (1, 2, 3). Similarly, the effects of moisture content, severe environmental factors, state of stress, and compaction process on the moduli response of subgrade soil have been studied. Part of the research results concerned with field and laboratory characterization of subgrade soils will be presented later. In this paper, however, the use of a nondestructive method of subgrade soil evaluation and the correlation of results with laboratory measured properties are discussed. An attempt is made to demonstrate the applicability of Dynaflect deflection measurements to performance evaluation of compacted subgrade soils.

SCOPE OF FIELD INVESTIGATION

This study is concerned with the development of a new or modified methodology that permits a rapid evaluation of soil compaction process and determination of pertinent design variables. To achieve this objective, five sites representing different geographical and climatic conditions were selected in Ohio. The subgrade compaction process and the construction activities were monitored, and raw and undisturbed materials were collected for laboratory soil characterization. The nature of the terrain and the road profile were also carefully reviewed to provide additional design inputs.

A number of observation stations were chosen in each project site so that information could be obtained with respect to variations of moisture and density with depth.

Publication of this paper sponsored by Committee on Compaction.

The moisture density readings were obtained just after construction and for a period of about 2 years. Each project site had about twelve 5-ft nuclear probe access tubes located just off the edge of the shoulder. Special care was given to the installation and maintenance procedures. Soil properties (i.e., moisture and density) were measured indirectly by radiation backscatter phenomena in both the moisture and density probes. Calibration curves were obtained from the manufacturer and from soil type calibration procedures developed by Moore and Haliburton (5). The density readings obtained were by volume and for wet density and were converted to weight and dry density basis.

Typical moisture versus depth curves for one station are shown in Figure 1. In addition to the moisture and density setup, thermocouples were also installed at various depths to record pavement temperatures.

The field observation also included the measurements of dynamic deflections for each station at chosen test sites. The Dynaflect equipment used in the field measurements consists of a dynamic force generator, a sensor assembly, and a calibration unit (geophone). The purpose of the system is to permit rapid and precise measurement of roadway dynamic deflections while the trailer is halted briefly at successive test locations. These deflections are sensed by a series of geophones located on a line perpendicular to the wheel axis (Fig. 2).

These dynamic deflection measurements were carried out as soon as the subgrade was compacted and proof-rolled prior to resurfacing. In such cases where completed subgrade remained unprotected (i.e., without surfacing for a period of time), periodic deflection measurements were conducted to detect changes in the subgrade support condition.

In each project site, by using the results of deflection measurements, a preliminary evaluation of subgrade support condition was made, and the subgrade support was then categorized into regions of expected, poor, fair, and satisfactory performance. Undisturbed soil samples were then obtained from each of these regions to be used for laboratory evaluation of soil performance parameters.

ANALYSIS

The following analysis procedures were pursued so that an interrelation between field and laboratory measured soil characteristics and a methodology for field soil compaction evaluation could be developed: (a) determination of in situ soil support condition, (b) validation of in situ measured properties, and (c) validation of laboratory-simulated field conditions.

Determination of In Situ Soil Support Parameters

Dynaflect-deflection measurements can be used to determine the subgrade soil support condition. The maximum deflection and the shape of the deflection profile are indicative of the relative stiffness of the subgrade soil. Figure 3 shows typical deflection profiles representing soils with poor and excellent support conditions. When the results of the deflection measurements are used, the dynamic modulus of subgrade soil can be calculated by means of computer programs developed for this purpose.

This method of analysis uses the multilayer elastic theory, which has been extensively used for determination of stresses and displacement in pavement layered systems. According to this method, the moduli of the pavement layers and in some cases the layer thickness are calculated by using measured surface deflections. A number of other investigators (4) have already presented similar analysis techniques and programs for pavement moduli determination, which require that the thickness of the pavement structure be known and a Poisson's ratio of 0.5 be assumed.

However, the method of analysis and the computer program developed in this paper deal with the pavement structure system more realistically and provide more flexibility in the selection of design variables. Specifically, this method can estimate the thickness of the compacted subgrade and deals with materials with Poisson's ratios different from the ideal 0.5. It also has a considerably reduced execution time. In this analysis, the layers are assumed to be homogeneous and isotropic elastic mate-

Figure 1. Pavement moisture content and depth.

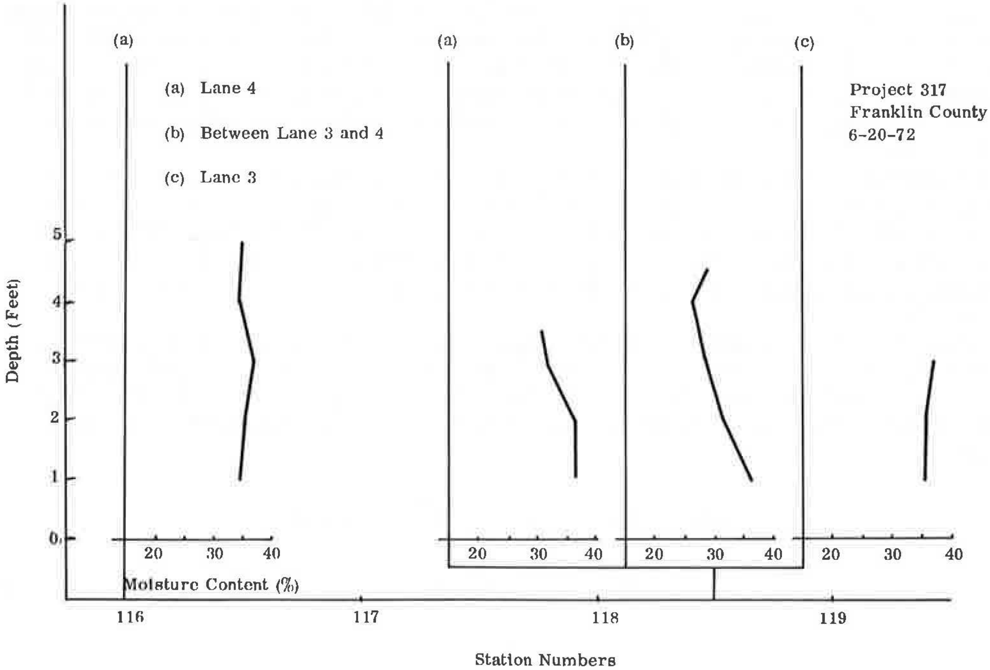
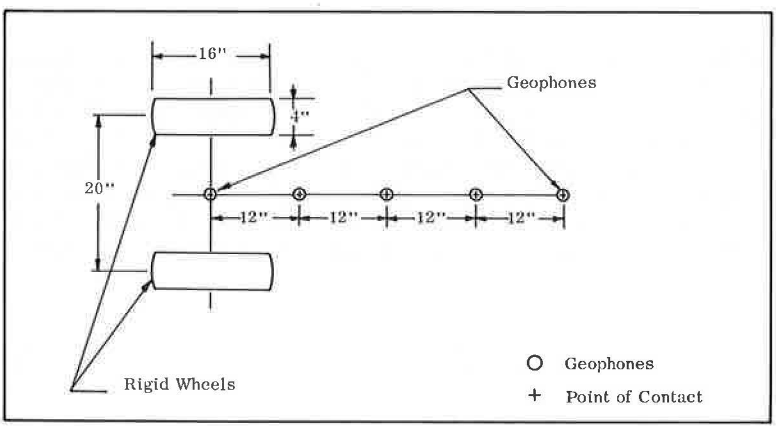


Figure 2. Sensor array.



rials and infinite in extent. The bottom layer is assumed to be infinite in depth. The interfaces between layers are considered rough so that the assumption of continuity of stresses and displacements is fulfilled. The pavement surface is also assumed to be stress-free except for the vertical Dynaflect load distributed over a circular area of known radius. The method of solution is the stress function approach, which leads to and can be evaluated by a relation between deflection, stress, material, and geometrical variables.

The calculation of moduli from surface deflections is considerably more complicated than the determination of deflection from known material and geometric conditions. The complication is caused by the nonlinear inhomogeneous functional relation existing between modulus and deflection. A number of methods exist for the solution of such nonlinear functions, among which is Newton-Raphson's method, which offers more promise.

An inspection of the deflection integral equation, as shown in Eq. 1, indicates that the ratio of deflection w_i/w_j is independent of moduli and only depends on Poission's ratios, layer thickness, and the modulus ratio, $L_1 = (1 + \nu_1)/(1 + \nu_2) \cdot E_2/E_1$. This representation reduces the number of variables by one and considerably simplified calculations.

$$w_2(r) = \frac{1 + \nu_1}{E_1} a \int_0^{\infty} \frac{1}{m} J_0(mr) J_1(ma)$$

$$f(m,\ h_1\ \nu_1,\ \nu_2\ L_1)\ dm \tag{1}$$

where

a = load radius,
h_1 = layer thickness,
ν_1 and ν_2 = Poisson's ratios, and
J_0 and J_1 = Bessel's functions.

Application of the moduli calculation program to subgrade soils, however, requires assumptions differing from those used in the analysis of pavement layered systems. As was pointed out earlier for design purposes, the subgrade soil is often considered as a homogeneous, isotropic elastic layer of infinite depth. In such cases, the analysis of a pavement system requires only independent determination of the modulus and Poisson's ratio. A subgrade soil represented by a one-layer system exhibits a unique deflection profile that is characterized by a spreadability ratio of 49.9 percent. The spreadability is defined as the average deflection expressed as a percentage of maximum deflection and is given by

$$\text{SP percent} = 20 \sum_{i=1}^{5} w_i/w_1 \tag{2}$$

where

w_1 = maximum deflection and
w_i = deflection of sensors 1 to 5.

The spreadability ratio is a complex function of moduli ratio E_1/E_2, pavement thickness, and Poisson's ratio. The higher spreadability ratios are indicative of greater system rigidity and ability to distribute load.

The experimental data indicate that, in subgrade soil compaction evaluation, the assumption of a one-layer system is not always justified, that is, in most instances, the

Figure 3. Dynaflect-deflection profile.

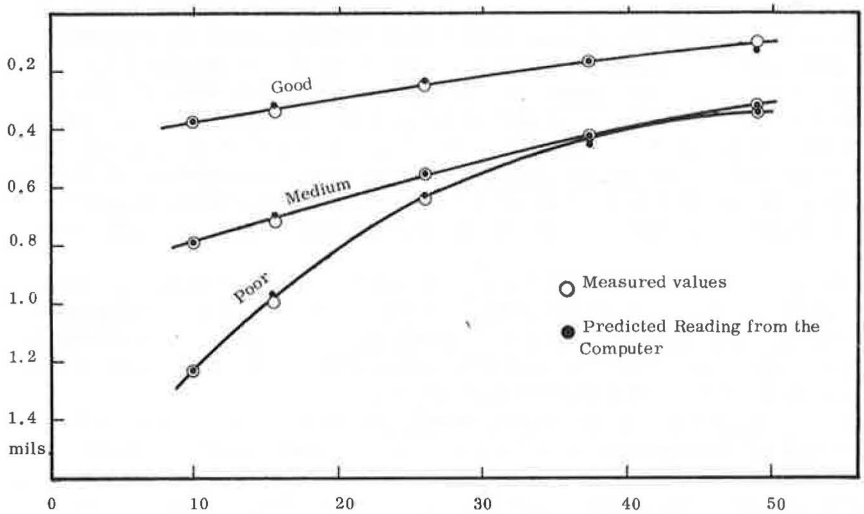


Table 1. Comparison of laboratory and field determined parameters (Pike County SR-124).

Measurements	Date	Location Depth (in.)	Water Content, W/C (percent)	Density, γ_d (pcf)	Modulus, E* (10^3 psi)
Undisturbed field samples	8/15/72		14.6		19.0
		15.0 to 20.0	14.5		16.2
		8.0 to 15.0	16.1		14.0
	4/24/73	6.0 to 12.0	16.7	114.7	7.0
		5.0 to 10.0	18.7	109.8	6.2
		20.0 to 26.0	23.9	102.2	3.3
		2.0 to 8.0	19.2	110.2	5.0
		12.0 to 16.0	18.9	109.8	7.2
Field (nuclear and Dynaflect)	11/11/72	2 ft avg.	23.0	114.5	15.0
	3/21/73	2 ft avg.	22.0	105.0	

Note: 1 in. = 0.0254 m; 1 pcf = 16.018 46 kg/m^3; 1 psi = 6.894 757 kPa.

Table 2. Comparison of laboratory and field determined parameters (Franklin County I-70).

Measurements	Date	Location Depth (in.)	Water Content, W/C (percent)	Density, γ_d (pcf)	Modulus, E* (10^3 psi)
Undisturbed field samples, Station 223+15	7/2/72		8.6	129.9	16.7
			8.9	122.0	25.1
			6.4	123.8	17.7
	4/16/73	8.0 to 15.0	8.1	137.0	10.0
		18.0 to 25.0	7.7	137.6	22.6
		4.0 to 11.0	11.2	123.8	6.8
		11.0 to 22.0	6.1	139.3	18.2
		4.0 to 10.0	9.0	130.7	9.5
		14.0 to 19.0	8.2	131.0	27.6
Field (nuclear and Dynaflect), Station 212+11	7/20/72		9.3	124.4	
	4/16/73	8.0 to 12.5	14.7	120.6	5.1
		19.0 to 24.0	15.5	138.1	9.4
		8.0 to 15.0	12.7	125.4	7.9
		6.0 to 12.0	15.2	113.6	8.9
		12.0 to 18.0	15.3	116.3	7.2

measured deflection data exhibit characteristics of multilayer structure. Whenever soil is compacted on the dry side of optimum moisture content, the top few inches of soil exhibit greater stiffness and better load distribution capacity than the remaining soil media. This results in a greater apparent slab action by the subgrade soil and a higher spreadability value. In such cases, the analysis of deflection data results in two modulus values E_1 and E_2, where E_1 might be many times greater than E_2. Under such conditions, often the strength and quality of the upper few inches of subgrade conceal the real weaknesses of the underlying layers. Surface observations of moisture and density measurements cannot reflect weaknesses of the soil support characteristics. Variations of subgrade modulus in depth can be determined by using undisturbed soil specimens.

Similarly, the analysis of deflection data can yield information pertaining to the relative support characteristics of various subgrade strata. Generally, a relatively high maximum deflection is indicative of a low E_2 modulus, whereas the spreadability or surface curvature index (sci), which is the difference between the deflection of the first two sensors, is indicative of the moduli ratio E_1/E_2.

Contrary to the previous case, during the subgrade compaction process in some instances, the upper part of the compacted soil might attain support characteristics smaller than lower strata. In such cases, the subgrade is represented by a layer system of $E_1 < E_2$ where the thickness of the uppermost layer might vary from a few inches to a few feet.

The analysis program developed for use in this study is also applicable, as was indicated previously, to subgrades consisting of two strata differing in moduli and with an unknown layer thickness. By using deflection data, the program attempts to find the unknown parameters h_1, E_1, and E_2 satisfying the mathematical requirements of the theory. Depending on the h_1, E_1/E_2 ratio, the solution might not be always unique but at most two solutions exist. The program attempts to find all the possible solutions. Determination of variables is achieved by comparing the experimental and calculated deflection data. This fit can be carried out using either maximum deflection and deflection of sensor 2, deflection of sensors 1 and 3, or sensor 1 and spreadability. Figure 3 shows the comparison of the calculated and experimental data.

These analyses can also be carried out with a least squares fit to the deflection data. However, this is rather expensive, and the analysis time is many times greater than with other procedures.

The modulus values calculated using deflection data are given in Tables 1, 2, and 3. Figures 4, 5, 6 and 7 show typical variations of maximum deflection and sci along two typical roadways. Figures 4 and 5 show the maximum deflection and sci of a section of roadway showing localized problems. As is indicated, the overall subgrade deflection is rather low except for the stations between 1568 and 1573, where erratic subgrade response is noted. Figure 6 shows the maximum deflection of another section of the same project and extreme variability throughout the entire length of the section is indicated. Figure 7, on the contrary, represents the deflection response of a subgrade exhibiting good support characteristics as well as excellent material uniformity.

The correlation of measured deflection and in situ field condition has frequently shown that field problem areas can be easily detected with these measured parameters. And extreme variability in the deflection measurements is more often observed in the cut to fill transition sections than in the fill sections.

Validation of Field Measurements

To check the validity of in situ dynamic modulus measurements required that undisturbed subgrade samples be obtained soon after construction and after 1 or more years of service. The in situ measurements of moisture and density and the analysis of undisturbed samples indicated seasonal changes in the physical and engineering properties of subgrade soil. As given in Tables 1, 2, and 3, most subgrades gained moisture after construction and suffered loss of density and modulus. Undisturbed soil samples were obtained at a few stations for each site. The results of analysis,

Table 3. Comparison of laboratory and field determined parameters (Gallia County SR-554).

Measurements	Date	Location Depth (in.)	Water Content, W/C (percent)	Density, γ_d (pcf)	Modulus, E* (10^3 psi)
Undisturbed field samples	10/31/72	1.5 to 5.0	12.0	120.4	10.5
		10.5 to 15.0	9.1	130.4	10.1
		1.5 to 6.0	12.7	91.3	10.1
		6.0 to 10.0	14.3	121.3	9.7
		15.0 to 21.0	10.7	126.5	10.1
		2.0 to 8.0	7.6	127.7	
	4/26/73	10.0 to 12.0	14.6	116.3	2.4
		13.0 to 18.0	15.7	112.8	3.6
		18.0 to 22.0	16.6	110.1	5.1
		7.0 to 12.0	16.7	114.8	2.9
		14.0 to 19.0	18.3	108.1	3.8
		20.0 to 25.0	19.7	112.2	5.3
		6.0 to 12.0	18.1	113.7	2.6
		22.0 to 27.0	13.6	123.6	7.9
Field (nuclear and Dynaflect)	10/31/72	2 ft avg.	15.5	113.5	
	4/26/73	2 ft avg.	17.7	Not available	

Figure 4. Maximum deflection and sci of a test section of roadway showing localized problems (less erratic subgrade response).

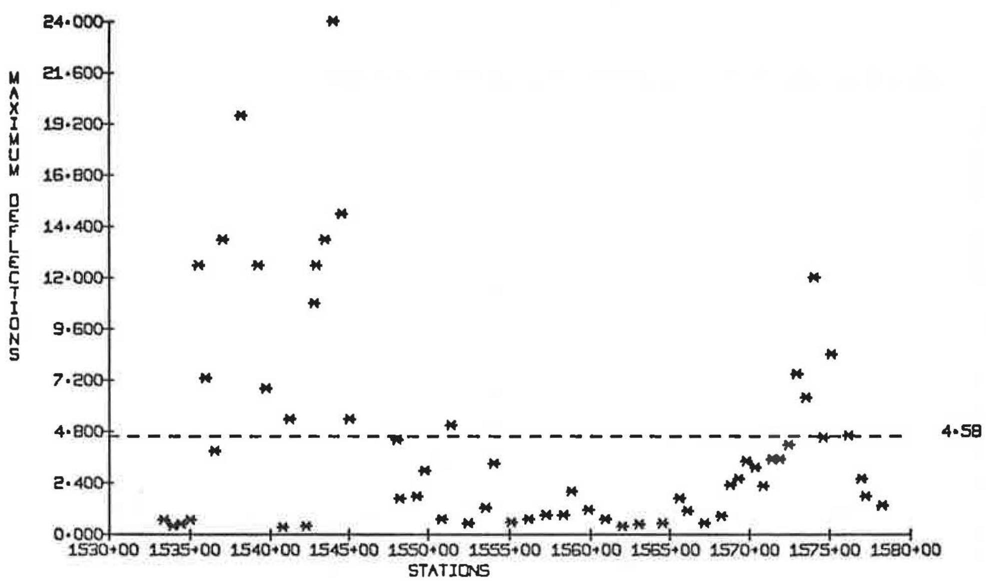


Figure 5. Maximum deflection and sci of a test section of roadway showing localized problems (more erratic subgrade response).

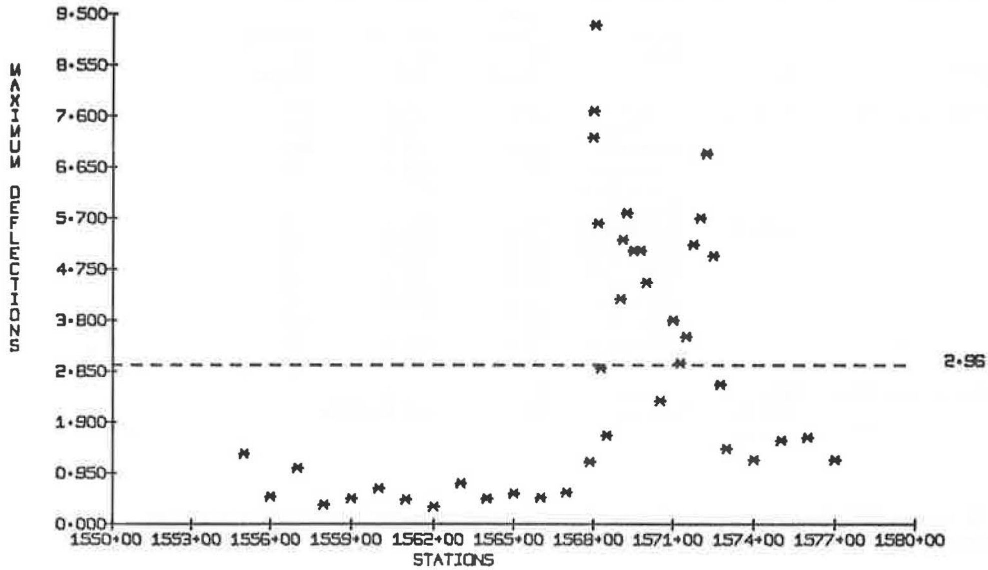


Figure 6. Maximum deflection and sci with extreme variability in subgrade response.

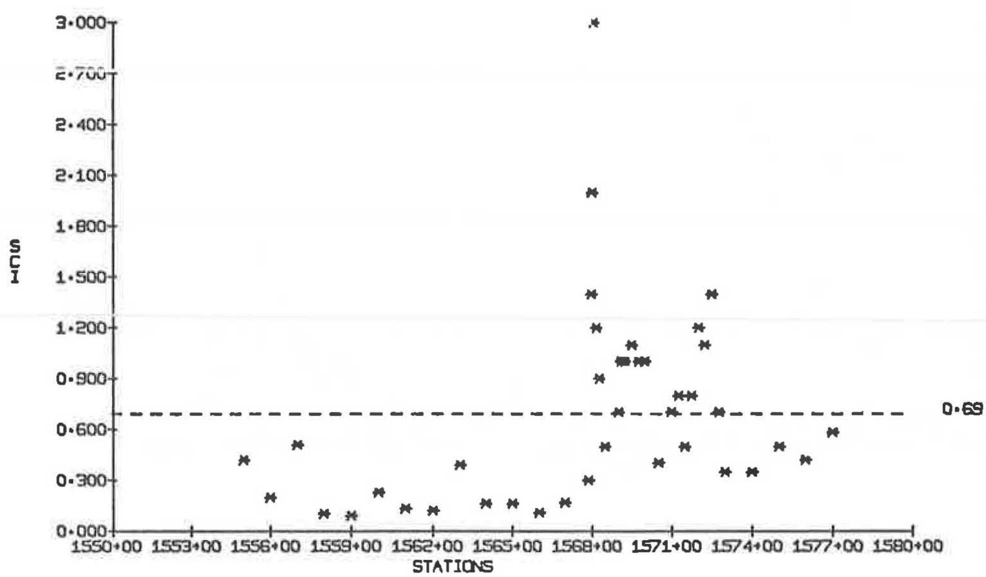


Figure 7. Maximum deflection and sci with good support characteristics and material uniformity.

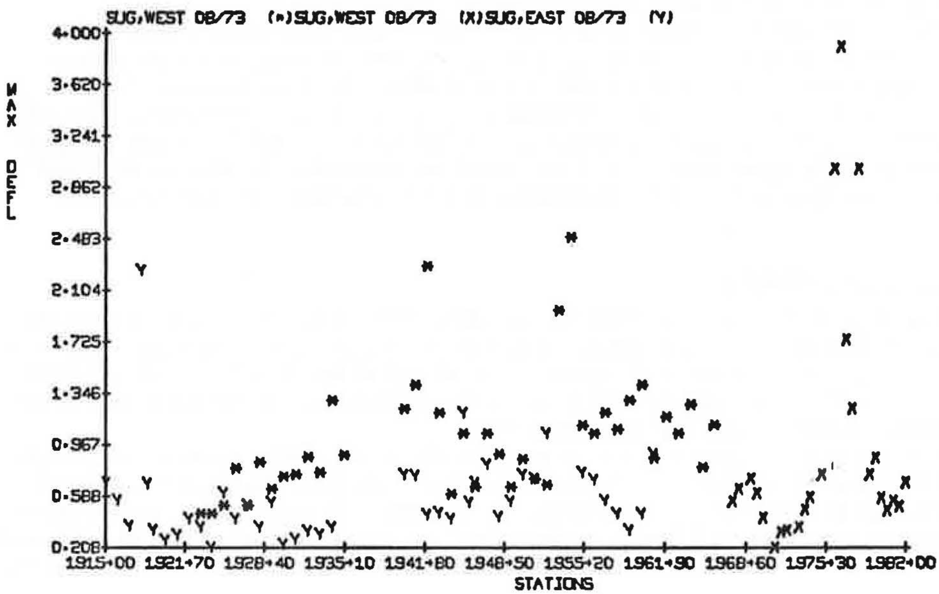
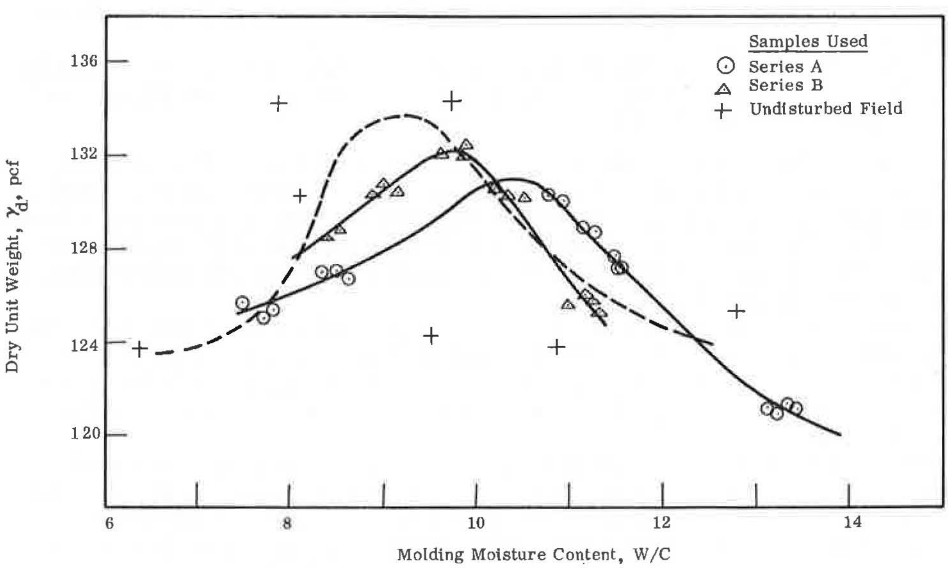


Figure 8. Relation between dry unit weight and molding moisture content.



wherever possible, are tabulated to show variations of soil characteristics with depth. There appears to be a good correlation between these observations.

Similarly, in verifying the validity of field in situ modulus measurements, undisturbed soil samples were subjected to dynamic loads, and the modulus of resilience and the dynamic modulus were calculated with procedures discussed in previous reports (2). The values of the dynamic modulus calculated with field deflection data and data of undisturbed field samples are compared in Tables 1, 2, and 3. There appears to be an excellent correlation between these measured parameters. For undisturbed field samples, the dynamic modulus was calculated for samples obtained at various depths.

Simulation of Field Conditions

In this phase, research was carried out to evaluate the effect of simulated environmental field conditions on the engineering characteristics of compacted soils. Laboratory experiments were carried out to evaluate the effect of increases of moisture content, due to saturation and freeze-thaw cycles after saturation, on the strength and deformation characteristics of compacted soils (6).

Laboratory compacted specimens were prepared by using drop hammer compaction covering a wide range of molding moisture contents. Soil specimens were saturated by simulating the boundary condition existing in the pavement subgrade. Soil samples were also subjected to several freeze-thaw cycles by using the open system that allowed additional moisture to enter the system and be drawn toward the ice lens as a result of freezing.

The properties considered were (a) parameters associated with physical properties—dry density and moisture content, (b) primary response parameters describing nonfailure behavior—complex modulus E* and resilient modulus M_R, and (c) ultimate response parameters describing failure conditions—shear strength and permanent deformation at failure.

The combined effects of compactive effort, moisture content at compaction, and its increase due to saturation and freeze-thaw cycles after saturation on the physical and mechanical properties are analyzed as follows:

1. The results showed that the percentage of dry unit weight loss suffered during capillary wetting and several freeze-thaw cycles was small, indicating that the soaking procedure was such that the increase in saturation occurred with no appreciable change in volume. On the other hand, the moisture content after saturation increased after several freeze-thaw cycles. This increase in moisture content significantly influenced the parameters investigated (Fig. 8).
2. There was a decrease in the value of unconfined compressive strength and an increase of the permanent deformation at failure due to saturation and freeze-thaw of the specimens. The loss in strength was greatest in those specimens with the lowest initial moisture content because of their greater absorption capacity and structural effect (Figs. 9 and 10).
3. The resilient modulus, M_R, was determined by using a constant stress level and samples with different molding moisture contents. With an increase of moisture content to full saturation, the soil loses an appreciable amount of its strength and approaches a minimum value of the resilient modulus. Again it was shown that the decrease in resilient modulus is greatest for those specimens with the lowest initial moisture content (Fig. 11).

For the design of a pavement structure, the variation of modulus of elasticity or the modulus of resilience with the environment and load is the critical factor because this parameter controls serviceability and deformation characteristics of the different components of the pavement structure. Therefore the input data gathered through environmental simulation are valuable for pavement performance analysis.

Figure 9. Relation between unconfined compressive strength and molding moisture content.

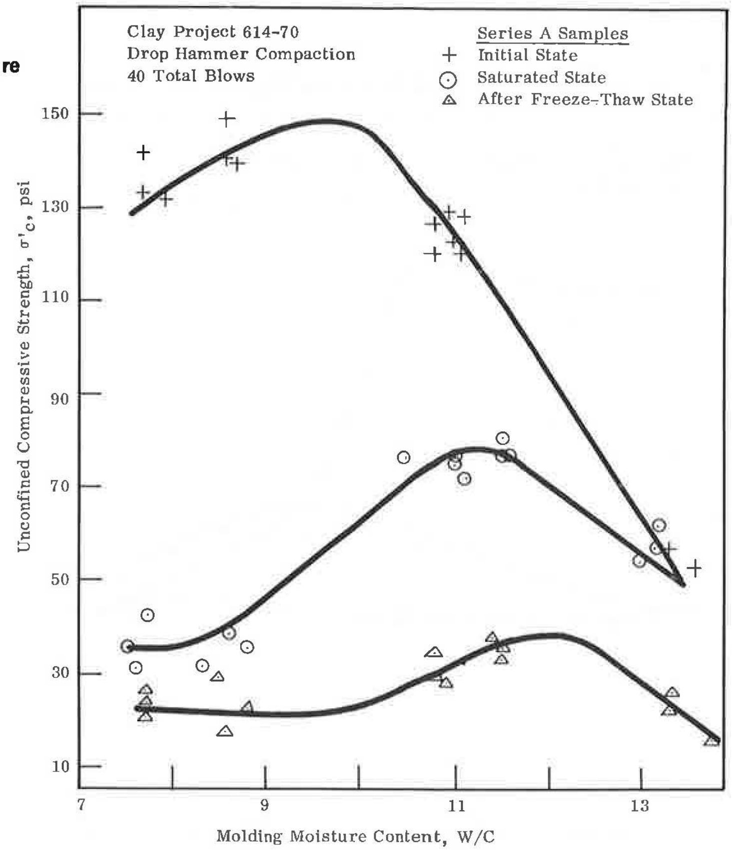


Figure 10. Relation between permanent deformation at failure and molding moisture content.

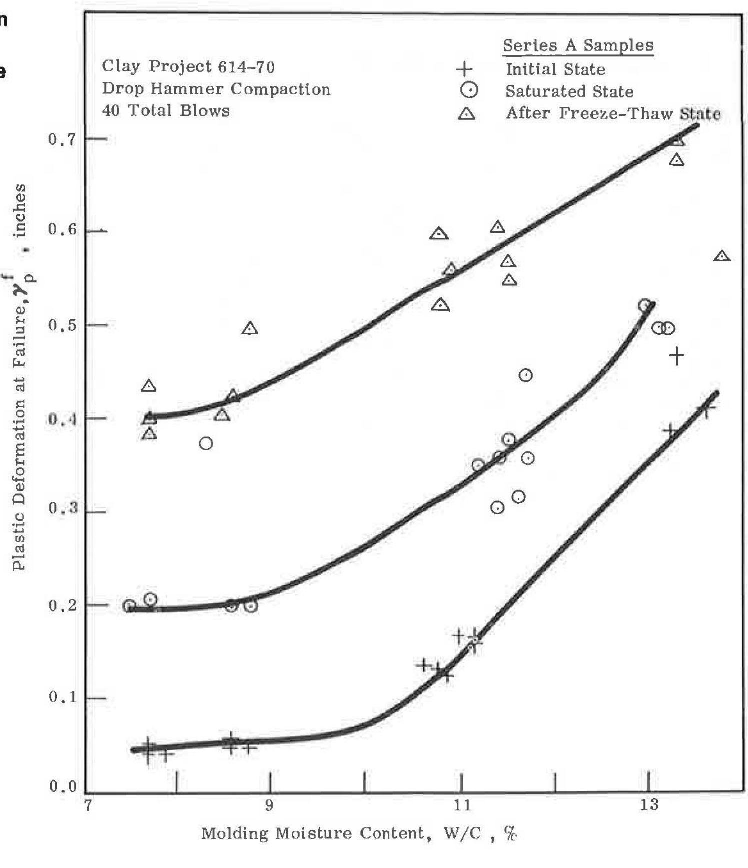


Figure 11. Relation between resilient modulus and molding moisture content.

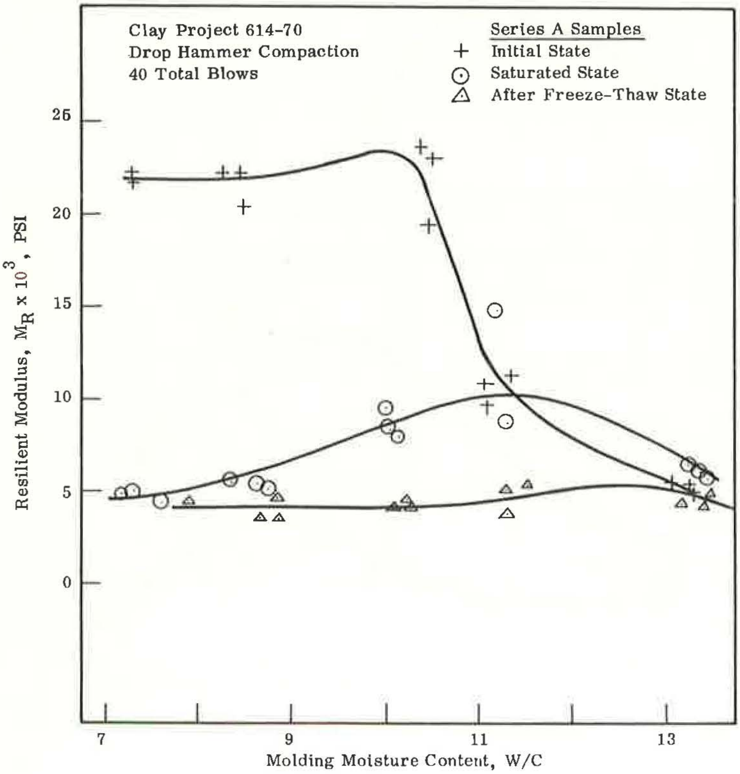


Table 4. Field samples (Franklin County SR-317).

Station	Location Depth (in.)	Water Content, W/C (percent)	Density, γ_d (pcf)	Unconfined Compressive Strength, σ_c'	Dynamic Modulus, $M_R \times 10^3$ psi: Laboratory	Dynamic Modulus, $M_R \times 10^3$ psi: Dynaflect	Subgrade Thickness (in.)
116+00	12 to 18	17.2		132.3	16.0	4.1	0 to 8.71
116+00	18 to 24	16.3		142.0	23.0	24.3	8.71 on
118+50	16 to 22	13.0		123.0	18.4	3.3	0 to 11.34
118+50	20 to 24	14.8		129.9	18.0	20.3	11.34 on
118+50	26 to 32	12.2		121.8	11.10		
121+00	9 to 15	13.2		130.7	18.9	27.4	0 on
126+00	7 to 13	10.6		61.7	11.3	27.6	0 on
129+00	10 to 16	13.6		122.6	15.7	5.0	0 to 10.4
129+00	24 to 30	12.8		125.8	15.9	19.1	10.4 on
130+00	6 to 12	15.4	110.07	77.9	23.5	5.7	0 to 14.33
130+00	12 to 18	16.0	110.43	64.9	24.2	21.1	14.33 on
138+00	8 to 14	12.8	114.22	108.4	31.4	—[a]	—[a]
150+00	6 to 12	10.1		114.4	19.0	4.4	0 to 10.3
150+00	18 to 24	11.7		123.4	20.1	21.8	10.3 on

[a]Data cannot be calculated.

CONCLUSIONS

The applicability of Dynaflect deflection measurements for use in in situ subgrade soil characterization and modulus evaluation and the use of maximum deflection and the shape of deflection profile, as presented by the spreadability concept, are discussed. Present data indicate that the Dynaflect can provide an accurate representation of quality of compaction, detect changes in pavement support as they occur, and point out areas of future problems.

A comparison of the results of undisturbed field samples with those obtained from laboratory testing indicates a very close agreement between various measured parameters. (Compare Table 2 with Figs. 8, 9, 10, and 11 for the I-70 project.)

The findings show that the subgrade undergoes seasonal changes in moisture and density detected from undisturbed samples, field nuclear measurements, and Dynaflect data. Most subgrades gained moisture and suffered loss of modulus and density after construction. Typical results are given in Tables 1, 2, and 3.

The moduli predicted by using Dynaflect measurements directly on the subgrade show close agreement with laboratory measured values (Table 4). Measurements on completed pavements showed seasonal trends but resulted in moduli somewhat higher than expected from this test. This can be attributed to the difference in the deviatory stress in each case and to the slab effect of pavement layers that will eliminate lateral movements that increase confining pressures. In addition, the Dynaflect measures the average subgrade support within a considerable depth, whereas those undisturbed field samples are obtained at the top of the subgrade and will only reflect variations in the upper few feet. Further research is in progress to establish the exact correlation between actual subgrade moduli and those obtained from Dynaflect measurements on pavement surfaces.

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FIELD EVALUATION PROGRAM OF CEMENT-TREATED BASES

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The purpose of this study is to improve or develop design criteria for cement-treated bases by means of field experiments using recommendations for implementation from laboratory studies on criteria for strength and shrinkage control of cement-treated bases and crack control in cement-treated bases. This paper presents information on the background of the research study, laboratory work, and field construction of experimental sections. It also includes a limited analysis of the laboratory and field test results. Tentative conclusions indicate that the present design criteria are valid and need no modification. Consideration should be given to using lime additive when high clay content (16 percent or more) is found in the base material. Sugar is a good retarding agent but also creates large cracks. If the cracks can be formulated and retained as fine hairline cracks, they will not be reflective on the pavement surface. Use of expansive cement and lime additive (two mixings) creates such fine hairline cracks. The undisturbed curing and the artificial traffic sections have recorded numerous fine cracks on the soil-cement but only a small amount on the pavement surface. From the standpoint of construction economy, providing the 7-day undisturbed curing practice is very expensive and inconvenient. Therefore, it is recommended that the undisturbed curing requirement be deleted and that construction traffic and necessary local traffic be allowed on the soil-cement during the 7-day period. These conclusions are preliminary pending a longer performance record of field experimental sections.

•TWO research studies using extensive laboratory testing and dimensional and model analyses were conducted by K. P. George (1, 2) on the construction and testing of cement-treated base materials. Following are recommendations for implementation from his research work and discussion of these recommendations by the Research and Development Division of the Mississippi State Highway Department (MSHD).

Recommendations pertaining to the selection of soil and design of soil-cement mixtures are as follows:

1. Because shrinkage stresses are a function of the maximum shrinkage, which in turn is solely controlled by the -2μm clay particles, an effort should be made to use soils with as small a quantity of clay as possible and still remain consistent with the clay requirement for proper cohesion and strength. If the clay mineral in the soil is kaolinite, the clay content should not exceed 15 percent; however, the clay content should be limited to about 8 percent if the clay mineral is montmorillonite. If the soil contains both kaolinite and montmorillonite in some proportion, the ceiling should accordingly be interpolated between these limits.
2. Inasmuch as montmorillonite soil shrinks much more than kaolinite, its shrinkage potential warrants extensive investigation.
3. The use of large aggregates [at least 1 in. (25.4 mm) in diameter] in a soil-cement matrix often exhibits excessive shrinkage and therefore should be discouraged.

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4. The cement requirement stipulates that amounts of cement equal to or greater than that specified by freeze-thaw test criteria (ASTM D 560) be used.
5. Type II cement is recommended over Type I cement.
6. For minimal shrinkage cracking, it is highly desirable to replace 1 to 2 percent of cement with an equal amount of lime.
7. Expansive cement admixture is very effective in well-graded coarse grain soils.
8. Cracking can be minimized when a mixture of sugar and lime is added to the soil-cement mixture.

George also recommended the following to ensure sound construction and curing procedures.

1. As high frictional subgrade may serve to more evenly redistribute stresses caused by shrinkage and thereby reduce the incidence of cracking, it is proposed that soil-cement base be placed over rough subgrade. Accordingly, mix-in-place soil-cement construction is desired.
2. Soil-cement base should be compacted to the highest density possible. A minimum of 95 to 100 percent AASHTO T-180 is recommended. Soils with AASHTO T-99 density below approximately 115 lb/ft^3 (1842 kg/m^3) should be used with extreme precaution. Also, it is extremely important that soil-cement be compacted at the dry side of optimum moisture.
3. Most important of all, cracking can be minimized by adequately extended curing. The evaporation rate of water from the surface of a fresh soil-cement base is the most important factor influencing shrinkage and shrinkage cracking, and that rate is influenced by the temperature, relative humidity, and wind velocity of the air and temperature of the soil-cement. Accordingly, specifications should be changed to discourage casting of a soil-cement base at low humidity and in windy and hot weather.
4. By increasing the stiffness of the base shrinkage, cracking can be controlled. Because the stiffness is proportional to the third power of the thickness, the advantage in increasing the thickness is obvious.
5. When the design calls for a thickness of about 7 in. (178 mm) or more, it is desirable to compact the base in two layers that are properly bonded at the interface.

Based on George's recommendations MSHD conducted an informal study to determine the predominate clay mineral to be used for soil-cement construction. To date, 60 samples have been X-rayed, and all were found to be predominately kaolinite with trace illite or montmorillonite. The clay content of these 60 samples ranged from 8 to 16 percent. These met all the requirements George outlined in his recommendations. Inasmuch as the 60 samples come from different districts of Mississippi and represent a general condition of the base material throughout, it does not seem that the highway department will have any problem in the selection of soils.

At the request of the department, George conducted a special study on three samples to evaluate and compare the department's procedure of determining cement content with those developed by the ASTM and PCA. The present department design criteria are based on 7- and 14-day unconfined compressive strengths. The results (Table 1) indicate that the department's criteria are about 1 percent higher than those specified by the freeze-thaw (ASTM D 560) test. This also meets George's recommendation.

The present specification requires a specified density of 97 to 101 percent AASHTO T-134 for the compaction control of soil-cement bases. Department engineers involved in the study felt that the 95 to 100 percent AASHTO T-180 compaction effort George recommended is not feasible under the present field construction practice. It may also cause damage to the subgrade soil.

Most soil-cement construction in Mississippi is done by mix-in-place operation. During mixing operations, the pulverizing machine actually roughens the subgrade to a certain degree, and this could be considered as a frictional surface. The two-layer, soil-cement construction used in the past years was quite unsatisfactory (Fig. 1). The department no longer uses the two-layer construction design. In another study at the Virginia Highway Research Council (3), it was found that sugar-lime admixture can be used successfully for retarding the hardening of cement-treated soils in highway construction.

Table 1. Comparison of cement content determination using MSHD, ASTM, and PCA procedures.

Soil Number and County	Predominant Clay Mineral	AASHTO Classification	Cement Percentage on Volume Basis: Recommended by Highway Department	PCA Shortcut Procedure	ASTM Freeze-Thaw Requirement
1-A, Attala	Kaolinite with trace illite and montmorillonite	A-2	8	9	7
2-A, Attala	Kaolinite with trace montmorillonite	A-2	6½	8	6½
3-A, Carroll and Montgomery	Kaolinite with trace montmorillonite	A-2	7½	8	6½

Figure 1. Two-layer soil cement failure.

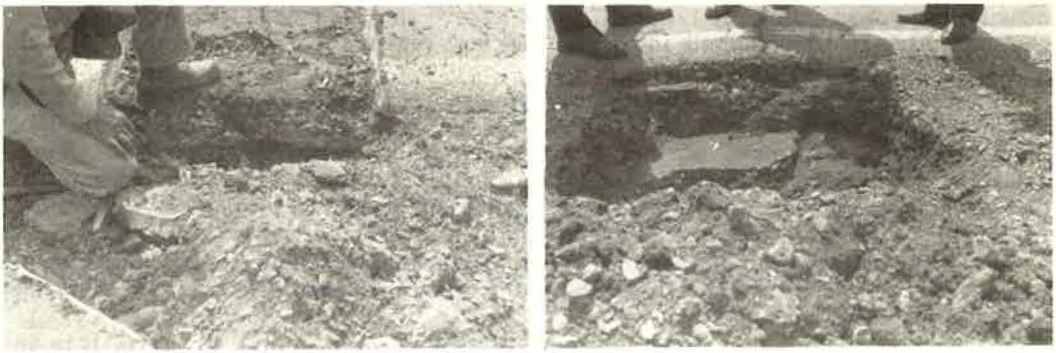


Table 2. Physical properties of soils used.

Item	Number
Percent passing	
No. 10	100.00
No. 40	87.00
No. 60	50.00
No. 200	21.00
No. 270	20.00
Silt, percent	6.00
Clay, percent	14.00
Colloids, percent	13.00
Dust ratio	24.06
Plasticity index, percent	N.P.
Raw soil standard dry density, pcf	115.6
Raw soil optimum moisture content, percent	12.8
Cement required to stabilize, percent	6.5

Table 3. Standard density and optimum moisture contents of control and experimental sections.

Section	Standard Density (pcf)	Optimum Moisture (percent)
Control	113.5	13.6
Experimental		
Lime (two mixings)	112.5	15.4
Lime (one mixing)	115.0	14.0
Sugar	115.7	13.9
Type II cement (same as control)	114.3	14.3
Type K cement	111.3	14.3
7-day undisturbed curing	115.7	13.5
Artificial traffic	115.7	13.5
Less cement	112.2	14.3
Less cement and increased thickness	115.8	13.1

Note: 1 pcf = 16.018 46 kg/m^3.

RESEARCH PROJECT DESIGN

The site selected for this research study is located in Winston County on Miss-395. Cement contents used in all the experimental sections are expressed in percentage by volume of raw soil, and lime and sugar contents are expressed in percentage by weight of raw soil.

From station 140+00 to 150+00 there will be a control section. Thickness of this section will be 6 in. (152 mm), and cement content will be 6.5 percent.

The experimental sections are as follows:

1. Station 150+00 to 160+00 (with lime additive)—thickness = 6 in. (152 mm), cement content = 4.5 percent, and lime content = 10.5 lb/yd^2 (5.7 kg/m^2) or 2 percent. Lime will be placed and mixed first and allowed to mellow 6 days, then cement will be added, and the total base remixed.
2. Station 160+00 to 170+00 (with lime additive)—thickness = 6 in. (152 mm), cement content = 4.5 percent, and lime content = 10.5 lb/yd^2 (5.7 kg/m^2) or 2 percent. Lime and cement will be applied and mixed in one mixing operation.
3. Station 170+00 to 180+00 (with sugar additive)—thickness = 6 in. (152 mm), cement content = 6.5 percent, and sugar content = $\frac{1}{3}$ lb/yd^2 (0.18 kg/m^2) or $\frac{1}{16}$ percent.
4. Station 180+00 to 190+00 (with Type II cement)—thickness = 6 in. (152 mm), and Type II cement content = 6.5 percent.
5. Station 190+00 to 200+00 (with Type K expansive cement)—thickness = 6 in. (152 mm), and expansive cement content = 6.5 percent.
6. Station 200+00 to 210+00—thickness = 6 in. (152 mm), and cement content = 6.5 percent. This section requires undisturbed curing. During the first 7 days after completion of the cement-treated base on this section (including the curing membrane), all traffic and equipment will be kept off the cement-treated base and routed around another road.
7. Station 210+00 to 219+00—thickness = 6 in. (152 mm), and cement content = 6.5 percent. This section requires artificial traffic during the curing period in addition to any traffic that must be maintained under the contract. After the required density has been obtained on this section and the curing membrane has been placed, the surface will immediately be sanded as lightly and as uniformly as necessary to prevent "picking up" the curing membrane. Thereafter at least three complete coverages by the pneumatic tire roller, or by something comparable used in obtaining the required density, will be made once in the morning and in the afternoon of each of the next successive 7 days in the curing period. In addition, a dual wheel dump truck loaded to the maximum load will traverse each lane of the section 10 times during each day. Such loads will cover the entire width of the cement-treated course, exclusive of the outside 1 ft (0.3 m). If, at any time during such maximum loadings, there is evidence that permanent damage to the base is beginning to occur, such maximum dual wheel loading will be stopped.
8. Station 219+00 to 228+00—thickness = 6 in. (152 mm), and cement content = 4.5 percent. This experimental section uses less cement.
9. Station 228+00 to 236+58—thickness = 8 in. (203 mm), and cement content = 4.5 percent. This section uses less cement and has increased base thickness.

LABORATORY TESTING AND RESULTS

Soil used for the cement-treated base is a uniform A-2(0) sand clay material that has about 14 percent clay content. The X-ray diffraction pattern indicated that the clay mineral was in the form of kaolinite.

The average daily traffic for this project is less than 1,000, and according to the department's roadway design procedure, the thickness of the soil-cement base is 6 in. (152 mm). Physical properties of soils are given in Table 2. The cement content (6.5 percent by volume) for the control section was determined by the present department design criteria, which are based on the 7- and 14-day unconfined compressive strength. The moisture-density relationships for the control and experimental sections are given in Table 3.

Effect of Delay Mixing on Soil-Cement Strength

Test specimens were made at different delayed mixing times, ranging from 1 to 48 hours, so that the effect of delay mixing on the strength of soil-cement could be studied. These samples were cured for 7 days and then tested for unconfined compressive strength. Figure 2 shows the results of these tests.

Sugar was tried at a different percentage and with and without lime. The sugar and lime percentage shown in Figure 3 is calculated from the weight of the raw soil. The mixture with $4\frac{1}{2}$ percent cement, 2 percent lime, and $\frac{3}{8}$ percent sugar almost completely killed the cementing action and provided extremely low strength. The mixture with $4\frac{1}{2}$ percent cement, 2 percent lime, and $\frac{1}{16}$ percent sugar provided a strength of about 150 psi (1.04 MPa). This mixture provided 150 psi (1.04 MPa) throughout the delayed period from 1 to 7 hours. The curve also shows that, when delayed 24 to 48 hours, the mixture had a gain of strength of about 40 psi (0.28 MPa). The mixture with $4\frac{1}{2}$ percent cement and 2 percent lime (one mixing) gives a curve that provides a strength of 250 psi (1.72 MPa) throughout the delayed period from 1 to 7 hours. The control mixture design with $6\frac{1}{2}$ percent cement only showed a high strength of 500 psi (3.45 MPa) at normal mixing; however, the strength dropped sharply to about 200 psi (1.38 MPa) when delayed 3 hours. This indicates that it is very important in soil-cement construction for the contractor to compact the soil-cement mixture as soon as possible so that the designed strength can be achieved.

The experimental mixture with $6\frac{1}{2}$ percent cement and $\frac{1}{16}$ percent sugar provided the same 7-day strength as the control section when it was tested at normal mixing and, yet, at the end of the 7-hour delay, it still had a strength of 250 psi (1.72 MPa).

7- and 14-Day Strengths of Experimental Mixtures

Table 4 gives the 7- and 14-day strengths of all experimental mixtures. The department's current design procedure requires a strength of 500 psi (3.45 MPa) or above; however, on this experimental project, the various experimental designs also varied in strength, which ranged from 300 psi (2.07 MPa) to 500 psi (3.45 MPa).

CONSTRUCTION OF FIELD EXPERIMENTAL SECTIONS

Construction for the experimental soil-cement base section began on July 23, 1972, and was completed on August 2, 1972. A Bros Roto Mixer was used for the mixing operation. A sheepsfoot roller and ruber-tired roller were used for the compaction of the soil-cement. The 1-in. (25.4 mm) hot plant mix pavement was completed in September 1972. No special problem was encountered during the field construction.

Experimental section 5 using Type II cement was constructed with the same type of cement used in section 1, the control section, because the cement that the contractor used met the AASHTO specification for Type I and II cement.

FIELD MEASUREMENTS AND RESULTS

Time Lapse Between Mixing and Compaction

The present specification requires that water supply and pressure distributing equipment will be provided, which will permit the application, within 1 hour, of all water required to bring the section being processed to the required moisture content. Each increment of water added during mixing will be incorporated into the mix for the full depth to avoid concentration of water near the surface, and no portion of the mixture will remain undisturbed for more than 30 min before compaction. Initial compaction will begin immediately, and machining and compacting will continue in such manner that, and until, the entire depth and designated width of the cement-treated material is compacted to the required density within 2 hours from the time of beginning the mixing.

An effort was made during the field construction to study the effect of time lapse between mixing and compaction on the strength of soil-cement.

Figure 2. Effect of delay mixing on strength of soil-cement.

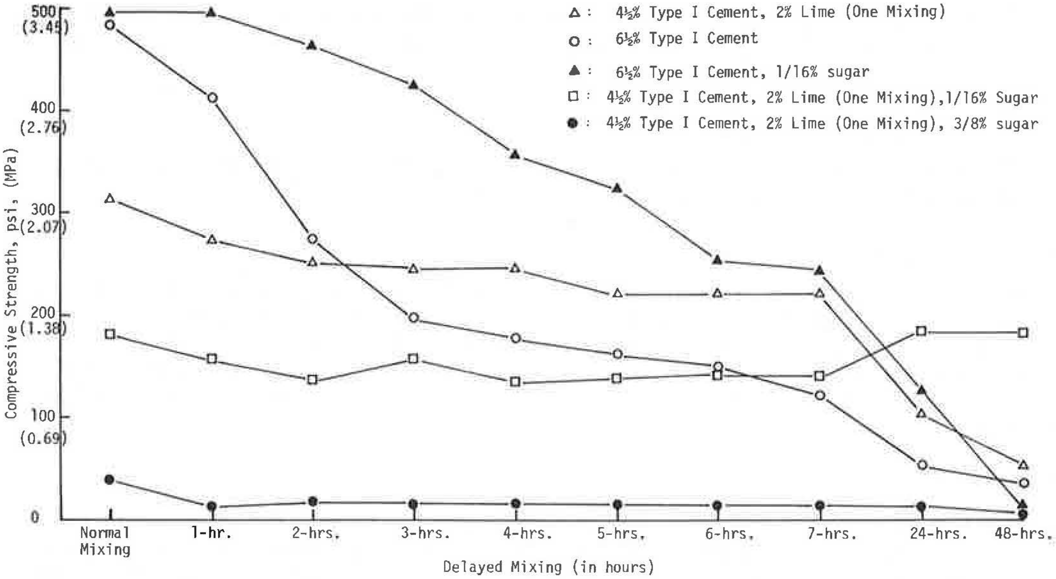
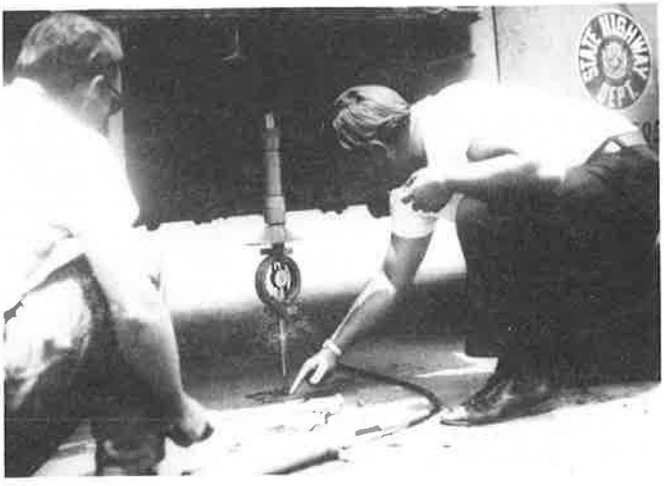


Table 4. All experimental mixtures at 7- and 14-day compressive strengths.

Experimental Mixtures	7-Day Strength (psi)	14-Day Strength (psi)
6½ percent Type I cement	485	577
4½ percent Type I cement, 2 percent lime (two mixings)	426	
4½ percent Type I cement, 2 percent lime (one mixing)	314	374
4½ percent Type I cement, 2 percent lime (one mixing), 3/8 percent sugar	40	32
6½ percent Type K expansive cement	370	
4½ percent Type I cement	346	
4½ percent Type I cement, 2¼ percent lime (one mixing), 1/16 percent sugar	183	263
6½ percent Type I cement, 1/16 percent sugar	497	549

Note: 1 psi = 0.006 894 757 MPa.

Figure 3. Penetration resistance test apparatus.



Field Moisture and Density

Field moisture and density measurements were obtained by nuclear and conventional methods (sand cone density and speedy moisture content were used as conventional methods).

Penetration Resistance Test

A homemade penetrometer apparatus was used during field construction to measure the penetration resistance. The apparatus is shown in Figure 3.

Cement Contents

So that the uniformity of the spreading and mixing operation could be studied, samples for cement contents determination were obtained during the compaction operation between stations 140 and 150. Cement contents were determined by X-ray.

In January 1973, cores were obtained between stations 140 and 150. The unconfined compressive strength and cement content of these cores were determined in the laboratory.

Deflection

Limited information on the deflection of the soil-cement was obtained during the field survey using the Dynaflect unit. The Dynaflect unit measures pavement deflection induced by an applied load. It is an electromechanical system consisting of a dynamic force generator, a motion measuring system that is mounted in a towed trailer, and five motion sensing geophones suspended from the towing arm of the trailer. The Dynaflect-measured deflections have good correlation with the Benkleman beam deflection measurements. Benkleman beam deflection is equal to about 20 times the Dynaflect deflections (unit in mils or mm).

Dynaflect deflection readings were conducted on the experimental section 7 days after its completion. At least four readings were obtained from each section.

Cracking

All visible cracks on the control and experimental sections were mapped when the soil-cement was 7 days old (in July 1972). The pavement surfaces were first mapped during September 1972, but no cracks were found. In June 1973, another field survey was made and cracks were mapped. Considerable cracking was recorded during this survey. Figures 4 through 13 show cracking maps of a 100-ft (30.5 m) section of the soil-cement and superimposed pavement selected from the control and experimental sections.

PRELIMINARY ANALYSIS OF RESULTS

A summary of results is given in Table 5. All the sections have about the same elapsed time (except the sugar section), penetration resistance, and deflection. The field moisture content and density are all slightly lower than the specified values. The only data that show a considerable difference among the 10 sections are the crackings. The cracking index is calculated from all the cracks located on each control or experimental section by using the total footage of cracks divided by the total area. The unit for the cracking index is therefore in ft/ft^2 (m/m^2).

The present design criteria for soil-cement mixtures based on the 7-day compressive strength of 500 psi (3.45 MPa) for DBST or concrete pavement and 600 psi (4.14 MPa) for hot plant mix pavement appear to be on the high side. However, considering the uniformity of spreading operation and time lapse during the mixing and compacting, this high strength requirement provides a safety factor for the field mass production. Therefore, it is recommended that these criteria not be changed at the present time.

Sugar is a good additive to retard the setting time of soil-cement mixtures and thus provides the contractor more time to mix and compact the material. However, sugar also causes larger cracks on the soil-cement, which is most undesirable.

Lime should be a very useful additive when higher clay contents are found in the base

Figure 4. Cracking pattern of soil-cement base and superimposed pavement for control section.

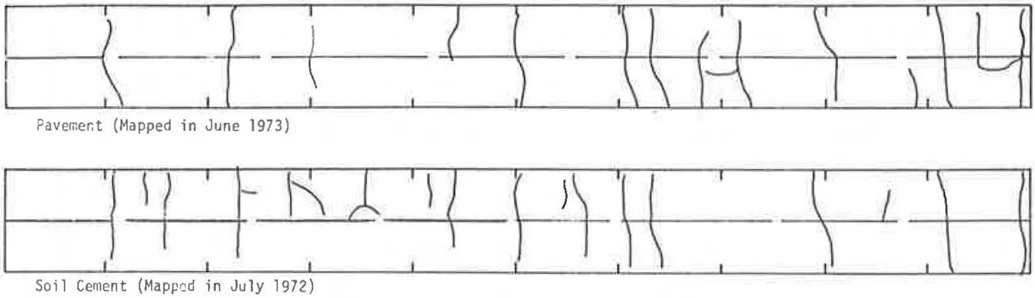


Figure 5. Cracking pattern of soil-cement base and superimposed pavement for experimental section with lime additive (two mixings).

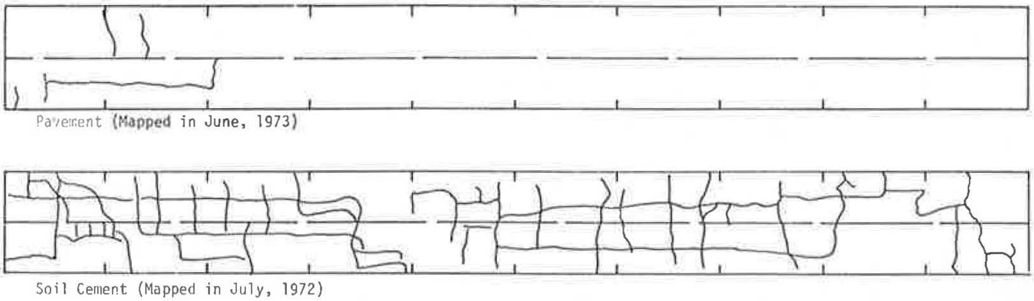


Figure 6. Cracking pattern of soil-cement base and superimposed pavement for experimental section with lime additive (one mixing).

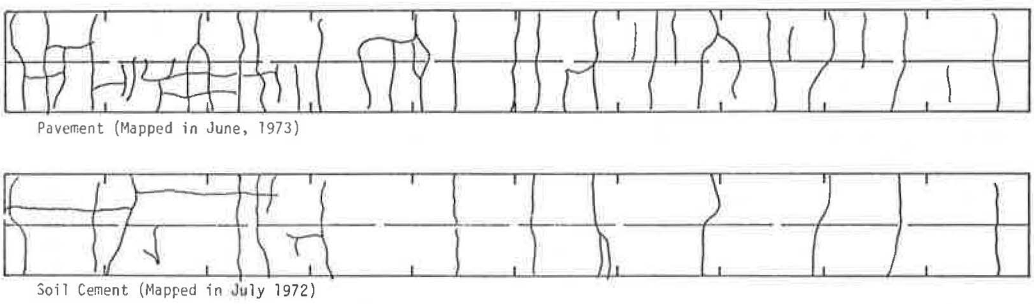


Figure 7. Cracking pattern of soil-cement base and superimposed pavement for experimental section with sugar additive.

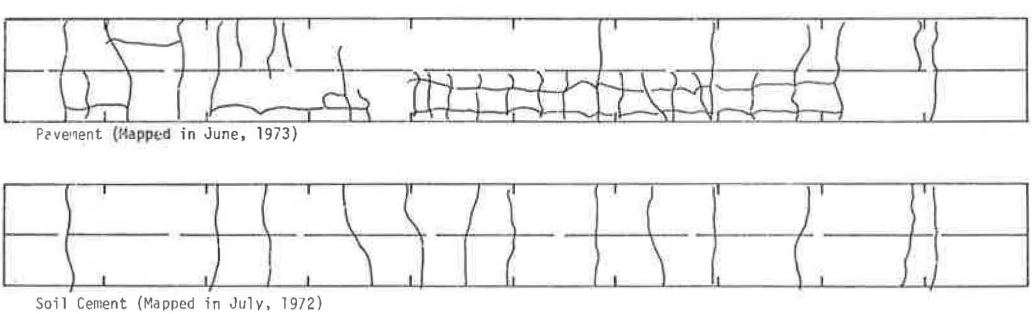


Figure 8. Cracking pattern of soil-cement base and superimposed pavement for experimental section with Type II cement (same as control section).

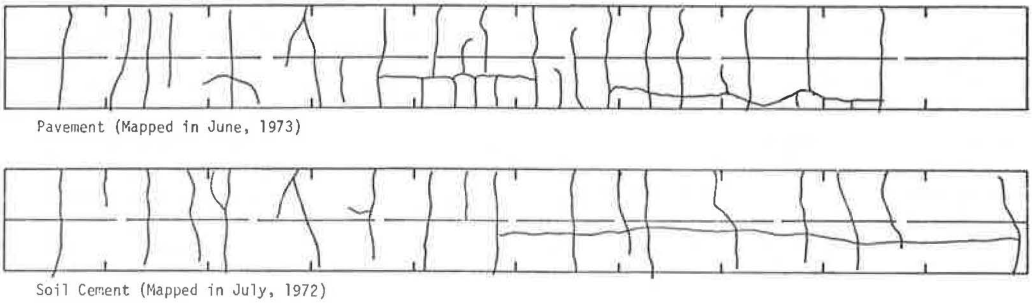


Figure 9. Cracking pattern of soil-cement base and superimposed pavement for experimental section with Type K expansive cement.

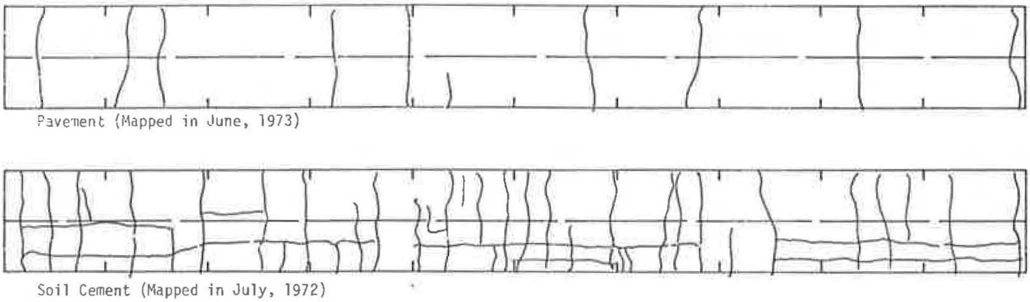


Figure 10. Cracking pattern of soil-cement base and superimposed pavement for experimental section with 7-day undisturbed curing period.

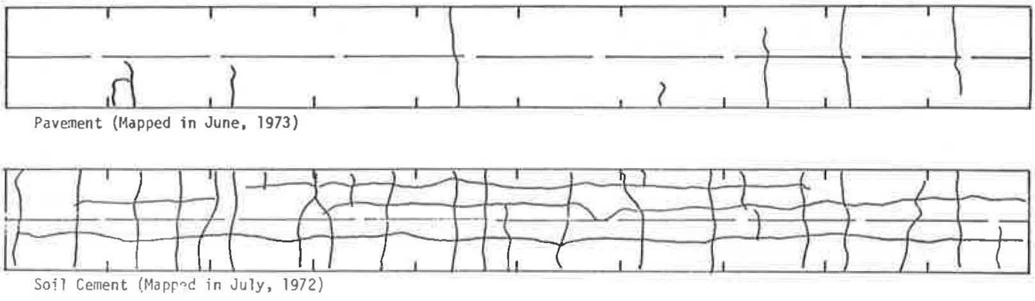


Figure 11. Cracking pattern of soil-cement base and superimposed pavement with artificial traffic.

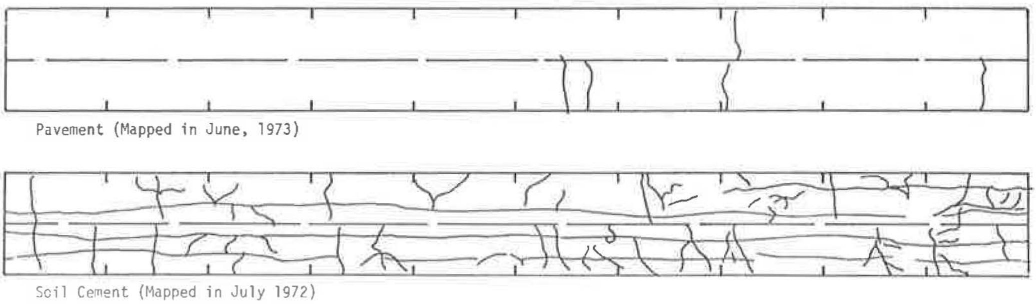


Figure 12. Cracking pattern of soil-cement base and superimposed pavement for experimental section with less cement content.

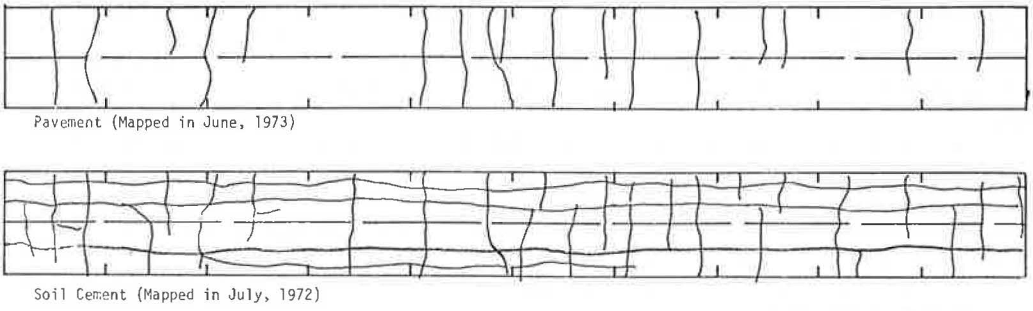


Figure 13. Cracking pattern of soil-cement base and superimposed pavement for experimental section with less cement and increased thickness.

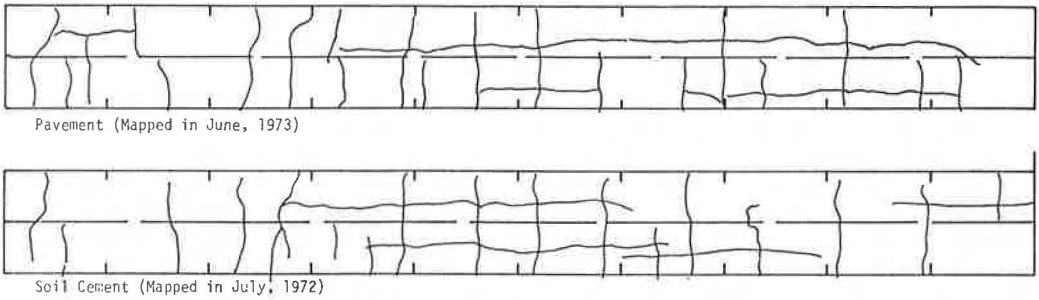


Table 5. Summary of results.

Section	Elapsed Time (min)	Field Moisture[a] (pcf)	Field Density[a] (pcf)	Penetration Resistance at 30 Hours (psi)	Field Cement Content (percent)	Core Strength (psi)	Deflection (mils)	Cracking Index on Soil-Cement (ft/ft^2)	Cracking Index on Pavement (ft/ft^2)
Control	75	-1	-2	6,000 to 7,000	3.7 to 10.7	199 to 549	1.434	0.119	0.102
Experimental									
Lime (two mixings)	105	-2	-2	5,000 to 7,000	—	—	1.680	0.237	0.031
Lime (one mixing)	55	-1	+2	5,000 to 5,500	—	—	1.635	0.072	0.127
Sugar	200	-2	-3	2,500 to 3,000	—	—	1.718	0.128	0.132
Type II cement (same as control)	85	-2	-1	6,000 to 7,000	—	—	1.480	0.095	0.104
Type K cement	90	-4	-1	6,000 to 7,000	—	—	1.538	0.257	0.061
7-day undisturbed curing	55	-1.5	-2	6,000 to 6,500	—	—	1.567	0.325	0.032
Artificial traffic	55	-2	-2	8,000 and above	—	—	1.345	0.227	0.021
Less cement	35	-3	+2	6,000 to 6,500	—	—	1.660	0.264	0.137
Less cement and increased thickness	50	-1	-1	6,000 to 6,500	—	—	1.670	0.153	0.166

Note: 1 pcf = 16.018 46 kg/m^3; 1 psi = 0.006 894 757 MPa; 1 ft/ft^2 = 3.2808 m/m^2.

[a]Deviations from specified density and optimum moisture.

material. Lime is also a retarding agent and when used in one mixing operation often creates wider cracks. However, when used in the two mixing operations, it creates more but finer cracks.

When expansive cement is used in the soil-cement mixture, it creates numerous hairline cracks on the soil-cement. Data obtained to date show that these cracks are not reflective.

The 7-day undisturbed curing section created about the same amount of cracks on the soil-cement as the control section, but these are also very fine cracks, and to date only a few cracks can be found on the pavement surface.

The section with artificial traffic created numerous hairline cracks on the soil-cement, and field survey indicated that these cracks were not reflective.

Sections using less cement are not desirable because they created larger cracks and most of them are reflective.

TENTATIVE CONCLUSIONS AND RECOMMENDATIONS

The present design criteria for the soil-cement mixture are valid and need no modifications. However, consideration should be given to adding a small percentage of lime to the soil-cement mixture when high clay contents (16 percent or more) are found on the base material. The benefit of lime additive is not conclusive from this study because the base material has only 14 percent clay content.

It is very important in soil-cement construction to have a uniform spreading of cement and to compact the soil-cement mixture as soon as possible to achieve the designed strength. Sugar is a good retarding agent but creates large cracks.

The crack survey indicates that it is almost impossible to keep the soil-cement from cracking under the field mass production operation. However, if the cracking can be formulated and retained as fine hairline cracks, it will not be reflective on the pavement surface. This is true even when the cracks are numerous. Therefore, research should be directed toward finding out how to keep the cracks small and numerous, rather than how to eliminate them.

Plans will be made in the next 2 years of observation to drill cores at the locations of the fine hairline cracks to study if the cracks are watertight and have zero or minimum growth. The undisturbed curing and artificial traffic sections have recorded numerous fine cracks on the soil-cement but only a small amount of cracks on the pavement surface. Providing a 7-day undisturbed curing practice is very expensive and inconvenient; therefore, it is recommended that the undisturbed curing requirement in the present specification be deleted and that construction traffic and necessary local traffic be allowed on the soil-cement during the 7-day period.

Based on these conclusions, the use of a lime additive (two mixings) and expansive cement should be incorporated in the design and construction of other projects so that their validity may be ascertained.

ACKNOWLEDGMENT

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The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Mississippi State Highway Department or the Federal Highway Administration.

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DISCUSSION

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The efforts of the authors in preparing this report on a very well-designed and comprehensive field experiment on cement-treated bases are sincerely appreciated. Engineers engaged in this field of research are looking forward to further reports.

The remark on the unsatisfactory performance of the two-layer, soil-cement construction aroused additional interest. In South Africa this form of construction was, and is, extensively used, and its abandonment in Mississippi may influence its future use. The comments of the authors on the following questions would therefore be extremely helpful:

1. Why was the two-layer design abandoned?
2. On how many cases was the decision based? How frequently did soil-cement distress appear?
3. What was the form of the distress or failure that was observed?
4. There are numerous factors controlling the performance and failure of a soil-cement base. Why was the use of a two-layer construction isolated and chosen as the cause of the failure?
5. How was the lower layer cured? How could this have influenced the performance?
6. What was the extent to the bond between the two cement-treated layers? Did you observe any signs of slippage or horizontal movement between the two layers?

AUTHORS' CLOSURE

Otte's comments are very much appreciated. It is acknowledged that the reason why Mississippi abandoned the two-layer, soil-cement design should be discussed in greater detail. In an effort to be completely clear and to avoid repetition, we offer the following comments in answer to Otte's questions.

Generally, the reasons for abandoning the two-layer system are threefold. Because of the advancement in design of construction equipment, certain thicknesses of soil-cement design can now be constructed as a single (monolithic) layer rather than as two layers. Single-layer construction is much more economical than two-layer construction. A few two-layer construction projects (traveling plant mixing) experienced the type of distress shown in Figure 1, where either the top or the bottom layer resulted in a distinct shear type of failure. From the field excavation, it appears that when the top layer failed, it was under some compressive force and yielded in shear. The expansion and the resulting compressive stress may be due to the several cracks that extended deep into the soil-cement base. During the cold weather when the cracks were at their maximum width, foreign materials may have crept into these cracks and, subsequently, when the temperature rose, the soil-cement slabs could not expand freely, thereby subjecting the slab to a compressive force. Second, alternate shrinkage and expansion resulting from drying and wetting could also create compressive force attributable to the same mechanism.

In other areas where the bottom layer has undergone typical shear failure and the top layer has exhibited only a minor crack, compressive failure may have been caused by the expansion in the bottom layer at a weak point that is possibly associated with constructed joints. This expansion and that of the top layer occur because of the same reasons.

During the field investigation, considerable moisture was accumulated at the interface between the top and bottom layers. That the bottom face of the top layer was poorly cemented was important. This caused the two 5-in. (127 mm) layers of soil-cement base to act separately as two layers rather than as one 10-in. (254 mm) base course to support the pavement. However, no sign of slippage or horizontal movement between the two layers was observed.

On a few two-layer projects that were constructed by the central plant mixing operation, many more cracks were observed than in the single-layer base course. On Miss-6 at the Oxford Bypass, which was constructed in 1965, near the University of Mississippi, both two-layer and single-layer designs were used. The two-layer portion was constructed with central plant mixing, and the single-layer portion was made with traveling plant mixing. Before the soil-cement base was covered with the bituminous pavement, the project engineer noticed that the two-layer portion had many more cracks than the single-layer portion (4). This observation was not documented; therefore, the authors specially requested the Department of Civil Engineering of the University of Mississippi to conduct field cracking surveys on the pavement. This survey (5) indicated that the two-layer section showed more transverse and longitudinal cracks than the single-layer section. The crack density (length of cracks per area) is also higher for the two-layer section than for the single-layer section. Results of this survey are given in Table 6. Figure 14 shows the general view of pavement surface of the single-layer section and Figure 15 shows the two-layer section.

Inasmuch as the two-layer section was constructed with the central plant mixing operation, there should be no problem in bonding the two layers. No sign of slippage or horizontal movement between the two layers was observed. If one assumes that the two layers were properly bonded and that they act as a monolithic layer, the hypothesis

Table 6. Crack survey, Miss-6, Oxford Bypass.

Pavement Type and Location	Total Length of Cracks[a] (ft)	Crack Density (ft/ft^2)	Number of Transverse Cracks[a]	Longitudinal Cracks, Ft per 100 Ft of Pavement
Two-layer, 400 ft east of railroad crossing, north lane	518	0.211	11	246
Two-layer, 300 ft east of railroad crossing, north lane	520	0.212	12	220
Two-layer, 100 ft east of second interchange from west end, south lane	580	0.237	14	250
Two-layer, 800 ft west of first interchange, north lane	510	0.208	12	226
Two-layer, 900 ft west of first interchange, north lane	420	0.171	11	125
Two-layer, 800 ft west of first interchange, south lane	515	0.210	12	215
One-layer, 1,000 ft east of Miss-7 crossing, south lane	400	0.163	9	185
One-layer, 200 ft eadt of Miss-7 bypass, south lane	330	0.134	8	110

[a]Per pavement section, 100 ft long; 24 ft, 6 in. wide.

Figure 14. Pavement surface of single-layer section.

Figure 15. Pavement surface of two-layer section.

for the two-layer section to have more cracks is that the top and bottom layers created the usual soil-cement cracking pattern. The top layer not only produced the regular cracking pattern but also showed the reflective crackings that stemmed from the bottom layer.

For curing the layers, the specifications of the Mississippi State Highway Department require that each course, top or bottom, of the completed cement-treated material be covered with a bituminous curing seal. The curing seal should be applied as soon as possible. The entire surface should be kept continuously moist until the curing seal is applied. The curing seal used should consist of a rapid or medium curing cut-back asphalt (grade as designated by the project engineer), which is applied at a minimum rate of 0.2 gal/yd^2 (0.9 litres/m^2). The seasonal limitation for placement of prime coat does not apply to use of bituminous material as a curing seal.

The asphalt curing seal applied on any course should be continuously maintained intact and applied as many times as necessary during the 7-day curing period. We did not make any statement about the curing in the paper and do not believe that the curing method Mississippi used had any influence on the performance of the soil-cement base course.

We realize there are numerous factors controlling the performance and failure of a soil-cement base. Mississippi stopped using the two-layer design as a result of the unsatisfactory performance of several two-layer design projects. However, the two-layer design and construction practice, which involves relatively thin layers, was never isolated or chosen as the only cause for soil-cement distress.

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PROCEDURE FOR ECONOMIC DEVELOPMENT OF SOIL-CEMENT MIX DESIGN

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ABRIDGMENT

The purpose of this paper is to provide design engineers and planning agencies with a method that incorporates economic considerations in developing the mix design of a soil-cement for highway construction. Modifications of the procedure can be made for other types of projects such as airfields, dam facings, and erosion control projects. The procedure is a step-by-step method incorporating planning, field sampling, preliminary laboratory testing, cost analysis, and final testing to develop a cement factor for construction.

• THE BASIC approach used for development of a successful soil-cement program considers two fundamental criteria: the durability and economy of structural load-carrying capacity. There have been many research projects, technical papers, standard test methods, and development of construction practices that ensure a structurally satisfactory job, but little is available in printed form to aid in the selection of economic considerations. The program that is discussed integrates both of these factors to cover the basic essentials of soil-cement construction.

Constructing soil-cement involves mixing the soil, cement, and water. Compacting, finishing, and curing complete the operation. In considering the economics involved, only a few variables are applicable: the materials to be used in the mix and the method of construction. In the material selection, water and cement are least variable from an economic standpoint. To ensure that the cement is sufficient to produce desired results requires specification of the type of cement to be used (e.g., normal Type I, conforming to ASTM C 150). Nearly all producers of cement meet this requirement. Variances in cost are normally concerned with the cost of transporting the cement. The water supply is usually specified to be free from substances that could interfere with the hardening of the soil-cement. If the water is safe for drinking, it generally meets this requirement. Again, the economic considerations generally are reduced to the cost of transportation. The third component of the mix, soil, presents a wide choice of sources or combinations of sources to achieve an economic mix design and will be discussed in detail.

METHODS OF CONSTRUCTION

There are two basic methods widely used in soil-cement construction that will produce consistently good results.

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The original manuscript on which this abridgment is based is available in Xerox form at the cost of reproduction and handling from the Transportation Research Board. When ordering, refer to XS-49, Transportation Research Record 501.

Mixed-in-Place Method

This method uses part or all of the in-place soils in the mix. After the roadway has been prepared to the lines and grades on the plans, a blanket of cement is applied to the surface corresponding to the cement factor specified for the thickness and type of soil being used. The soil and cement are mixed together by single-pass or multipass machines. Water is added and mixed in, and the mixture is compacted, shaped, and cured to complete the soil-cement process. How expensive elimination of the cost of procuring and transporting all or part of the materials is should be considered.

Central Plant Mix Method

This method must use 100 percent borrow material that is transported to the roadway in dump trucks and that is proportioned with the specified cement factor and water in a batch plant or in a continuous mix plant. The mixture is spread (through mechanical spreaders), compacted, shaped, and cured. Because of the 30-min limitation on transporting the mixture, the plant must normally be moved when the haul distance exceeds about 5 miles (8 km). The economic consideration for this method is the ease of quality control during construction.

During the investigation and evaluation of materials used for construction, both the in-place and central plant mix method should be considered to determine the economics involved. If there is not a great deal of difference, the contractors should be permitted to bid them as alternates.

SOILS

It has been claimed that there are more different soils in the world than there are people. This might present us with an insurmountable task for selecting the most economic soil for a given project. Fortunately, however, the total range of soils can be broken down into a few groupings by physical and mineral characteristics that greatly simplify the problem. Each group requires a limited range of cement factors that can be determined by standard test methods. Better than 80 percent of the earth's soils or soil mixtures can be treated successfully with less than 12 percent cement. The design engineer must select from the soils available to him that soil, or combination of soils, which will produce the desired results by balancing the three cost factors involved with that soil:

1. Cost of buying, excavating, and transporting soil;
2. Physical properties of the soil that affect the costs of mixing, compacting, and finishing; and
3. The cost of the amount of cement necessary to give the mixture the desired structural properties.

TYPES OF CEMENT STABILIZATION

There are several classifications of cement stabilization, which are intended to solve a particular problem for the design engineer. They all follow many of the same procedures for testing and evaluation. All are based on the performance of the end product, which, of course, involves the properties of structural strength and durability. The amount of cement required to satisfy these requirements has been developed into standard testing methods adopted by ASTM and AASHTO.

Soil-Cement

Soil-cement is the most commonly used type of cement stabilization. It is intended to be a hard, durable, structural element of a pavement structure that fits somewhere between the load-carrying characteristics of flexible or rigid pavements. It is normally called a semirigid or semiflexible pavement. It is used as a base course under an asphalt wearing surface or as a subbase under a concrete pavement. Because of the abrasion losses from traffic, it should never be used as a wearing surface.

The performance of soil-cement, used as either a base or subbase, has been standardized through research and field evaluation. Regardless of the type of soil used, standard laboratory tests can develop the structural strength and durability desired. In other words, this standard of performance for over 80 percent of the earth's soils can be achieved in the laboratory by varying the amount of cement to meet the requirements.

Cement-Treated

Cement-treated, rather than soil-cement, denotes use of a better grade of soil aggregate in the mix. This is normally a pit-run or river-run gravel with a cement content that produces a higher strength and more durability. Cement treatment is normally used as a subbase under concrete pavement for high-density jet airports. The testing and construction procedures are the same as for soil-cement.

Plastic Soil-Cement

Where restricted construction areas prevent the use of normal construction equipment, some use of soil-cement has been made to overcome this rather specialized situation. Sufficient water is added to the mix to permit placement and finishing in much the same manner as that used for concrete. The tests for determining the cement content are run in a plastic state with the addition of 4 percent cement added to offset the lack of density that results from the excess water used in placement.

Cement-Modified Soil

This classification of cement-stabilized soils is limited to changing some of the physical properties of a soil, and it does not result in a hard, durable structural element. Normally, cement-modified soil is used in much the same manner as lime treatment. Either hydrated lime or cement can be used to lower the plasticity index of a plastic subgrade soil to correct deficiencies in the subgrade and to provide a building platform for subsequent pavement layers. In a highly plastic soil, subject to high volume change, cement-modified treatment, like lime treatment, will provide a relatively impervious layer. This will permit the groundwater to reach a state of substantial equilibrium so that the volume change can take place before subsequent structural pavement layers are applied. In design, it is a good practice not to assign a structural load-carrying capacity to a layer for purposes of reducing the subsequent pavement thickness. This layer is normally considered only as an improved subgrade. The test procedures to determine the amount of cement to be used depend on those properties of the soil that the design engineer wants to modify. As an example, to reduce the PI of a clay, the Atterberg limits test would be run with different amounts of cement to determine the best cement content to achieve the desired results.

Soil-Cement Revetments

In recent years, soil-cement has been used much more for erosion control, ditch linings, levee construction, and dam facings. Because these uses are not subject to abrasion by traffic, they normally require no wearing surface. Tests to determine the cement content to be used are the same as those used for normal soil-cement. Two percent cement is added for structure areas subject to wetting and drying cycles and wave action.

Because soil-cement, used as a base or subbase for highways, airports, streets, and parking areas, is by far the most commonly used type of cement stabilization, the discussion will be directed to a procedure for the mix design that is applicable to this process. To arrive at the most economical pavement structure requires that two variables be evaluated: (a) soils to be used in the mixture, and (b) the method of construction. These two variables are interdependent, with the selection of the soils probably being the more important for the sake of economy.

PROGRAM FOR MIX DESIGN

Soils Evaluation

The design of the mix to be used for construction will follow a procedure of laboratory tests, evaluation of costs, and a study of data developed previously.

Use of Available Data—When the design engineer has decided to use soil-cement for a particular highway project, several tools will be available that should prove very useful for selecting materials for construction:

1. Aerial photographs and topographic, geological, Agricultural Soils Association, and soils series maps are useful in identifying soils in the highway right-of-way and for the location of prospective borrow pits.
2. The soils boring program establishes subgrade bearing values for design.
3. Visual inspection of cuts along the right-of-way, showing the soil profile, and inspection of existing borrow pits and riverbeds should help develop an overall picture of materials available in the highway area.

Tables of average cement contents for different soils can be found in the Soil-Cement Laboratory Handbook, published by the Portland Cement Association. From these tables, one can see that the better graded granular soils require far less cement than do silts and clays for the same standard of load-carrying capacity and durability. Possibly there could be well-graded, granular soils in the roadbed, but this does not occur often, except in an old, granular-base road that is to be reconstructed with soil-cement. More often, the existing subgrade will consist of a material that, by itself, cannot be treated as economically as a borrow soil or a combination of a borrow soil and the in-place soil.

Cement Versus Soil Gradation—To better understand the factors that determine the amount of cement required for a soil, it helps to picture what takes place with several different types of soil. First, in the case of a well-graded, pit-run gravel (AASHTO A-1-a), 3 to 5 percent cement normally will provide the desired strength and durability. The makeup of this soil has just enough of each size of soil particle, from gravel size to sand, silt, and clay sizes, to fill most of the voids when the mass is compacted to a high density. In this condition, then, very little cement is required to permanently cement one soil particle to another. In the case of a uniformly graded sand (AASHTO A-3), the cement contents range from 7 to 11 percent, which is more than double the cement required for an AASHTO A-1-a soil. This results because considerable amounts of voids between soil particles after compaction must be filled with cement if the soil particles are to be cemented together.

In the case of a fine-grained soil, clay (AASHTO A-7-5 or A-7-6), the cement contents range from 13 to 16 percent. The soil has far more soil particles than the same mass of a well-graded gravel. More cement is required to stick all of these fine particles together. Because the amount of cement required for any particular soil is a function of the gradation of that soil, it is easy to see that any improvement of the gradation will result in a lower cement factor. Take the case of an existing, uniform sand in the roadway that requires 7 to 11 percent cement (AASHTO A-3) and an available borrow pit with a clay sand (AASHTO A-2-6) that requires 5 to 9 percent cement. If these soils were blended together, using 50 percent of each, an improvement of both soils that would require about 4 to 8 percent cement would result.

Planning of Economic Mixes

Obviously, the more material in place in the roadway that is suitable for the mixture, the less costly the mixture will be. However, use of in-place soils also dictates that the construction procedure must use mixed-in-place construction. More commonly, all borrow soils used are from pits located on or near the project. It all boils down to balancing the cost factors involved in acquiring and handling soils versus the cost of the cement required.

Procedure for Selection, Testing, and Evaluation of Materials

A workable method for selection of materials to be considered for final mix designs follows:

1. Evaluate existing data from sources previously discussed (e.g., soils maps, aerial photographs) to determine locations of prospective materials to be sampled and tested.
2. Send a field crew out to selected locations to obtain representative samples from each site.
3. Perform preliminary laboratory tests consisting of gradation, Atterberg limits, and standard AASHTO moisture-density relationships.
4. Make combinations of soils, from results of preliminary tests, that will improve the properties of each.
5. Mold sets of three cylinders each at 6 percent cement for each prospective mix, cure for 7 days, and break to determine the average compressive strength for each set of cylinders. Normally, a compressive strength of less than 200 psi would eliminate any of the prospective mixtures from further consideration. (For silts or clays, a higher cement factor can be used.)
6. Submit the remaining prospective mixtures to the shortcut test method for sandy soils.
7. Estimate the costs from the results of this test to determine the most economical mixtures for further consideration.
8. Determine the cement factor to be used for construction by testing the remaining one or two mixtures for each given section of the project with the standard ASTM or AASHTO method.

Inasmuch as most soils can be adequately hardened to produce a given standard of structural strength and durability by varying the amount of cement, it follows that economics will be the deciding factor. For sizable projects, it is certainly worth the time, effort, and expense to logically evaluate all the materials that show a promise for use in construction.

Tests for Economic Mix

The use of a simple compressive strength test using 6 percent cement enables the engineer to substantially reduce the overall testing program. The soil mixes above a compressive strength of 200 psi are now run through the shortcut test method for sandy soils.

This test provides a safe cement factor for the soil mixes that may be the minimum cement factor possible or 1 to 3 percent above it. There is no way to determine the minimum cement factor used in construction without running the standard ASTM or AASHTO test for wet-dry and freeze-thaw (ASTM D559-57 and D560-57 and AASHTO T135-57 and T136-57). The difference between 1 and 3 percent cement for large projects can be considerable for cement that is not necessary for the quality of the end product. Therefore, the cement factor for the final mixture must be determined by these tests to achieve the most economic mixture. To complete the 12 cycles required for these tests normally takes between 6 and 8 weeks. To run more than the most promising mixtures through this phase of testing would be a waste of time, effort, and money. This hypothetical project, like most projects, has many prospective mixtures that could be used in construction, and elimination of all but the most promising from the time-consuming and costly standard tests is desirable.

Elimination can be accomplished readily with a simplified cost analysis based on the comparative cement factors, developed in the shortcut test for sandy soils, and on the projected costs of materials delivered to the job.

Because we are still in the preliminary phase of the investigation, we assume that samples of soils taken substantially represent the materials in the roadway and in the selected borrow pits. It is now advisable to substantiate these assumptions for both the quality and quantity of the materials to meet the project requirements.

This requires a more thorough investigation of the two borrow pits selected and of the soils in the roadway. A recommended procedure to test a prospective borrow pit is to take undisturbed samples, with Shelby tubes or split-spoon samples with a sufficient number of borings to effectively determine the limits of the desirable materials in the pit. From this boring log, the soil profile of the pit can be drawn, and a calculation made of the quantity of materials available. Analysis of the soil profiles establishes the basic requirements the contractor must follow in excavating the borrow pit to ensure that materials taken from the pit have the same proportions from each horizon or layer as those used in mix design testing.

Boring is performed on the undisturbed samples to be used in the mixture to prove that the materials in the roadway are substantially the same as those used in the mix design test. Normally, this program consists of borings every 300 to 500 ft (90 to 150m) in a staggered pattern—right side, centerline, left side, centerline, right side, and so on. If there is an apparent soil change between borings, the area of change is determined by intermediate borings. If the changed areas are relatively minor, it is most often more economical to undercut and remove a soil that would require more cement than it is to change the cement factor for a short distance.

Final mix design samples are collected from the borrow pits and the roadway and are combined in the same proportions as those used in the preliminary mix design to see if their physical characteristics agree with those of the original testing.

The normal project calls for payment for cement as a separate bid item. This eliminates the gamble, on the part of the contractor, for the amount of cement used in construction and makes the engineer responsible for determining what material will be used and what cement factor will be required for construction. To put this responsibility on the contractor would necessitate duplicate laboratory testing programs that would increase the cost of the project, in addition to requiring the contractor to hire the personnel or equipment to perform this phase of engineering design.

Final Mix Design

We have established the sources and mixtures of materials used in construction and we must now determine the most economical cement factor to be used. For the average soil-cement project, the wet-dry and freeze-thaw tests, as established in ASTM and AASHTO procedures, will prove adequate for construction. The results of the tests take into account minor variations, soil conditions, and construction methods. During a construction project, the contractor should perform the work substantially within the limits specified. However, it is unreasonable to expect that these limits will not be occasionally violated. Construction specifications are regarded as a guide, and minor variations are expected and are accounted for in mix design testing.

Laboratories that are not equipped with apparatus to perform the freeze-thaw test often substitute a minimum compressive strength test in lieu of the freeze-thaw test. A workable criterion in this case would be to run the wet-dry test in conjunction with an unconfined compressive strength test requiring a minimum strength of 350 psi at 7 days.

Procedures to Be Avoided

Experience indicates a few factors in mix design testing and evaluation procedures that should be avoided to assure a successful construction project.

1. The engineer should insist that laboratory tests be run to determine the cement factor. Guessing at a proper cement factor or assuming that tests run on a similar soil will require the same amount of cement without check testing is very dangerous. It is better not to use soil-cement unless an adequate testing program is used in the mix design and unless there is quality control of construction.

2. Testing procedures are all based on the widely used normal soils test: the moisture-density relationship (ASTM D 558-57 and AASHTO T 134-57). This calls for a standard compactive effort using a $\frac{1}{30}$-ft^3-capacity Proctor mold with three layers compacted at 25 blows per layer with a $5\frac{1}{2}$-lb hammer falling 12 in. Basic research to determine the cement factor has been based on this compaction. It can be disastrous to assume that, if standard AASHTO compaction is adequate, modified

AASHTO is better for the mix design test. With most soils, a 1 to 3 percent lower cement factor will be indicated with a modified AASHTO compactive effort; this can cause failures in the construction phase.

3. Locally developed tests that are substantiated by PCA, ASTM, and AASHTO standards should be avoided.

4. The most economical soil mixtures generally fall into the granular material category. Heavy clays are impractical because they require a high cement factor and are difficult to adequately mix with cement. Pretreating a high-PI clay with hydrated lime can eliminate the disadvantages in extreme cases where granular soils are not available. Organic topsoils are normally avoided; however, they, too, can be used by neutralizing the harmful effects of the organic acids with an additive of calcareous material such as calcium chloride.

Testing procedures to determine the most economical cement factor for construction are easy to use and give consistently reliable results. Following the described investigation program to accompany these test procedures should produce a project that is not only satisfactory structurally but also economically.

COLD WEATHER LIME STABILIZATION

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In recent years, lime stabilization of poor quality subgrade soils to upgrade quality or provide acceptable subbase material has increased in popularity as a construction alternative. However, in many situations, specifications relative to cutoff dates for lime stabilization construction have rendered the alternative unfeasible. Previous studies have revealed that soil-lime mixtures cured prior to subjection to freeze-thaw conditions undergo autogenous healing that resulted in continued strength gain. The purpose of this study was not to evaluate the ramifications of autogenous healing of soil-lime mixtures but to evaluate the behavior of soil-lime mixtures subjected to cold weather stabilization. The basic premise involved in this study was that soil-lime mixtures subjected to freeze-thaw conditions immediately after compaction would not undergo pozzolanic reactions until favorable curing conditions were attained. Soil-lime reaction that would not occur during cold weather treatment would then be renewed under favorable conditions to produce latent strength gains. The scope of this study involved the investigation of the behavior of only one soil subjected to cold weather lime stabilization. The selected soil was evaluated at only one lime content, which was established as the stabilization lime content for that soil.

•STRENGTH GAINS associated with lime stabilization of clayey soils are derived primarily from cementitious bonds developed as a result of pozzolanic reactions. Pozzolanic reactions are lime-soil reactions involving calcium ions that are provided in the form of high-calcium lime and natural pozzolans in clayey soils that are hydrates of silica and alumina. The rate of reaction is highly dependent on reaction temperature as are other less complex chemical reactions.

Anday (1, 2) conducted studies on curing lime-stabilized Virginia soils and revealed that pozzolanic reactions, thus strength gains, were curtailed below temperatures of approximately 50 F (283 K). Through these studies, he found that only one strength-maturity curve could be expected for a given lime-stabilized soil provided that the minimum temperature required for lime-soil reaction exists. Inasmuch as 50 F (283 K) was observed to be approximately the minimum requirement, his data revealed that a maturity of 750 degree-days [calculated with 50 F (283 K) as the datum temperature] is required to develop strength gains in the field. (A requirement for development of approximately 750 degree-days after lime stabilization should be used only when cementation by pozzolanic reaction is required or desired.)

Autogenous healing is the phenomenon that occurs when pozzolanic reactions occur along newly exposed reaction surfaces, thus producing cementitious bonding along fractures and failure planes. Studies conducted by Thompson and Dempsey (3) revealed that autogenous healing occurred in cured lime-soil mixtures after they were subjected to freeze-thaw cycles. Samples tested in this study were rapidly cured for 2 days at 120 F (322 K) before freeze-thaw testing. Unconfined compression tests were conducted immediately after freeze-thaw cycles and at various time periods after completion of freeze-thaw tests. Results of these tests revealed increased strength with time as a result of autogenous healing.

Evaluation of frost parameters in freeze-thaw testing of soil-lime mixtures proved to be quite variable in relation to climatic conditions, geographical location, and the

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position of stabilized materials in the pavement section (4, 5). From these results it is evident that, with current knowledge, the testing of lime-stabilized materials should be correlated with local climatic conditions. Although there exists a wide variety of frost parameters throughout the United States, a standard laboratory testing procedure for freeze-thaw testing should be developed and standardized for lime-stabilized materials.

MATERIALS AND TESTS

The residual limestone material used in the study was obtained from a depth of approximately 10 ft (3.0 m) below the surface in an excavation located west of Knoxville, Tennessee. The soil was classified as a CH material according to the unified classification system and as an A-7-6(11) material according to the AASHTO classification system. Some of the physical properties of the soil are as follows: specific gravity, 2.72; liquid limit, 64; plastic limit, 40; plasticity index, 24; and percentage of material passing sieve No. 10, 100; No. 40, 96; No. 80, 78; No. 200, 50; and 0.02 mm, 42.

The percent high-calcium, hydrated lime used in stabilizing the soil was established by means of the pH method recommended by Eades and Grim (6). Figure 1 shows the results of the pH tests on soil-lime slurries. Although 4 percent by dry weight of hydrated lime produced a pH of 12.5, a lime content of 6 percent was used for stabilization because this percentage was found to produce maximum 28-day strength gain under normal curing conditions.

A standard procedure was adopted for preparing soil-lime mixtures to ensure uniform moisture distribution during the compaction process. All specimens were compacted with a Harvard miniature compaction apparatus that used kneading compaction. Moisture contents and densities of all samples were held within 1 percent of optimum moisture content and 1 lb/ft^3 (16.01 kg/m^3) maximum density respectively. Test specimens were wrapped and sealed in wax immediately after compaction. Control specimens were subjected to normal curing for periods of 7, 14, 21, and 28 days. Other test specimens were subjected to various cycles of freezing and thawing.

Because no recommended or standard procedure for freeze-thaw testing of lime-soil mixtures exists, a procedure similar to that for soil-cement mixtures (AASHTO T 136-57) was adopted. Specimens were subjected to a temperature of 0 F (255 K) for a period of 24 hours before being thawed for the same period in a moist room at 73 F (296 K). Thus, a period of 48 hours was required for a complete freeze-thaw cycle. During the freeze-thaw testing, no attempt was made to prevent volume change of the specimens. Moisture contents and compacted moisture contents were the same after freeze-thaw testing.

Test specimens were subjected to freeze-thaw cycles ranging from one to six. Subsequent to freeze-thaw treatment, samples were tested by unconfined compression at intervals of 0, 7, 14, 21, and 28 days after normal curing.

At least three test specimens were used to establish each strength data point. Consequently, 18 specimens were tested from each sequence of freeze-thaw treatment.

RESULTS

Soil-lime mixtures subjected to adverse curing conditions immediately after compaction lay dormant until curing conditions conducive to lime-soil reactions were present.

Figure 2 shows compressive strengths for samples that were subjected to various cycles of freeze-thaw after curing under normal conditions. Because specimens subjected to freeze-thaw cycles were not restrained from volume change, there exists a significant loss of strength in all specimens at zero curing time. Samples subjected to three and six cycles of freezing were found to possess negligible compressive strength at 0 day's curing. Subsequent curing of freeze-thaw specimens produced rates of strength increase nearly the same as those for control specimens.

Although the freeze-thaw tests to which the samples were subjected are more severe than field conditions, results indicate that 28-day strength gains under normal curing conditions will be attained by lime-soil mixtures subjected to adverse curing conditions immediately after compaction, if conducive curing conditions do occur at some later

Figure 1. Results of pH tests on soil-lime slurries.

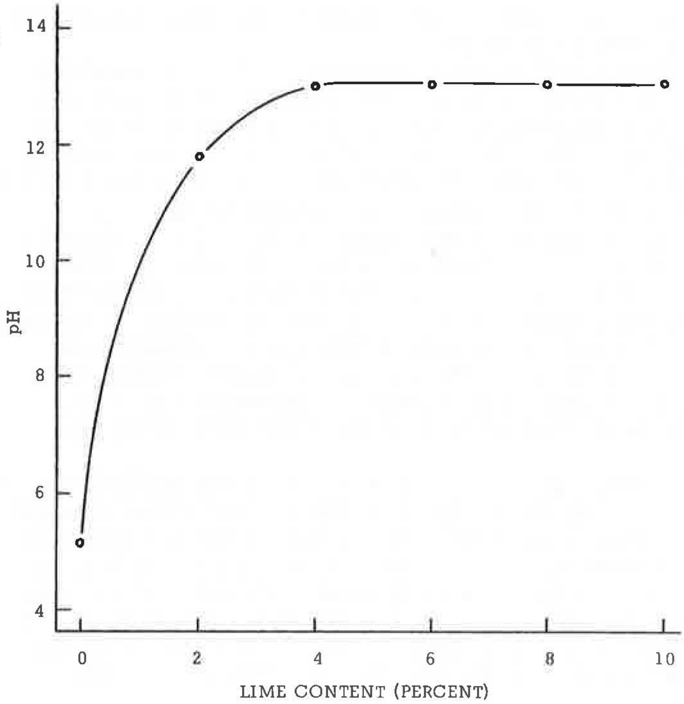
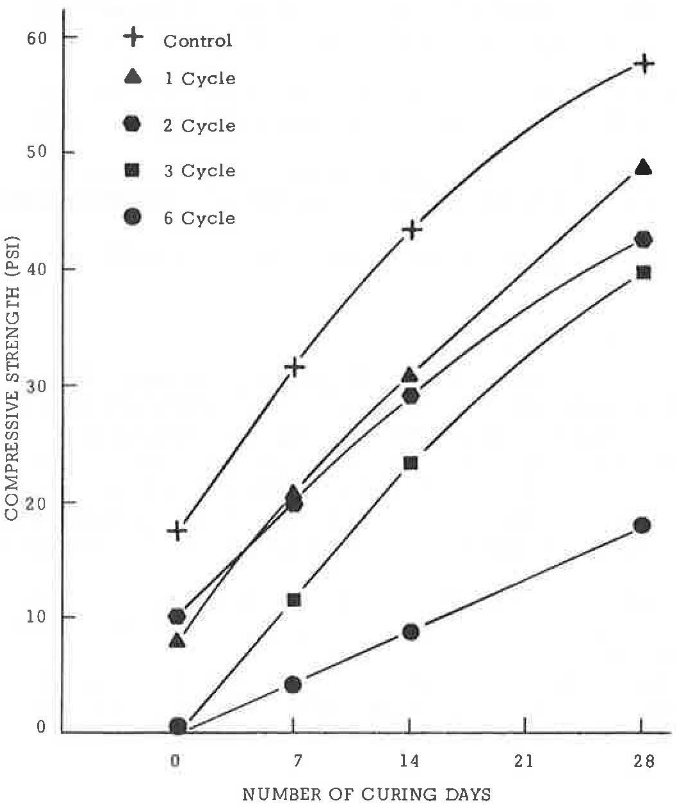


Figure 2. Compressive strengths of samples after normal curing and freeze-thaw cycles.



date. Consequently, pozzolanic reactions may be dormant during low temperatures but regain reaction potential under suitable temperatures.

Because 28-day compressive strength of lime-clay mixtures has become somewhat standard for relative strength comparisons, the 28-day strength gain of all specimens was analyzed as a percentage of the control specimens for various freeze-thaw cycles. Figure 3 shows the results of that evaluation. Results for this particular soil indicate that almost a linear relationship exists between relative strength and freeze-thaw cycles.

As previously mentioned, Anday found that approximately 750 degree-days [50 F (283 K)] were required for soil-lime mixtures to reach maturity in the field. Based on this hypothesis, an investigation of curing conditions in the Knoxville area was undertaken. When average air temperatures were used, lime stabilization could feasibly be continued into late fall if freezing temperatures during hours of construction did not prevent adequate mixing operations. Inasmuch as an accumulation of degree-days would continue well into winter months, no adverse effects should be expected relative to strength gains at maturity. Because ground temperatures are generally somewhat higher than air temperatures, the estimations of cold weather lime-soil curing are conservative.

If a relationship similar to that shown in Figure 3 were developed for particular soils, an approximation could be made regarding the curing time required to produce strength gains comparable to those under normal conditions. Generation of this type of information would allow cold weather lime treatment to be conducted when other climatological factors permitted. Soil-lime mixture would lie dormant during extreme weather conditions only to be renewed in favorable curing conditions. Construction practices of this nature would allow lime stabilization to be conducted well into winter months in areas where favorable climatic conditions exist, with the exception of ambient temperatures that may fall below freezing at night.

CONCLUSIONS

Based on data from this study, the following conclusions may be drawn:

1. Lime-stabilized soils subjected to extreme freeze-thaw conditions immediately after compaction undergo significant strength gains when curing conditions become favorable for pozzolanic reactions;
2. Residual strength gains obtained after severe freeze-thaw testing indicate that cold weather lime treatment may extend construction cutoff dates under certain conditions;
3. Development of realistic cutoff dates for lime stabilization should be based on local climatic conditions, soils, and performance requirements for lime-stabilized subgrades; and
4. Development of a standard procedure for freeze-thaw testing of lime-soil mixtures should be investigated.

CASE HISTORY

Expansion of the McGhee-Tyson Airport in Knoxville, Tennessee, involved the construction of a new terminal building with aprons and taxiways to service the new facility.

Pavement designs and compaction requirements for the aprons and taxiways were based on a 350-kip (1 556 878 N) dual-tandem wheel load, gross aircraft weight. The rigid pavement design consisted of 14 in. (35.6 cm) of portland cement concrete, 7 in. (17.8 cm) of cement-treated subbase, and 6 in. (15.2 cm) of lime-stabilized subgrade. The rigid pavement design was based on a modulus of subgrade reaction of the stabilized subgrade of 300 pci (8.3 kg/cm^3).

The natural subgrade soil used in construction of the lime-stabilized layer was a residual soil weathered from dolomitic limestone. This soil is highly plastic and is classified as an E-8 or an A-7-6 material.

The lime-stabilized subgrade was designed for 6 percent by dry weight of hydrated lime. Twenty-eight-day unconfined compressive strengths obtained from this design averaged 400 psi (28.1 kg/cm^2) when compacted at modified AASHTO density.

Figure 3. Results of evaluation of 28-day strength gain as a percentage of control specimens.

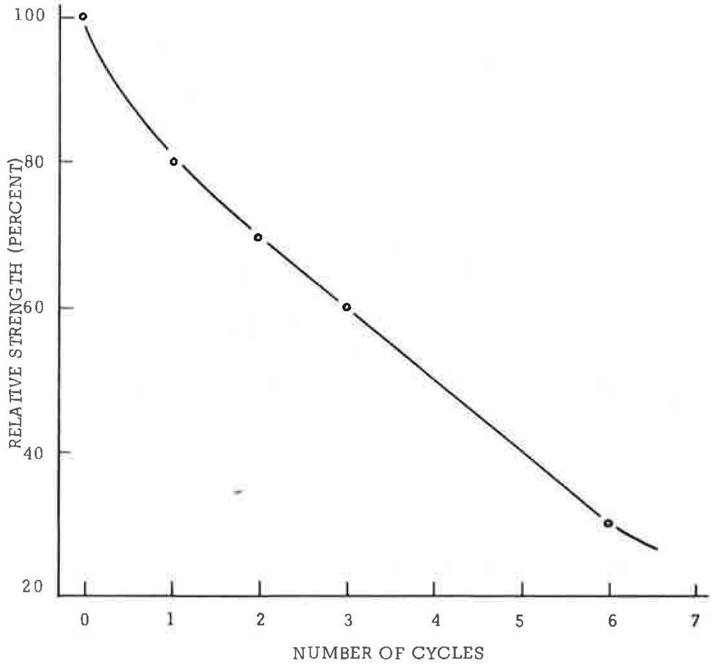


Figure 4. Results of plate bearing test for lime-stabilized subgrade.

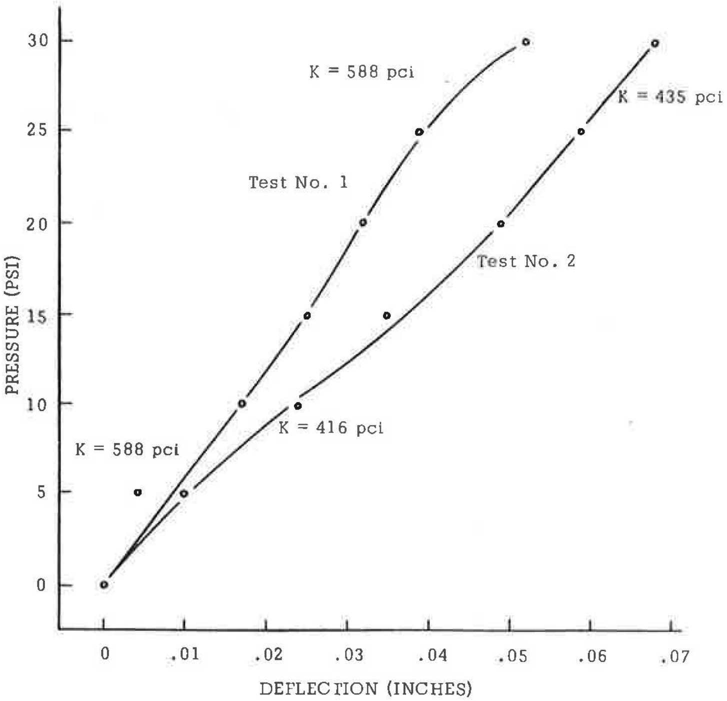


Figure 5. Cumulative degree-days versus time curves.

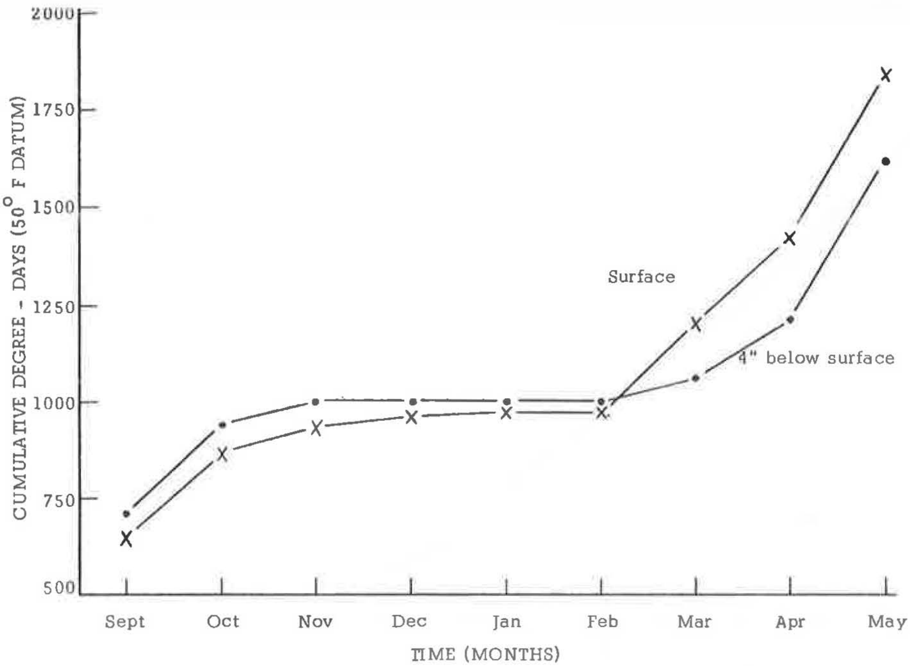
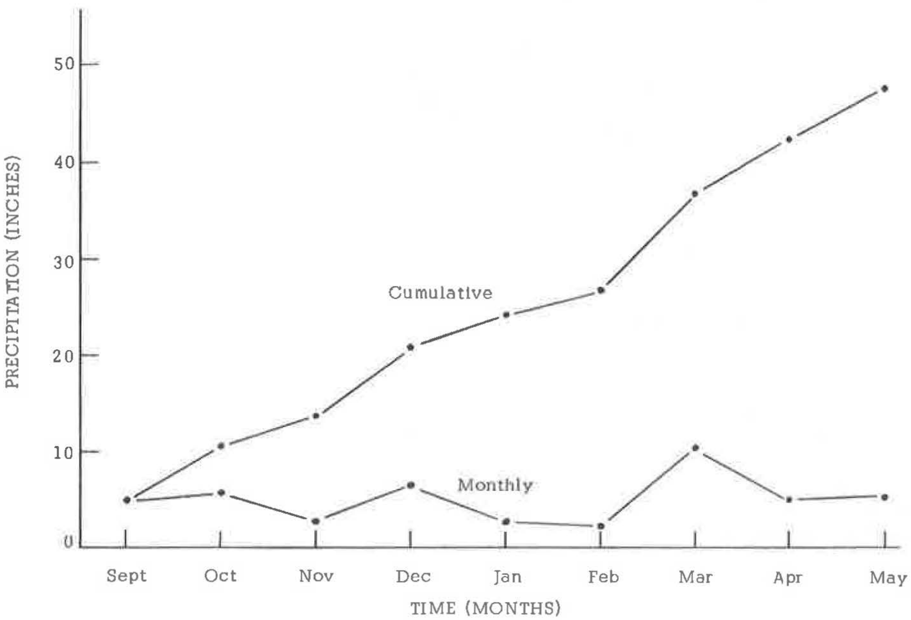


Figure 6. Precipitation versus construction time for McGhee-Tyson Airport.



Before the initiation of the lime stabilization construction, a test section was built to establish construction procedures and to ensure that a modulus of subgrade reaction in excess of 300 pci (8.3 kg/cm^3) would be obtained with the lime stabilization design. Figure 4 shows the results of the plate bearing test conducted on the test section. As indicated, the K-values obtained from two tests were in excess of 400 pci (11.1 kg/cm^3) with an average of 511 pci (14.2 kg/cm^3) resulting from the test. Modulus of subgrade reaction values obtained from six tests on natural soil averaged 100 pci (2.8 kg/cm^3).

Construction of the lime-stabilized subgrade began around September 1, 1972, and continued to mid-November 1972. During this period, approximately half the lime-stabilized subgrade was completed. Inspection of the cumulative degree-days versus time curves (Fig. 5) reveals that almost no curing occurred from November 1972 until February 1973 when the temperatures were such that degree-days of curing began to accumulate rapidly.

Performance of the lime-stabilized subgrade that was unprotected throughout the winter and spring clearly demonstrated the advantages of cold weather lime stabilization. During the period of dormancy, the lime-stabilized subgrade was subjected to a series of adverse conditions relative to both temperature and precipitation. As shown in Figure 6, approximately 48 in. (122 cm) of precipitation accumulated from September 1972 until May 1973 when construction was resumed. In March 1973, 10.25 in. (26 cm) of rainfall occurred giving rise to both a 50- and 100-year flood in east Tennessee.

Despite the extremely adverse conditions, the unprotected lime-stabilized subgrade protected the final grade from excessive erosion and saturation. The lime-stabilized subgrade layer was in excellent condition at the beginning of the 1973 construction season. After the excessive rainfalls in the spring, the stabilized subgrade was readied for application of the cement-treated subbase by rerolling to tighten the surface that had fluffed during periods of freezing.

Experience at the McGhee-Tyson Airport has demonstrated that lime-stabilized subgrade preparation late in the construction season is beneficial relative to protecting finished grades and expediting continued construction operations at the beginning of a new construction season.

ACKNOWLEDGMENT

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IMMEDIATE AMELIORATION OF WET COHESIVE SOILS BY QUICKLIME

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Immediate amelioration of wet cohesive soils by lime, a current practice in earthwork operations for highway construction in Belgium, is performed with 1 to 1.5 percent quicklime. Such small doses are effective because they agglomerate the soil into crumbs that are stable in free water and retain their individuality after kneading and compaction. Procedures for evaluating the crumb stability and the amount of lime needed for adequate CBR value have been developed. Additions of 1 percent quicklime produce not only the immediate amelioration effects but long-term strength gains as well. Strontium and barium hydroxides produce the same immediate effects as equivalent amounts of lime but far lower long-term strength gains. Lime percentage has far less incidence on immediate amelioration than on long-term strength gains.

• IN BELGIUM, and in neighboring parts of France and Germany, large areas are covered with loess soils, which are moderately plastic clayey silts. Summer in northwestern Europe is rather wet and cool, and the moisture content of the loess at depths of 2 ft or more under the natural surface remains close to the plasticity limit the year round. This limit is about 20, about 8 percent higher than the optimum water content for compaction. When these soils are excavated from hills, they are too wet for appropriate compaction; therefore they were discarded in the past and replaced by borrowed materials for the construction of highway fills and embankments.

Improvement of loess by a lime additive has become a current process of earthwork operations, resulting in appreciable cost and time savings in highway construction.

Research findings of the Centre de Recherches Routières on the lime treatment of loess soils are discussed in this paper. The investigation, initiated in 1970, deals primarily with the short-term ameliorating effects of the lime; more research devoted to the long-term effects is still in progress.

LIME TREATMENT OF SOILS IN BELGIUM

Since 1960, much research has been devoted to the effects of lime treatment of cohesive soils. However, as pointed out by Neubauer and Thompson (1), most of that research has considered the long-term effects, or cured strength, of the lime. Fewer findings are available on the immediate effects that result in an amelioration of the geotechnical properties of uncured lime-soil mixtures as compared to the natural soil.

In Belgium, improvement of wet cohesive soils by quicklime has been used for earth work operations in highway construction since 1968.

Powdered quicklime is generally used in Belgium for the treatment of soils because it is cheaper than slaked lime. Hazards of using quicklime are largely assuaged by pneumatic handling, and use of protective clothing and spectacles is recommended (4, 5). The grade of quicklime currently available is approximately 100 percent passing No. 10 (2 mm) sieve and 40 to 50 percent passing No. 200 (75 μm) sieve. The free

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CaO content of the lime, measured by the "rapid sugar method" (ASTM C 25-67), is 90 to 95 percent. Slaking in water (ASTM C 110-67) is rapid (less than 10 min).

When mixed with a wet soil, the calcium oxide in the powdered quicklime is readily converted into calcium hydroxide, which dissolves in the soil water in exactly the same way (about 1.4 grams/litre) as if slaked lime had been added. From then on, the effects of the addition of 1 part quicklime of the grade described are identical to those brought about by 1.3 parts slaked lime of the same degree of purity.

DRYING EFFECTS OF QUICKLIME AND EXPOSURE

Use of quicklime, which is an advantage when wet soil is treated, results in a decrease in the water content of the mixture. This drying effect is brought about by the chemical fixation of the hydration water, by evaporation of some water by a part of the heat of hydration, and by the increase in weight of the solids caused by the lime addition (2). The drying effect was shown by Van Ganse (3, 4, 5), by laboratory and field experiments, to be about 0.65 percent loss in moisture content per gram of commercial quicklime added to 100 grams of dry soil. Field experiments have shown (4, 5) that exposure of the fresh, loose lime-soil mixture to sunshine and wind can result in a much larger loss of moisture, up to 2.5 percent, than can be obtained by the action of the usual small dose of quicklime.

IMMEDIATE EFFECTS OF SMALL DOSES OF QUICKLIME

Immediate effects of additions of quicklime to Belgian loess soils have been reported (4, 5, 6, 7) and are comparable to the findings of other authors (1, 8, 9, 10, 11). The following have been observed on many types of loess and other clayey soils: (a) an increase of the plastic limit; (b) improvement of trafficability of the soil; (c) a shift of the moisture-density relationship toward a lower maximum dry density and a higher optimum water content, allowing a higher relative dry density to be obtained by compaction; and (d) a substantial increase in the CBR value of the compacted lime-soil mixtures.

Evidence was obtained, however, that the instant improvements of the soils do not increase proportionally with the amount of lime. In fact, about two-thirds of the effects produced by 3 percent quicklime are obtained with an addition of 1 percent, at the same water content.

The actual decrease in water content, initiated by the slaking of the quicklime and by exposure of the mixture before compaction, enhances the immediate effects of the lime. However, a very high initial water content of the soil, even when not impeding long-term strength gain by a large lime dose, may prevent sufficient improvement to expedite the earthwork.

PROCEDURE FOR EVALUATING MINIMUM LIME DOSE

A standard procedure has been developed (12) to evaluate the amount of quicklime needed to ameliorate a given soil (with its natural water content) to obtain a sufficient CBR value after compaction. CBR values considered suitable in Belgium are 10 percent for the bulk of highway embankments and 15 percent for 30-cm-thick ameliorated subgrades. The procedure plots moisture content against CBR values (immediately after compaction) for 2-hour-old mixtures of soil and arbitrary (e.g., 3 percent) doses of quicklime. If allowance is made for the loss of moisture that occurs on the construction site during the 2-hour exposure of the loose mix before compaction, the probable CBR strength can thus be evaluated. If the predictable CBR strength is higher than the required value, the test is resumed with a smaller amount of lime.

This laboratory procedure, which has been checked by field trials, has shown that in the majority of cases adequate amelioration of Belgian loess soils with their natural water content is obtained with 1 to 1.5 percent quicklime. In favorable cases where the soils were somewhat drier than usual, quicklime doses as low as 0.5 percent have been effective.

In fact, from 1968 to 1971, over 12 million cubic meters of soil were treated in Belgium with an average dose of 1.65 percent commercial quicklime, with excellent results (5). The actual average quicklime percentage used has decreased from 3 percent in 1968 to about 1 percent today.

EFFECT OF LIME ON SOIL STRUCTURE

The effectiveness of small doses of lime in the instant amelioration of cohesive soils is explained by the primary instant effect of the lime as a modification of the soil structure.

Unsaturated cohesive soils in a loose condition do not fall apart into elementary particles like sand does. They form aggregates, called clods, lumps, or crumbs, according to size. The soil particles may be glued together by substances such as humus or cemented by calcium carbonate, for example. Physicochemical forces in the ionic layer surrounding the soil particles may also act. The main binding force, however, is the traction exerted by menisci of capillary water on adjoining particles. Most soil aggregates disintegrate when immersed in water because capillary forces disappear when saturation arises.

Therefore, the structure or aggregate division of a soil mainly depends on the moisture content. Structure is promoted by drying and shearing and deteriorated by moisture and compression. (Soil structure is important in agriculture because permeability for air and water, which is vital for the root functions in plant life, depends on the voids between aggregates. Tilling the soil is aimed mainly at restoring deteriorated soil structure.)

When a small amount of lime is mixed into a moist, though unsaturated, fine-grained cohesive soil, striking changes appear almost instantaneously. The color of the soil becomes lighter, its gloss duller, and its consistency pulverulent. These changes are brought about by the formation of soil crumbs (granules), into which the mixture readily divides. The size of the crumbs ranges from 0.20 to 0.5 cm (Fig. 1).

Mixing under standard conditions has shown that the mean diameter of the lime-soil granules increases with the water content for a given soil and with the plasticity index for different soils. Freshly prepared lime-soil granules are soft and can be crushed easily between the fingers; however, they do not quite disintegrate when immersed in water.

MEASUREMENT OF CRUMB STABILITY

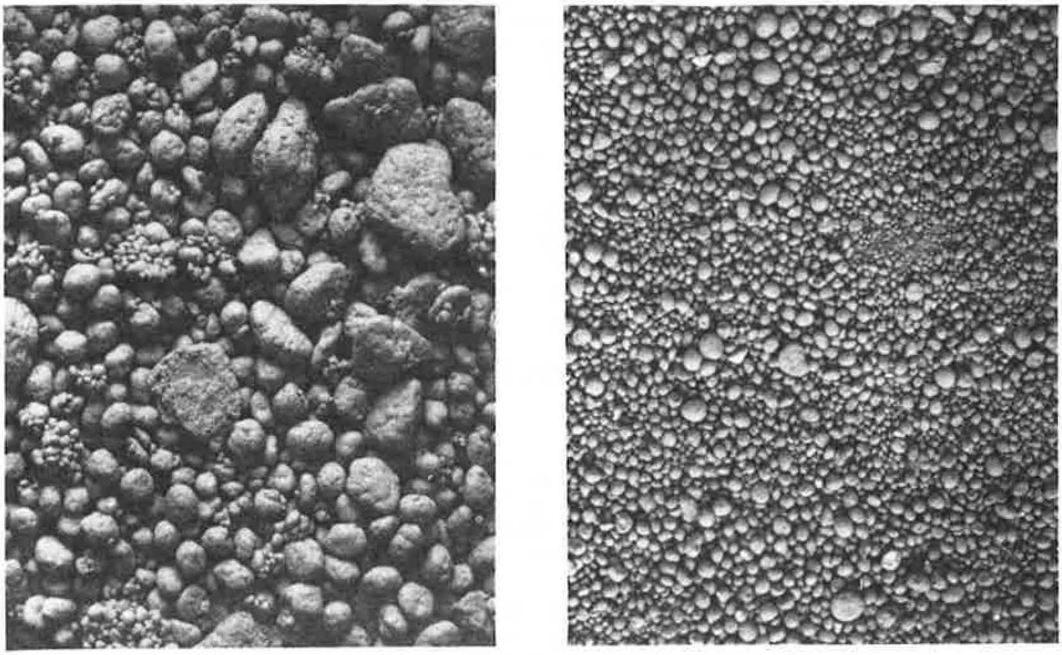
Crumb stability in the presence of free water can be measured by the percentage of dry weight of crumbs retained on a suitable sieve after repeated immersion (13). The size of the crumbs and their stability depend on soil type and moisture content. The stability of lime-soil crumbs depends also on the dose of lime and increases with time because of pozzolanic reactions. A standard procedure for measuring the crumb stability (fixing all the variables) and a suitable automatic sieve apparatus have been developed by the Centre de Recherches Routières (14).

Crumbs of an untreated soil do not, as a rule, disintegrate completely when soaked in water for a rather short time. In the test procedure, a soaking time of only 20 min is usually sufficient to show the stabilizing effect of lime, without unduly increasing the time required for the test.

Comparison of crumb stabilities measured on an untreated soil 2 hours after addition of 32.8 meq of CaO per 100 grams of dry soil (1 percent quicklime of 92 percent grade) is an adequate, simple, and rapid method for evaluating the fitness of that soil for instant amelioration. On 84 Belgian soils that have actually been treated successfully with lime, crumb stability increases of 30 percent or more were found, except in a few cases where high stability of the natural soil crumbs precluded detection of such a large increase.

In many cases, the crumb stability test gives a univocal answer, its main advantages being that only a 1-kg soil sample and 1 hour of an operator's time are needed.

Figure 1. Structure of Belgian loess soil: (a) w_P = 19, I_P = 8, loosened with w = 16 percent; and (b) after mixing with 1 percent quicklime.



a.　　　　　　　　b.

Figure 2. Rupture faces of (a) untreated loess soil, (b) loess soil with 1.5 percent quicklime in the laboratory, and (c) loess soil with 1.5 percent quicklime on a construction site.

EFFECTS OF LIME-INDUCED CRUMB STABILITY

The instant improvements of geotechnical properties of cohesive soils by addition of lime have been explained in relation to an increase of the plastic limit by Hilt and Davidson (15). A more general explanation, however, is suggested because the water-stable crumbs of a lime-treated soil retain their individuality to a large extent when the lime-soil mixture is subjected to kneading and rolling (as in the plastic limit test) or to compaction.

Figure 2 shows rupture faces of broken compacted specimens, which had been prepared from an untreated soil and from the same soil treated with 1.5 percent quicklime. Almost no structure is visible in the specimen of untreated compacted soil, but the treated specimens clearly show the granulated structure of the compacted lime-treated soil with a visible macroporosity along the edges of the crumbs. This macroporosity is thought to be the primary cause of the increase of the plastic limit and of the shift of the Proctor curve toward a lower dry density and a higher optimum moisture content. On the other hand, the increase in CBR strength at the wet side of optimum is also because of the stability of the crumbs, which opposes parallel orientation of clay mineral particles.

NATURE OF IMMEDIATE REACTION

The nature of the immediate reaction of calcium hydroxide dissolved in soil pore water with the clay minerals has been the object of many speculations. Diamond and Kinter (11) reviewed several hypotheses and concluded that cation exchange, flocculation, and carbonation cannot be retained as essential factors of instant modification of soil. They believe that the primary phenomenon is a very rapid physical adsorption of both calcium and hydroxyl ions on the clay surfaces, even when these are already calcium-saturated. They suggest that an immediate reaction occurs between the alumina-bearing edges of clay particles and the lime adsorbed on the faces of adjacent particles. This results in the formation of bonds of tetracalcium alumina hydrate and, possibly, calcium silicate hydrate, which link the clay particles together.

The appearance of water-stable soil crumbs is entirely consistent with the hypothesis of such a linking process. To date, however, evidence of the instant formation of calcium aluminate or calcium silicate hydrate or both does not seem to have been demonstrated. Another type of link, made up by the bivalent calcium ion itself, has been suggested by Grossmann (16).

An essential requirement of the instant reaction is alkalinity of the medium. A neutral calcium salt like $CaCl_2$ exerts, notwithstanding its potent flocculation action, no instant amelioration effects on clayey soils. It is sufficient however to add, with the $CaCl_2$, an equivalent amount of NaOH to obtain the same effects as those produced by lime. The high pH value of a $Ca(OH)_2$ solution obviously implicates increased solubility and reactivity of silica and alumina at the faces and edges of clay mineral particles.

Some experimental work has been performed by Van den Bergh (17) in connection with instant reactions. These experiments involved (a) substitution of Ca^{++} ions by Sr^{++} and Ba^{++} ions, (b) measuring the amounts of hydroxide consumed in early and in later stages, and (c) measuring the respective unconfined compression strengths.

The experiments were performed on five typical Belgian loess soils. Table 1 gives their main characteristics. The main differences between these soils are the calcium carbonate content, which is high in soil 361, and the percentage of particles less than 2 μm in size, which is determined by ASTM D422-63.

Crumb Stability

Crumb stability was measured (14) on the untreated soils and 2 hours after adding a mixture of 1 percent quicklime containing 92 percent free CaO or equivalent quantities of strontium hydroxide and barium hydroxide. The dose in each case was 32.8 meq per 100 grams of dry soil (Table 2). The coefficient of variation (standard deviation divided by mean) of the measured crumb stabilities is about 0.05.

No significant difference appears between the crumb stabilities induced in a given soil by the three hydroxides. The increase in crumb stability in all cases is in the range 30

Table 1. Characteristics of five typical Begian loess soils.

Soil No.	Origin	Horizon	Specific Gravity (g/cm³)	$CaCO_2$ (percent)	Organic Matter (percent)	Liquid Limit	Plastic Limit	Plasticity Index	Percent Passing No. 200	Percent Passing No. 10
361	Sterrebeek	B	2.706	12.1	0.05	27	20	7	98	13
362	Hingeon	B	2.702	2.4	0.04	31	20	11	98	25
363	Orbais	B	2.705	1.8	0.01	28	20	8	98	25
364	Lavoir	B	2.685	1.7	0.25	26	19	7	97	18
365	Tihange	Mixture	2.730	2.6	0.21	32	21	11	93	30

Note: The Belgian loess soils are predominantly illitic and contain chlorite.

Table 2. Crumb stability of loess soils, untreated and treated with hydroxides of Ca, Sr, and Ba.

Soil No.	Crumb Stability (percentage of dry weight) Untreated	Ca	Sr	Ba
361	11	49	55	51
362	51	84	92	92
363	3	66	—	—
364	8	54	63	56
365	57	96	—	—

Table 3. Fixation, in meq, of hydroxides in soil-hydroxide mixtures.

Soil No.	Cation	Hydroxide Fixation (meq/100 g) 2 Hours, 20 C	24 Hours, 20 C	28 Days, 20 C	28 Days, 40 C
361	Ca	8	9	13	27
	Sr	5	11	16	18
	Ba	5	12	14	17
362	Ca	10	15	20	26
	Sr	10	10	19	24
	Ba	10	17	18	24
363	Ca	15	—	19	24
	Sr	—	—	—	—
	Ba	—	—	—	—
364	Ca	9	13	17	28
	Sr	8	10	19	23
	Ba	7	10	23	24
365	Ca	9	—	25	25
	Sr	—	—	—	—
	Ba	—	—	—	—

Table 4. Unconfined compressive strength of untreated soils and hydroxide-soil mixtures.

Soil No.	Cation	Compressive Strength (bars) for Curing at 2 Hours, 20 C	24 Hours, 20 C	28 Days, 20 C	28 Days, 40 C
361	Untreated	1.1	1.2	1.5	1.5
	Ca	2.1	2.8	7.1	17.2
	Sr	1.7	1.7	1.4	1.7
	Ba	1.7	1.6	1.3	1.4
362	Untreated	1.7	1.8	2.3	2.3
	Ca	3.7	5.7	6.5	12.5
	Sr	3.2	2.0	3.4	4.7
	Ba	3.1	2.7	4.9	6.4
363	Untreated	2.3	2.3	2.3	2.3
	Ca	3.6	—	5.1	10.8
	Sr	—	—	—	—
	Ba	—	—	—	—
364	Untreated	1.2	1.2	1.3	1.3
	Ca	2.2	2.5	2.9	5.8
	Sr	2.0	2.1	1.8	2.7
	Ba	1.9	1.7	2.9	3.8
365	Untreated	1.9	1.9	2.0	2.0
	Ca	5.9	5.9	5.9	9.9
	Sr	—	—	—	—
	Ba	—	—	—	—

Table 5. Development of unconfined compressive strength in lime-soil mixtures, with 5 percent quicklime.

Soil No.	Compressive Strength (bars) for Curing at 2 Hours, 20 C	24 Hours, 20 C	28 Days, 20 C	28 Days, 40 C
361	2.4	3.8	11.0	41.5
362	5.5	7.9	11.3	57.1
363	4.2	7.8	9.8	52.3
364	4.3	5.3	6.4	32.9
365	12.0	14.8	17.7	28.1

to 60, indicating a strong instant reaction. This suggests that the three hydroxides react in the same way, in spite of the large differences in size between Ca, Sr, and Ba ions. This in turn suggests that the instant reaction occurs outside the clay mineral particles.

Rate of Hydroxide Consumption

The quantity of hydroxide remaining that was available was determined by ASTM C 25-67. By subtracting from the amount added, the amount of hydroxide consumed could be calculated. These figures, reproducible within about ±2-meq limits, are given in Table 3.

Although the rate of fixation of hydroxide in the first 2 hours depends on the soil, no important differences between the fixation rates of the three hydroxides are evident. The only possible exception is soil 361, which has a fairly high calcium carbonate content, in spite of which it consumes more calcium hydroxide than strontium or barium hydroxide in the ultimate stage of curing. Curing for 28 days at 40 C is roughly equivalent to about 1 year at 20 C.

Experiments not given in Table 3 have shown that the lime fixation within the first 10 min after mixing is almost as much as within the first 2 hours. Therefore, in well-mixed laboratory samples, the immediate reaction is almost completed within a few minutes and from then on the pozzolanic reactions are much slower.

Note that the data in Table 2 were obtained with additions of hydroxides equivalent to 1 percent commercial quicklime. Larger additions of lime produce higher rates of fixation in the early stage as well as later on. More detailed observations of fixation rates will be available after completion of research on the long-term effects of lime.

Unconfined Compressive Strength

Cylindrical specimens, 5 cm in diameter and 10 cm high, were molded from the untreated soils and from mixtures with three hydroxides with water contents at the wet side of optimum so that in each case 95 percent of the maximum dry density of the modified Proctor USCE test could be obtained.

After various periods of curing in sealed wrappings at 20 C and 40 C, the unconfined compressive strength of the specimens was measured, with a rate of strain of 0.127 cm/min. The remains were used to determine the available free hydroxide.

Table 4 gives the mean results of a series of five identical specimens. The coefficient of variation of the results of individual specimens is about 10 percent. It is apparent from these results that breaking strengths of the specimens of 2-hour-old hydroxide-soil mixtures are roughly twice those of the untreated soils and that the effects of strontium and barium hydroxides are practically the same as those of lime in this early stage.

After curing, large gains in strength appear in the lime-treated specimens. However, strontium and barium hydroxides produce insignificant gains in strength, in spite of the fact that consumption of strontium and barium hydroxides progresses, during curing, at rates comparable to those of lime consumption. This suggests that similar low-speed reactions may occur, but that the reaction products are different, perhaps for steric reasons.

For accelerated curing, 40 C rather than 48.9 C has been chosen to obtain strength-time curve shapes and chemical compositions of the reaction products that more closely approximate the behavior of the lime-treated soils at 20 C.

LONG-TERM REACTION OF SMALL AMOUNTS OF LIME

It may seem surprising that an addition as small as 1 percent of 92 percent grade quicklime produces not only instant ameliorations of geotechnical properties but also a relatively high long-term gain in strength. This evidence was obtained even though the lime dose added was much smaller than the "lime fixation (or retention) point" [i.e., the amount of calcium hydroxide beyond which the plastic limit is not increased by a larger dose of lime (15, 18, 19)]. This critical amount of lime, the modification optimum (20), "contributes to the soil workability but not to increases in strength, while amounts of lime beyond this dose cause the formation of cementing materials" (15). The fact that even a small amount of lime does evolve both these results is not consistent with this theory.

PRACTICAL CONSIDERATIONS

Long-term effects of an admixture of 1 percent quicklime are far inferior to those brought about by larger amounts of lime. Table 5 gives unconfined compressive strengths of specimens of mixtures of five Belgian soils of Table 1 with 5 percent quicklime. This is not the "optimal" dose for long-term stabilization of these soils; optimal doses may be larger.

Comparison of Tables 4 and 5 shows that, as far as immediate effects are considered, the strength after 2 hours at 20 C obtained with 5 percent quicklime is by no means five times but at most two times as large as the strength brought about by 1 percent. Only after a rather long curing time will the ratio of 5 percent to 1 percent strength become really large, justifying the use of relatively high and expensive amounts of lime in long-term stabilization for structural purposes (21).

In earthwork operations, however, the only purpose of the lime treatment is immediate amelioration of the soil (1). Long-term gain in strength may then supply a factor of safety, but it can also become a nuisance when trenching work is to be performed in a lime-treated embankment, for example.

The Belgian practice in highway construction is to treat wet clayey fine-grained soils with the lowest dose of quicklime, ensuring adequate workability and a sufficient bearing value after compaction 2 hours after mixing. This practice appears to be justified from the technical as well as from the economical viewpoint. Extensive descriptions of the actual earthwork procedures involved, of the equipment used, and of quality control test results have been published elsewhere (4, 7, 22).

CONCLUSIONS

The following describe the usefulness of quicklime when mixed with loess soils.

1. Instant improvement of wet loess soils by an admixture of 1 to 1.5 percent quicklime proved to be sufficient, not only in expediting earthwork operations, but also in allowing loess soils to be used, instead of discarded, in the construction of highway embankments.
2. A decrease in moisture content of about 0.65 percent per 1 percent quicklime is evolved, but drying of the loose mixture by exposure on the site is often more important.
3. A procedure has been developed for evaluating the amount of lime necessary to ameliorate a given soil just enough to obtain a suitable CBR value after compaction.
4. Structuring the soil into water-stable crumbs is the basic instant effect of lime. A simple and rapid procedure for measuring the increase of crumb stability, which indicates the fitness of the soil for instant amelioration, has been developed.
5. The instant ameliorations of geotechnical properties by lime treatment are caused by the experimental fact that lime-soil crumbs retain their individuality to a large extent when a lime-soil mixture is kneaded and compacted.
6. Hydroxides of strontium and barium produce the same immediate effects as an equivalent amount of lime. In later stages also, the rates of consumption of the three hydroxides remain nearly equal; however, long-term strength gain is far higher with calcium hydroxide than with strontium hydroxide or barium hydroxide.
7. Both immediate and long-term effects are brought about by small amounts of lime. This fact is not consistent with the lime retention point theory.
8. Lime percentage has less incidence on immediate effects than on long-term strength gains.

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DISCUSSION

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The experiences reported by Van Ganse in his paper indicate that the application of Verhasselt's technique (23) for measuring crumb stability may help in easily determining the minimum lime content required for immediate and economical amelioration of loessial soils containing illite. Unconfined compressive strength data generally confirmed crumb stabilities measured on 1 percent lime-treated soil after a 2-hour period at 20 C; no crumb stability data were given on cured specimens.

Analysis of unconfined compressive strength data in relation to -2μ clay content (13 to 30 percent) and organic matter content (0.01 to 0.25 percent) indicates the following:

1. Total ineffectiveness of a higher percentage of lime (>1 percent) on the lowest clay-content (13 percent) specimen. Strength gains at higher clay contents are a function of increasing clay content.
2. A marked decrease in effectiveness of a given lime percentage on higher clay-content specimens containing organic matter (0.21 and 0.25 percent).
3. For the higher organic content specimens, total ineffectiveness of 20 C curing for more than 1 day, with lime treatment of 1 percent. Unconfined compressive strength data for 5 percent lime-treated specimens after the longest (28-day), higher temperature (40 C) curing period reveal that specimens containing more than 20 percent clay have only one-fourth the strength because of a fourfold increase in the organic content.
4. $Ca(OH)_2$ consumption levels at 1 percent lime treatment of specimens, uncured and cured at 20 C, generally increasing with increases in clay content. However, data on specimens of intermediate clay content, cured at 40 C, indicate greatest consumption occurs for the one containing the highest organic matter. Although all specimens examined had approximately equal lime concentrations (grams of lime/litre of water), higher quantities of lime are used by those with higher organic matter content. Furthermore, strength gains normally associated with increased clay content are reduced.

Data collected by this discusser and other investigators of soil materials containing various amounts of montmorillonite relate to these findings.

Immediate ameliorative effects of hydrated lime treatment on materials tested at 20 C for cohesion and unconfined compressive strength are reported by Neubauer and Thompson (1). Materials contained approximately the same range of -2μ clay percentages (13 to 46 percent) and were molded at water contents (w) near the plastic limit (PL), as Van Ganse had done. Those data indicate the maximum rate of cohesion improvement occurred for the lowest level of treatment used, 2 percent, and there were decreasing benefits at higher treatment levels.

Direct tensile strength data by Glenn (24) indicate that immediate amelioration occurs on 90 percent bentonite clay (Na^+) with greatest strength gains from about 1 percent treatment with hydrated calcitic lime. Lower strengths are obtained with higher levels of treatment (3, 6, and 9 percent) on specimens molded near optimum moisture content for the untreated material. The necessary sensitivity to measure effects of minute changes in specimen constituents is available in newly developed apparatus (25). Tensile strengths are also greatest for 1 percent lime treatment at w greater than 40 percent, exceeding that for the untreated specimen at 40 percent w by 100 percent.

Indirect tensile strength data by Kennedy and Moore (26) were taken on a marl clay containing illite, montmorillonite, and kaolinite (in proportions of 5:3:1). Although the data for specimens cured for 3 weeks at 5 C are not strictly comparable, they are the only other published work available. Analysis of w versus tensile strength for the 50 percent clay-content specimens, molded at w near 18 percent PL, provided conclusions comparable to those of Verhasselt (23). In proportion to higher clay-content percentages, strengths increased with increasing lime treatment level but at a decreasing rate of gain. Indirect tensile testing method is not easy to use with w much above PL; therefore, no higher w data are reported. Extrapolation of the curves plotted from the tabulated data indicates that higher strengths would occur with higher w.

Higher temperature data from the curing of treated specimens (24, 26) show similar tensile strength response to the 1 percent lime treatment. Optimum lime percentages, giving maximum cured strengths, are 5 percent or more. The trend is toward lower strengths for treatments above the optimum amount of lime.

One may then correctly conclude that commonly found constituents in expansive and nonexpansive clayey soils are amenable to immediate amelioration by relatively low lime percentages. However, in design, one should consider the longer term effects of the variables involved: the immediate effects of clay and organic matter content percentages and their bearing on cured strengths.

Higher strengths for 1 percent lime treatment may be accounted for by the additional strength of lime-flocculated structures as compared to the dispersed structures for untreated clay. This is indicated also at higher water contents by the higher strengths that occur at still higher void ratios, suggesting the fuller development of edge-to-face bonds with increased w.

The concept that increased macroporosity in clayey soils occurs with increased lime follows directly from the flocculating effect of lime on loess soils and the consequent increase of void ratio. Higher flocculated structure strengths, indicated by increased tensile strength, are also associated with higher PL, increased CBR, and cohesiometer measurements; higher interparticle bond strengths are developed. The influence on the PL may be noted by considering those physicochemical phenomena that operate in the system with decreasing w as the specimen is remolded to smaller diameters. Decrease in w alone leads to a decrease in soil-water tension, associated with decreased numbers of interparticle menisci, which may lead to a decrease in strength.

On the other hand, there is the concomitant increase in cation concentration with reduction in w. The resulting reduced interparticle repulsion that leads to the flocculated structure may compensate for the loss in menisci with increased strength. This, of course, leads to crumb development, which is seen in the breakup of the $\frac{1}{8}$-in.-diameter bead in the rolling- and drying-out process at the PL. This clearly demonstrates the greater strength of the flocculated structure in the crumbs or clods that form in the bead as compared to that of the soil-water continuum that preserves the stable cylindrical specimen geometry at w>PL.

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AUTHOR'S CLOSURE

The influence of -2μ clay content and organic matter content on unconfined compressive strength and that of lime consumption on mixtures of illite-containing loess soils with lime are, as Glenn states, very important. Development of cured strength is retarded, not prevented, by organic matter. The delay during which unconfined compressive strength does not progress much may amount to several months at 20 C. This delay may lead to consideration of some soils that respond normally to lime in immediate amelioration as unfit for long-term stabilization. This conclusion may be sound from a practical viewpoint when a fairly rapid increase in strength is desired; however, after the delay period, during which harmful organic constituents are destroyed, strength increase is resumed, provided enough lime is still available.

An important feature of the retarding action of organic matter is the content in "blocked" nitrogen compounds (compounds, not immediately attacked by lime, with no ammonia being given off by fresh lime-soil mixture). However, these compounds become unblocked during the curing period, and while unconfined compressive strength gain is retarded, ammonia is emitted by the cured specimens. The blocked nitrogen compounds are not eliminated by hydrogen peroxide, and as a consequence partly escape the organic matter determination with H_2O_2. However, the H_2O_2 treatment does unblock these compounds, that is when the H_2O_2-treated soil is mixed with lime, it immediately emits ammonia. A 0.1 percent nitrogen soil content (determined by Kjehldahl attack) has been found to be effective in producing the described phenomena.

ERODIBILITY AND DURABILITY OF CEMENT-STABILIZED LOAM SOIL

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M. R. Akky, Woodward-Lundgren and Associates, Oakland

ABRIDGMENT

A cement-stabilized loam soil (A-4) was used to study the weight loss per unit surface area of samples containing various amounts of cement (i.e., 6, 8, 10, 12, and 14 percent by weight). These samples were subjected to repeated weathering (freeze-thaw or wet-dry cycles). At the end of each treatment cycle, the weight loss caused by brushing or erosion was measured. Erosion tests were carried out in a rotating cylinder apparatus with a chosen speed of 1,060 rpm and a duration of erosion of 2 min. The results show that resistance to weathering and subsequent erosion increases as cement content in the samples increases. It was also observed that a steady state of soil erosion can be achieved in samples containing higher cement contents. A comparison of weight loss caused by brushing and erosion indicates that the brushing procedure, as specified in durability test methods, appears to be more damaging to soil-cement samples than the shear stress generated by the rotating fluid. It is believed that by using the rotating cylinder apparatus, this study has provided a more rational approach to relating the durability test results to erodibility of cement-stabilized soils that are not accounted for in the current soil-cement design criterion.

• DURING the last two decades, there has been increasing use of soil-cement (cement-stabilized soils) to reduce surface erosion and seepage loss in hydraulic structures such as highway drainage ditches, reservoir and channel lining, and earth dam facing. The first 10-year performance of the Bonny reservoir test embankment (1) proved that durable, low-cost slope protection for dam or other earth embankments and linings can be built with natural soils and cement using construction procedures and equipment similar to those used in soil-cement road construction. During the design stage of the Bonny test section, the engineers were confronted with deciding on the amount of cement suitable for different soils for slope protection. There was no information available at that time about the interaction between erosion resistance and weathering of cement-stabilized soils; therefore, it was logical to borrow experience from the road construction industry. Although recognizing that the critical forces experienced by the soil are different in roadway construction than in slope protection, the use of durability tests designed primarily for roads to determine the cement content for the construction of hydraulic structures was not altogether unfounded. Furthermore, to minimize possible risks because of lack of knowledge, 2 and 4 percent more cement were added to the amount determined from durability tests (ASTM D 559-57 and D 560-57) for granular soil and fine-grained soil respectively.

Unfortunately, in almost 25 years since the construction of the Bonny test section, our fundamental knowledge of cement-stabilized soil in hydraulic structure construction has not improved significantly. Information gathered from performance studies is scarce. The absence of any major failure thus far seems to satisfy most engineers

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to adopt the borrowed design criterion permanently. Furthermore, the adequacy of the design criterion was substantiated in a laboratory study (2). However, in design thus far, the more basic factors relating the interaction of stabilized soil, flowing water, and the environment have not been thoroughly studied.

More recently, Akky and Shen (3) studied the erodibility of a cement-stabilized sandy soil. For uncycled samples (subjected to no environmental attack) of low cement contents, they reported that a simple relationship can be established between the 7-day cured unconfined compressive strength and the critical shear stress at which erosion is initiated. Further results (4) indicated that this relationship holds for a variety of soils with low cement contents. For samples subjected to various freeze-thaw cycles, erodibility cannot be related directly to the unconfined compressive strength. Expansion of water in the pore space, which is due to repeated freezing and thawing, causes the soil surface to heave. This weakens the bond strength between cementing particles and consequently reduces the soil's erosion resistance. From the results of their study, Akky and Shen concluded that the alternating weathering and erosion cycle is responsible for the deterioration of cement-stabilized soils and is essential to understanding the interaction of water, stabilized soil, and the environment. This paper gives the results as a part of a continuing study on soil-cement erodibility (i.e., the weight loss of a cement-stabilized soil caused by either erosion in a rotating cylinder or brushing as specified in the standard durability tests. It is hoped that by relating these two types of soil losses a better picture may be obtained to translate the durability test results to erosion resistance of cement-stabilized soils.

EXPERIMENTAL PROGRAM AND TEST RESULTS

The experimental program tested a loam soil for durability and erosion. After each treatment cycle (either freeze-thaw or wet-dry), soil weight loss due to either brushing in the case of durability test samples or erosion in the case of erodibility test samples was recorded.

The loam soil (A-4) used in this study was a local soil known as Yolo loam (35 percent sand, 55 percent silt, and 10 percent clay). Based on standard AASHTO compaction, the optimum water content was approximately 17 percent and the corresponding maximum dry density was 1.74 grams/cm^3. Commercially available Type II cement was used to mix with the soil. Five cement content levels were used in sample preparation: 6, 8, 10, 12, and 14 percent by dry weight of the soil. Durability test samples were compacted according to the standard procedure, whereas erosion samples were compacted by kneading in two layers in a 7.62-cm-diameter by 8.76-cm-high steel mold. All samples were compacted to a dry density of 1.74 grams/cm^3 at a molding water content of about 17 percent. For samples scheduled for the erosion test, a 1.90-cm hole was drilled axially along the length of the sample. All samples were then cured in the moisture room for 7 days (95 percent humidity and 22 C) before specified tests were performed.

The freeze-thaw and wet-dry durability tests were carried out according to the standard methods (ASTM D 559-57 and D 560-57). Erosion tests were performed after each cycle of treatment. A detailed description of the testing apparatus and procedure is given by Akky (5). Figure 1 shows an overall view of the testing apparatus. A fixed rotating speed of 1,060 rpm was chosen in this study; it is equivalent to a shear stress of approximately 0.8 grams/cm^2 acting on the surface of the sample. The erosion cycle was set for 2 min, which, according to previous experience, is sufficient to cause the erodible surface material to separate from the rest of the sample.

The weight loss per unit surface area after each erosion cycle is shown for wet-dry (Fig. 2) and freeze-thaw (Fig. 3) samples. Samples having higher cement contents underwent a total of 18 treatment cycles. For these samples the amount of weight loss per unit surface area remained more or less constant in the last few cycles indicating that the steady state of erosion loss is reached under the given set of testing parameters. In all cases the resistance to weathering (treatment cycles) and subsequent erosion increases as the cement content increases. The 6 percent wet-dry sample and the 6, 8, and 10 percent freeze-thaw samples showed excessive soil loss, and testing of those samples was discontinued before the completion of 18 cycles.

Figure 1. Erosion apparatus.

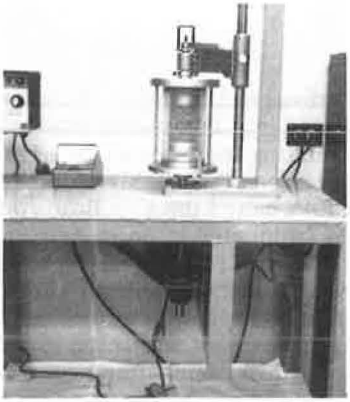


Figure 2. Erosion test results—wet-dry cycles.

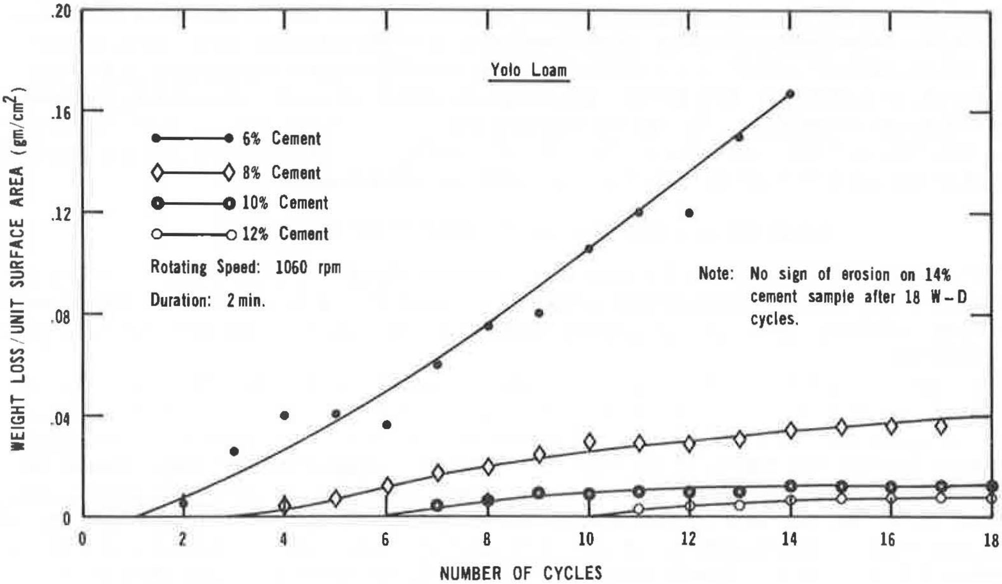


Figure 3. Erosion test results—freeze-thaw cycles.

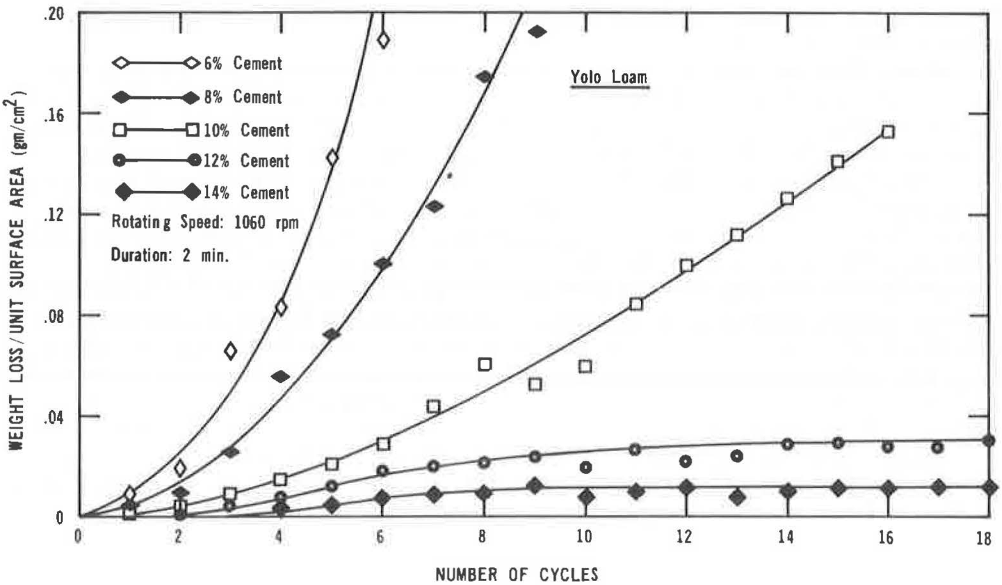


Figure 4. Comparison of soil loss—10 percent.

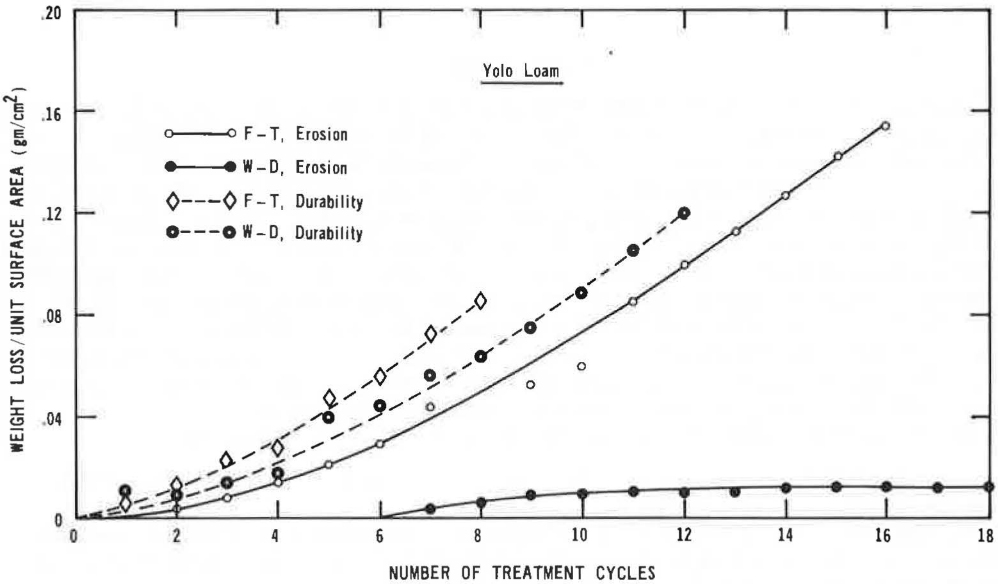
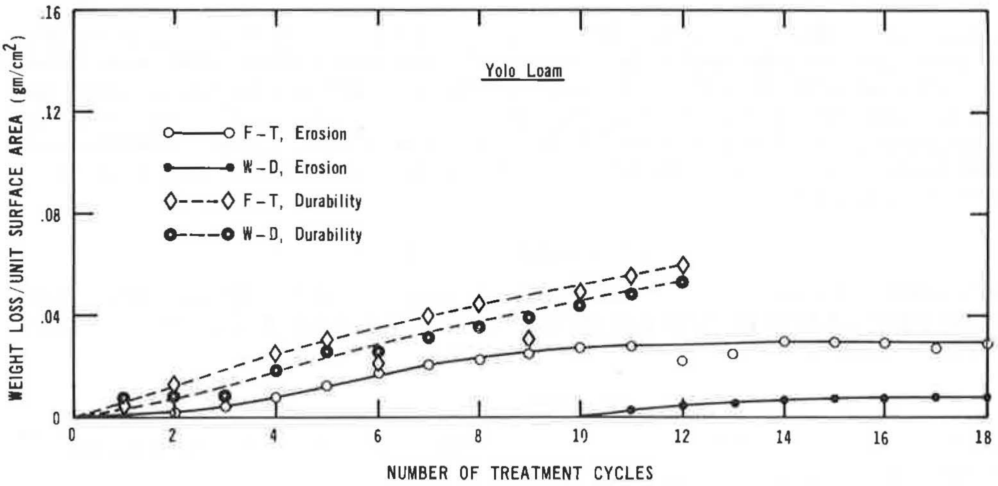


Figure 5. Comparison of soil loss—12 percent.



Durability test results showed that a minimum of 12 percent cement is required for soil-cement.

CONCLUSIONS

Because the current design of soil-cement slope protection for earth embankments and linings is essentially based on the durability criterion in which erodibility of soil-cement is not being considered, examination and comparison of the results of weight loss due to both brushing and erosion may provide necessary information to relate the durability test results to erosion resistance of cement-stabilized soils. Figures 4 and 5 show typical comparisons of weight loss per unit surface area of the various samples caused by brushing and erosion. These figures indicate that (a) the brushing procedure is more damaging to samples than the shear stress generated by the rotating fluid, and that (b) there is more weight loss due to erosion on freeze-thaw samples than on wet-dry samples. By changing the rotating speed and, to a lesser extent, the time of erosion, the weight loss due to erosion will be different. Therefore, the comparisons are limited to describing the test results obtained from this study.

On the basis of this study the following may be tentatively concluded:

1. By using the rotating cylinder apparatus it is possible to relate soil-cement erodibility to durability test results.
2. Resistance to weathering and subsequent erosion of cement-stabilized soil increases as cement content in the soil samples increases. A steady state of erosion loss is achieved in samples of higher cement contents.
3. For the testing parameters, the brushing procedure used in durability tests caused more severe damage (higher soil loss) to samples than the shear stress generated by the rotating fluid.

Furthermore, there are many other factors that could detrimentally affect the performance of cement-stabilized soils that are not considered in this study—most notably field construction variables such as mixing procedure, compaction control, construction scheduling, interface treatment, and curing method. The use of weight loss comparison caused by brushing and erosion, however, is believed to have provided a more rational approach to realistically relate the durability test results to erodibility of cement-stabilized soils.

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