HIGHWAY RESEARCH BOARD

Bulletin 353

Stabilization of Soils with Portland Cement

Design, Testing, Properties, Admixtures

National Academy of Sciences—
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A Cement-Treated Base for Rigid Pavement

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The new Jackson (Mississippi) Municipal Airport is the first jet-age municipal airport to use a cement-treated base under rigid pavement. Because of its geological setting in an area of few natural deposits of satisfactory base materials, the existence of a good sandy soil on the site, and the need to pay special attention to the highly expansive clay on the site, the use of the cement treatment was found desirable. Results of laboratory tests of various mixes of the natural materials with cement are presented and discussed in describing the selection of the design mix. Methods of working the borrow pit and constructing the cement-treated base are discussed in detail.

• THE COMING of the jet age in air transportation found the airport runways of the City of Jackson, Miss. too short a length. A thorough study was made to determine whether it was feasible to expand the existing field or whether an alternate site should be developed. The study showed that the existing facilities could be expanded only at considerable expense, and even then they would not be entirely satisfactory. A group of possible new sites were studied minutely with respect to proximity to passenger origin, availability of land, approach clearances, topographic features, finances required, and many other details.

One site rated higher than the others with regard to most of the criteria considered and was chosen. This site was located in an adjoining county just across the Pearl River Valley from Jackson.

The selected site lies at the eastern edge of the alluvial valley of the Pearl River in the adjoining low, rounded hills. The alluvium in this area is predominantly silt. The surface covering of most of the hill section is a thin layer of lean clay; however, remnants of sandy river terraces are found on the tops of a few of the highest hills on the site. These surface layers are underlain by a thick, very expansive montmorillonitic, marine deposit known as Yazoo Clay.

PAVEMENT DESIGN CONSIDERATIONS

Borings showed that Yazoo Clay would be exposed in most of the cut sections, and laboratory tests and past experiences indicated this to be the most critical material from the design standpoint on the site. Results of the tests showed the following ranges of values: liquid limit, 70 to over 100; plasticity index, 40 to 70; volume change from LL, 80 to over 200 percent; and swell (compacted samples soaked 4 days), 10 to 20 percent. These values indicate this to be a treacherous material with which to deal. The soil is stable in its natural state, but swells considerably on being rewet after having dried. No instances of changed conditions have been observed where about 7 ft of lean clay overburden exists.

These factors were studied and satisfactory means of keeping the Yazoo Clay at its in situ condition during construction and subsequent years of use were considered. It was estimated that very little temperature change, if any, would occur more than 3 ft below the light-colored surface of a concrete pavement; therefore, little or no moisture change would occur because of temperature variation. This was considered to be the only type of protection against moisture change in the completed structure necessary as no ground water was encountered on the site. Thus, a thickness of select subgrade, subbase, base course, and pavement of 3.5 ft was believed to provide adequate thermal insulation for the Yazoo Clay. Cut sections had to be undercut 19 in. below subgrade elevation to provide the 3.5-ft thickness. To keep the exposed Yazoo Clay from drying during construction, it was specified that the lean clay backfill should be placed immediately, or the Yazoo Clay sprinkled sufficiently to prevent drying until the backfill could be placed. The subgrade was overlaid with 12 in. of subbase and base and 11 in. of pavement, as shown in Figure 1.

All fills under areas to be paved were constructed of the lean clay; the Yazoo Clay was deposited only in median areas where it was very unlikely that any pavement would ever be constructed.

A deposit of fine, clean sand containing thin clay and silt lenses was located near one end of the runway, and offered a source of base course material. Visual examination and laboratory classification tests indicated that it was a material of adequate stability, but, on further consideration, it was thought highly improbable that sufficient mixing could be effected to produce a uniform base on which to perform paving operations. Small pockets of clean sand would tend to ravel under paving equipment. Also, the material was fine enough to be easily transported by pumping action under moving aircraft if water entered the base. In its natural state, the sand was considered unsuitable for base course immediately beneath rigid pavement.

A study of the costs involved showed that because of haul distances, it would be cheaper to mix the sand with a moderate amount of cement to produce a satisfactory base than to import another material. Also it was believed that the cemented layer would provide additional protection and insulation over the Yazoo Clay. These facts were the basis for the decision to make a design study of cement stabilization in the top 6 in.

CEMENT TREATMENT DESIGN

Examination of samples and boring logs from the sand deposit proposed for use as base material showed that the amount of silt and clay in the final base material could be controlled to a considerable extent by the manner in which the material was taken from the pit. It was proposed that the material be removed by a shovel or dragline cutting up a vertical face. In so doing, the number of silt and clay layers cut could be controlled, thereby controlling the amount of material passing the No. 200 sieve.

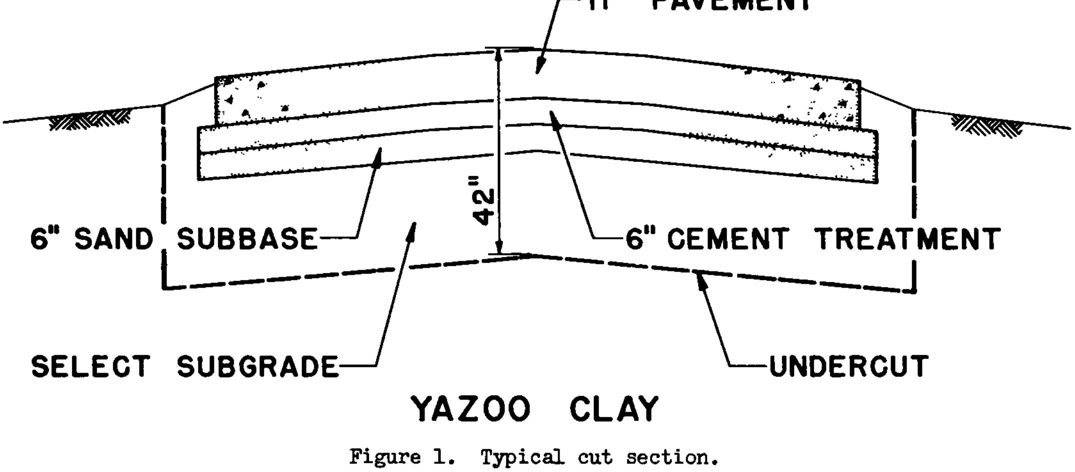

Figure 1. Typical cut section.

Laboratory moisture-density curves had been produced for untreated soil for four samples selected to cover the range of materials available in the borrow area in connection with subbase design. The characteristics of the four samples are given in Table 1. Because of time limitations it was necessary to compact specimens containing cement only at about the optimum moisture contents determined previously for the untreated soils. The specimens were compacted in a standard 4-in. diameter mold, using modified AASHO compactive effort, and were broken by compression at the end of 7 and 28 days of moist curing.

Plots of compressive strength vs percentage of cement are shown in Figure 2 for the individual samples. These data indicate that the compressive strength obtained varies with the amount of cement used and also with the sample tested.

TABLE 1

CHARACTERISTICS OF SAMPLES OF MATERIALS AVAILABLE IN BORROW AREA

Sample No.	Sample	Percent Passing			LL	PI	Max. Dens.	Opt. Moist.
		No. 40	No. 60	No. 200				
1	Clayey sand	100	99	27	30	4	118.0	12.6
2	Clayey sand	97	52	18	25	3	122.7	10.6
3	Sandy silt	99	90	53	29	7	123.6	11.6
4	Silty sand	98	83	13	NP	NP	116.7	11.2

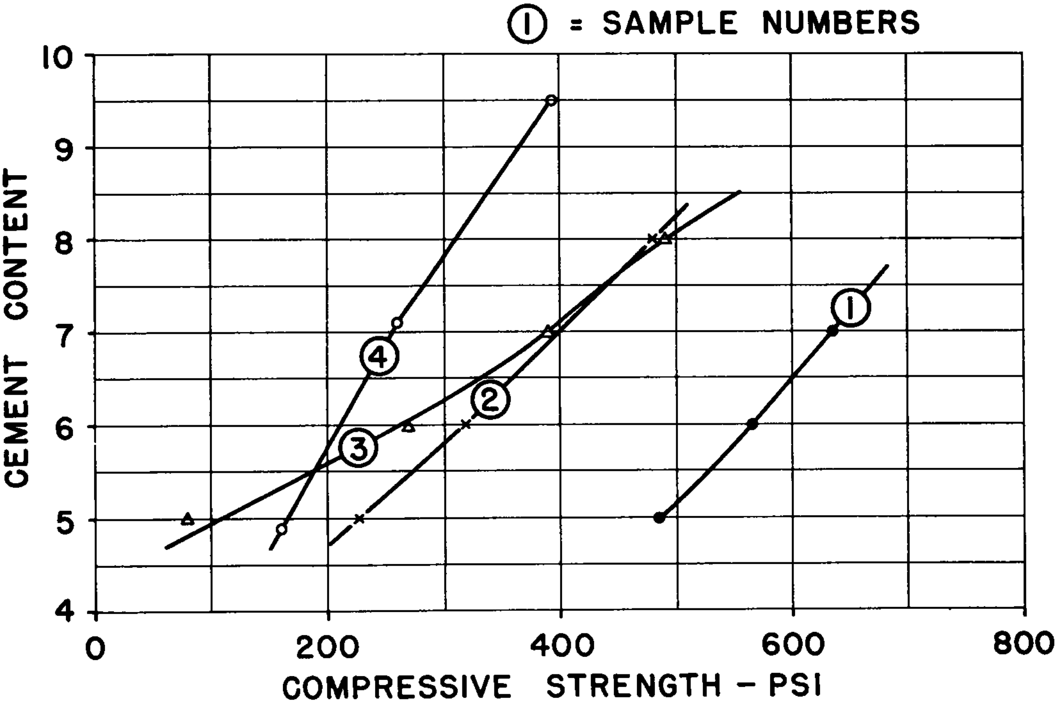

Figure 2. Compressive strength vs cement content.

The cause of variation in strength with sample tested was analyzed by plotting the percentage of material passing the No. 200 sieve against the compressive strength at 6 percent cement (Fig. 3). This curve indicates that maximum strength for cement used is obtained with about 28 percent passing the No. 200 sieve.

At this point it was necessary to decide on the minimum acceptable compressive strength. Based on long experience the Portland Cement Association recommends a minimum of 300 psi at 7 days for use with flexible pavements. It was felt that this figure could be reduced slightly for rigid pavements inasmuch as the design considerations do not envision such strength. Examination of laboratory specimens indicated that those breaking at 200 psi or more were well-cemented. This material so treated would be unlikely to pump, and would provide a stable working surface for paving operations. However, to add a little conservatism to the requirements, a minimum strength of 250 psi at 7 days was selected.

To arrive at the most economical and practical soil-cement mix which would provide the selected minimum compressive strength a plot of cement content vs percentage of material passing the No. 200 sieve was made from Figure 2 for a compressive strength of 250 psi and is shown in Figure 4.

Examination of this plot shows that the cement required to produce minimum strength varies inversely with the percentage passing the No. 200 sieve. This observation is based on samples 1, 2, and 4. Sample 3 was not considered in the analysis because the quantity of this material was limited and could only be used for mixing with the sand in cases of deficiency in material passing the No. 200 sieve. A study of the boring logs for the pit indicated that a material with 20 to 30 percent passing the No. 200 sieve would be produced in most cases by pit operation and that little mixing of additional fines would be required. Figure 4 shows that such a material would require about 4 to 5 percent cement to obtain the desired strength.

The laboratory investigations just described were all performed on well-pulverized and blended samples. It was realized, however, that field conditions would probably be different, and it was specified that the contractor should build a test section at the

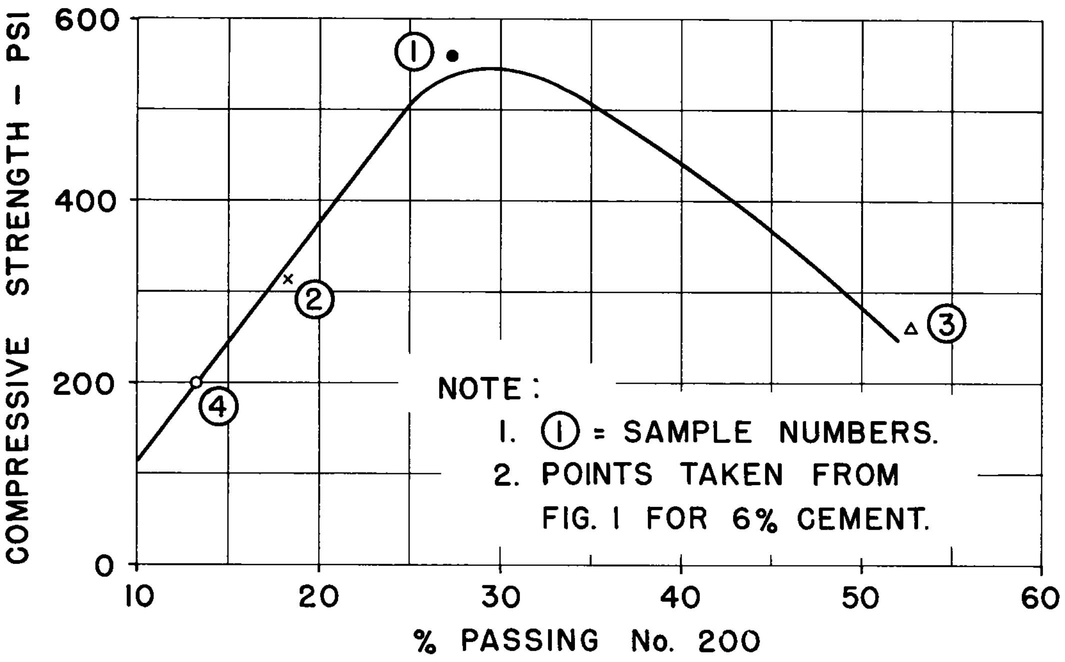

Figure 3. Effect of fines on compressive strength.

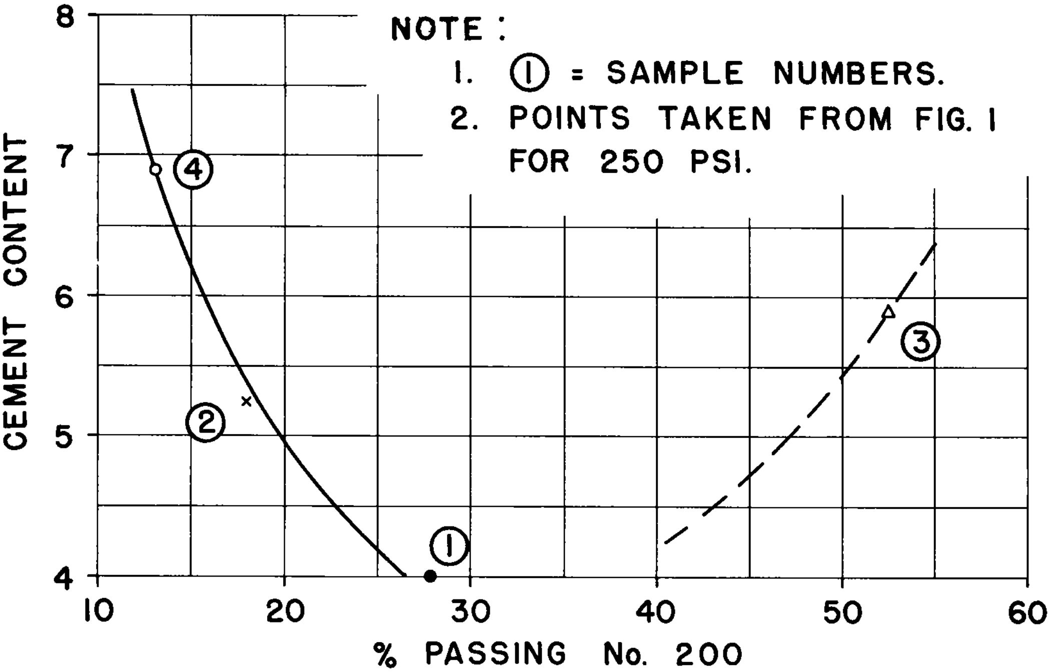

Figure 4. Cement required for 250 psi.

beginning in which operating procedures would be established. Construction of the test section showed that the pulverization requirements of 100 percent passing the 1-in. sieve and 75 percent passing the No. 4 sieve could be met consistently, but that the field behavior of this material differed considerably from the well-pulverized laboratory material.

One of the first problems encountered during construction was that of obtaining sufficient blending of material on the fill to produce a reasonably uniform soil. This was shown to be necessary by the discovery of a number of small spots (less than 100 sq ft) lacking sufficient fines and other small spots having excessive fines. The low amount of cement (5 percent) did not provide sufficient cement to produce minimum strength requirements in these areas. Moisture requirements for each of these spots differed considerably from that needed in most of the base, and it was found almost impossible to produce satisfactory moisture conditions in the whole area at one time.

Before beginning construction of the untreated subbase, laboratory moisture-density relationships had been determined on a typical sample meeting the specification requirements. It was observed that the action of the hammer on the small lifts in the laboratory mold produced further pulverization that was not duplicated in the field by the action of vibratory, pneumatic, and sheepsfoot rollers. A study of the effects of pulverization on density is shown in Figure 5. Curve 1 represents a completely pulverized sample. The sample represented by curve 2 received a moderate amount of pulverization in the field during excavation, and limited additional pulverization in the laboratory. Curves 3, 4, and 5 represent samples with limited field and laboratory pulverization.

Sheepsfoot, pneumatic, and vibratory rollers were tried in various weights and in various combinations, and the moisture content of the material was varied considerably during early stages of subbase construction. Almost regardless of equipment, the density obtained compared favorably with that of curves 3, 4, and 5 of Figure 5. The clay layers in the pit were generally in a very firm condition and at a moisture content of about optimum or a little less. When compaction could be completed without

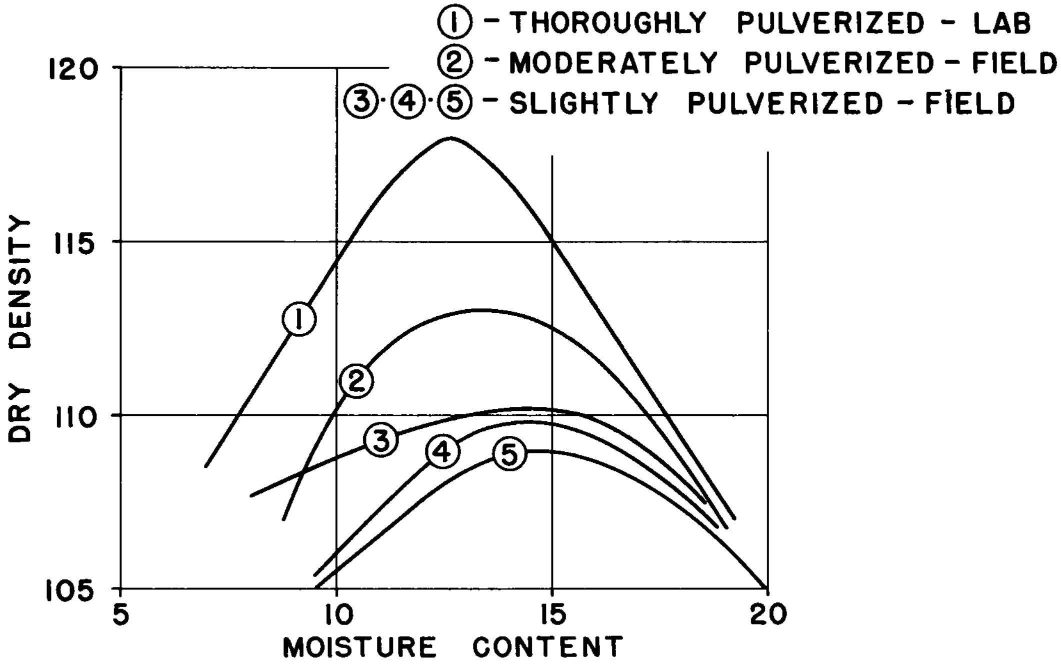

Figure 5. Moisture-density curves, subbase material.

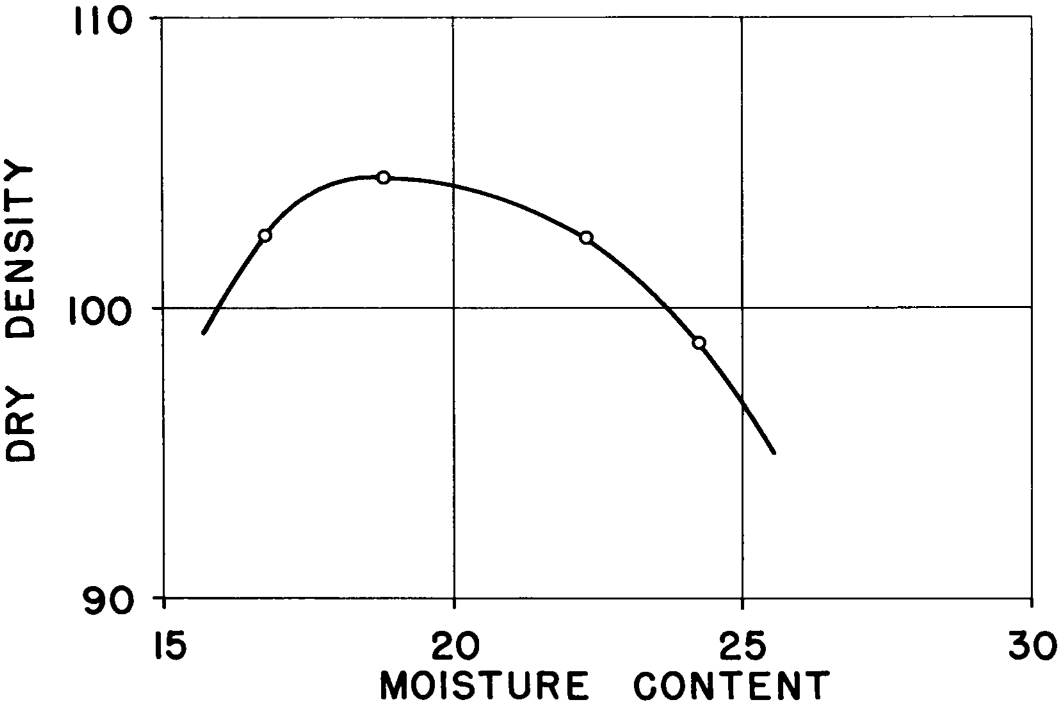

Figure 6. Moisture-density curve, cement treatment (5.5 percent cement).

the clay becoming wetter, the clay balls showed little deformation under the rollers. However, when the moisture content of the balls increased appreciably, they flattened considerably under the rollers and produced a spongy condition. It then became necessary to dry the clay sufficiently to bring about stable conditions before satisfactory compaction could be obtained.

Based on this finding, a laboratory curve for compaction control purposes was produced by adding 5.5 percent cement to a sample that had limited field and laboratory pulverization (Fig. 6). Test section construction indicated this to be about the maximum density obtainable in the field with reasonable construction effort. The compaction effort consisted of 4 to 6 coverages of a sheepsfoot roller producing a bearing of about 300 psi, two coverages of a vibratory roller, and 2 coverages of a light penumatic roller.

Cores taken from the test section showed that compressive strengths were generally a little below the 250 psi required, ranging from about 150 to 270 psi. From these tests it was apparent that additional pulverization or additional cement would be required to produce a base course with the desired compressive strength. It was decided that blending would be accomplished, the material pulverized more, and 6.5 percent cement added for another trial.

At this point in the history of this project (early November 1961) a rainy season began and lasted well into December. It appears that weather conditions will obviate further cement treatment work until the spring of 1962.

In conclusion, the design procedures required for the cement treatment for this project were of a routine nature based on thoroughly pulverized laboratory samples. After the relationships were established on ideal samples, however, it was necessary to perform laboratory tests on samples which were comparable to the mixes and blends produced in the field to provide control standards. The critical item in this problem has proved to be the degree of pulverization of the clay lumps, which must be taken into account during design testing to predict what can be obtained in the field with satisfaction.

Alternate Methods for Measuring Freeze-Thaw and Wet-Dry Resistance of Soil-Cement Mixtures

R. G. PACKARD, Chief, Soil-Cement Laboratory, Paving Bureau, Portland Cement Association

Three alternate methods for measuring freeze-thaw and wet-dry resistance in the standard soil-cement tests are studied: length change, compressive strength, and pulse velocity. Length change and compressive strength methods show promise of development into desirable alternate procedures. Data obtained by the alternate measures provide new information that will be helpful in further studies to improve soil-cement testing procedures.

• SINCE THE START of soil-cement paving in 1935 the purpose has been to produce a dependable, long-life paving material. Consideration of long-range performance is an inherent part of the design of soil-cement pavements. The standard soil-cement testing procedures* originally developed in the late 1930's have as their objective the selection of the cement content, moisture content, and density that will insure excellent, long-range performance. The service record of 450,000,000 sq yd of soil-cement in this country shows that the stability is maintained, and even increased, over years of weathering and increasing traffic. The level of cement contents selected by the standard testing procedures is largely responsible for the field performance of soil-cement and the resulting, present acceptance and widespread use of this material.

In recent years, other soil-cement testing procedures have been developed for selecting cement contents. It is fair to assume, if the cement content is lower, that some of these newer procedures may not produce the long-range performance expected and long associated with soil-cement. If a level of cement content different from that established by standard procedures is used it may produce an entirely different product. The long-range performance must be established for specific combinations of climate, soil type, and the cement contents determined by these other procedures.

The standard soil-cement testing procedures produce the long-range performance required for all soil types and for any climate. This paper is concerned with the development of faster and easier methods of determining the cement content that will produce the same field performance obtained when the cement content is determined by the standard freeze-thaw and wet-dry tests.

Studies to improve soil-cement testing procedures have been in progress in the Soil-Cement Laboratory of the Portland Cement Association. These studies can be classified into three phases:

1. Developing a better understanding of the effects of freezing-thawing and wetting-drying on soil-cement mixtures to determine the specific physical and physico-chemical properties evaluated by the standard freeze-thaw and wet-dry tests.

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*"Methods of Test for Moisture-Density Relations of Soil-Cement Mixture," ASTM Designation D 558-57; AASHO Designation T 134-57; "Methods of Wetting-and-Drying Test of Compacted Soil-Cement Mixtures," ASTM Designation D 559-57; AASHO Designation T 135-57; and "Methods of Freezing-and-Thawing Test of Compacted Soil-Cement Mixtures," ASTM Designation D 560-57; AASHO Designation T 136-57.

2. Examining new yardsticks (length change, pulse velocity, and compressive strength) for measuring the physical changes taking place in soil-cement mixtures during the freeze-thaw and wet-dry tests to determine if they have advantages over the standard yardsticks such as greater accuracy, sensitivity, ease of operation, or requirement of less procedural control.
3. Examining all details of the freeze-thaw and wet-dry testing procedures to determine if cycling time periods, the number of cycles, or temperatures can be modified so that the same cement content can be established in a much shorter time.

This paper is a progress report on the first two phases. The information obtained is expected to be valuable and necessary to development of the third phase, acceleration of procedures.

TEST METHODS

Soil-cement specimens made from four soils were conducted through three series of environmental conditions: alternate freezing and thawing, alternate wetting and drying, and continuous moist-curing. Weights, lengths, and pulse velocities were determined during these tests at each environmental change. At the completion of each series, compressive strengths were determined and the specimens were inspected visually for deterioration.

Freeze-thaw and wet-dry tests were conducted on specimens that were brushed by the standard procedure and on companion specimens that were not brushed. The continuously moist-cured specimens were used to establish control values of length, weight, compressive strength, and pulse velocity to compare with corresponding values determined on the freeze-thaw and wet-dry specimens. Details of the testing program are given in Table 1.

Length measurements were made on unbrushed specimens in a length comparator equipped with a dial gage graduated to 0.0001 in. The equipment is shown in Figure 1. After some difficulty, a technique was developed for attaching reference points that would hold through the various environmental conditions. An epoxy resin was used to cement stainless steel bearings to the top and bottom of the specimens. Length measurements with this precision provide data for an improved analysis of the shrinkage and expansion of specimens during freezing-thawing and wetting-drying.

Pulse velocities were determined on all specimens during the tests with a soniscope, (1) a device that measures the velocity with which a shock pulse travels through a material. Normally, the pulse velocity increases as the strength of the material increases. The equipment is shown in Figure 2.

Compressive strengths were determined on all specimens after each series of tests. The specimens were capped and broken in compression after a soaking period. Rate of load application was 20 psi per sec.

Weights of all specimens were recorded during and after the tests, permitting calculation of weight losses and moisture changes.

The freeze-thaw and wet-dry specimens were examined for hardness after test. This was done by picking with an ice pick and sounding with a hard object. Significant penetration of the pick and a punky sound indicate inadequate hardness.

Standard size specimens, 4.0-in. diameter by 4.6-in. height, were molded at optimum moisture and maximum density for each soil. A range of cement contents above and below the cement requirement as determined by weight loss criteria (2) was studied. Duplicate specimens were molded for each cement content and for each condition within each series of the testing program.

MATERIALS

Four soils representing a range of textural types were used in this study: a gravelly loamy coarse sand, a loam, a clay loam, and a silt loam. In this report the four soils will be referred to as gravelly sand, loam, clay, and silt. The 0.005-mm clay contents of these soils are 6, 24, 45, and 7 percent, respectively. Gradations, Atterberg limits, classifications, and moisture-density relations are given in Table 2.

The Type I portland cement used was a blend of four commercial brands purchased on the open market in the Chicago area. Cement contents are reported as percent by weight of oven-dry soil.

TABLE 1

TESTING PROGRAM

Test Series	Specimens Labelled	Cure			12 Cycles of Test			Water Soak, Hours	Weight & Velocity Determined	Length Determined	Capped & Broken in Compression
		Days of Moist-Cure	Weight & Pulse Velocity Determined	Length Determined	Brushed	Weight & Velocity Determined	Length Determined				
Freeze-Thaw	A	7	Yes	No	After each thaw	After each thaw	No	24	Yes	No	Yes
	B	7	Yes	Yes	No	After each thaw	After each freeze and thaw	24	Yes	Yes	Yes
Wet-Dry	A	7	Yes	No	After each dry	After each wet & dry	No	24	Yes	No	Yes
	B	7	Yes	Yes	No	After each wet & dry	After each wet & dry	24	Yes	Yes	Yes
Controls for Strength & Pulse Velocity	E	7	Yes	Yes	No			4	Yes	Yes	Yes
	F	39	Yes	Yes	No			4	Yes	Yes	Yes

TABLE 2

PHYSICAL PROPERTIES OF SOILS TESTED

Soil	Sampling Location	Gradation - per cent smaller than								Color	USDA Textural Class	Atterburg Limits		AASHO Soil Class	Moisture-Density Relations by AASHO T 134-57	
		No. 4 Sieve	No. 10 Sieve	No. 40 Sieve	No. 60 Sieve	No. 200 Sieve	0.05 mm	0.005 mm	0.002 mm			L. L.	P. I.		Opt. Moisture %	Max. Dry Density Lb. per cu. ft.
Gravelly Sand	Plainville, Illinois	70	59	30	19	12	11	6	5	Brown	Gravelly Loamy Coarse Sand	NP	NP	A-1-b(0)	8.3	134.5
Loam	Lake County, Illinois	100	97	92	89	61	54	24	18	Tan	Loam	19	1	A-4(5)	13.1	116.7
Clay	Cook County, Illinois	100	100	95	89	74	70	45	33	Brown	Clay Loam	32	12	A-6(10)	18.8	105.7
Silt	Colfax County, New Mexico	100	100	100	99	91	77	7	4	Light Grey	Silt Loam	NP	NP	A-4(8)	13.9	109.8

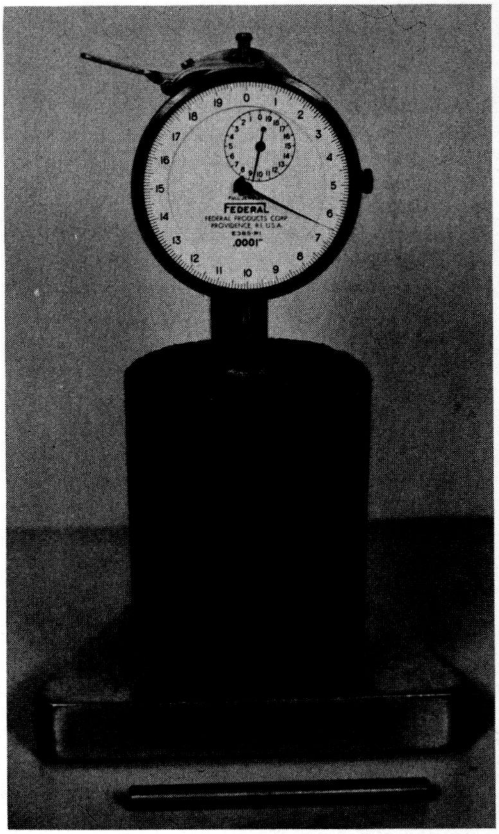

Figure 1. Comparator used for length measurements.

RESULTS OF FREEZE-THAW TESTS

Weight Losses, Moisture Changes, Condition of Specimens

Weights of brushed and unbrushed specimens were determined after the thawing portion of each cycle and at the end of test. These are given in Tables 3 through 6 along with computed weight losses, moisture changes, and conditions of specimens.

During the freeze-thaw test, the maximum moisture contents for specimens of the gravelly sand, loam, and clay were, respectively, 1.2, 3.5 and 2.9 percentage points higher than the molded moisture content. These moisture gains occurred at the lower cement contents. Specimens at higher cement contents showed smaller moisture gains. None of the specimens for these three soils reached complete saturation during the test.

WEIGHTS, MOISTURE CONTENTS, AN

Weight LB.			2% Cement				3% Cement		
			A1	A2	B1	B2	A1	A2	B1
As Molded, Wet			4.92	4.92	4.90	4.90	4.88	4.90	4.9
After Cure			4.93	4.94	4.93	4.94	4.81	4.88	4.9
Cycle No.	2	After Thaw	4.80	4.82	4.94	4.94	4.84	4.87	4.9
	4		4.56	4.57	4.94	4.95	4.82	4.85	4.9
	6		4.30	4.23	4.94	4.92	4.79	4.82	4.9
	8		4.06	4.03	4.94	4.93	4.77	4.80	4.9
	10		3.78	3.79	4.92	4.91	4.74	4.77	4.9
	12		3.51	3.65	4.92	4.90	4.72	4.75	4.9
After Soak			3.51	3.65	4.93	4.90		4.75	4.9
Weight Loss, %			29	26	0	0	3	3	
Maximum Moisture Gain, % by dry wt.					1.0	1.2			0.2
Condition of Specimens(1)			F	F	F	FO	O	O	G

Weight LB.		2% Cement				3% Cement		
		A1	A2	B1	B2	A1	A2	B1
As Molded, Wet		4.91	4.91	4.90	4.89	4.91	4.90	4.8
After Cure		4.87	4.83	4.94	4.87	4.87	4.86	4.9
Cycle No.	2 Wet	4.81	4.81	4.92	4.90	4.85	4.83	4.9
	2 Dry			4.57	4.58			4.6
	4 Wet	4.68	4.67	4.92	4.90	4.87	4.79	4.9
	4 Dry			4.57	4.57			4.5
	6 Wet	4.45	4.36	4.91	4.89	4.73	4.84	4.9
	6 Dry			4.60	4.56			4.5
	8 Wet	4.01	3.96	4.90	4.88	4.36	4.54	4.9
	8 Dry			4.57	4.57			4.5
	10 Wet	3.88	3.82	4.89	4.86	4.32	4.56	4.9
	10 Dry			4.52	4.54			4.5
	12 Wet	3.76	3.68	4.87	4.85	4.29	4.56	4.8
	12 Dry			4.52	4.53			4.5
After Soak		4.03	3.94	4.86	4.84	4.60	4.85	4.8
Weight Loss, %		17	19	0	0	6	0	
Condition of Specimens (1)				FO	FO			O

COM

Weight LB.	2% Cement				3% Cement		
	D1	D2	E1	E2	D1	D2	E1
As Molded, Wet	4.87	4.91	4.88	4.90	4.91	4.90	4.8
After 7 Day Cure	4.79	4.74	4.89	4.89	4.78	4.78	4.9
And Soak	4.85	4.90			4.90	4.89	
After 39 Day Cure			4.90	4.90			4.8
And Soak			4.91	4.92			4.9

(1) Condition of Specimens After Test by Visual Inspection

P - Poor
F - Fair   Intermediate conditions indicated as FO -
O - OK
G - Good
E - Excellent

E 3

ITION OF SPECIMENS FOR GRAVELLY SAND

ZE-THAW SPECIMENS

	5% Cement				7% Cement			
2	A1	A2	B1	B2	A1	A2	B1	B2
89	4.93	4.90	4.92	4.92	4.92	4.89	4.88	4.90
90	4.90	4.88	4.94	4.95	4.89	4.83	4.90	4.89
88	4.89	4.86	4.92	4.93	4.88	4.80	4.89	4.90
88	4.88	4.85	4.92	4.93	4.84	4.81	4.88	4.93
88	4.88	4.85	4.92	4.93	4.87	4.79	4.89	4.94
89	4.88	4.84	4.93	4.93	4.87	4.79	4.89	4.90
89	4.87	4.83	4.93	4.93	4.86	4.78	4.90	4.89
89	4.86	4.83	4.93	4.93	4.87	4.80	4.90	4.90
90		4.86	4.94	4.95	4.87	4.85	4.92	4.94
0	2	2	0	0	1	2	0	0
0			0.2	0.2			0.5	1.0
	E	E	E	E	E	E	E	E

T-DRY SPECIMENS

	5% Cement			
2	A1	A2	B1	B2
90	4.91	4.92	4.91	4.89
91	4.91	4.85	4.94	4.84
91	4.88	4.89	4.93	4.92
59			4.62	4.63
90	4.81	4.88	4.91	4.89
58			4.65	4.61
89	4.74	4.87	4.93	4.91
56			4.68	4.64
89	4.39	4.57	4.94	4.91
54			4.68	4.65
89	4.36	4.54	4.95	
55			4.63	4.61
87	4.34	4.55	4.94	4.90
54			4.67	4.64
87	4.64	4.86	4.93	
0	5	1	0	0
			G	G

VE STRENGTH SPECIMENS

	5% Cement				7% Cement			
	D1	D2	E1	E2	D1	D2	E1	E2
88	4.91	4.90	4.87	4.88	4.91	4.92	4.90	4.90
88	4.85	4.81	4.82	4.88	4.84	4.89	4.92	4.91
	4.89	4.88			4.89	4.91		
91			4.78	4.86			4.85	4.88
93			4.86	4.90			4.90	4.92

OK.

T

WEIGHTS, MOISTURE CONTENTS, A

Freez

Weight, Lb.			3% Cement				5% Ceme	
			A1	A2	B1	B2	A1	A2
As Molded, Wet			-	-	4.40	-	4.39	4.39
After Cure			4.42	4.39	4.26	4.42	4.38	4.43
Cycle No.	2	After Thaw	4.33	4.33	4.41	4.46	4.40	4.44
	4		4.22	4.21	4.42	4.41	4.40	4.42
	6		4.12	4.12	4.36	4.40	4.36	4.37
	8		4.04	4.04	4.37	4.38	4.33	4.32
	10		3.94	3.93	3.92	4.34	4.29	4.26
	12		3.85	3.84	3.74	4.31	4.23	4.16
After Soak			3.86	3.86	3.63	4.32	4.23	4.16
Weight Loss %			14	14	-	-	6	8
Maximum moisture gain, % by dry wt.			-	-	-	-	-	-
Condition of Specimens (1)			F	FO	F	F	O	O

Wet-

Weight, Lb.		3% Cement				5% Ceme	
		A1	A2	B1	B2	A1	A2
As Molded, Wet		4.39	4.32	4.38	4.37	4.40	4.43
After Cure		4.47	4.34	4.40	4.39	4.49	4.49
Cycle No.	2 Wet	4.43	4.39	4.45	4.42	4.41	4.42
	2 Dry	3.91	3.84	3.93	3.95	3.89	3.90
	4 Wet	4.38	4.31	4.44	4.41	4.38	4.39
	4 Dry	3.84	3.76	3.92	3.90	3.89	3.90
	6 Wet	4.30	4.20	4.43	4.40	4.25	4.38
	6 Dry	3.79	3.67	3.96	3.94	3.76	3.88
	8 Wet	4.22	4.10	4.43	4.39	4.17	4.37
	8 Dry	3.72	3.59	3.93	3.90	3.69	3.87
	10 Wet	4.14	4.02	4.42	4.38	4.16	4.35
	10 Dry	3.62	3.50	3.91	3.87	3.69	3.86
	12 Wet	4.05	3.93	4.40	4.36	4.16	4.33
	12 Dry	3.55	3.43	3.90	3.86	3.69	3.85
After Soak		4.01	3.87	4.40	4.36	4.16	4.33
Weight Loss %		9	11	0	0	6	2
Condition of Specimens (1)		FO	FO	O	O	FO	FO

Compressi

Weight, Lb.	3% Cement				5% Ceme	
	D1	D2	E1	E2	D1	D2
As Molded, Wet	4.34	4.36	4.35	4.36	4.40	4.41
After 7 Day Cure	4.42	4.42	4.40	4.48	4.34	4.48
and Soak	4.44	4.45	-	-	4.43	4.48
After 39 Day Cure			4.21	4.30		
and Soak			4.43	4.43		

(1) See footnote 1 of Table 3

4

NDITION OF SPECIMENS FOR LOAM

pecimens

	7% Cement				10% Cement				13% Cement			
B2	A1	A2	B1	B2	A1	A2	B1	B2	A1	A2	B1	B2
37	4.42	4.42	4.41	4.42	4.42	4.39	4.39	4.38	4.38	4.38	4.40	4.41
36	4.44	4.48	4.45	4.46	4.50	4.47	4.44	4.41	4.43	4.47	4.47	4.45
40	4.42	4.46	4.46	4.47	4.48	4.46	4.45	4.44	4.44	4.46	4.48	4.46
46	4.41	4.44	4.46	4.47	4.47	4.44	4.45	4.46	4.43	4.44	4.48	4.46
45	4.40	4.42	4.46	4.47	4.45	4.42	4.44	4.45	4.42	4.43	4.47	4.46
45	4.39	4.41	4.45	4.47	4.44	4.41	4.45	4.45	4.42	4.42	4.47	4.46
46	4.38	4.40	4.45	4.47	4.44	4.41	4.45	4.46	4.42	4.42	4.47	4.43
47	4.37	4.37	4.44	4.47	4.42	4.40	4.45	4.46	4.41	4.41	4.47	4.43
47	4.38	4.38	4.45	4.48	4.43	4.41	4.46	4.46	4.42	4.42	4.47	4.45
-	3	3	-	-	2	2	-	-	1	1	-	-
3	-	-	1.2	1.2	-	-	1.5	2.0	-	-	1.8	1.2
)	O	O	G	G	G	E	E	G	E	E	E	E

cimens

	7% Cement				10% Cement			
B2	A1	A2	B1	B2	A1	A2	B1	B2
.39	4.41	4.37	4.41	4.40	4.38	4.40	4.42	4.38
.46	4.48	4.49	4.47	4.49	4.49	4.52	4.49	4.49
.43	4.43	4.42	4.41	4.43	4.41	4.41	4.38	4.35
.92	3.93	3.90			3.94	3.96	3.94	3.91
.42	4.40	4.38						
.94	3.95	3.90						
.41	4.37	4.35						
.91	3.92	3.88						
.41	4.37	4.35						
.92	3.92	3.87						
.40	4.37	4.34						
	3.91	3.86						
.40	4.37	4.33	4.45	4.44	4.39	4.38	4.41	4.38
.95	3.93	3.85	4.01	3.99	3.97	3.98	4.01	4.00
.39	4.37	4.33	4.45	4.43	4.38	4.37	4.40	4.36
)	2	2	0	0	0	2	0	0
)	O	O	G	G	G	G	G	G

th Specimens

	7% Cement				10% Cement				13% Cement			
E2	D1	D2	E1	E2	D1	D2	E1	E2	D1	D2	E1	E2
.41	4.39	4.41	4.40	4.40	4.42	4.40	4.37	4.40	4.40	4.40	4.40	4.43
.44	4.40	4.48	4.48	4.42	4.42	4.41	4.43	4.44	4.45	4.39	4.44	4.48
-	4.43	4.49	-	-	4.46	4.47	-	-	4.46	4.44	-	-
.47			4.23	4.31			4.32	4.32			4.36	4.49
.47			4.38	4.40			4.38	4.38			4.40	4.49

TABLE 5

WEIGHTS, MOISTURE CONTENTS, AND CONDITION OF SPECIMENS FOR CLAY

Freeze-Thaw Specimens

Weight, Lb.		3% Cement		5% Cement				7% Cement				10% Cement				13% Cement			
		B1	B2	A1	A2	B1	B2	A1	A2	B1	B2	A1	A2	B1	B2	A1	A2	B1	B2
As Molded, Wet		-	-	4.19	4.13	4.13	4.12	4.15	4.16	4.19	4.17	4.19	4.18	4.16	4.17	4.22	4.22	4.22	4.22
After Cure		4.07	4.03	4.19	4.13	4.12	4.10	4.18	4.21	4.23	4.18	4.20	4.20	4.16	4.18	4.24	4.23	4.24	4.23
Cycle No. After Thaw	2	4.12	4.10	4.15	4.11	4.12	4.10	4.15	4.18	4.23	4.18	4.17	4.17	4.15	4.17	4.22	4.20	4.23	4.22
	4	4.18	4.14	4.07	4.08	4.14	4.11	4.10	4.15	4.21	4.19	4.16	4.15	4.15	4.17	4.20	4.19	4.22	4.22
	6	4.20	4.19	3.99	3.98	4.16	4.11	4.03	4.07	4.21	4.18	4.14	4.13	4.14	4.16	4.20	4.18	4.21	4.20
	8	3.85	3.90	3.81	3.80	4.18	4.12	3.90	3.94	4.20	4.18	4.13	4.12	4.14	4.16	4.18	4.17	4.21	4.20
	10			3.65	3.55	4.21	4.15	3.83	3.88	4.21	4.19	4.12	4.11	4.15	4.17	4.18	4.17	4.22	4.21
	12			3.44	3.25	4.23	4.19	3.76	3.80	4.21	4.19	4.10	4.10	4.15	4.17	4.17	4.16	4.21	4.20
After Soak				3.43	3.24	4.20	4.18	3.77	3.80	4.21	4.18	4.10	4.10	4.16	4.18	4.18	4.17	4.22	4.21
Weight Loss, %				19	23	0	0	10	9	0	0	2	2	0	0	1	1	0	0
Condition of Specimens (1)		P	P	F	F	FO	F	FO	FO	O	O	O	O	G	G	G	E	G	E
Maximum Moisture Gain, % by Dry Wt.				-	-	2.9	2.0	-	-	1.1	0.7	-	-	0	0	-	-	0.2	0.5

Wet-Dry Specimens

Weight, Lb.		3% Cement				5% Cement				7% Cement				10% Cement			
		A1	A2	B1	B2	A1	A2	B1	B2	A1	A2	B1	B2	A1	A2	B1	B2
As Molded, Wet		4.10	4.09	4.12	4.13	4.21	4.17	4.16	4.18	4.18	4.19	4.16	4.15	4.17	4.19	4.18	4.19
After Cure		4.23	4.27	-	-	4.24	4.17	4.17	4.20	4.20	4.21	4.19	4.10	4.17	4.19	4.20	4.21
Cycle No.	2 Wet	4.11	4.01	4.16	4.17	4.15	4.12	4.14	4.17	4.14	4.15	4.16	4.09	4.14	4.14	4.16	4.17
	2 Dry	3.45	3.29	3.53	3.55	3.76	3.74	3.53	3.56	3.59	3.74	3.69	3.48	3.80	3.82	3.74	3.74
	4 Wet	3.91	3.78	4.12	4.10	4.09	4.09	4.14	4.16	4.13	4.11	4.09	4.04	4.09	4.08	4.12	4.11
	4 Dry	3.26	3.04	3.47	3.46	3.55	3.52	3.53	3.57	3.59	3.63	3.61	3.46	3.67	3.72	3.76	3.66
	6 Wet	3.76	3.49	4.06	4.06	4.10	4.07	4.13	4.15	4.12	4.09	4.14	4.03	4.08	4.10	4.13	4.08
	6 Dry	3.13	2.90	3.42	3.40	3.52	3.46	3.54	3.54	3.55	3.55	3.63	3.47	3.58	3.61	3.65	3.64
	8 Wet	3.59	3.35	3.98	3.96	4.07	4.01	4.13	4.14	4.11	4.05	4.12	4.04	4.06	4.08	4.11	4.10
	8 Dry	2.93	2.80	3.36	3.36	3.52	3.45	3.66	3.58	3.57	3.59	3.55	3.39	3.62	3.66	3.70	3.73
	10 Wet	3.36	3.12	3.92	3.89	4.01	3.94	4.11	4.12	4.05	4.01	4.09	3.96	4.01	4.02	4.07	3.97
	10 Dry	2.75	2.58	3.31	3.29	3.49	3.39	3.49	3.60	3.56	3.58	3.66	3.49	3.62	3.62	3.80	3.59
	12 Wet			3.89	3.87	3.99	3.90	4.03	4.12	4.03	4.01	4.01		4.01	4.04	4.00	4.01
	12 Dry			3.28	3.26	3.44	3.28	3.50	3.51	3.51	3.55	3.48		3.58	3.60	3.62	3.67
After Soak				3.93	3.90	3.98	3.84	4.01	4.12	4.05	4.06	4.06		4.05	4.07	4.07	4.07
Weight Loss, %		20	25	6	6	6	7	2	0	1	1	1	-	0	0	0	0
Condition of Specimens (1)		P	PF	F	F	F	F	F	FO	FO	FO	FO	FO	FO	0	OG	O

Compressive Strength Specimens

Weight, Lb	3% Cement				5% Cement				7% Cement				10% Cement			13% Cement			
	D1	D2	E1	E2	D1	D2	E1	E2	D1	D2	E1	E2	D1	D2	E1	D1	D2	E1	E2
As Molded, Wet	4.07	4 07	4.06	4 09	4 19	4 15	4 15	4 15	4 15	4.15	4.19	4.20	4.18	4 19	4.19	4.22	4.22	4.17	4.22
7 Day Cure	4.14	4.21	4.06	4.15	4.22	4.19	4.19	4.16	4.15	4.14	4.20	4.21	4.20	4.19	4.15	4.23	4.25	4.21	4.19
And Soak	4.17	4.21	-	-	4.23	4.19	-	-	4.15	4.16	-	-	4.20	4.19	-	4.24	4.24	-	-
39 Day Cure	-	-	4.06	-	-	-	4.22	4.21	-	-	4.22	4.24	-	-	4.23	-	-	4.23	4.21
And Soak	-	-	4.13	4.19	-	-	4.23	4.21	-	-	4.22	4.24	-	-	4.23	-	-	4.23	4.21

TABLE 6

WEIGHTS, MOISTURE CONTENTS, AND CONDITION OF SPECIMENS FOR SILT

Freeze-Thaw Specimens																
	8% Cement				10% Cement				12% Cement				14% Cement			
Weight Lb	A1	A2	B1	B2	A1	A2	B1	B2	A1	A2	B1	B2	A1	A2	B1	B2
As Molded, Wet	4.15	4.16	4.16	4.15	4.17	4.12	4.18	4.16	4.17	4.17	4.15	4.15	4.14	4.15	4.11	4.13
After Cure	4.26	4.26	4.27	4.33	4.23	4.22	4.23	4.22	4.26	4.29	4.22	4.25	4.29	4.30	4.32	4.39
Cycle No. After Thaw 2	4.27	4.29	4.35	4.33	4.33	4.31	4.35	4.34	4.35	4.31	4.34	4.33	4.39	4.38	4.40	4.40
4	4.25	4.27	4.35	4.33	4.31	4.29	4.35	4.34	4.33	4.31	4.35	4.34	4.37	4.38	4.39	4.39
6	3.83	3.81	4.36	4.34	4.10	4.09	4.36	4.34	4.32	4.30	4.35	4.35	4.37	4.35	4.38	4.39
8	3.68	3.50	4.06	4.16	3.88	3.88	4.36	4.34	4.29	4.29	4.36	4.35	4.36	4.34	4.39	4.39
10	3.33	3.20	4.05	3.77	3.80	3.77	4.37	4.35	4.17	4.19	4.36	4.36	4.35	4.33	4.39	4.40
12	2.91	2.69	3.96	3.75	3.69	3.59	4.29	4.08	3.96	4.00	4.37	4.23	4.35	4.33	4.39	4.39
After Soak	2.88	2.69	3.93	3.76	3.70	3.55	4.17	4.03	3.98	4.01	4.23	4.10	4.36	4.34	4.40	4.40
Weight Loss %	34	38			15	18			8	7			2	2		
Maximum Moisture Gain, % by Dry Wt			5.5	5.2			5.2	5.2			6.0	5.5			7.9	7.4
Condition of Specimens (1)	F	F	FO	FO	F	O	O	F	FO	O	O	O	G	G	G	E

Wet-Dry Specimens																
	4% Cement				6% Cement				8% Cement				10% Cement			
Weight Lb	A1	A2	B1	B2	A1	A2	B1	B2	A1	A2	B1	B2	A1	A2	B1	B2
As Molded, Wet	4.14	4.14	4.14	4.14	4.14	4.15	4.15	4.14	4.13	4.13	4.15	4.15	4.13	4.13	4.14	4.16
After Cure	4.21	4.36	4.29	4.25	4.21	4.19	4.21	4.16	4.18	4.24	4.30	4.25	4.24	4.22	4.17	4.18
Cycle No 2 Wet	4.28	4.29	4.32	4.31	4.24	4.25	4.28	4.33	4.26	4.26	4.28	4.29	4.26	4.26	4.28	4.29
2 Dry	3.62	3.64	3.71	3.71	3.67	3.67	3.70	3.72	3.68	3.68	3.69	3.72	3.69	3.69	3.71	3.71
4 Wet	4.17	4.19	4.31	4.30	4.21	4.22	4.28	4.33	4.24	4.24	4.28	4.29	4.24	4.25	4.29	4.29
4 Dry	3.51	3.52	3.69	3.71	3.61	3.61	3.71	3.72	3.64	3.64	3.70	3.73	3.66	3.67	3.72	3.72
6 Wet	4.04	4.05	4.29	4.29	4.15	4.16	4.29	4.33	4.21	4.21	4.28	4.29	4.22	4.23	4.29	4.29
6 Dry	3.42	3.44	3.70	3.70	3.57	3.58	3.73	3.74	3.61	3.62	3.72	3.74	3.65	3.68	3.74	3.74
8 Wet	3.94	3.97	4.30	4.29	4.11	4.11	4.28	4.33	4.18	4.18	4.28	4.29	4.20	4.21	4.28	4.29
8 Dry	3.36	3.38	3.70	3.70	3.53	3.55	3.73	3.73	3.60	3.61	3.71	3.74	3.63	3.66	3.74	3.74
10 Wet	3.86	3.88	4.29	4.29	4.05	4.07	4.28	4.33	4.14	4.15	4.28	4.29	4.18	4.20	4.29	4.29
10 Dry	3.26	3.29	3.68	3.71	3.49	3.52	3.73	3.72	3.58	3.60	3.72	3.74	3.63	3.66	3.74	3.74
12 Wet	3.76	3.79	4.27	4.18	3.99	4.04	4.29	4.32	4.13	4.13	4.28	4.30	4.17	4.19	4.30	4.30
12 Dry	3.20	3.22	3.68	3.61	3.44	3.49	3.72	3.72	3.56	3.57	3.70	3.73	3.62	3.63	3.72	3.74
After Soak	3.75	3.77	4.26		3.99	4.04	4.28	4.31	4.14	4.13	4.28	4.29	4.17	4.18	4.29	4.28
Weight Loss %	12	12	0	0	7	6	0	0	3	3	0	0	2	2	0	0
Condition of Specimens (1)	FO	FO	FO	O	O	O	G	O	O	O	G	G	OG	OG	G	G

Compressive Strength Specimens																								
	4% Cement				6% Cement				8% Cement				10% Cement				12% Cement				14% Cement			
Weight Lb	D1	D2	E1	E2	D1	D2	E1	E2	D1	D2	E1	E2	D1	D2	E1	E2	D1	D2	E1	E2	D1	D2	E1	E2
Molded Wet	4.15	4.16	4.15	4.15	4.13	4.13	4.15	4.15	4.14	4.13	4.12	4.15	4.13	4.12	4.14	4.14	4.13	4.12	4.15	4.15	4.14	4.15	4.15	4.14
7 Day	4.24	4.33	4.11	4.20	4.35	4.16	4.13	4.33	4.24	4.24	4.22	4.20	4.19	4.14	4.18	4.15	4.18	4.20	4.16	4.20	4.20	4.25	4.25	4.39
and Soak	4.30	4.35			4.36	4.26			4.26	4.27			4.27	4.27			4.26	4.27			4.35	4.34		
39 Day			4.27	4.22			4.20	4.28			4.24	4.21			4.16	4.23							4.46	4.30
and Soak			4.31	4.30			4.31	4.32			4.32	4.33			4.33	4.32			4.32	4.32			4.46	4.39

(1) See footnote 1 of Table 3

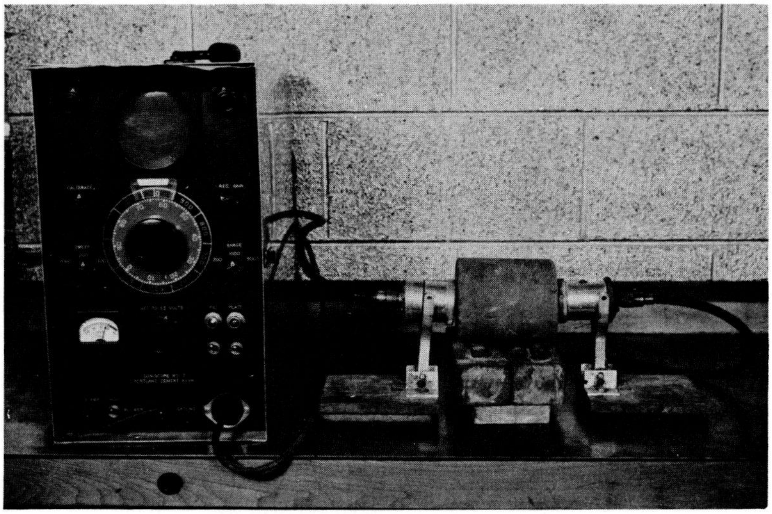

Figure 2. Soniscope for determining pulse velocity.

Soil-cement specimens of the silt had maximum moisture gains ranging from 5.2 to 7.9 percentage points above the molded moisture content, more than enough moisture for saturation. The greater portion of these moisture gains occurred during the 7-day moist-curing period, where the specimens were exposed to 100 percent relative humidity but were protected from contact with free water. It is assumed that expansion accompanied, or perhaps caused, these large moisture gains during the moist-curing period. The extent of expansion is not known because lengths were not measured before the seventh day. Ordinarily, moisture gains above saturation indicate that the specimens are deteriorating. In this case the large moisture gains during cure did not cause deterioration later in the freeze-thaw test. They occurred on all the silt specimens including those passing the test with low weight losses, excellent strengths, and adequate hardness.

The silt soil is a "scaler" in the freeze-thaw test. Specimens at the lower cement contents developed a surface shell which scaled off or was easily removed by picking. At higher cement contents, no shell could be observed or removed by vigorous picking. However, a shell was observed when these specimens were broken in compression after freeze-thaw test.

It would be interesting to determine, for a number of soils, if the development of a surface shell during the freeze-thaw test is related to large moisture gains. However, it is more significant to note that the existence of a shell, which cannot be removed, does not indicate deterioration, because the strength and hardness of specimens in this specific condition were excellent.

Based on recommended weight loss criteria adequate freeze-thaw resistance was obtained for specimens containing the cement contents given in Table 7.

TABLE 7

CEMENT CONTENTS OF SOIL SAMPLES FOR FREEZE-THAW TESTS

Sample	Lowest Cement Content Meeting Weight Loss Criteria (% by wt.)
Gravelly sand	3
Loam	5
Clay	10
Silt	12

Specimens at these and higher cement contents passed the test; specimens at lower cement contents failed. In this report, these are referred to as acceptable specimens and failed specimens. This is the basis of judgment for evaluating the suitability of length change, compressive strength, and pulse velocity methods for measuring freeze-thaw resistance.

Length Changes

Lengths of unbrushed specimens were determined after cure, after each freeze and each thaw portion of the cycles and after soaking at the end of test. Length changes from the 7-day moist-cured length are given in Tables 8 through 11. Positive values indicate an expansion above the cured length and negative values indicate shrinkage.

Figure 3 shows the length changes during the freeze-thaw test for the clay. This is the typical pattern for all four soils. Acceptable specimens shrank on freezing and expanded back to near the cured length on thawing, frozen lengths and thawed lengths remaining practically constant throughout the test. Failed specimens expanded above the cured length on thawing at some time during the test. Badly deteriorated specimens expanded on freezing.

Length changes at the end of test are shown in Figure 4 for all four soils. It can be seen that all failed specimens expanded more than 0.0075 in. on thawing. Badly deteriorated specimens expanded on freezing above the thawed lengths. Expansions up to 0.0300 in. were obtained depending on the degree of deterioration.

For acceptable specimens, no serious expansion occurred. In addition, the frozen lengths were always less than the thawed lengths. The gravelly sand at 3 percent cement, an adequate cement content, expanded 0.0040 in. on thawing, which may indicate that deterioration was imminent.

In any case, all acceptable specimens had length changes during the test of 0.0040 in. or less, and frozen lengths were less than thawed lengths.

Soil type was the primary factor influencing the freezing shrinkage of acceptable specimens. These shrinkages varied from -0.0025 to -0.0150 in. depending on clay content. Thawed lengths also varied directly with the clay content of the soils.

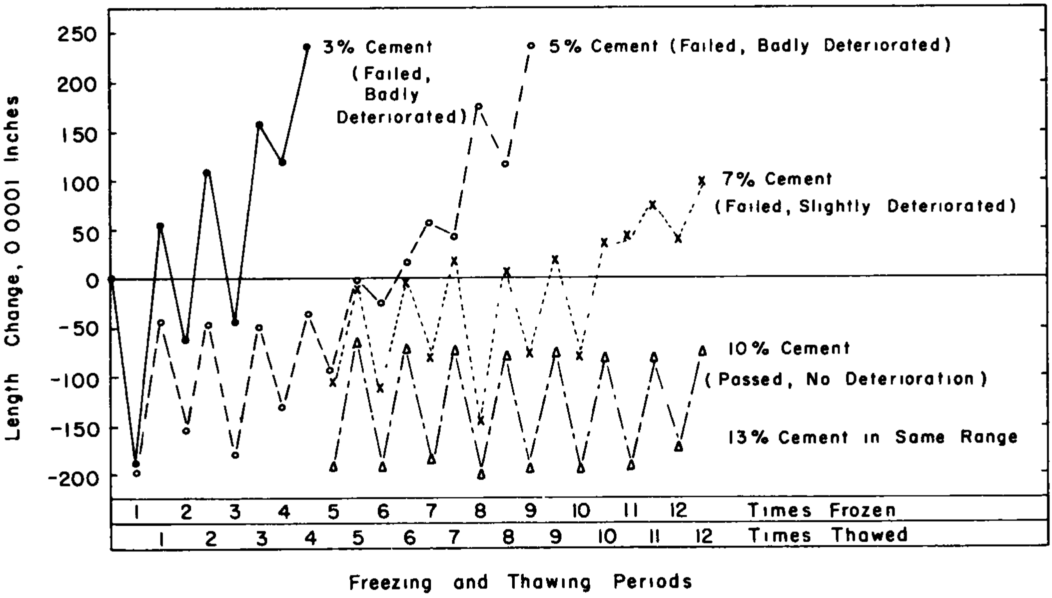

Figure 3. Length changes during freeze-thaw test, clay.

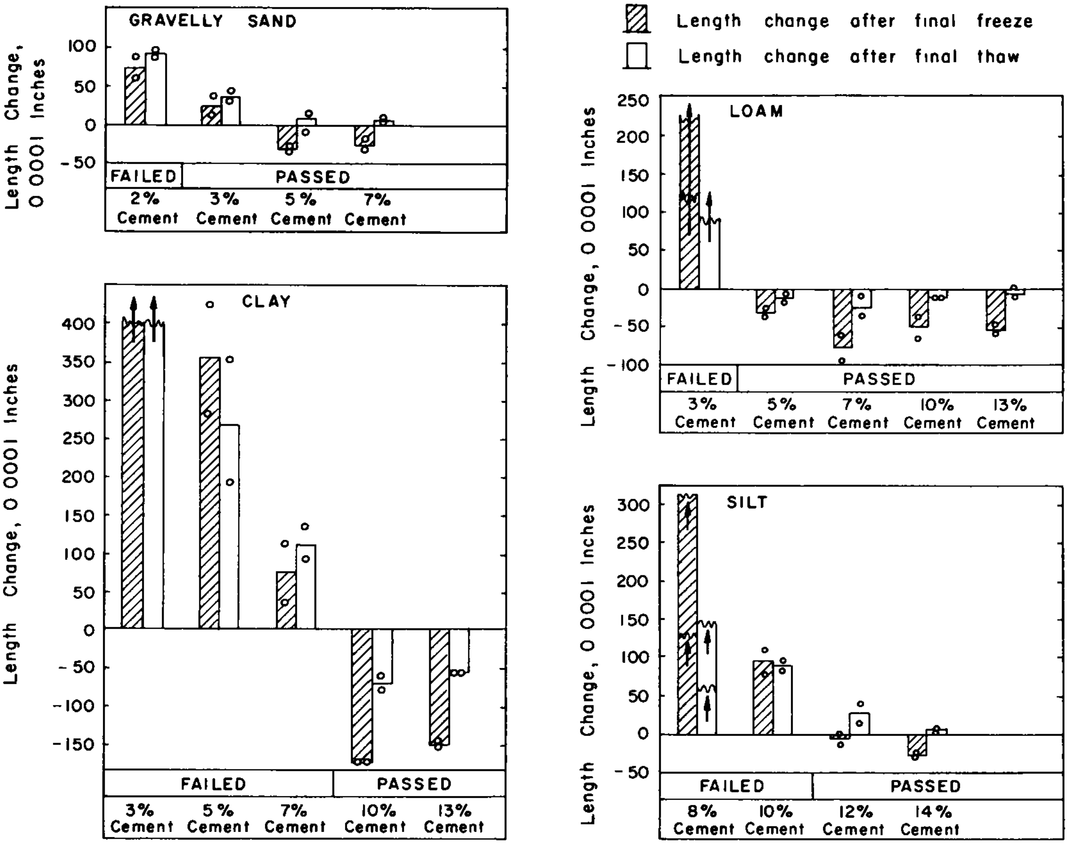

Figure 4. Length changes at the end of freeze-thaw test.

Excellent sensitivity to deterioration is indicated for length change measurements. For example, referring again to Figure 3, expansion trends show that deterioration is imminent at the first cycle for 3 percent cement and at the fifth cycle for 5 percent cement.

Compressive Strengths

Compressive strengths of soaked specimens after the freeze-thaw test are given in Table 12. The strengths-after-test are also plotted in Figure 5 where they are compared with strengths at 7 and 39 days. The strengths after 7 and 39 days of continuous moist cure represent control values for the beginning and end of the freeze-thaw test.

The strength values for adequate freeze-thaw resistance varied considerably for the different soil types. For specimens with the minimum cement content passing the test, the strengths vary, for the four soils, from 250 to 770 psi at 7 days, from 400 to 1,250 psi at 39 days, and from 350 to 1,000 psi after the freeze-thaw test.

Strengths after the freeze-thaw test for all adequately hardened specimens were greater than the 7-day strengths and less than the 39-day strengths. These strength gains during the test are evidently due to additional moist-curing in the 24-hr thaw periods.

The compressive strengths-after-test for acceptable specimens are, as a minimum, halfway between the 7- and 39-day strength values. The strengths after-test of failed specimens are less than the value halfway between the 7- and 39-day strengths, in fact,

TABLE 6

LENGTH CHANGES, GRAVELLY SAND

Length Changes of Freeze-Thaw Specimens, 0001 In

	2% Cement				3% Cement				5% Cement				7% Cement			
	B1		B2		B1		B2		B1		B2		B1		B2	
	F	T	F	T	F	T	F	T	F	T	F	T	F	T	F	T
After Cure		0		0		0		0		0		0		0		0
1		5	-12	5		0		0		0		0	1	0	-35	2
2			11	9		3		2		-1		-9		1		
3	13	30				2		3		-1		-7		1	-11	10
4			33	35		7		3		3		-9		4	-24	6
5			47	66		5		10		3		-6		1	-17	12
6	19	66	54	68		17		23		15		4		1	-23	11
7	44	76	43	60	-2	16	-3	21	-29	14	-32	0	-30	1	-31	8
8	54	77	50	59	0	14	9	24	-31	13	-39	-2	-37	2	-32	7
9	61	84	61	74	-1	17	10	29	-26	16	-36	-1	-35	1	- 7	10
10	74	93	68	77	8	20	19	27	-23	16	-35	1	-31	1	-20	15
11	70	127	63	77	2	26	18	36	-27	21	-33	-2	-37		-31	7
12	66	100	78	87	16	30	39	40			-29	-10	-33	3	-20	12
After Soak		84		85												9

Cycle No

F - After Freeze

T - After Thaw

Length Changes of Wet-Dry Specimens, 0001 In

	2% Cement				3% Cement				5% Cement			
	B1		B2		B1		B2		B1		B2	
	W	D	W	D	W	D	W	D	W	D	W	D
After Cure	0		0		0		0					
1	-1	x	3	-35	1	-43	5	-68	-8	-46	5	-23
2	x-15	x-6	-41	-17	-79	x	-65	-63	-53	-33	-30	- 9
3	x-13	x-4	-31	-13	x-7	x-10	-68	-62	-50	-48	-32	-22
4	x-16	x-1	-37		x-20	x-12	-69	-63	-61	-39	-34	-16
5	x-14	x-7	-36	-23	x-21	x-14	-71	-43	-55	-41	-34	-21
6	x-17	x-3	-29	-1	x-22	x+3	-69	-60	-42	-27	-29	-28
7	x-16	x+2	-19	-22	x-19	x-1	-76	-61	-27	-33	-29	-31
8	x-23	x-2	-18	8	x-30	x-9	-71	-56	-31	-34	-24	
9	x-15	x-4	-5	11	x-20	x-7	-68	-55	-50	-36		
10	x-15	x+10	-11	9	x-20	x-6	-67	-37	-42	-29		
11	x-16	x+6	-18	2	x-23	x+9	-59	-53	-56	-31		
12	x-1	x+9	-12	4	x-20	x+4	-58	-43	-50	-31		
After Soak	x-8		-10		x-20		-45		-34			

Cycle No

W - After Wet

D - After Dry

x - new reference points attached to specimen

Length Changes of Compressive Strength Specimens, 0001 In

	2% Cement		3% Cement		5% Cement		7% Cement	
	E1	E2	E1	E2	E1	E2	E1	E2
39 Day Cure	-6	-9	0	-5	-3	-5	0	-3
39 Day Cure and Soak	0	-7	-1	-7	-9	-5	-4	-4

TABLE 9

LENGTH CHANGES, LOAM

Length Changes of Freeze-Thaw Specimens, .0001 in.

F-After Freeze
T-After Thaw

Cycle No.	3% Cement				5% Cement				7% Cement				10% Cement				13% Cement			
	B1		B2		B1		B2		B1		B2		B1		B2		B1		B2	
	F	T	F	T	F	T	F	T	F	T	F	T	F	T	F	T	F	T	F	T
After Cure		0		0		0		0		0		0		0		0		0		0
1		8	-92	43	-97	2		-30	-76	-17	-86	-28	-67	-3	-66	-3	-58	-19	-61	-5
2	-11	-6	-15	100	-56	13	-102	-39	-81	-18	-98	-37	-65	-5	-56	-2	-72	-18	-62	-7
3	-53	1	32	96	-44	4	-103	-46	-107		-90	-38	-61	-6	-49	-2	-67	-16	-57	-6
4	-53	7	64	99	-65	-6	-114	-48	-69	-17	-91	-36	-60	-4	-48		-66	-20	-57	-7
5	-21	19			-47	-4	-111	-47	-67	-9	-89	-37	-61	1	-49		-69	-15	-61	-2
6	-32	24	42	90	-52	-2	-108	-48	-61	-10	-89	-37	-53	2	-38	-1	-61	-15	-52	-2
7	44	39	89	81	-17	-3	-90	-40	-52	-4	-85	-36	-55	-1	-32		-55	-14	-48	4
8	129	66	120	85	-50	-1	-94	-40	-62	-5	-101	-37	-49	-1	-40	-2	-56	-10	-47	3
9	238	84			-20	-3	-74	-34	-50	-3	-99	-38	-60	-3	-35	-1	-23	-1	-20	5
10					-35	-16	-48	-30	-57		-100	-34	-57	-2	-31	-2	-40	-4	-40	-5
11					-28	-5	-25	-15			-103	-41	-60	-2	-41	-4	-44	-4	-48	-2
12					-37	-2		-12			-94	-36	-62	-4	-35	-4	-59	-2	-46	3
After Soak						5		-16				-34	-55	1		-3		1		3

Length Changes, Wet-Dry Specimens, .0001 in.

W-After Wet
D-After Dry

Cycle No.	3% Cement				5% Cement				7% Cement				10% Cement			
	B1		B2		B1		B2		B1		B2		B1		B2	
	W	D	W	D	W	D	W	D	W	D	W	D	W	D	W	D
After Cure																
1	11	-187	12	-162	23	-184	40	-147	58	-111	73	-28	62	-90	42	-102
2	-172	-189		X	-163	-206	-144	-160	-115	-132	-107	-159	-77	-116	-101	-134
3	-174	-183	X+12	X-1	-198	-188	-154	-150	-124	-121	-151	-147	-106	-105	-127	-124
4	-176	-185	X+9	X-10	-187	-196	-149	-150	-120	-124	-145	-148	-101	-110	-121	-130
5	-175	-191	X-1	X-16	-208	-206	-156	-158	-122	-133	-144	-154	-107	-116	-126	-138
6	-174	-185	X-6	X-8	-208	-194	-158	-154	-130	-134	-155	-157	-116	X	-142	-138
7	-164	-180	X-1	X-5	-206	-203	-143	-139	-134	-135	-154	-153	X+2	X+7	-139	-135
8	-172	-179			-202	-214	-143	-130	-132	-134	-150	-165	X+7	X-13	-134	-146
9	-162	-186			-193	-211			-137	-119	-158	-147	X-6	X+4	-142	-135
10	-157	-180			-210	-203			-133	-128	-158	-163	X-5	X-10	-140	-146
11	-150	-168			-211	-194			-138	-121	-159	-149	X-7	X	-143	-134
12	-129	-152			-206	-206			-134	-126	-156	-157	X-2	X-7	-138	-145
After Soak	-126				-208				-137		-156		X-5		-144	

X---New reference points attached to specimen.

Length Changes, Compressive Strength Specimens, .0001 in.

	3% Cement		5% Cement		7% Cement		10% Cement		13% Cement	
	E1	E2	E1	E2	E1	E2	E1	E2	E1	E2
39 Day Cure	-85	-96	-54	-54	-77	-37	6	-67	5	5
39 Day Cure and Soak	-53		-57	-55		-15	-8	-86	3	3

TABLE 10

LENGTH CHANGES, CLAY

Length Changes, Freeze-Thaw Specimens, .0001 in

	3% Cement				5% Cement				7% Cement				10% Cement				13% Cement			
	B1		B2		B1		B2		B1		B2		B1		B2		B1		B2	
	F	T	F	T	F	T	F	T	F	T	F	T	F	T	F	T	F	T	F	T
After Cure		0		0		0		0		0		0		0		0		0		0
1	-189	60	-184	45	-198	-40	-212	-45			-107	-2	-147	-28	-136	-17	-118	-22	-122	-26
2	-66	126	-153	87	-159	-46	-188	-49			-57	3	-175	-42	-167	-37	-162	-44	-161	-35
3	-46	161	-61	177	-181	-48	-195	-47			-37	8	-185	-59	-172	-51	-166	-54	-155	-52
4	124	237	108	220	-132	-36	-138	-36			-14	16	-190	-64	-180	-59	-166	-67	-160	-52
5	774	451	535	291	-92	-1	-136	-23	-109	7	-10	40	-193	-68	-191	-58	-170	-62	-158	-54
6					-25	13	-94	-10	-114	4	-16	36	-192	-75	-186	-68	-164	-65	-155	-63
7					61	43	-41	1	-85	14	1	45	-186	-74	-182	-67	-155	-64	-149	-71
8					175	117	-5	27	-142	5	24	46	-199	-81	-192	-74	-167	-66	-165	-68
9					235	134	32	52	-76	18	10	66	-192	-75	-186	-69	-162	-68	-153	-65
10					374	184	70	73	-80	26	7	72	-191	-82	-186	-64	-164	-61	-156	-61
11					509	331	167	130	38	73	104	103	-194	-82	-189	-64	-165	-64	-161	-61
12					435	357	285	189	36	94	116	134	-175	-78	-174	-63	-154	-60	-149	-62
After Soak						294		182		99						-60		-56		-55

Cycle No.

F-After Freeze

T-After Thaw

Length Changes, Wet-Dry Specimens, .0001 in

	3% Cement				5% Cement				7% Cement				10% Cement			
	B1		B2		B1		B2		B1		B2		B1		B2	
	W	D	W	D	W	D	W	D	W	D	W	D	W	D	W	D
After Cure																
1		-463		-483	1	-319	0	-386		-363	-1	-303	2	-368	1	-395
2	-363	-479	-337	-492	-260	-322	-347	-411			-257	-312	-322	-412	-340	-437
3	-373	-513	-361	-511	-277	-357	-364	-426			-256	-335	-357	-425	-377	-458
4	-385	-515	-380	-508	-269	-332	-371	-410			-278	-320	-377	-410	-408	-454
5	-381	-492	-383	-488	-232	-311	-370	-373			-269	-305	-362	-398	-411	-453
6	-387	-500	-382	-495	-226	-311	-371	-387			-253	-313	-353	-410	-415	-461
7	-316	-473					-380	-426			-245	-308	-371	-405	-410	-460
8	-320	-435					-394	-412			-232	-317	-354	-397	-407	-448
9	-247	-425					-401	-419			-235	-325	-342	-385	-404	-476
10	-202	-254					-377	-403			-246	-291	-329	-366	-420	-465
11							-387	-429			-230	-322	-321	-389	-420	-470
12							-395	-426			-231	-305	-320	-370	-414	-452
After Soak											-229		-314		-414	

Cycle No.

W-After Wet

D-After Dry

Length Changes, Compressive Strength Specimens, .0001 in.

	3% Cement				5% Cement				7% Cement				10% Cement				13% Cement			
	D1	D2	E1	E2	D1	D2	E1	E2	D1	D2	E1	E2	D1	D2	E1	E2	D1	D2	E1	E2
7 Day Cure & Soak																				
39 Day Cure			-137				-6	-4			-7	-7			-7	-7			-7	-7
39 Day Cure & Soak			-131	-122			-9	-9			-8	-7			-7	-7			-7	-8

TABLE 11

LENGTH CHANGES, SILT

Length Changes, Freeze-Thaw Specimens, .0001 in.

	8% Cement				10% Cement				12% Cement				14% Cement			
	B1		B2		B1		B2		B1		B2		B1		B2	
Cycle No.	F	T	F	T	F	T	F	T	F	T	F	T	F	T	F	T
After Cure		0		0		0		0		0		0		0		0
1	-28	3	-31	3	-31	3	-30	3	-29	2	-28	3	-34	2	-36	3
2	-29	4	-38	1	-38	2	-37	3	-38	1	-37	1	-42	3	-48	-8
3	-17	13	-31	5	-31	7	-30	7	-32	3	-33	4	-43	1	-47	7
4	-2	21	-29	9	-28	8	-28	10	-30	7	-31	6	-30	2	-38	-3
5	27	32	-23	9	-26	9	-25	10	-23	7	-25	7	-31	2	-31	-1
6	58	46	-23	11	-28	9	-27	10	-28	6	-30	6	-35	3	-37	4
7	124	66	43	50	-30	10	-25	12	-28	8	-28	9	-31	3	-29	4
8			138	96	-30	10	-25	12	-28	8	-20	12	-36	4	-35	6
9			263	144	-26	17	-21	20	-28	13	-17	10	-31	1	-29	5
10			318	LB	11	31	35	38	-26	16	-2	17	-31	6	-29	12
11					89	45	84	45	-25	11	2	22	-27	5	-22	13
12					111	95	80	85	-16	17	4	39	-29	2	-25	6
After Soak						100		100		26		43		7		10

F - After Freeze
T - After Thaw

Length Changes, Wet-Dry Specimens, .0001 in.

	4% Cement				6% Cement				8% Cement				10% Cement			
	B1		B2		B1		B2		B1		B2		B1		B2	
Cycle No	W	D	W	D	W	D	W	D	W	D	W	D	W	D	W	D
After Cure	0		0		0		0		0		0		0		0	
1	0	-22	0	-19	2	-31	7	-36	2	-26	2	-26	3	-30	-1	-36
2	-3	7	-7	-1	-22	-13	-24	-33	-24	-17	-22	-23	-15	-17	-18	-24
3			0	-1		-30	-18	-22	-22	-20	-22	-19	-14	-11	-16	-14
4			-4	-3		-30	-18	-28	-18	-11	-19	-23	-13	-15	-16	-19
5			-4	-3	-25	-27	-25	-33	-9	-13	-20	-16	-11	-6	-19	-9
6			-13	-2	-26	-25	-26	-25	-12	-6	-20	-18	-9	-10	-22	-18
7			-6		-27	-27	-24		-11	-7	-18	-16	-10	-10	-19	-19
8				1	-33	-26		X	-16	-13	-23	-15	-15	-7	-24	-15
9			6	33	-31	-27	X+7	X	-15	-12	-19	-13	-13	-10	-21	-19
10			38	65	-29	-23	X+11	X-2	-12	-8	-13	-8	-10	-5	-19	-15
11			65	84	-29	-18	X+8		-11	-2	-9	-1	-8	0	-16	
12			82		-28	-27			-11	-12	-11	-10	-9	-12		
After Soak					-27				-9		-4		-9			

W - After Wet
D - After Dry

Length Change, Compressive Strength Specimens, .0001 in.

	4% Cem.		6% Cem.		8% Cem.		10% Cem.		12% Cem.		14% Cem.	
	E1	E2	E1	E2	E1	E2	E1	E2	E1	E2	E1	E2
39 Day Cure	11	17	-5	5	31	6	7	10	-1	2		
39 Day Cure & Soak		17	3	12	27	13	13	16	8	10		

X---New reference points attached to specimen.

TABLE 12
COMPRESSIVE STRENGTHS

GRAVELLY SAND

COMPRESSIVE STRENGTH, PSI

	2% Cement		3% Cement		5% Cement		7% Cement	
	Spec No	PSI	Spec No	PSI	Spec No	PSI	Spec No	PSI
7 Day Cure and Soaking	D1	190	D1	312	D1	702	D1	1123
	D2	197	D2	326	D2	702	D2	1120
39 Day Cure and Soaking	E1	377	E1	605	E1	1131	E1	1831
	E2	332	E2	581	E2	1154	E2	1791
After Freeze-Thaw Test and Soaking	A1	158	A1	358	A1	1162	A1	1820
	A2	174	A2	487	A2	916	A2	1385
	B1	151	B1	596	B1	1218	B1	1688
	B2	110	B2	475	B2	1011	B2	1268
After Wet-Dry Test and Soaking	A1	365	A1	820	A1	1457		
	A2	337	A2	693	A2	1346		
	B1	380	B1	698	B1	1130		
	B2	237	B2	764	B2	1135		

LOAM

COMPRESSIVE STRENGTH, PSI

	3% Cement		5% Cement		7% Cement		10% Cement		13% Cement	
	Spec No	PSI	Spec No	PSI	Spec No	PSI	Spec No	PSI	Spec No	PSI
7 Day Cure and Soaking	D1	200	D1	224	D1	341	D1	577	D1	867
	D2	216	D2	243	D2	403	D2	586	D2	851
39 Day Cure and Soaking	E1	293	E1	426	E1	684	E1	1010	E1	1273
	E2	296	E2	423	E2	656	E2	1042	E2	1416
After Freeze-Thaw Test and Soaking	A1	250	A1	384	A1	481	A1	740	A1	987
	A2	235	A2	407	A2	525	A2	796	A2	1002
	B1		B1	350	B1	541	B1	804	B1	1120
	B2	244	B2	430	B2	543	B2	747	B2	844
After Wet-Dry Test and Soaking	A1	*	A1		A1	753	A1	1098		
	A2	354	A2	464	A2	676	A2	1161		
	B1	407	B1	703	B1	1050	B1	1352		
	B2	380	B2	636	B2	1050	B2	1368		

CLAY

COMPRESSIVE STRENGTH, PSI

	3% Cement		5% Cement		7% Cement		10% Cement		13% Cement	
	Spec No	PSI	Spec No	PSI	Spec No	PSI	Spec. No	PSI	Spec No	PSI
7 Day Cure and Soaking	D1	92	D1	215	D1	325	D1	477	D1	573
	D2	84	D2	240	D2	363	D2	504	D2	506
39 Day Cure and Soaking	E1	78	E1	273	E1	407	E1	608	E1	652
	E2	92	E2	271	E2	461			E2	678
After Freeze-Thaw Test and Soaking			A1	153	A1	261	A1	573	A1	660
			A2	160	A2	280	A2	528	A2	648
	B1		B1	156	B1	391	B1	533	B1	644
	B2		B2	179	B2	347	B2	533	B2	679
After Wet-Dry Test and Soaking	A1	*	A1	342	A1	347*	A1	551*		
	A2	*	A2	181*	A2	629	A2	492*		
	B1	85*	B1	202*	B1	400*	B1	655		
	B2	87*	B2	259	B2		B2	811		

SILT

COMPRESSIVE STRENGTH, PSI

	4% Cement		6% Cement		8% Cement		10% Cement		12% Cement		14% Cement	
	Spec No	PSI	Spec No	PSI	Spec No	PSI	Spec No	PSI	Spec No	PSI	Spec No	PSI
7 Day Cure and Soaking	D1	188	D1	310	D1	530	D1	693	D1	780	D1	1060
	D2	209	D2	327	D2	525	D2	640	D2	772	D2	948
39 Day Cure and Soaking	E1	294	E1	358	E1	822	E1	1082	E1	1280	E1	1392
	E2	210	E2	425	E2	771	E2	1051	E2	1345	E2	1562
After Freeze-Thaw Test and Soaking					A1	335	A1	952	A1	1208	A1	1350
					A2	227	A2	737	A2	1170	A2	1280
					B1		B1	682	B1	1050	B1	1197
					B2	377	B2	802	B2	1115	B2	1340
After Wet-Dry Test and Soaking	A1	97*	A1	405	A1	893	A1	1115				
	A2	81*	A2	479	A2	908	A2	1155				
	B1	245	B1	681	B1	877	B1	1130				
	B2	190	B2	621	B2	980	B2	1117				

*Specimen cracked into 2 or more pieces, repaired, if possible, for strength tests

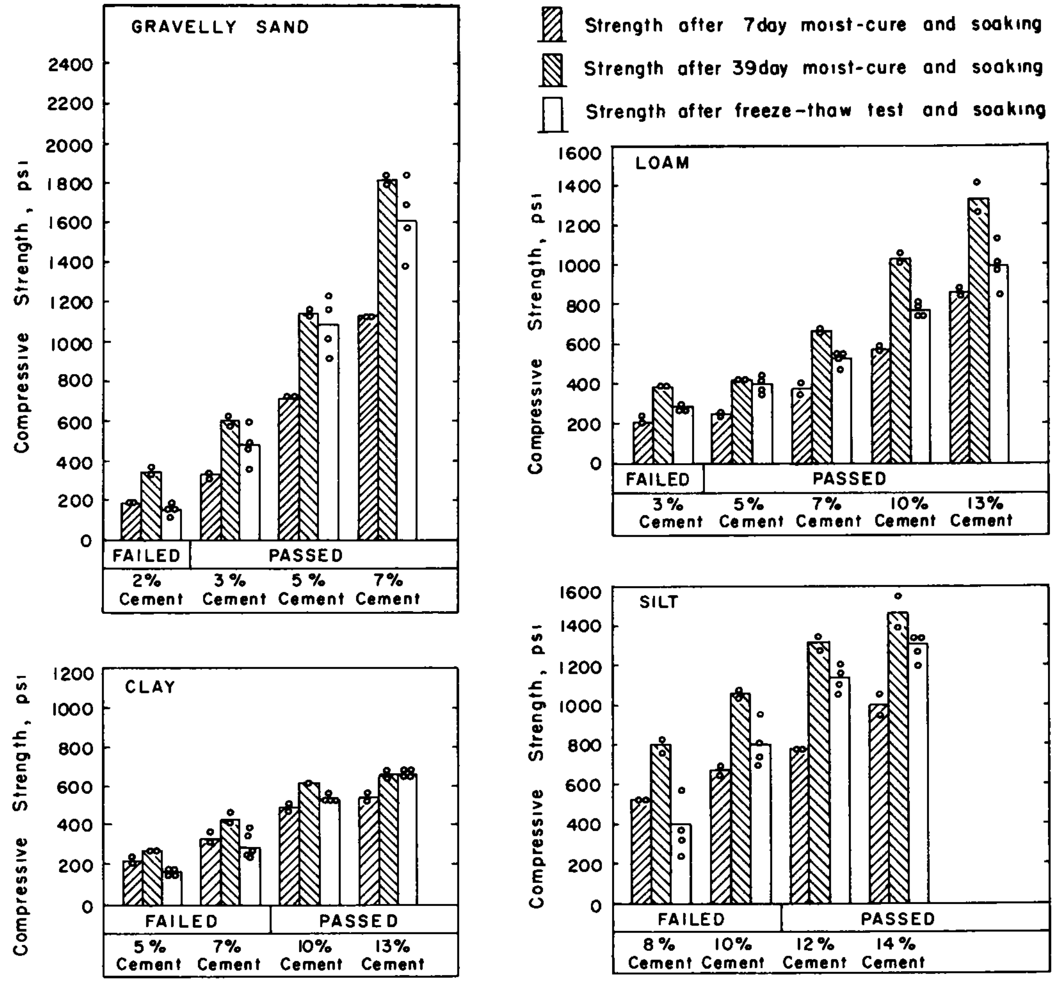

Figure 5. Compressive strengths after freeze-thaw test compared to 7 and 39 day strengths.

strengths-after-test are less than the 7-day strengths for two-thirds of the failed specimens.

Pulse Velocities

Pulse velocities were determined on brushed and unbrushed specimens after each thaw portion of the cycles and at the end of test and are given in Tables 13 through 16.

Figure 6 shows the pattern of pulse velocities through the test for the gravelly sand. This is the typical pattern for the four soils. Acceptable specimens showed increases or only slight decreases in velocity through the test. In general, velocities of inadequately hardened specimens decreased appreciably. The velocity data are subject to considerable variation on the same specimens at different cycles and between duplicate specimens on the same cycle.

Velocities-after-test for all four soils are shown in Figure 7 where they are compared to velocities of specimens moist-cured for 7 and 39 days. The velocity values for adequate freeze-thaw resistance vary considerably for the different soil types. Velocities at the lowest cement contents passing the freeze-thaw test vary, depending on soil type, from 7,000 to 8,500 fps at 7 days, from 7,000 to 11,000 fps at 39 days, and from 6,500 to 9,500 fps after the freeze-thaw test.

TABLE 13

PULSE VELOCITIES, GRAVELLY SAND

			Pulse Velocities of Freeze-Thaw Specimens, 100 Ft per Sec															
			2% Cement				3% Cement				5% Cement				7% Cement			
Specimen No			A1	A2	B1	B2	A1	A2	B1	B2	A1	A2	B1	B2	A1	A2	B1	B2
After Cure			82	82	82	80	83	82	98	89	98	101	101	104	104	93	120	107
Cycle No	2	After Thaw	66	80	89	96	87	87	96	91	123	116	123	120	123	120	127	116
	4		55	63	80	77	91	98	112	109	123	120	132	123	127	132	147	109
	6		56	56	67	75	77	78	98	96	87	87	116	116	123	109	120	124
	8		49	65	65	66	87	85	93	91	106	109	106	106	112	116	116	124
	10		50	64	68	64	92	92	104	101	112	112	123	127	120	120	137	107
	12		50	57	63	58	93	96	101	96	112	116	112	116	120	116	127	120
After Soak					68	65												132

		Pulse Velocities of Wet-Dry Specimens, 100 Ft per Sec											
		2% Cement				3% Cement				5% Cement			
Specimen No		A1	A2	B1	B2	A1	A2	B1	B2	A1	A2	B1	B2
After Cure		70	64	82	68	85	83	83	91	99	91	103	99
Cycle No	2 Wet	51	48	60	65	85	87	74	77	103	102	103	116
	2 Dry			44	48			73	66			87	96
	4 Wet	56	51	59	51	66	60	71	78	82	72	93	91
	4 Dry			44	43			65	67			80	91
	6 Wet	64	47	56	50	75	56	80	82	63	87	104	104
	6 Dry			53	45			68	71			85	87
	8 Wet	66	42	54	44	65	78	91	80	58	75	91	91
	8 Dry			53	49			83	75			101	99
	10 Wet	53	41	49	47	62	75	66	71	62	75	103	101
	10 Dry			38	45			66	67			87	87
	12 Wet	48	40	48	44	67	78	65	66	62	82	91	106
	12 Dry			44	45			67	72			87	91
After Soak		48	42		45	71	89	58	61	77	89	89	

	Pulse Velocities of Compressive Strength Specimens, 100 Ft per Sec															
	2% Cement				3% Cement				5% Cement				7% Cement			
Specimen No	D1	D2	E1	E2	D1	D2	E1	E2	D1	D2	E1	E2	D1	D2	E1	E2
7 Day Cure	72	63	68	66	77	77	71	74	107	98	85	80	109	107	85	87
and Soak	72	61			70	74			89	87			98	104		
39 Day Cure			101	96			98	113			107	113			116	116
and Soak			101	96			104	107			113	120			120	128

TABLE 14

PULSE VELOCITIES, LOAM

Pulse Velocities of Freeze-Thaw Specimens, 100 Ft per Sec

			3% Cement				5% Cement				7% Cement				10% Cement				13% Cement			
Specimen No.			A1	A2	B1	B2	A1	A2	B1	B2	A1	A2	B1	B2	A1	A2	B1	B2	A1	A2	B1	B2
After Cure			53	51	48	47	70	75	64	72	72	76	76	78	87	87	83	83	93	91	91	91
Cycle No	2	After Thaw	45	45	49	46	67	67	66	67	66	71	72	74	80	81	81	81	85	87	89	89
	4		49	50	46	40	75	75	64	65	72	76	78	78	82	85	72	87	98	98	96	96
	6		53	48	43	43	75	75	64	72	67	72	72	72	81	81	80	80	85	87	85	87
	8		48	48	49	44	66	68	65	72	66	74	74	74	83	83	83	81	89	89	89	89
	10		48	52	48	48	75	74	72	71	66	76	71	67	83	83	78	80	93	96	81	81
	12		48	51	48	54	83	66	48	68	70	72	70	74	83	85	80	81	89	89	89	91
After Soak			54	53	60	53	71	60	58	74	67	76	71	75	80	81	81	81	89	87	89	89

Pulse Velocities of Wet-Dry Specimens, 100 Ft per Sec

		3% Cement				5% Cement				7% Cement				10% Cement			
Specimen No		A1	A2	B1	B2	A1	A2	B1	B2	A1	A2	B1	B2	A1	A2	B1	B2
After Cure		66	61	57	57	74	78	74	72	82	82	78	78	85	96	83	83
Cycle No	2 Wet	39	39	40	39	56	56	56	48	67	56	56	65	72	82	66	66
	2 Dry	39	39	41	43	45	48	48	44	56	65	57	52	67	68	67	51
	4 Wet	42	42	41	43	49	56	57	57	65	57	65	57	68	61	68	68
	4 Dry	38	38	38	44	48	47	52	50	58	57	61	64	68	71	71	71
	6 Wet	42	42	42	43		51	55	53	66	64	62	61	70	70	68	68
	6 Dry	43	43	41	46		46	51	48	58	57	56	56	65	67	66	67
	8 Wet	47	47	41	42		50	56	56	62	58	64	66	70	72	70	72
	8 Dry	45	45	43	42		47	52	49	57	57	57	60	67	67	68	68
	10 Wet	43	43	39	39		51	51	51	56	58	60	61	67	70	72	71
	10 Dry	44	44	40	44		46	52		57	56	66	59	66	67	72	72
	12 Wet	35	35	41	42		48	56	56	63	63	65	65	75	75	80	80
	12 Dry	41	41	39	41		46	54	49	56	55	65	64	65	68	67	72
After Soak		25	24	28	33		49	57	56	58	58	58	62	68	70	68	67

Pulse Velocities of Compressive Strength Specimens, 100 Ft per Sec

	3% Cement				5% Cement				7% Cement				10% Cement				13% Cement			
Specimen No	D1	D2	E1	E2	D1	D2	E1	E2	D1	D2	E1	E2	D1	D2	E1	E2	D1	D2	E1	E2
7 Day Cure	58	61	61	64	61	70	70	68	71	77	74	70	80	80	85	85	83	82	85	87
and Soak	61	64			62	70			71	75			78	75			82	83		
39 Day Cure			58	58			67	71			75	75			80	80			85	96
and Soak			56	64			65	70			74	72			78	77			83	91

TABLE 15

PULSE VELOCITIES, CLAY

Pulse Velocities of Freeze-Thaw Specimens, 100 Ft per Sec

		3% Cement		5% Cement				7% Cement				10% Cement				13% Cement			
Specimen No		B1	B2	A1	A2	B1	B2	A1	A2	B1	B2	A1	A2	B1	B2	A1	A2	B1	B2
After Cure		40	36	64	65	60	60	68	70	70	70	71	71	68	70	71	72	68	68
Cycle No After Thaw	2	39	36	54	55	53	55	61	55	58	60	66	66	65	64	66	66	63	66
	4	40	35	48	43	53	54	57	50	51	61	71	71	66	70	70	70	66	66
	6	32	32	41	37	46	52	53	44	50	56	68	66	65	62	65	66	62	61
	8	27	26	38	29		48	51	35	51	56	68	66	64	64	68	67	63	64
	10			31	31	47	53	43	32	50	55	66	72	61	58	66	68	60	62
	12			30	33	35	39	45	39	58	57	67	68	64	65	66	68	64	62
After Soak				31	31	36	34	44	38	48	48	66	66	65	66	66	63	64	63

Pulse Velocities of Wet-Dry Specimens, 100 Ft per Sec

		3% Cement				5% Cement				7% Cement				10% Cement			
Specimen No		A1	A2	B1	B2	A1	A2	B1	B2	A1	A2	B1	B2	A1	A2	B1	B2
After Cure		53	45			65	65	61	61	68	68	67	67	72	71	70	60
Cycle No	2 Wet	44	29	31	27	43	43	27	25	43	50	31		57	56	55	48
	2 Dry	32	43	37	36	47	47	43	32	42	47	43		64	63	56	48
	4 Wet			29	31	44	39	35	31	44	44	42		56	56	56	48
	4 Dry			35	35	43	44	22	27	44	51	32		58	51	48	44
	6 Wet			27	31	44	44	35	27	47	47	32		47	55	49	48
	6 Dry			33	33	48	43	39	41	43	43	34		48	51	53	48
	8 Wet			27	35	44	30	17	25		44	27		51	50	49	28
	8 Dry			31	31	47	33	27	33		32	44		51	48	51	51
	10 Wet			30	33	43	30	33	24		44	22		50	50	47	39
	10 Dry			28	29	46	44	43	28		39	44		56	43	48	43
	12 Wet			28	29	48	33		25		43	32		56		47	43
	12 Dry			29	29	48	31		28		39	29		30		44	
After Soak						42	21		22		38					42	

Pulse Velocities of Compressive Strength Specimens, 100 Ft per Sec

	3% Cement				5% Cement				7% Cement				10% Cement			13% Cement			
Specimen No	D1	D2	E1	E2	D1	D2	E1	E2	D1	D2	E1	E2	D1	D2	E1	D1	D2	E1	E2
7 Day Cure	49	48	47	46	62	65	61	60	67	62	65	63	70	68	62	68	67	70	72
and Soak	51	51			64	65			68	62			70	68		68	67		
39 Day Cure			36	40			65	64			67	70			70			72	74
and Soak			35	35			65	65			65	67			70			72	

TABLE 16

PULSE VELOCITIES, SILT

Pulse Velocities of Freeze-Thaw Specimens 100 Ft. per Sec.

		8% Cement				10% Cement				12% Cement				14% Cement			
Specimen No.		A1	A2	B1	B2	A1	A2	B1	B2	A1	A2	B1	B2	A1	A2	B1	B2
After Cure		68	68	67	67	80	74	74	75	82	83	80	77	85	83	78	83
Cycle No. After Thaw	2	55	64	65	65	70	71	72	72	80	80	77	77	89	89	85	91
	4	63	60	66	66	74	75	77	77	78	82	80	80	85	85	85	85
	6	64	62	61	65	77	75	75	74	82	82	78	77	89	89	83	91
	8	65	67	74	66	80	82	87	80	91	89	85	85	85	87	85	83
	10	65	54	55		82	66	74	72	87	82	76	77	89	89	87	87
	12	62	57	63	68	80	68	67	70	87	87	76	70	101	91	87	91
After Soak		59		65	75	77	75	91	82	89	96	89	80	96	93	89	91

Pulse Velocities of Wet-Dry Specimens, 100 Ft. per Sec.

		4% Cement				6% Cement				8% Cement				10% Cement			
Specimen No		A1	A2	B1	B2	A1	A2	B1	B2	A1	A2	B1	B2	A1	A2	B1	B2
After Cure		47	44	45	46	55	56	56	56	63	63	61	64	68	66	68	67
Cycle No.	2 Wet	43	43	43	43	49	50	51	53	56	58	56	57	65	65	62	62
	2 Dry	44	48	54	51	55	60	53	56	65	63	63	63	76	63	68	68
	4 Wet	47	50	39	43	56	56	55	55	62	58	64	60	66	64	65	66
	4 Dry	48	48	29	47	51	53	56	58	65	57	59	55	66	63	63	66
	6 Wet	47	44	43	45	55	55	55	57	62	63	67	62	66	64	68	66
	6 Dry	47	51	46	47	56	56	64	65	59	56	66	66	65	71	74	73
	8 Wet	54	53	43	49	63	63	60	67	66	67	68	66	71	69	74	74
	8 Dry	53	51	31	51	56	58	61	59	61	62	69	69	71	70	71	72
	10 Wet	54	55	48	48	58	61	59	63	67	65	69	65	70	66	67	70
	10 Dry	55	50	34	46	56	58	66	66	66	66	69	66	71	70	82	80
	12 Wet	65	65	51	45	64	65	60	67	67	67	67	67	76	73	71	75
	12 Dry	51	52	50	51	66	65	67	66	68	78	71	69	76	68	77	83
After Soak				49		66	66	65	58	70	71	72	73	80	75	80	78

Pulse Velocities of Compressive Strength Specimens, 100 Ft. per Sec.

	4% Cement				6% Cement				8% Cement				10% Cement			
Specimen No.	D1	D2	E1	E2	D1	D2	E1	E2	D1	D2	E1	E2	D1	D2	E1	E2
7 Day Cure	44	46	50	47	56	56	58	57	62	62	67	67	67	67	71	75
and Soak	47	47			56	56			64	62			70	66		
39 Day Cure			60	58			62	62			72	66			75	77
and Soak			53				67	67			80	77			85	89

	12% Cement				14% Cement			
Specimen No.	D1	D2	E1	E2	D1	D2	E1	E2
7 Day Cure	74	72	78	78	85	87	80	83
and Soak	74	70			91	85		
39 Day Cure			80	85			93	98
and Soak			96	98			91	98

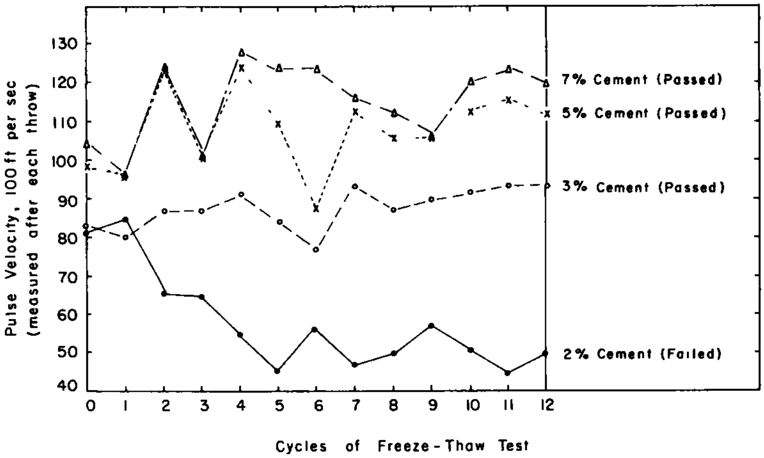

Figure 6. Pulse velocities during freeze-thaw test, gravelly sand.

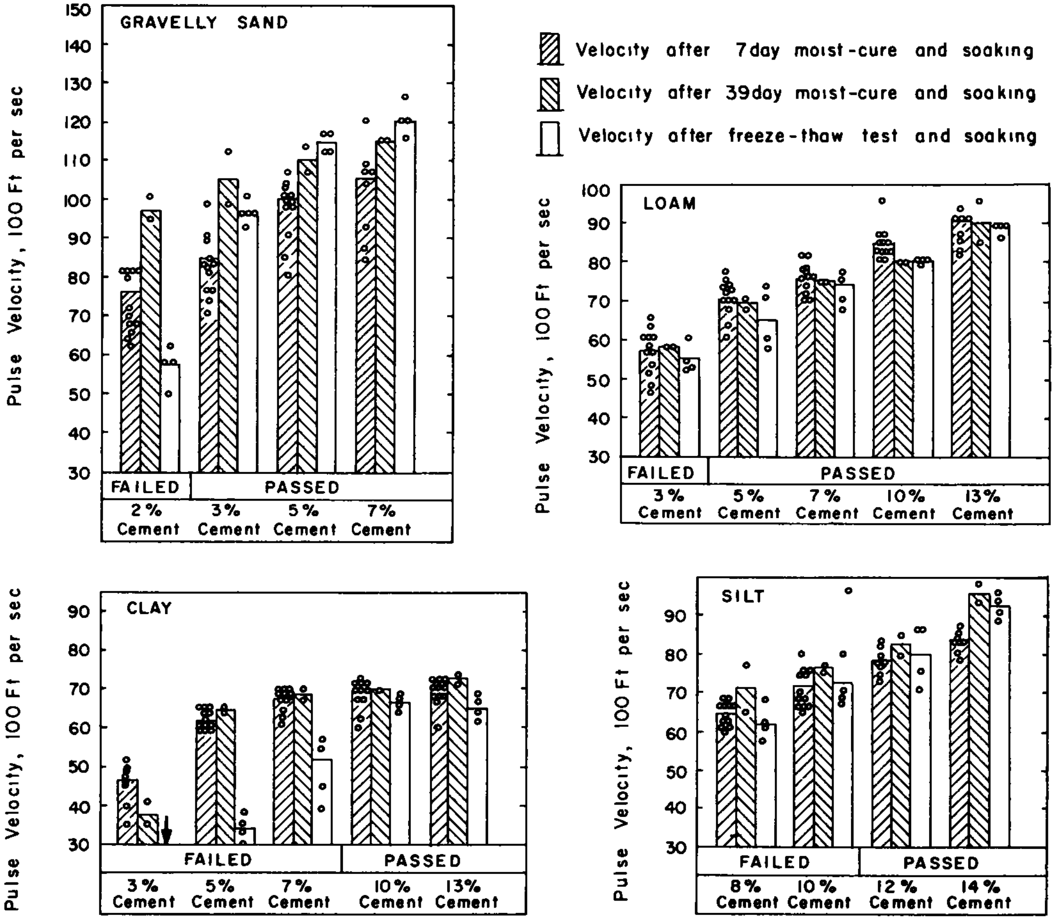

Figure 7. Pulse velocities after freeze-thaw test compared to 7 and 39 day velocities.

RESULTS OF WET-DRY TESTS

Weight Losses, Moisture Changes, Condition of Specimens

Weights of brushed and unbrushed specimens were determined after each wet and each dry period and at the end of test. These are given in Tables 3 through 6 along with computed weight losses, moisture changes, and condition of specimens during the wet-dry test.

Moisture contents for soil-cement specimens of the gravelly sand and the loam ranged from the molded moisture content on wetting to about 2 percent moisture on drying. The clay specimens had moisture contents on wetting that progressively, but slightly, reduced during the test to values of 1 to 3 percent below the molded moisture content at the end of test. The moisture content of the clay soil-cement specimens on each drying was from 2 to 6 percent. Specimens of the silt, on wetting, gained about 4 percent moisture above the molded moisture content and dried on each cycle to 2 percent moisture. There was no appreciable difference in the moisture changes between failed and acceptable specimens.

During the wet-dry test, no cracking occurred on specimens of the gravelly sand. For the loam, one specimen of the four tested at 3 percent cement cracked. For the clay, all four specimens at 5, 7, and 10 percent cement cracked. For the silt, two of four specimens at 4 percent cement cracked. Here, the term cracking refers to separation of the specimens into two or three separate pieces.

It has been the author's experience that cracking of specimens in the wet-dry test is not necessarily a sign of a deterioration that would occur under natural conditions. The cracking of the acceptable specimens of the clay soil is considered to be a laboratory phenomenon that does not represent conditions in a soil-cement pavement. However, if high weight losses and poor conditions of specimens accompany the cracking, the deterioration is considered to be related to natural conditions.

Based on recommended weight loss criteria, adequate wet-dry resistance was obtained for specimens containing the cement contents given in Table 17. Specimens at these and higher cement contents passed the test; specimens at lower cement contents failed. This is the basis of judgment for evaluating length change, compressive strength, and pulse velocity methods for measuring wet-dry resistance.

TABLE 17

CEMENT CONTENTS OF SOIL SAMPLES FOR WET-DRY TESTS

Sample	Lowest Cement Content Meeting Weight Loss Criteria (% by wt.)
Gravelly sand	3
Loam	3 or less
Clay	5
Silt	6

Length Changes

Lengths of unbrushed specimens were determined after cure, after each wet and dry period, and after test. Length changes from the 7-day moist-cured length are given in Tables 8 through 11.

Figure 8 shows the length changes during the wet-dry test for the four soils. Shrinkages on drying vary from -0.0025 to -0.0500 in. depending mostly on the clay content of the soils. All specimens exhibited a shrinkage on the first drying that was never fully recovered except on deteriorated specimens. The amount of length change on subsequent wetting and drying ranged from 0.0012 in. for low clay content soils to a maximum of 0.0100 in. for the high clay content soil.

It is indicated that acceptable specimens will not expand back, after first drying, to the original length whereas failing specimens tend to return to the original length. However, data in this respect are limited and may not warrant this as a definite conclusion.

Due to loss of reference points in the first trials of length measures, there is not a full set of length data. Loss of reference points occurred frequently in the wet-dry

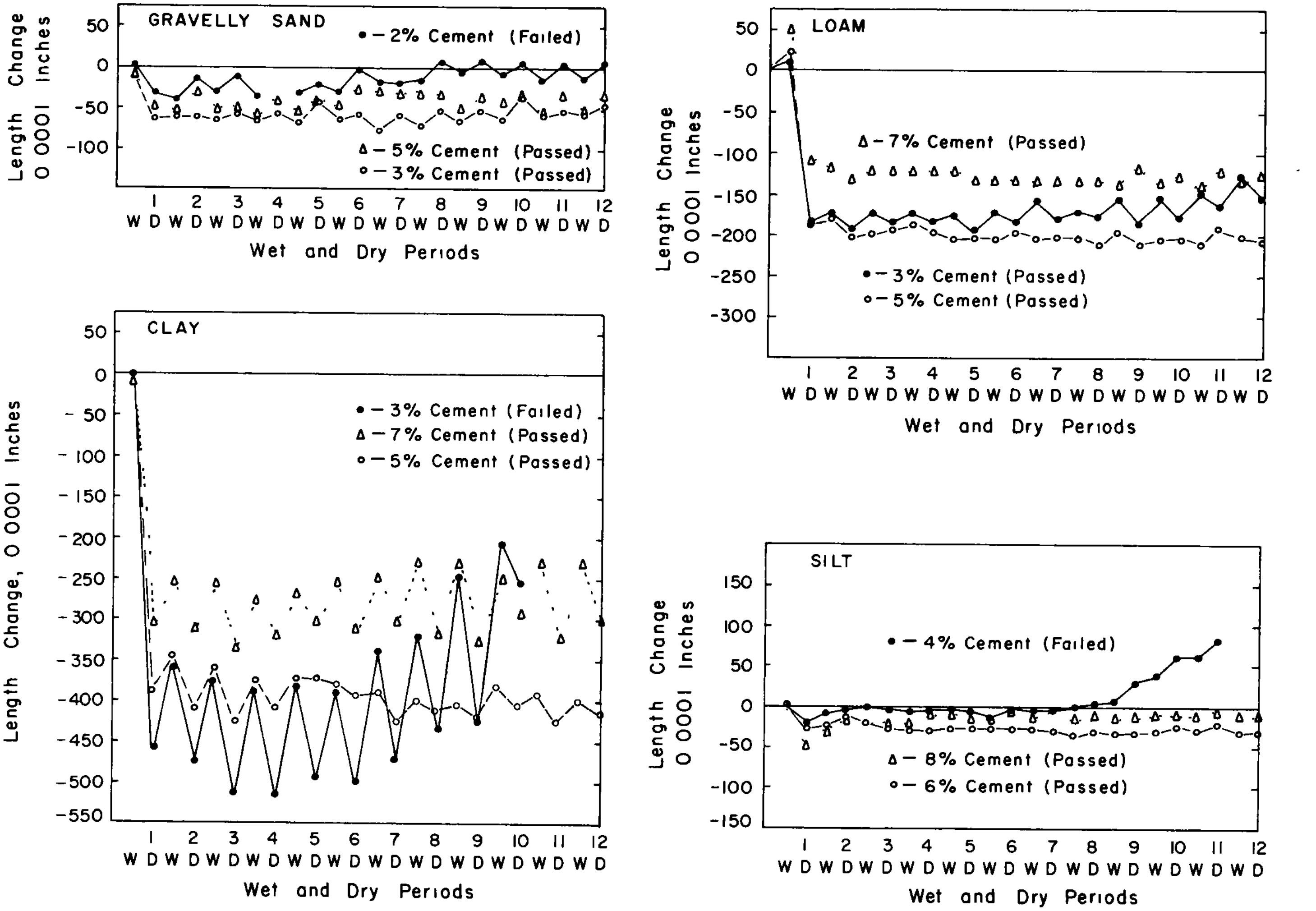

Figure 8. Length changes during wet-dry test.

test before the technique for attaching the points was fully developed. Additional development greatly reduced the loss of points. In most cases where original reference points were lost, relative length measurements were continued with new reference points; however, absolute lengths could no longer be determined. For those specimens on which a full set of data were obtained, the lengths of failing specimens showed an upward trend at some time after first drying.

Figure 9 shows the length changes at the end of test. It can be seen that the lengths at end of test, in themselves, are not as significant in measuring deterioration as the trend of the length changes during the test.

For the low clay content soils (gravelly sand and silt), after first drying, shrinkage occurred on wetting and expansion occurred on subsequent dryings, contrary to the expected reaction. This led to the hypothesis that, on drying, the small drying shrinkage of these low volume change soils was counteracted by thermal expansion at the 160 F drying temperature. An additional experiment on a few dry specimens indicated a reversible thermal length change of 0.0025 in. on heating and cooling through several cycles between 75 and 160 F. It is hypothesized that a reversible thermal expansion may overcome an irreversible (after first drying) shrinkage on drying if the drying shrinkage is not of sufficient magnitude.

This hypothesis is shown in Figure 10 with the data for the gravelly sand. If this concept is validated by additional tests, it might be appropriate to replace the high-temperature drying in the wet-dry test by drying at a lower temperature so that the

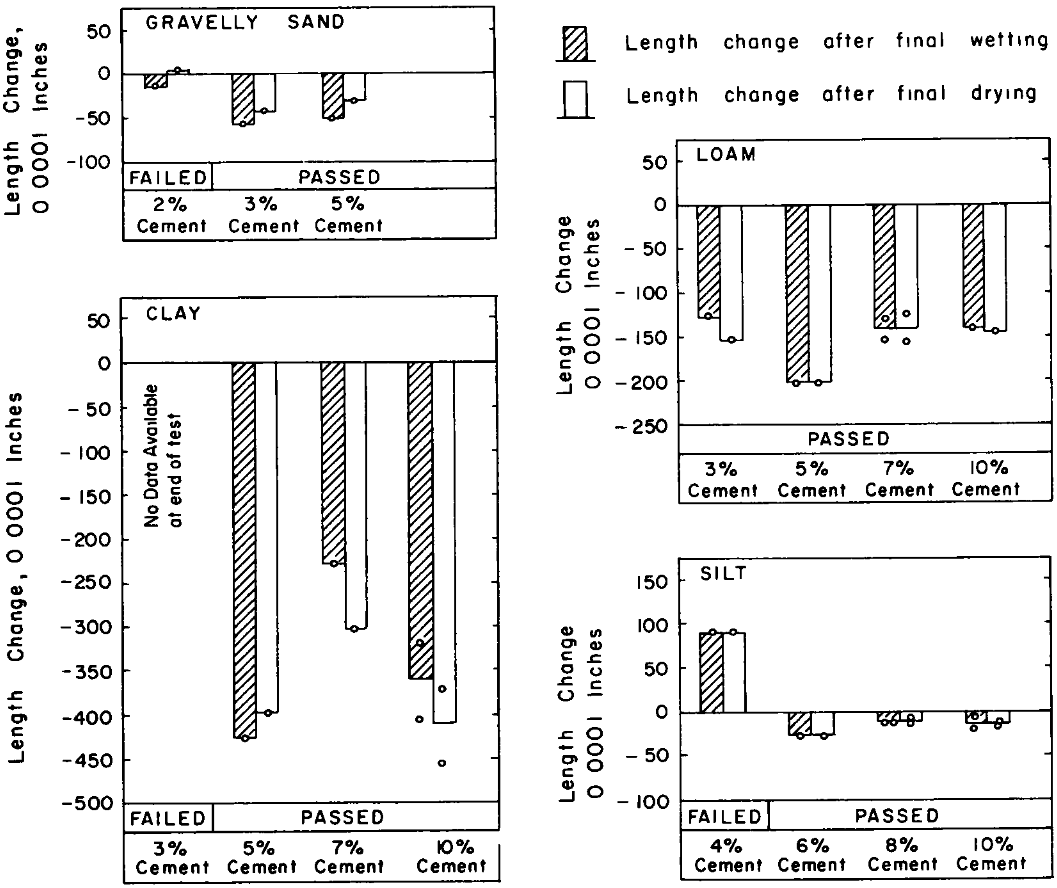

Figure 9. Length changes at the end of wet-dry test.

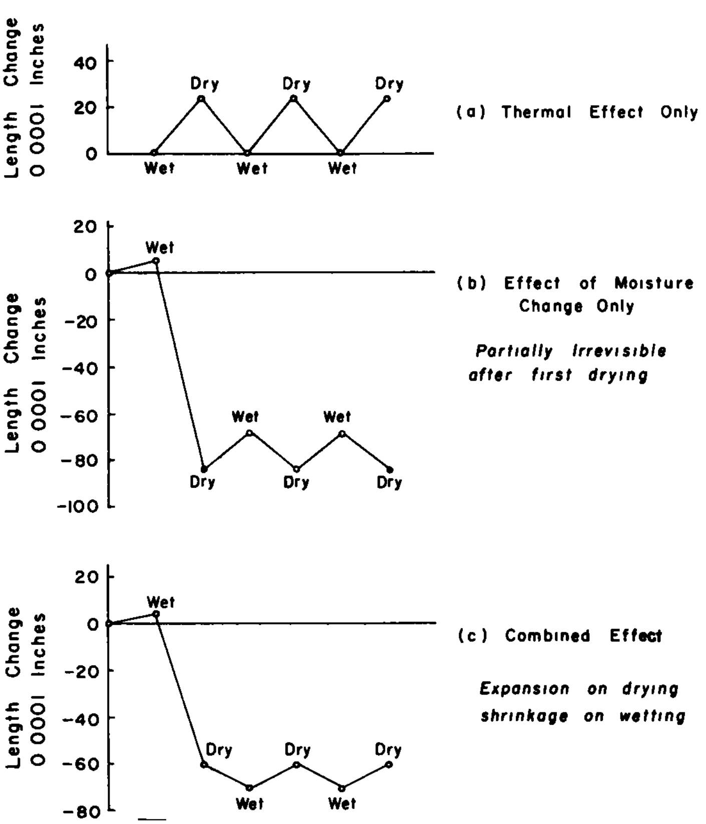

Figure 10. Effects of changes in temperature and moisture content on length of specimens in wet-dry test, gravelly sand.

volume changes due to wetting and drying are not canceled or reduced by thermal effects.

For the gravelly sand the volume changes due to wetting and drying were completely reversed by thermal effects. For the silt, the volume changes were canceled by thermal effects.

Compressive Strengths

Compressive strengths determined on brushed and unbrushed specimens after the wet-dry test are given in Table 12. The strengths are shown in Figure 11 where they are compared with 7- and 39-day strengths.

In general, failed specimens had strengths-after-test less than the 39-day strength. The strengths of acceptable specimens were close to or usually considerably above the 39-day strength. Two exceptions indicate that some minimum strength value after test might have to be included in criteria that may be developed.

In all cases strengths after the wet-dry test of acceptable specimens are greater than the 7-day strengths and, in most cases, are greater than the 39-day strengths. This indicates that there is an acceleration of strength during the 160 F drying cycles. This greater strength gain occurs at the higher temperature even though there is not as much water available as there is in the continuous moist-cure environment at room temperature. It is felt that this high-temperature strength gain may be unduly beneficial to specimens that would not otherwise withstand wetting and drying volume changes at lower temperatures.

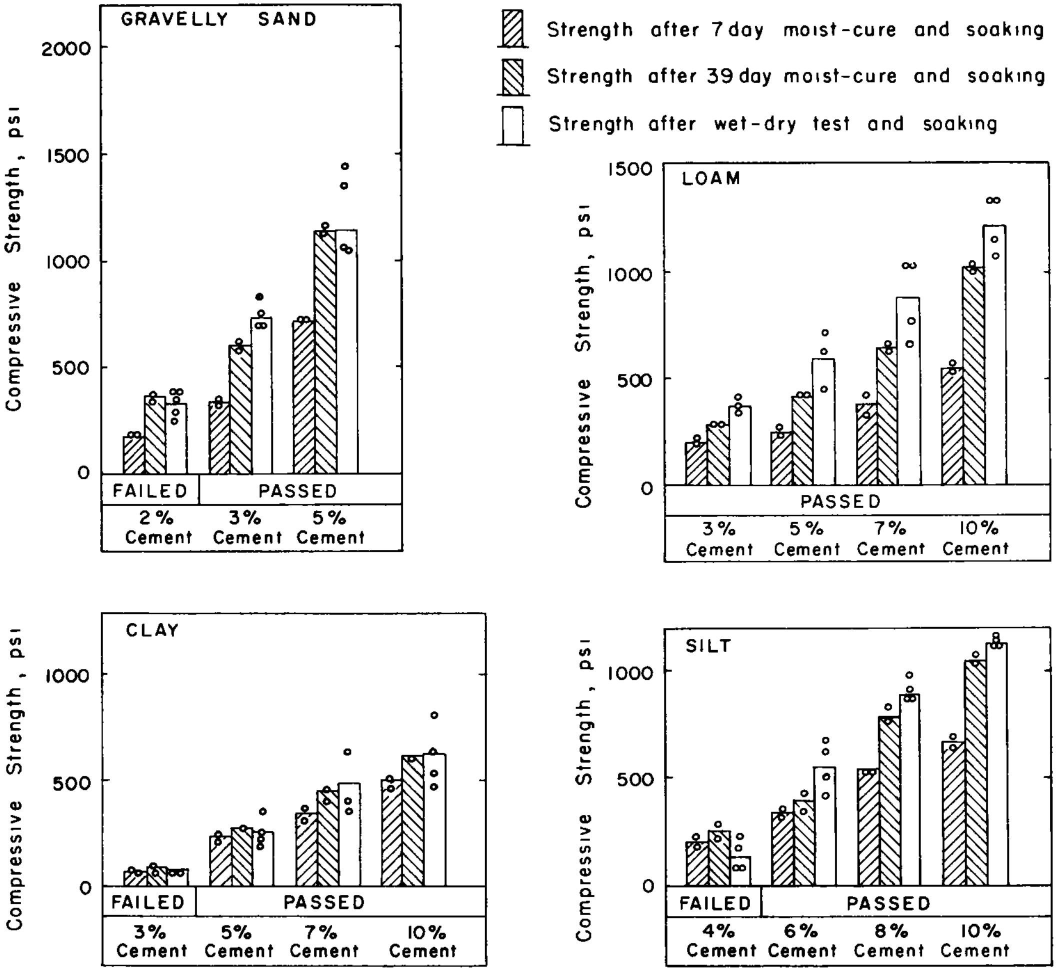

Figure 11. Compressive strengths after wet-dry test compared to 7 and 39 day strengths.

Pulse Velocities

Pulse velocities were determined on brushed and unbrushed specimens after cure, after each wet and each dry period and at the end of test. These are given in Tables 13 through 16.

Figure 12 shows the pulse velocities-after-test compared to velocities at 7 and 39 days. For the silt only, the pulse velocities-after-test were greater than the 39-day values, the same trend that was established for compressive strengths. Contrary to the trend of compressive strength, the pulse velocities after test for the gravelly sand, loam, and clay were considerably less than the 7-day values at all cement contents.

For freeze-thaw and for 7- and 39-day moist-cured specimens, the ratio of velocity to strength was roughly similar. For wet-dry specimens, a much lower ratio of velocity to strength was obtained. Some type of structural change must be indicated by these reduced velocities on three soils but the relation to the condition of the specimens is not understood. Therefore, further analysis of pulse velocity measurements in the wet-dry test is not attempted.

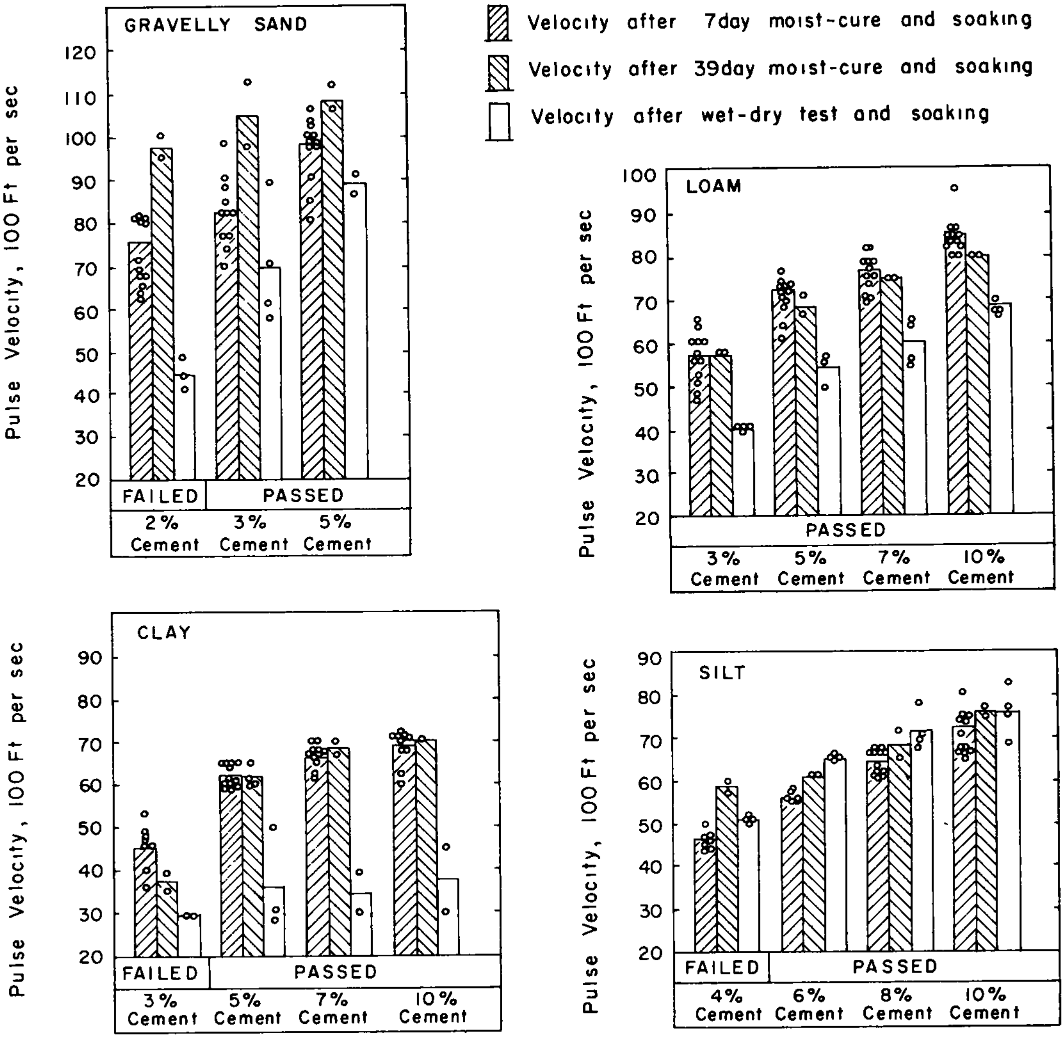

Figure 12. Pulse velocities after wet-dry test compared to 7 and 39 day velocities.

ANALYSIS OF RESULTS

It must be emphasized that suggestions of tentative criteria for the alternate methods of measuring freeze-thaw and wet-dry resistance are beyond the purpose of this study and are not warranted by the data. The purpose, in this repect, is to determine if criteria may be developed after investigating a large number of soils.

Freeze-Thaw Test

A correlation of all the measures is shown in Figure 13. The alternate measures shown in Figures 13a through 13f are compared with condition of specimens (Fig. 13g) and weight losses, (Fig. 13h). All of the measures can be used to detect deterioration. From this figure it is possible to evaluate quickly the ability of the alternate methods in separating failed specimens from acceptable specimens.

It is also possible to get an idea of what values of the alternate measures indicate deterioration on the four soils studied. For example, in Figure 13a, the maximum expansion on thaw of acceptable specimens for all four soils was 0.0030 in., whereas the minimum expansion on thaw of all failed specimens was 0.0080 in.

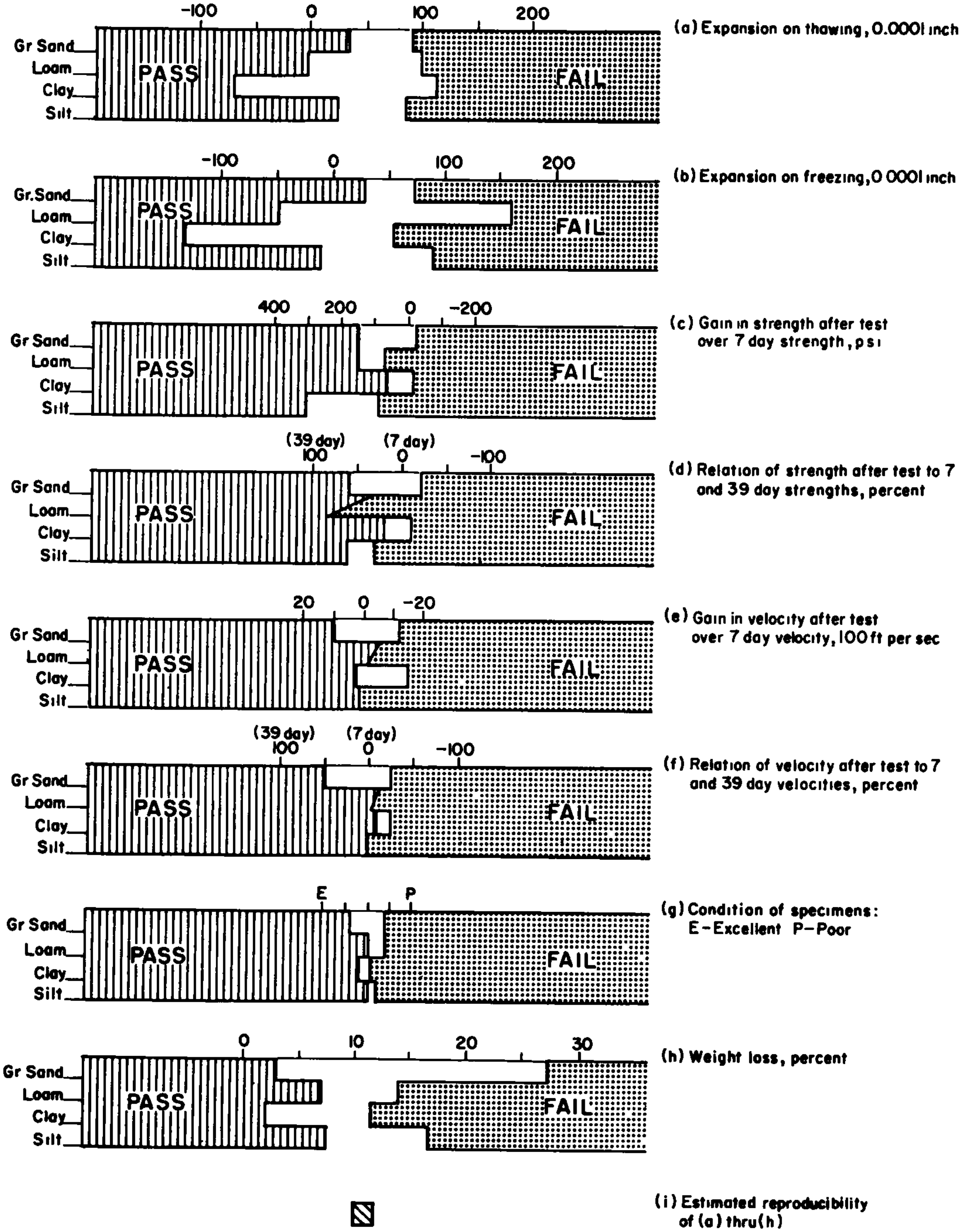

Figure 13. Correlation of length, strength and velocity measurements with weight losses and condition of specimens in freeze-thaw test.

The suitability of each measure in detecting deterioration is indicated by the width of the gap between the extreme value of passing and the extreme value of failing, the wider gap indicating more promise of developing criteria. A desirable feature is that the gap occur at the same values for the four soils. However, this is not completely necessary, because if a wide gap exists for each soil it may be possible to develop a separate criteria for each soil type.

Another consideration in evaluating the different measures is that the separation of values for acceptable and failed specimens must be, for each soil, greater than the reproducibility of the method. All the measures in Figure 13 are plotted to the same scale of estimate reproducibility. An idea of the relative reproducibility of each measure was obtained by observing the range of variations for each measure on duplicate specimens at the lowest cement contents passing the test. This scale of reproducibility is shown in Figure 13i. Most of the variations are less than that represented by this bar, and on further development of techniques, it is possible that the variations can be further reduced.

An inspection of Figure 13 enables evaluation of the alternate methods based on the four soils studied. The methods are rated from the standpoint of sensitivity, reproducibility, and prospects that criteria can be developed as follows: (a) length change, (b) compressive strength, and (c) pulse velocity.

The length change method appears to be the most worthy of further investigation on a large number of soils. As shown in Figures 13a and 13b expansion values clearly define failed and acceptable specimens.

The method appears to have immediate sensitivity to deterioration. It has other desirable features of ease of operation, accuracy, low cost of equipment, and lack of dependence on a great amount of procedural control.

Consideration of soil type in establishing criteria does not seem necessary but might further enhance accuracy. Figures 13a and 13b show that the gap shifts to the left with increasing clay contents.

Compressive strength testing also appears worthy of further study. As shown in Figures 13c and 13d, strength values separating failed and acceptable specimens are clearly defined for three soils, whereas there is an overlap of values for one soil. The overlap occurs for the loam soil at 3 percent cement where the strength-after-test was a comparatively low value (less than 300 psi). Acceptable specimens for all four soils had strengths-after-test considerably above 300 psi. Therefore, it appears that criteria might appropriately include a minimum strength-after-test value. This would eliminate the overlap shown in Figures 13c and 13d.

In the development of strength criteria it may be important to relate strength-after-test to both of the control strengths. The expected gain over the 7-day strength should be related to the potential gain indicated by the 39-day strength.

A desirable feature of the compressive strength method is that accuracy does not depend on a great amount of procedural control. However, variations in strengths-after-test indicate a need for testing replicate specimens. The method is destructive and can be used only at the end of test.

Pulse velocity values are also related to the quality of the specimens. As shown in Figures 13e and 13f,the velocity values separating failed and passed specimens are clearly defined for only two soils. For the other two soils the values abut or slightly overlap indicating that it may be more difficult to establish criteria. A need for testing replicate specimens is also indicated by variations in pulse velocity measurements. The test is nondestructive and easy to perform but the cost of pulse velocity equipment is high.

Wet-Dry Test

A correlation of length change, compressive strength, and pulse velocity measurements with weight losses and condition of specimens is shown in Figure 14.

All of the measures in the figure are plotted to the same scale of estimate reproducibility. Thus, it is possible to compare the suitability of the measures with the weight loss method. The alternate measures are rated from the standpoint of sensitivity, reproducibility, and prospects that criteria can be developed as follows: (a) length change, (b) compressive strength, and (c) pulse velocity. Length change and compressive strength measures both appear worthy of further study.

Figures 14a and 14b show length change measures seem to be more suitable than the compressive strength measures shown in Figure 14c. In addition, the length change method incorporates most of the previously discussed desirable features.

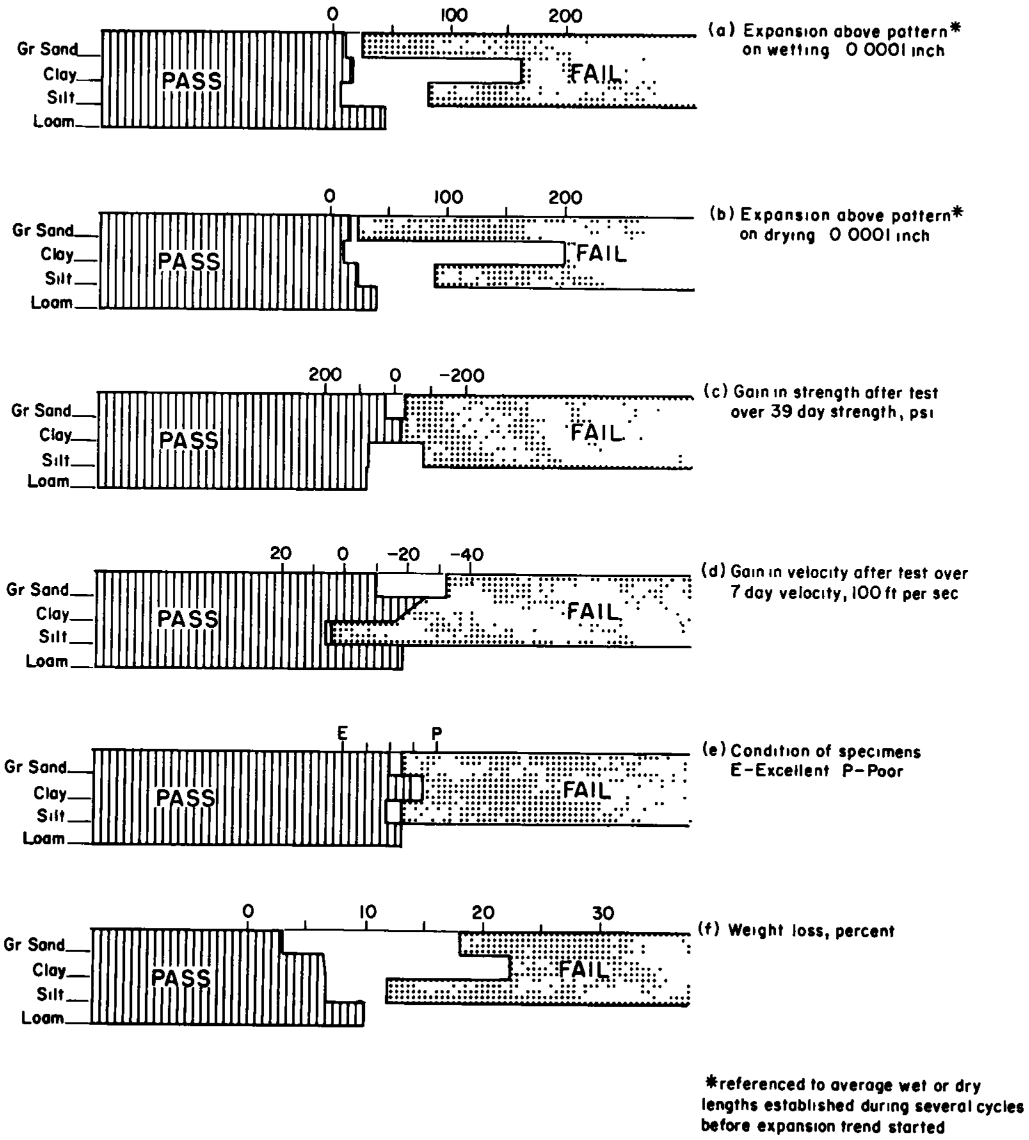

Figure 14. Correlation of length, strength and velocity measurements with weight losses and condition of specimens in wet-dry test.

It is indicated that criteria for length change may depend on soil type because the shrinkage on first drying depends greatly on the clay content.

Compressive strength values that indicate deterioration seem to be directly related to the 39-day control strength, as shown in Figure 14c. Variations in strength values indicate a need for testing replicate specimens.

Perhaps the information gained concerning the mechanics of the wet-dry test is more important, at this time, than an evaluation or correlation of the alternate measures. It is definitely indicated that the drying temperature used in the test is sufficiently high to cause an acceleration of strength gain which does not occur at lower temperatures. Also, it is hypothesized from the data, that drying at 160 F produces thermal expansions that may significantly counteract the wetting-drying volume changes.

SUMMARY

Exploratory data have been presented that indicate that the alternate measures—length change, compressive strength and pulse velocity—are valuable laboratory tools for studying the effects of freezing, thawing, wetting, and drying on soil-cement mixtures. A better understanding of these effects is helpful to the development or improvement of soil-cement test methods and criteria.

In addition, these data indicate that length change and compressive strength measurements show definite possibilities of development into desirable alternate methods.

The principal requirement for any method of selecting a cement content for soil-cement construction is that the method and criteria be related to the performance of pavements in service. This relationship is well established for standard test methods used with corresponding criteria established by the Portland Cement Association. Before other methods or variations of the standard methods can be considered reliable for all soil types, an investigation of a great number and variety of soils must be conducted and the relationships to field performance firmly established, either directly or by correlation with standard procedures.

Studies will be conducted to investigate further the effects of freezing, thawing, wetting, and drying on soil-cement specimens and to develop criteria for the promising alternate methods.

REFERENCES

1. Whitehurst, E.A., "Pulse-Velocity Techniques and Equipment for Testing Concrete." HRB Proc., 33:226-242 (1954).
2. "Soil-Cement Laboratory Handbook." 5th ed., Portland Cement Assoc., (1959).

Moisture-Density, Moisture-Strength and Compaction Characteristics of Cement-Treated Soil Mixtures

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A laboratory investigation was conducted to find relationships between strength and density for cement-treated soil mixtures compacted at different moisture contents. A dune sand and three clays were used to prepare sand-clay mixtures having different amounts and dominant kinds of clay minerals. Test specimens of each cement-treated mixture were molded to near standard and modified Proctor density, moist cured 7 or 28 days, and then immersed in water for 24 hour before being tested for unconfined compressive strength.

Test results show that the optimum moisture content for maximum density and the optimum moisture content for maximum unconfined compressive strength of cement-treated sand-clay mixtures are not necessarily the same. The moisture contents for maximum strengths are to the dry side for sand-clay mixtures dominant in sand, and to the wet side for sand-clay mixtures dominant in clay. As clay content increases, the optimum moixture content for both maximum density and maximum strength values decrease. Also presented are the increase in density and in strength for the different soils when the compaction effort is increased from standard to modified. The influence of different kinds of clay minerals on the relationships studied does not appear to be significant.

• THE COMPACTING MOISTURE CONTENT recommended in soil stabilization with cement is the one that gives a maximum density. A standard method used to find the optimum moisture content of soil-cement mixtures is that given in ASTM Designation D 558-57 and also in AASHO Designation T 134-57.

The optimum moisture for maximum density has sometimes been found not to be the same as the optimum moisture for maximum strength. For instance, Felt (1) reported that compressive strength increases to a maximum at slightly less than optimum moisture for maximum density for a sandy soil and a silty soil, and at greater than optimum for a clay soil.

This investigation was conducted to find, primarily, the relation between the moisture-density and the moisture-strength relationships in several basic soils or their mixtures. A sand and three clay soils (a kaolinitic, an illitic, and a montmorillonitic), alone or in sand and clay combinations were treated with different amounts of cement.

The soil-cement mixtures were compacted with the standard AASHO compaction and a few selected mixtures also with the modified AASHO compaction.

MATERIALS

Four soil samples were used: a dune sand, two commercially produced clays, and a highly plastic clay soil. The dune sand is clean, uniformly graded, and found in the Iowa drift plain in east central Iowa. The kaolinite clay, a naturally occurring Kaolin clay, is hydraulically mined and water processed from a deposit located in north central Florida by the Edgar Plastic Kaolin Company of Edgar, Fla. The illite clay, very rich in illite, is produced by the Illinois Clay Products Company of Chicago, Ill. The montmorillonite clay is Kansan gumbotil; in Iowa this gumbotil is believed to be a highly weathered fossil B horizon developed on Kansan glacial till. The physical and chemical properties of these soils are given in Table 1.

TABLE 1

CHARACTERISTICS OF SOILS USED

Property	Dune Sand	Kaolinite Clay	Illite Clay	Montmorillonite Clay
Lab Desig.	S-6-2	Kaolin	AR-9	528-8
Sampling location	Benton Co., Iowa	North central Fla.	Goose Lake region, Ill.	Keokuk Co., Iowa
Horizon	C	Probably C	Probably C	Fossil B
Sampling depth (in.)	18-132	Unknown	Unknown	78-102
Mechanical analysis (%):				
Sand (2-0.074 mm)	94.4	0.0	6.4	17.5
Silt (74-5μ)	1.6	18.0	18.6	8.5
Clay (<5μ)	4.0	82.0	75.0	74.0
Clay (<2μ)	3.8	60.0	59.3	71.0
Physical:				
Liquid limit (%)	19.0	51.0	54.8	87.1
Plastic limit (%)	NP	29.0	27.1	34.5
Plasticity index	NP	22.0	27.7	52.6
Std. Proctor density (pcf)	110.0	93.0	100.0	95.0
Opt. moist. content (%)	12.3	27.0	22.0	28.0
Spec. gravity	2.64	2.66	ND	ND
Chemical:				
CEC (No. 10 sieve) (meq/100 g)	-	ND	19.1	ND
CEC (No. 40 sieve) (meq/100 g)	-	ND	ND	45.3
pH	6.50	5.5	5.5	6.5
Carbonates (%)	0.02	ND	1.9	0.8
Organic matter (%)	0.04	ND	1.5	0.2
Predom. clay mineral	Montmorillonite and illite interlayer	Kaolinite	Illite	Montmorillonite
Classification				
Textural	Sand	Clay	Clay	Clay
AASHO	A-3(0)	A-7-6(16)	A-7-6(18)	A-7-5(20)
Unified	SP-SM	CH	CH	CH

Type I normal portland cement, obtained from the Penn-Dixie Cement Corporation of Des Moines, Iowa, was used in this investigation. This type of cement was selected because it is commonly used in soil-cement construction. The water used in all mixtures and tests was distilled water.

LABORATORY PROCEDURES

Proportioning of Mixtures

To compare the relative effects of various sand to clay ratios and the three kinds of clay minerals on optimum moisture contents, maximum density, and maximum 7- and 28-day strengths, the dune sand was used in the preparation of the sand-clay mixtures given in Table 2 with each of the clays.

These mixtures were molded at the standard AASHO compaction and those of 100 percent sand or clay and 75 percent sand plus 25 percent clay were also molded with the modified AASHO compaction.

The cement contents selected for study with each different combination of sand and clay were 8, 12, and 16 percent, dry weight of soil basis; some of the mixtures made with montmorillonitic clay were also treated with 20 percent cement. These cement contents bracketed the minimum cement requirements for soil-cement for each mixture. Each mixture was prepared at four or five different moisture contents, varying from dry to very wet.

Preparation of Mixtures

Predetermined amounts of air dry sand and clay passing the No. 10 sieve were weighed and dry mixed by hand (trowel); this step was not necessary in 0 percent sand or 0 percent clay batches. Next, the desired amount of cement was added to the soil and dry mixed by hand. Then the desired amount of distilled water was added and wet mixing was done in a modified Hobart kitchen mixer, Model C-100 at low speed for 1.5 min (4). Finally the mixing bowl was scrapped and the mixture hand mixed for about 30 sec to break up any clods that might have formed.

Molding and Testing of Specimens

Test specimens 2 in. in diameter by 2 in. in height were molded in the Iowa State compaction apparatus (2, 4, 5). The procedures that produce compactive efforts comparable to standard or modified AASHO tests are given elsewhere (2, 5).

Nine specimens were molded from each mixture; the first three were tested for unconfined compressive strength after 7 days curing, the second three specimens after 7 days curing plus 1 day immersion in distilled water, and the last three after 28 days curing. After each specimen was molded, it was wrapped in wax paper to reduce loss of moisture and placed in a humidity room for curing. The relative humidity and temperature maintained were 95 ± 5 percent and 71 ± 4 F.

The unconfined compressive strength test was made at a constant rate of loading of 0.1 in. per min. The strength and density values reported are averages of three test specimens.

RESULTS

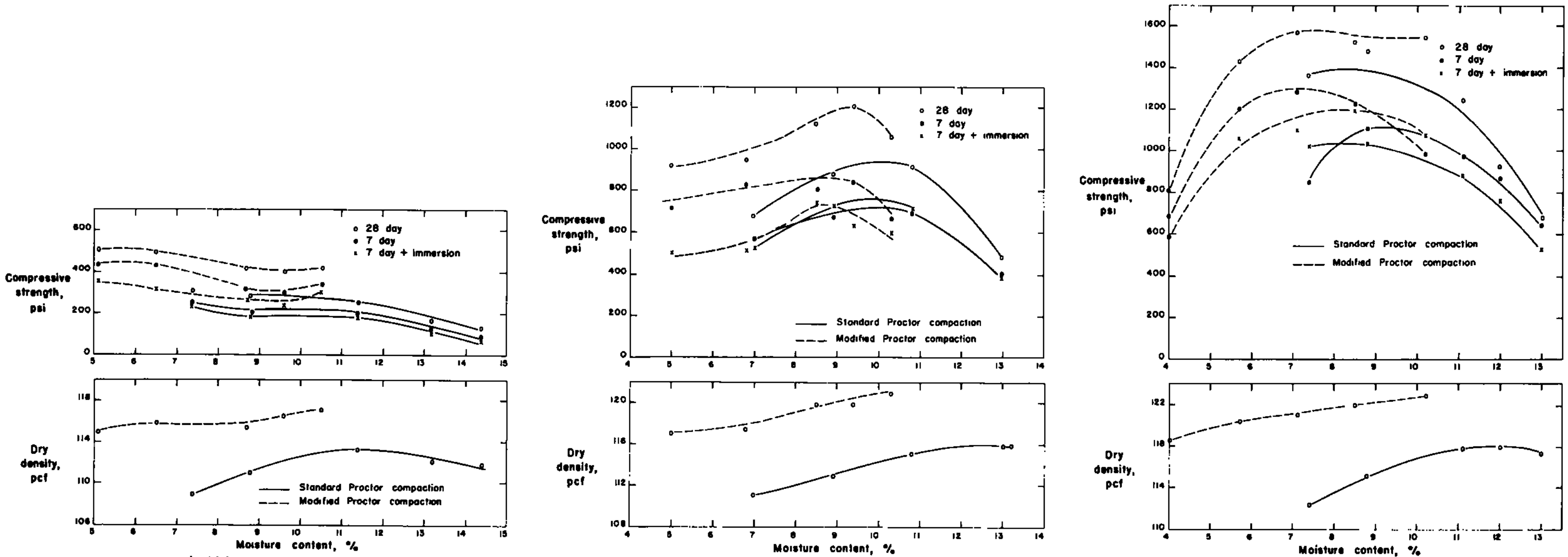
Moisture-Density-Strength Relationships

The data obtained are presented in Figures 1 through 13. For a given mixture the same moisture content does not necessarily give both maximum density and maximum strength. The moisture contents for maximum strength are generally on the dry side for the sand-clay mixtures dominant in sand, and on the wet side for mixtures dominant in clay (Table 3).

TABLE 2

SAND-CLAY MIXTURE PROPORTIONS

Sand (% by wt)	Clay (% by wt.)
100	0
75	25
50	50
25	75
0	100

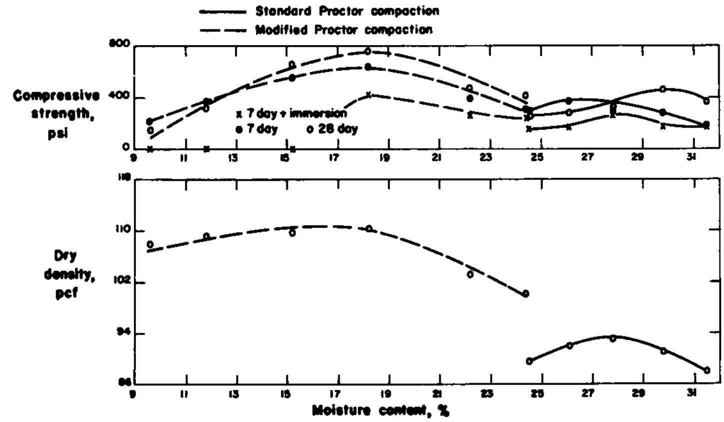


a) 100 % dune sand Cement, 8 %

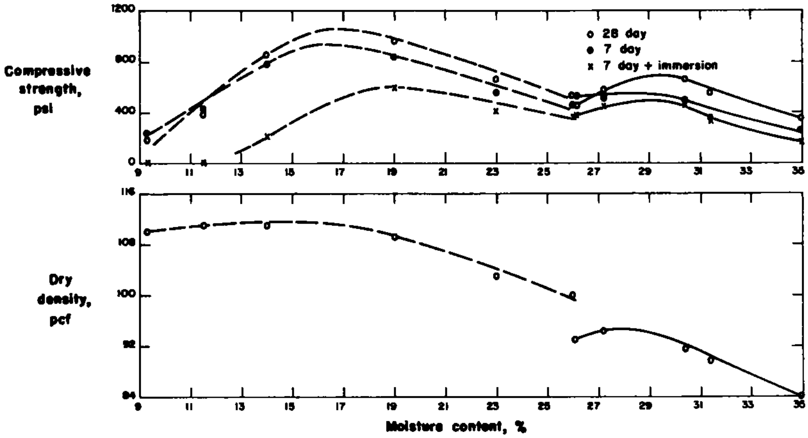
b) 100 % dune sand Cement 12 %

c) 100 % dune sand Cement, 16 %

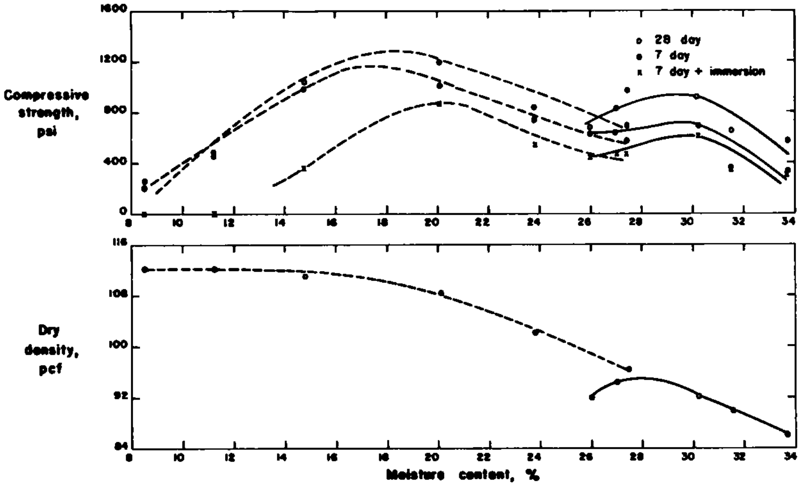
Figure 1.



a) 100% montmorillonitic clay Cement, 8%

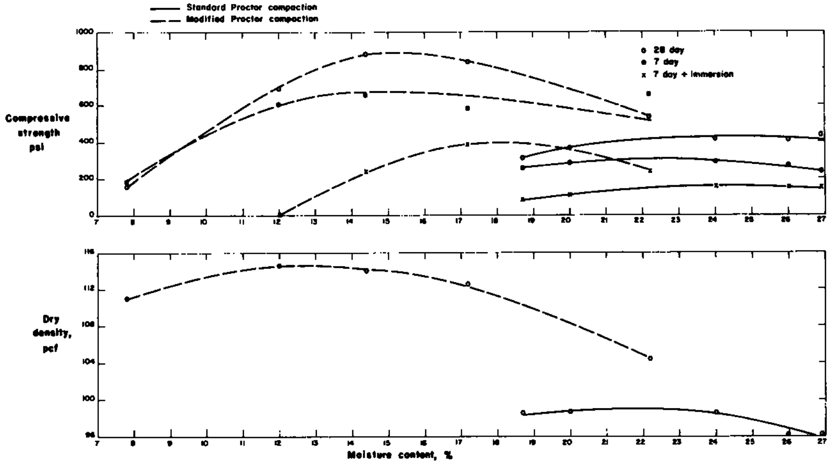


b) 100% montmorillonitic clay Cement, 12%

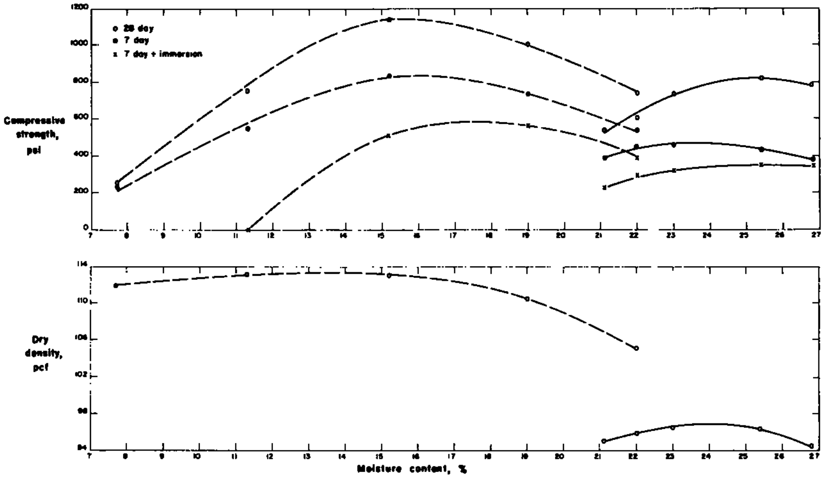


c) 100% montmorillonitic clay Cement, 16%

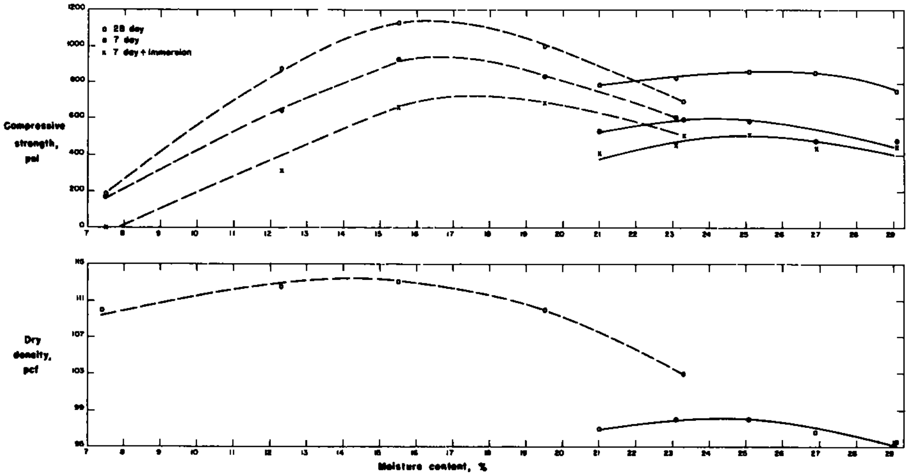
Figure 2.



a) 100% illitic clay Cement 8%

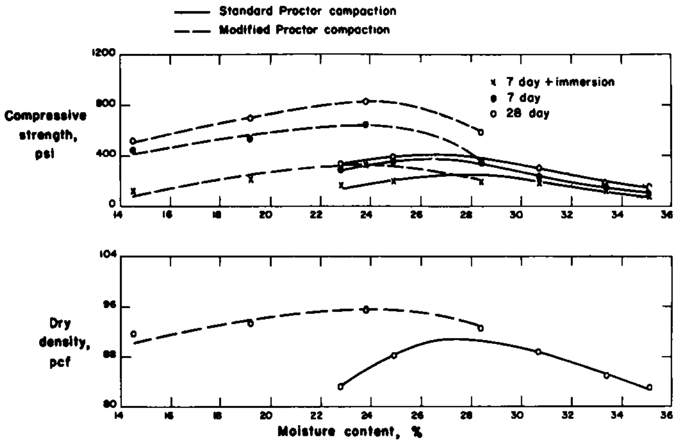


b) 100% illitic clay Cement 12%

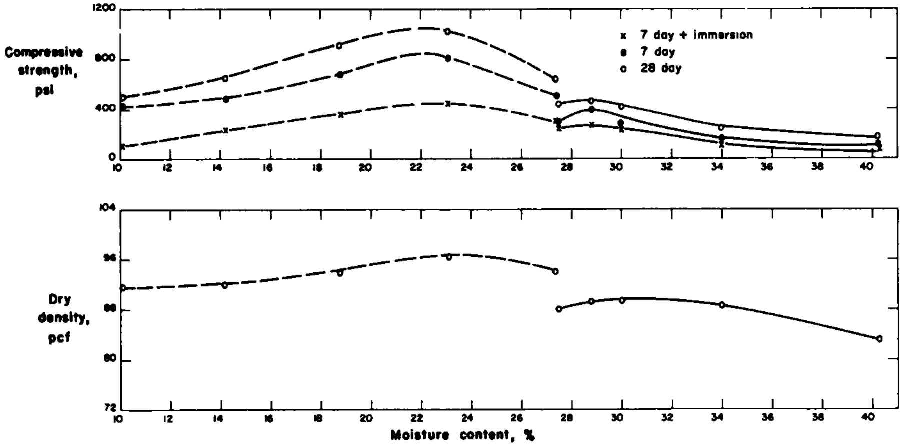


c) 100% illitic clay Cement, 16%

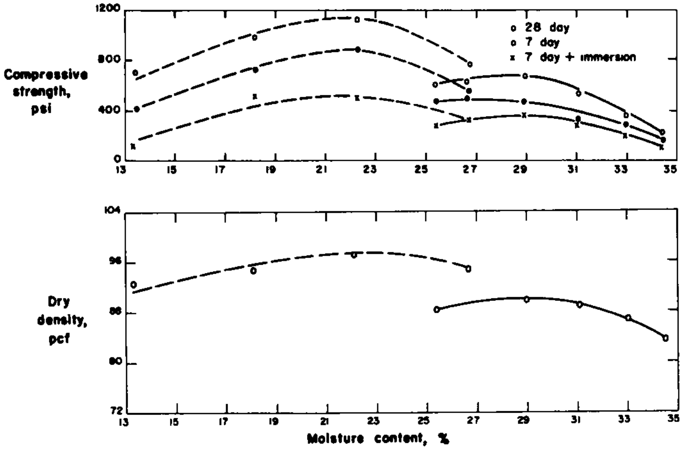
Figure 3.



a) 100% kaolinitic clay Cement, 8%

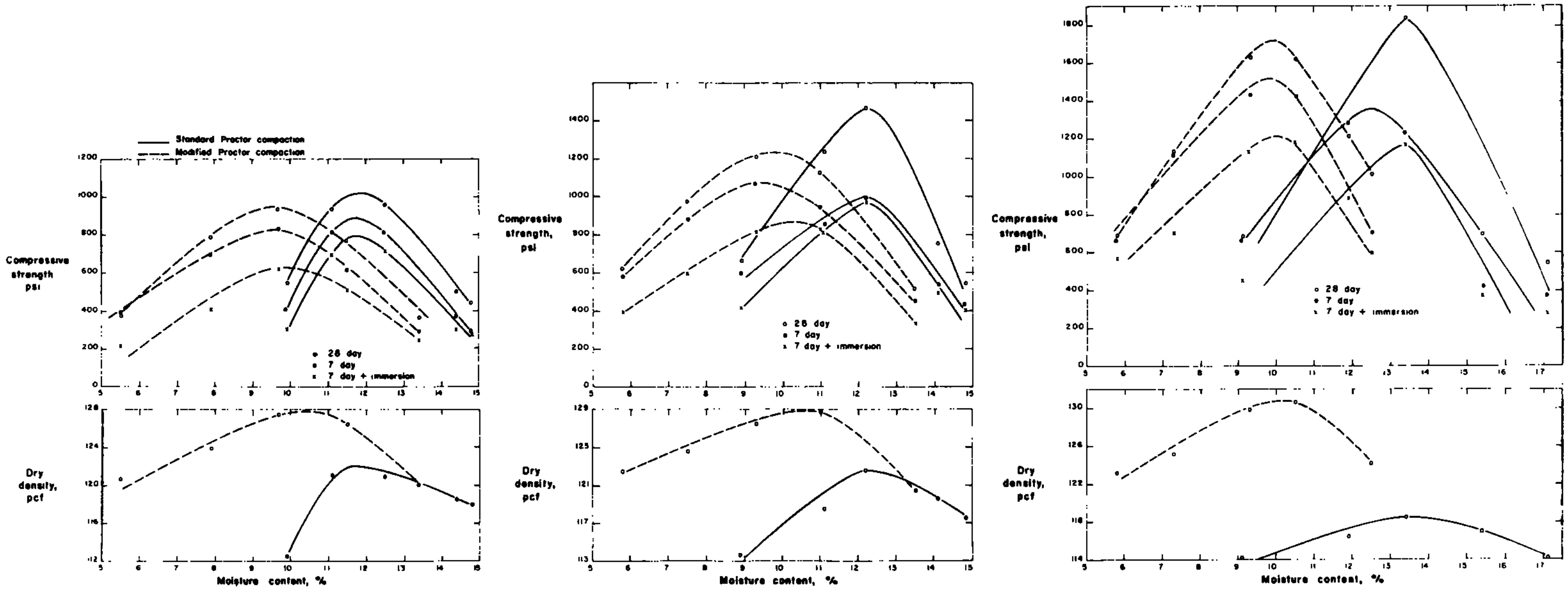


b) 100% kaolinitic clay Cement, 12%



c) 100% kaolinitic clay Cement, 16%

Figure 4.

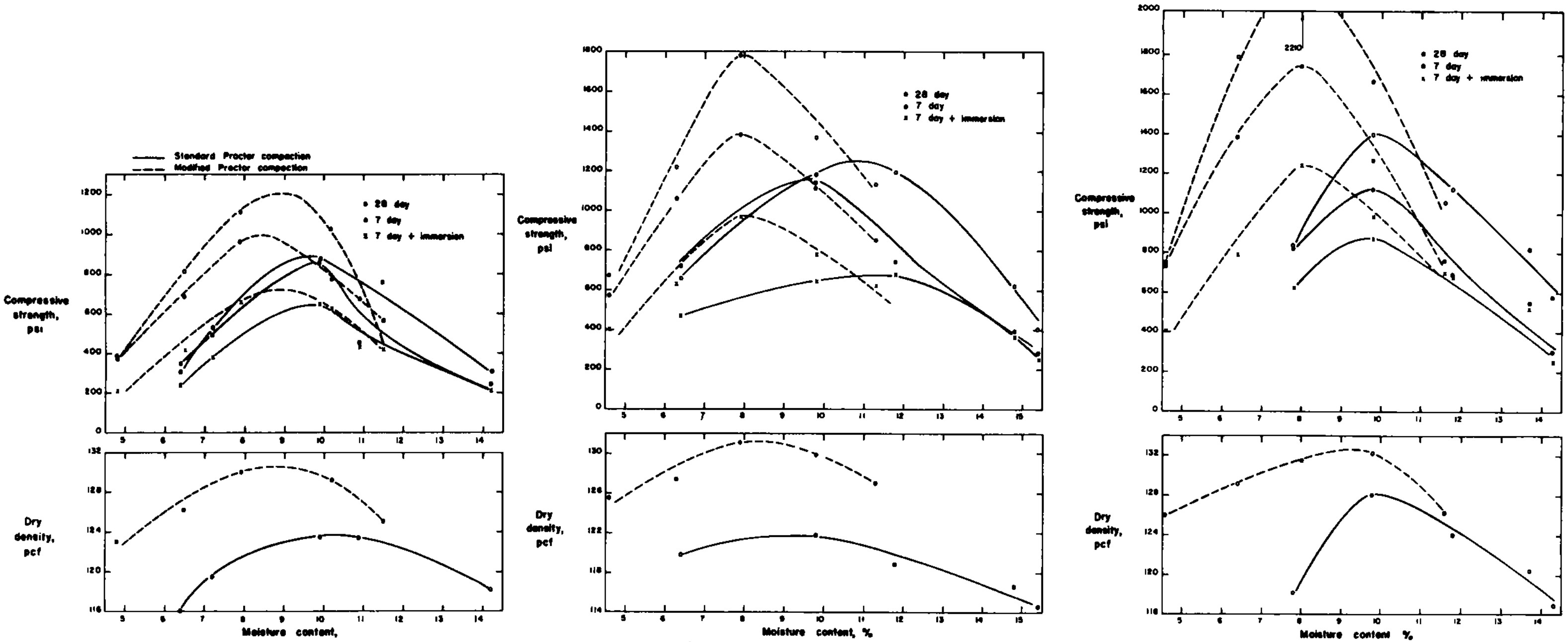


a) Blend 75 % dune sand and 25 % montmorillonitic clay Cement 8 %

b) Blend 75 % dune sand and 25 % montmorillonitic clay Cement 12 %

c) Blend 75 % dune sand and 25 % montmorillonitic clay Cement 16 %

Figure 5.

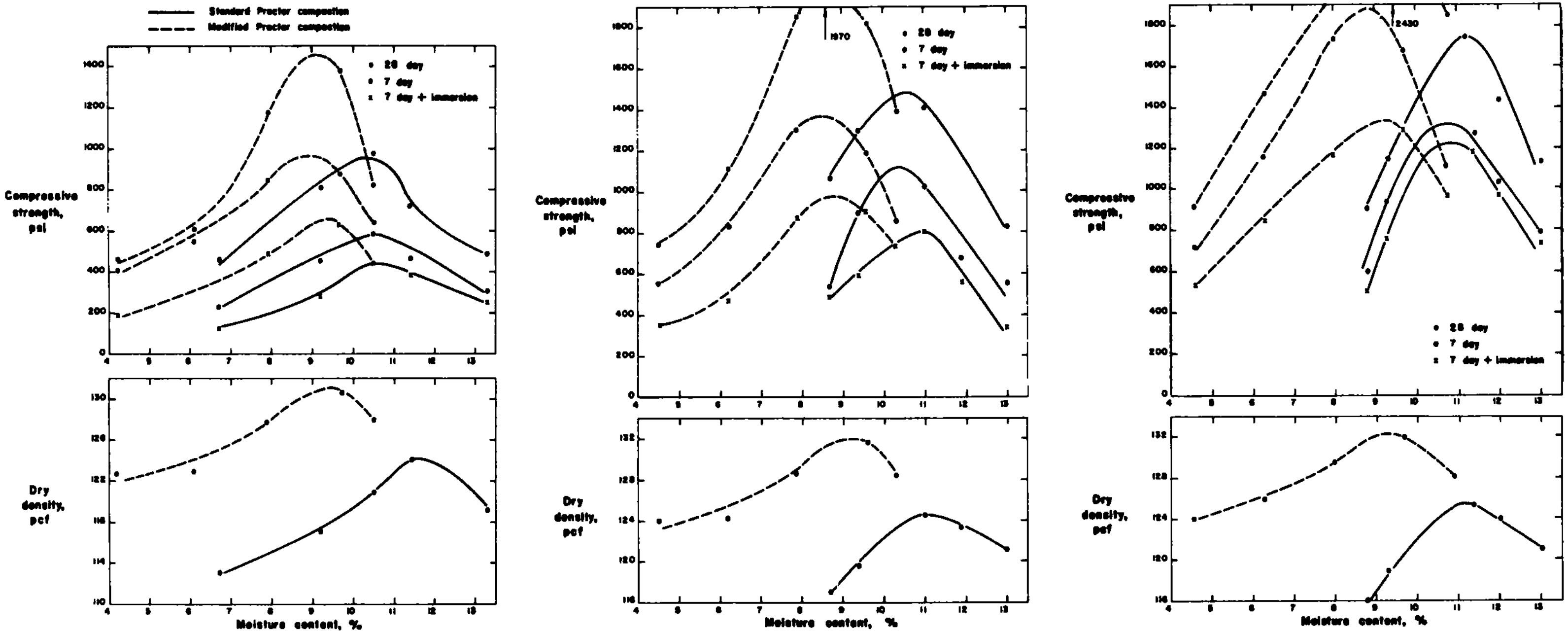


a) Blend 75% dune sand and 25% illitic clay
Cement 8%

b) Blend: 75% dune sand and 25% illitic clay Cement 12%

c) Blend: 75% dune sand and 25% illitic clay Cement 16%

Figure 6.

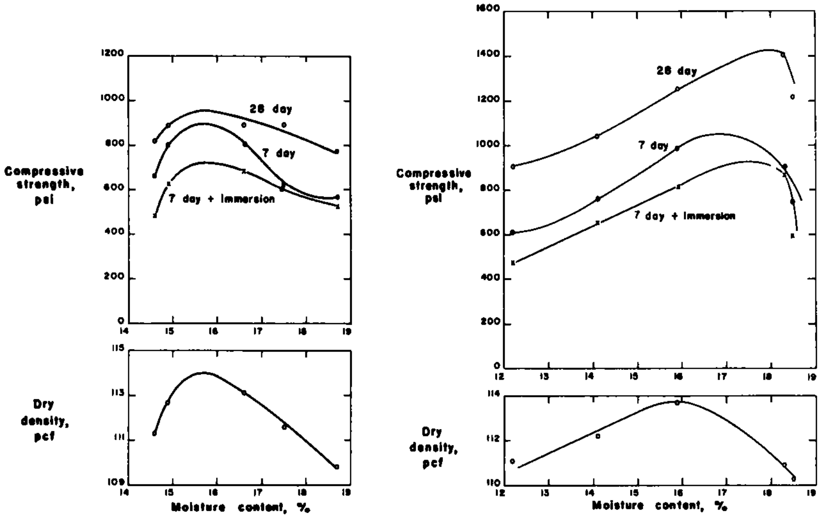


a) Blend: 75 % dune sand and 25 % kaolinitic clay. Cement content, 8 %

b) Blend: 75 % dune sand and 25 % kaolinitic clay. Cement content, 12 %

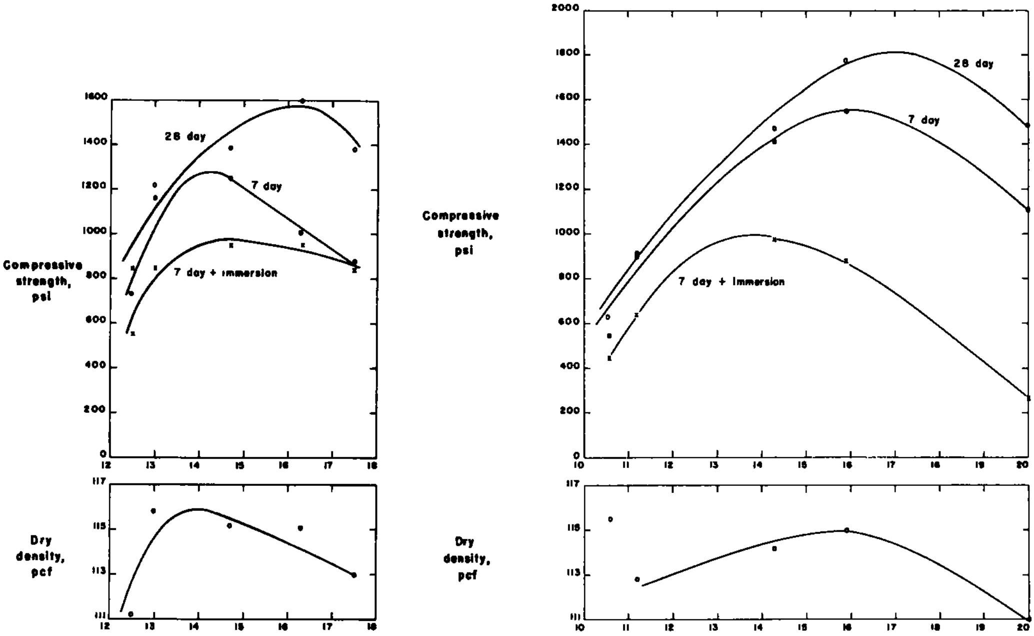
c) Blend: 75 % dune sand and 25 % kaolinitic clay. Cement content, 16 %

Figure 7.



a) Blend: 50 % dune sand and 50 % montmorillonic clay
Cement, 8 %

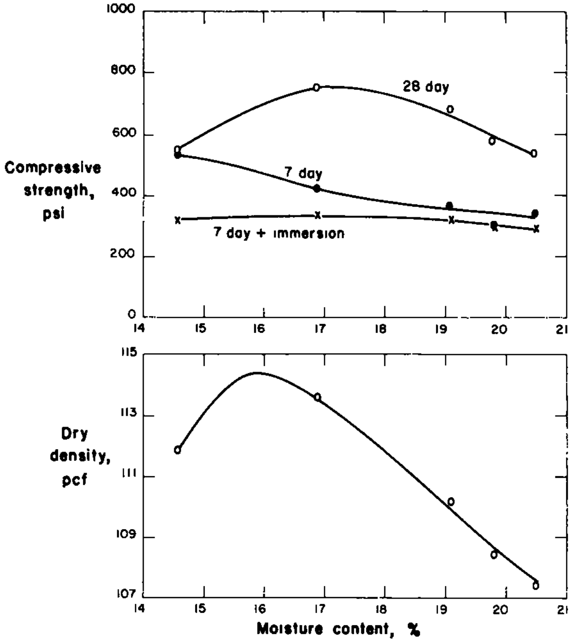
b) Blend 50 % dune sand and 50 % montmorillonitic clay
Cement, 12 %



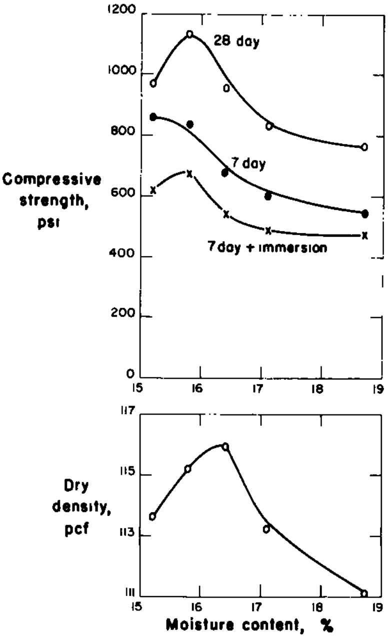
c) Blend 50 % dune sand and 50 % montmorillonitic clay
Cement, 16 %

d) Blend 50 % dune sand and 50 % montmorillonitic clay
Cement, 20 %

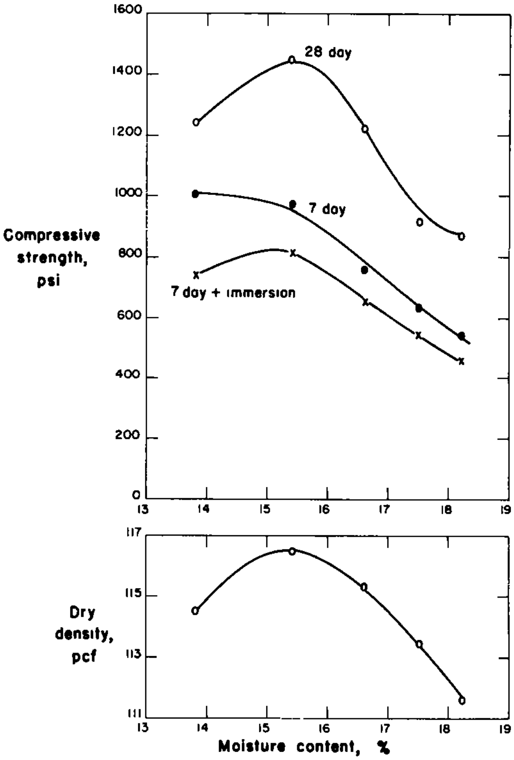
Figure 8.



a) Blend 50% dune sand and 50% illitic clay Cement, 8%

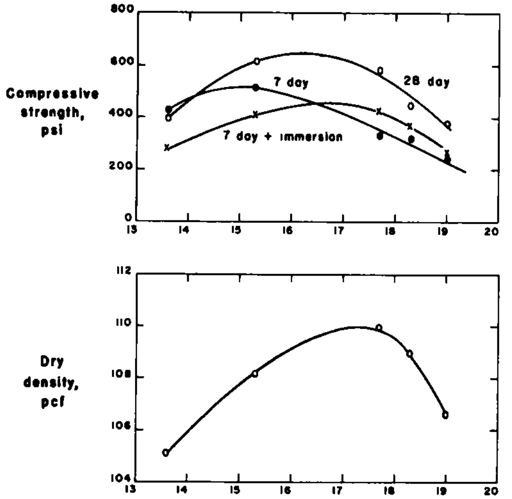


b) Blend 50% dune sand and 50% illitic clay Cement, 12%

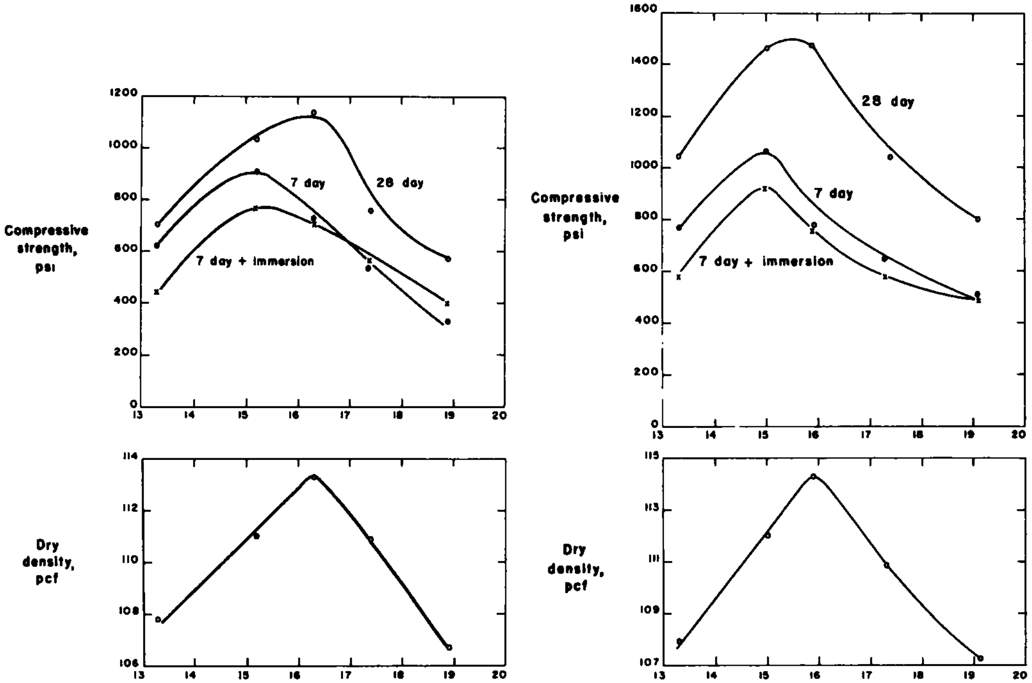


c) Blend 50% dune sand and 50% illitic clay Cement, 16%

Figure 9.



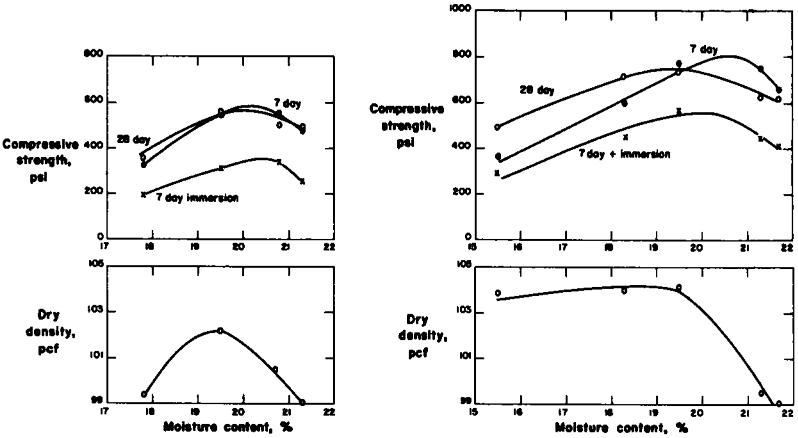
a) Blend 50 % dune sand and 50 % kaolinitic clay
Cement, 8 %



b) Blend 50 % dune sand and 50 % kaolinitic clay
Cement, 12 %

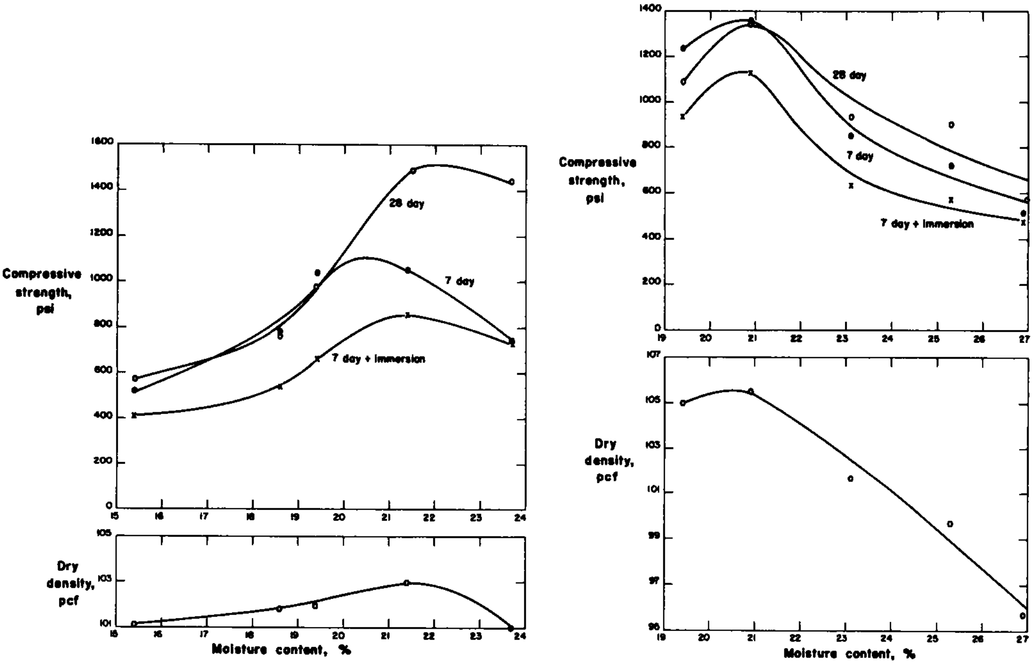
c) Blend 50 % dune sand and 50 % kaolinitic clay
Cement, 16 %

Figure 10.



a) Blend 25% dune sand and 75% montmorillonitic clay Cement, 8%

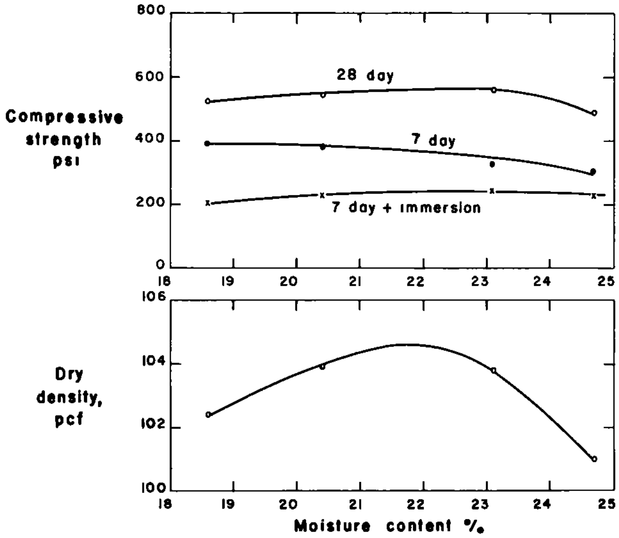
b) Blend 25% dune sand and 75% montmorillonitic clay Cement, 12%



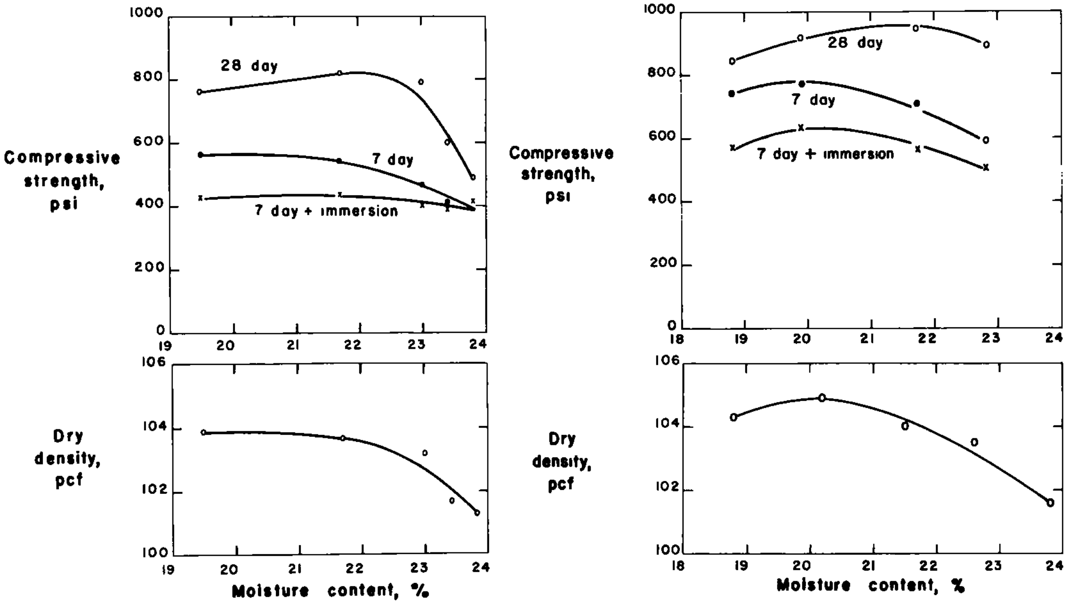
c) Blend: 25% dune sand and 75% montmorillonitic clay Cement, 16%

d) Blend 25% dune sand and 75% montmorillonitic clay Cement, 20%

Figure 11.



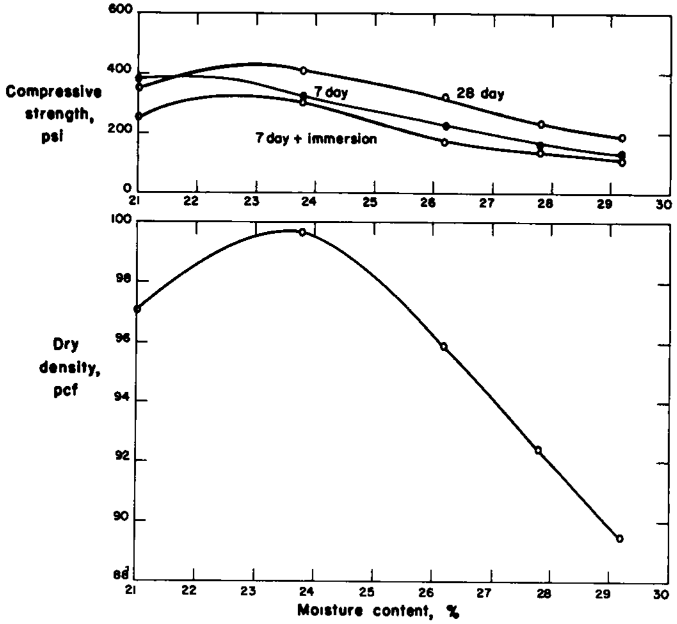
a) Blend 25% dune sand and 75% illitic clay
Cement 8%



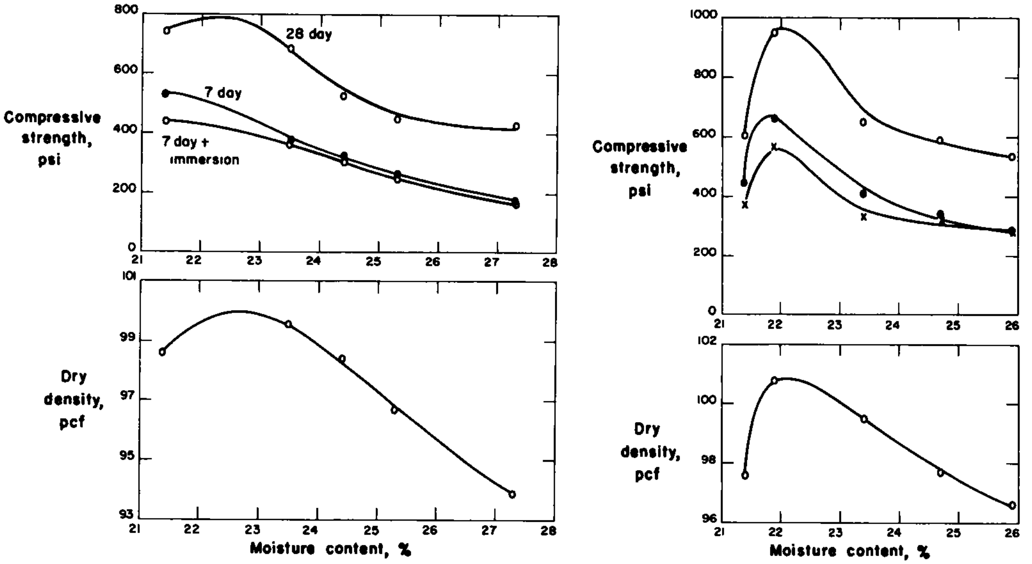
b) Blend 25% dune sand and 75% illitic clay
Cement, 12%

c) Blend 25% dune sand and 75% illitic clay
Cement, 16%

Figure 12.



a) Blend 25% dune sand and 75% kaolinitic clay Cement, 8%



b) Blend 25% dune sand and 75% kaolinitic clay Cement, 12%

c) Blend 25% dune sand and 75% kaolinitic clay Cement, 16%

Figure 13.

TABLE 3

OPTIMUM MOISTURE CONTENTS FOR MAXIMUM DENSITY AND MAXIMUM 7-DAY STRENGTH FOR CEMENT-TREATED SAND-CLAY MIXTURES AT STANDARD AASHO COMPACTION

Cement Content (%)	Dominant Clay Mineral[a]	Property	Opt Moist. Content (%) for Sand-Clay Ratios of				
			100:0	75:25	50:50	25:75	0:100
8	Kaolinite	Density	11.3	11.5	17.1	23.7	27.6
		Strength	6.8	10.4	15.3	22.2	26.5
	Illite	Density	11.3	10.3	15.9	21.8	21.9
		Strength	6.8	9.8	14.5	20.0	22.3
	Montmorillonite	Density	11.3	11.7	15.6	19.4	27.8
		Strength	6.8	11.7	15.8	20.2	26.6
12	Kaolinite	Density	12.2	11.0	16.3	22.7	30.4
		Strength	9.8	10.3	15.2	21.4	28.8
	Illite	Density	12.2	9.8	16.3	19.5	24.0
		Strength	9.8	9.6	15.1	19.5	23.9
	Montmorillonite	Density	12.2	12.2	15.9	19.3	28.2
		Strength	9.8	12.2	16.8	20.6	28.8
16	Kaolinite	Density	12.2	11.2	15.9	22.2	29.0
		Strength	9.3	10.7	15.0	21.8	28.0
	Illite	Density	12.2	9.8	15.3	20.2	24.3
		Strength	9.3	9.8	14.0	20.1	24.7
	Montmorillonite	Density	12.2	13.3	13.8	21.1	27.8
		Strength	9.3	12.5	14.3	20.6	29.8

[a] In sand-clay mixtures.

The variation between the optimum moisture contents for both maximum density and maximum strength may be related to the particle size of the soils. The surface area of sands is relatively small, and most of the lubrication water provided is available for hydration of the cement. The optimum moisture for maximum density may furnish a surplus of water for the hydration reaction, causing a reduction in strength analogous to that found in concrete with a high water to cement ratio; when less water is added, the proper hydration of the cement increases the over-all strength of the mixture, even though the density is slightly less than maximum. Clays, on the other hand, have a large surface area, and much of the water provided for lubrication may be adsorbed resulting in insufficient water available for hydration of the cement. In the sand and clay mixtures dominant in clay, the additional water to obtain proper cement hydration apparently raises the moisture content for maximum strength above that required for maximum density.

For the sand-kaolinite clay mixtures and kaolinite alone, maximum strength is always on the dry side of optimum moisture for maximum density, perhaps because kaolinite has a very low capacity for adsorbing water (3), and cement hydration requirements are satisfied with less water than is needed for compaction to maximum density. For the sand-illite and sand-montmorillonite clay mixtures, the optimum moisture content for maximum strength shifts to the wet side of optimum moisture for maximum density at the higher clay contents. Compared to kaolinite, illite and particularly montmorillonite clay have high capacities for adsorbing water (3), and as the amount of illite or montmorillonite clay in sand-clay mixtures is increased, the moisture requirement for cement hydration is not completely satisfied. Consequently the optimum water content for maximum strength is greater than that required for maximum density when the amounts of clay are large.

As the cement content of a given sand-clay mixture increases, the optimum moisture for maximum density and the optimum moisture for maximum strength does not change much. This suggests that the water necessary for hydration of cement may be small in comparison with the amount needed to obtain a maximum density.

Strength Increase Between 7 and 28 Days

The strength increase in the early curing period is important. The maximum strengths at 28 days are 100 to 160 percent of those at 7 days (Figs. 1 to 13 and Tables 4 and 5). No definite relationship is found between the strength increase and type of clay or other factors. However, the strength increase of the montmorillonite clay mixtures is generally less than that shown by the illite or kaolinite clay mixtures. This may be due to the greater affinity for water of montmorillonite clay, and a resultant shortage of water for cement hydration reactions.

Immersed and Unimmersed Strengths

Because stabilized soil courses in a pavement may become saturated, specimens cured for 7 days were tested in both immersed and unimmersed conditions. The 2- by 2-in. specimens were at near saturation moisture content after immersion in distilled water for 24 hr.

The unimmersed specimens have maximum strengths of from one to two times those of the corresponding immersed specimens (Figs. 1 to 13 and Tables 4 and 5). The ratio of unimmersed to immersed maximum strength varies with the type and amount of clay in the mixture, cement content, and the kind of compaction. Every mixture tested gives a different ratio of unimmersed to immersed strength. However, with standard AASHO compaction, mixtures with illite clay are more susceptible to immersion than mixtures made with either of the other two clays.

Effect of Compactive Effort on Strength

Modified compactive effort produced higher maximum strengths than standard compactive effort with all four basic soils and with the 75 percent sand-25 percent illite clay and 75 percent sand-25 percent kaolinite clay mixtures at all cement contents (Figs. 1 to 7 and Table 4). But with the 75 percent sand-25 percent montmorillonite clay mixture, standard compaction gave maximum strengths equal to or higher than

TABLE 4

MAXIMUM COMPRESSIVE STRENGTHS OF SOIL-CEMENT MIXTURES COMPACTED WITH STANDARD AASHO AND MODIFIED AASHO COMPACTION

Mixture	Cement Content (%)	Maximum Compressive Strength (psi)					
		Standard AASHO Compaction			Modified AASHO Compaction		
		7-Day + 1-Day Immersion	7-Day	28-Day	7-Day + 1-Day Immersion	7-Day	28-Day
100% sand	8	235	250	300	350	440	510
	12	760	720	940	740	860	1,205
	16	1,035	1,120	1,400	1,200	1,300	1,580
100% montmorillonite	8	280	380	470	420	640	760
	12	485	560	680	610	940	1,060
	16	610	715	940	880	1,160	1,280
100% illite	8	160	300	425	395	670	885
	12	355	475	825	585	840	1,140
	16	510	600	870	730	950	1,145
100% kaolinite	8	260	385	405	330	640	840
	12	280	400	480	450	830	1,045
	16	360	480	680	500	886	1,120
75% sand + 25% montmorillinoite	8	880	880	1,020	630	830	950
	12	960	1,000	1,460	870	1,080	1,240
	16	1,180	1,360	1,840	1,220	1,520	1,720
75% sand + 25% illite	8	640	860	890	720	1,000	1,200
	12	680	1,140	1,260	970	1,390	1,790
	16	880	1,180	1,400	1,240	1,740	2,210
75% sand + 25% kaolinite	8	440	580	960	660	970	1,455
	12	800	1,120	1,480	980	1,370	1,970
	16	1,220	1,320	1,740	1,330	1,880	2,430

TABLE 5

MAXIMUM COMPRESSIVE STRENGTH OF SOIL-CEMENT MIXTURES COMPACTED WITH STANDARD AASHO COMPACTION

Mixture	Cement Content (%)	Maximum Compressive Strength (psi)		
		7-Day + 1-Day Immersion	7-Day	28-Day
50% sand + 50% montmorillonite	8	720	890	960
	12	930	1,045	1,420
	16	970	1,280	1,580
	20	995	1,555	1,815
50% sand + 50% illite	8	335	535	760
	12	680	860	1,125
	16	825	1,010	1,440
50% sand + 50% kaolinite	8	460	520	645
	12	765	910	1,130
	16	930	1,060	1,500
25% sand + 75% montmorillonite	8	360	585	565
	12	560	805	750
	16	860	1,105	1,520
	20	1,135	1,340	1,360
25% sand + 75% illite	8	240	390	565
	12	430	565	820
	16	626	766	960
25% sand + 75% kaolinite	8	325	385	430
	12	440	530	790
	16	560	670	965

those for modified compaction (Fig. 5). This finding was at first questioned, and the experiment was carefully repeated. The results were the same. Further, more detailed studies are planned to obtain the data needed for an explanation.

Maximum strengths for modified AASHO compaction range between 71 and 247 percent of the maximum strengths for standard AASHO compaction, the highest percentage increasing occurring with 8 percent cement-treated illite clay. It is apparent that many variables affect the relationship between strength and compactive effort. It appears, however, that if maximum densities as great as those obtained by the modified AASHO compaction could be assured, soil-cement of superior quality would be produced. This is especially true for soils dominant in illite and kaolinite clays.

Influence of Clay Content on Strength

The maximum strengths with standard compaction for different sand-clay combinations treated with 8, 12, and 16 percent cement are shown in Figure 14. For the sand-clay combinations investigated, the strength values generally reach a peak for mixtures composed of about 75 percent sand and 25 percent clay. This strength peak corresponds to a density peak for about the same combination of sand and clay. It appears, therefore, that the blends of 75 percent sand and 25 percent clay form a well-graded soil mixture giving high maximum densities and high maximum strengths.

The relative influence of the different clays in the sand-clay mixtures is seen in Figure 14. For amounts of clay greater than 25 percent, the mixtures containing montmorillonite clay gave higher maximum strengths than comparable mixtures containing illite or kaolinite clays. This may be partly due to the high surface activity of montmorillonite and its participation in the reactions producing the cementing compounds.

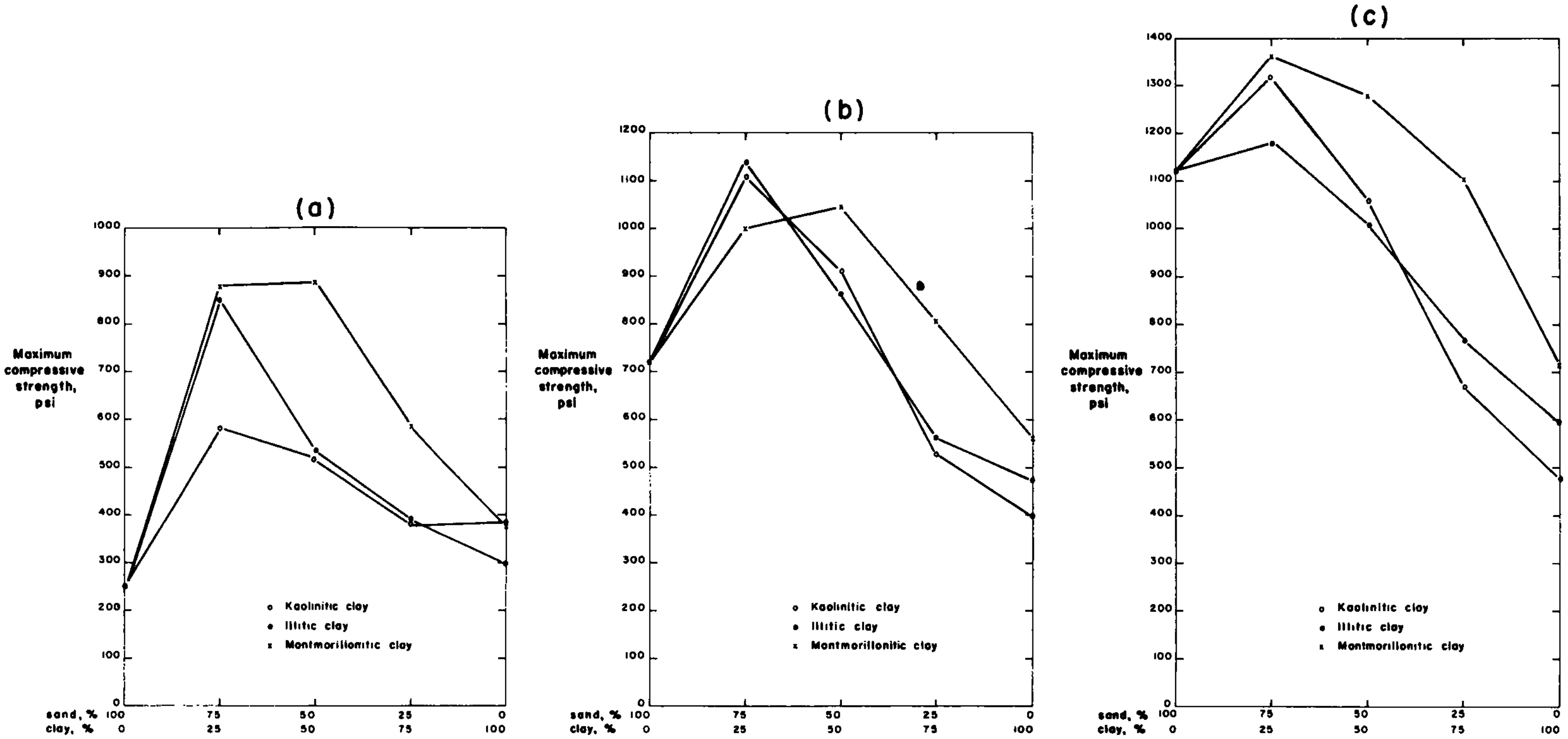


Figure 14. Maximum strengths of sand-clay mixtures compacted with the standard AASHO compactive effort and treated with (a) 8 percent concrete, (b) 12 percent concrete, (c) 16 percent concrete.

Effect of Clay Content on Density

The maximum density for a given combination of sand-clay is about the same with either montmorillonite or illite clays (Figs. 1 to 13). With these clays the densities are greater than with kaolinite clay.

Influence of Amount of Cement on Strength

The maximum strength increases with increase in cement (Fig. 14 and Tables 4 and 5), but the rate of strength increase varies for every sand-clay combination. In general, the maximum 7-day strength is increased 30 percent by increasing the amount of cement from 8 to 12 percent, from 12 to 16 percent, or, in mixtures with montmorillonite clay, from 16 to 20 percent. However, a 30 percent increase in strength with a 4 percent increase in cement can not be relied on because in some sand-clay mixtures the increase is as low as 3 percent or as high as 70 percent. An increase in cement content from 8 to 12 percent resulted in a very great improvement in maximum 7-day strength of cement treated dune sand with no clay, a threefold increase with standard compaction and a twofold increase with modified compaction.

If a minimum strength of 300 psi is required after 7 days curing and 1 day immersion, none of the four basic soils used were properly stabilized with 8 percent cement and standard AASHO compaction. However, when modified compaction is used, the four basic soils with 8 percent cement have 7-day cured, 1-day immersed strengths of over 400 psi.

All sand-clay mixtures treated with 8 percent cement and given standard compaction have 7-day cured, 1-day immersed strengths of over 300 psi, except the one composed of 25 percent sand and 75 percent illite clay.

CONCLUSIONS

The following conclusions appear applicable to the cement-treated soil or sand-clay mixtures used in this investigation:

1. The optimum moisture content for maximum density and the optimum moisture content for maximum unconfined compressive strength are not necessarily the same.
2. The moisture content for maximum strength of the sand is from 2.4 to 4.5 percentages drier than the moisture content for maximum density.
3. The moisture content for maximum strength of the sand-kaolinite clay mixtures is slightly drier than the moisture content for maximum density.
4. The moisture content for maximum strength of the sand-illite clay or the sand-montmorillonite clay mixtures is on the dry side of the moisture content for maximum density in mixtures dominant in sand, and on the wet side in mixtures dominant in clay.
5. A sand-clay mixture of about 75 percent sand and 25 percent clay is near optimum for both maximum density and maximum strength.
6. Modified compactive effort gives maximum strengths up to 247 percent higher than the standard, except with a mixture of 75 percent sand and 25 percent montmorillonite clay, for which standard compaction produced maximum strengths equivalent to or higher than those obtained with modified compaction.
7. The maximum strengths after 28 days of curing are from 100 to 160 percent greater than those obtained after 7 days of curing.
8. The maximum strengths after 7 days of curing, without immersion, are from 100 to 200 percent greater than after 7 days of curing plus 1 day of immersion.
9. It is recommended for construction purposes that the difference between optimum moisture for maximum density and optimum moisture for maximum strength be considered practically negligible for sand-clay mixtures containing about 25 percent clay or more. However for mixtures with sand alone, or sand and small amounts of clay, the optimum moisture content for maximum strength should be used.

ACKNOWLEDGMENTS

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Science and Technology. Project 449-S is under contract with the Iowa Highway Research Board of the Iowa State Highway Commission as their Project HR-82.

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Effect of Lime on Cement Stabilization of Montmorillonitic Soils

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Although lime as a secondary additive to soil-cement mixtures has been investigated before, the effectiveness of the addition has not been thoroughly explained. The present research is an attempt to fill this gap.

Five soils were studied. Though all contained montmorillonite as the dominant clay mineral, they varied texturally from friable loess to high-clay gumbotil. Various amounts of lime and cement were used, and the compressive strengths of compacted specimens were determined after 7, 28 and 84 days humid room curing and 1 day immersion in distilled water. The results obtained indicate that there is an optimum amount of lime which gives the maximum strength for a fixed amount of cement.

The increase of strength with addition of lime was higher for soils with high clay content and much higher when noncalcareous soils of similar gradation were used.

The effect of elapsed time between mixing and compacting was studied. The results showed a decrease of density and strength with the time elapsed between mixing and compacting when the molding moisture content was below or near the optimum moisture content. Use of lime minimized both the density and strength reductions due to this time lag. However, with molding moistures above the optimum moisture content an increase in strength was obtained when an interval of 2 hr was allowed between mixing and compacting.

Unimmersed unconfined compressive strengths within a period of 24 hr after mixing and sedimentation tests were performed on soil-lime-cement mixtures for tracing exchange reactions and sedimentation.

• PORTLAND CEMENT is the chemical most successfully used in soil stabilization for road construction. Almost all soils respond to treatment with cement; however, chemical conditions of some soils (by interfering in the hydration of cement) and high plasticity of others (by preventing an intimate mixture) have limited the use of portland cement. Another chemical widely used in soil stabilization is lime, which has been shown to improve greatly the workability of the soil and to increase the strength of the mixture.

Previous studies have pointed out beneficial results of a simultaneous use of lime and cement under certain conditions, both in field construction (7, 11) and in laboratory (4, 10). The purpose of the present investigation was to evaluate quantitatively and economically this treatment as a function of some soil characteristics and to attempt an explanation for some of the reactions involved.

MATERIALS

Five soils were used in the study (Table 1). In all five, montmorillonite was the dominant clay mineral, but the soils varied texturally from friable loess to high-clay gumbotil. The friable loess was sampled from the deep loess bordering the Missouri River. The unleached Kansan till was obtained in north western Iowa. The plastic loess, the gumbotil, and the leached Kansan till were sampled at different depths from the same site in southeastern Iowa.

As for the stabilizing agents, the properties of the type I portland cement, the main agent, are given in Table 2; and those of the hydrated lime, which was the secondary additive, are given in Table 3.

TABLE 1

PROPERTIES OF SOILS

Property	Friable Loess	Plastic Loess	Leached Kansan Till	Unleached Kansan Till	Gumbotil
I. E. E. S. sample designation	20-2	528-4	528-10	411	528-8
Sampling location (Iowa)	Harrison Co.	Keokuk Co.	Keokuk Co.	Page Co.	Keokuk Co.
Soil series	Hamburg	Mahaska	---	Shelby	---
Horizon	C	C	C_p	C	B_p
Sampling depth (in.)	467-480	36-77	>110	216-240	91-107
Textural composition[a]:					
Gravel (above 2 mm)	0.0	0.0	0.0	0.0	0.0
Sand (2 - 0.074 mm)	0.6	5.5	37.0	23.9	22.2
Silt (0.074 - 0.005 mm)	81.4	53.3	22.3	28.3	15.0
Clay (below 0.005 mm)	18.0	41.2	40.7	47.8	62.8
Clay (below 0.002 mm)	13.0	33.5	36.2	39.7	59.6
Physical (%):					
Liquid limit	33	52	44	45	76
Plastic limit	28	20	17	15	26
Plasticity index	5	32	27	30	50
Chemical:					
pH[b]	8.4	6.3	7.0	7.7	6.9
C.E.C.[c] (meq/100 g)	15.58	23.64	23.6	19.84	37.54
Carbonates[d] (%)	10.91	1.87	2.27	12.05	2.45
Organic matter[e] (%)	0.50	0.44	0.13	0.10	0.16
Predominant clay mineral	M[f]	M	M	M	M
Classification:					
Textural[g]	Silt loam	Silty clay loam	Clay loam	Clay loam	Clay
Unified	ML	CH	CL	CL	CH
BPR (AASHO)	A-4 (8)	A-7-6 (18)	A-7-6 (16)	A-7-6 (17)	A-7-6 (20)

[a] Textural gradation tests performed only on soil fraction passing No. 10 sieve All soils contained less than 5 percent gravel.

[b] Glass electrode method using suspension of 15 g soil in 30 cc distilled water.

[c] Ammonium acetate (pH = 7) method on soil fraction below 2 mm.

[d] Versenate method for total calcium.

[e] Potassium bichromate method

[f] M standing for montmorillonite

[g] USDA textural classification

TABLE 2

PROPERTIES OF PORTLAND CEMENT USED

Property	Value
Chemical analysis (%):	
Silicon dioxide, SiO_2	21.62
Aluminum oxide, Al_2O_3	5.05
Ferric oxide, Fe_2O3	2.97
Calcium oxide, CaO	64.05
Magnesium oxide, MgO	2.90
Sulfuric trioxide, SO_3	2.26
Insoluble residue	0.16
Loss on ignition	0.58
Specific surface (sq cm/g)	
Turbidimeter (Wagner)	1,855
Air permeability (Blaine)	3,395

TABLE 3

PROPERTIES OF HYDRATED LIME USED

Property	Value
Chemical analysis (%):	
Silicon dioxide, SiO_3	0.3
Aluminum and ferric oxide, $Al_2O_3 + Fe_2O_3$	0.6
Calcium oxide, CaO	73.8
Magnesium oxide, MgO	0.6
Sulfuric trioxide, SO_3	0.3
Loss on ignition	24.1
Percent passing No. 325 sieve	95.5

METHODS OF INVESTIGATION

The samples of soil were air dried and ground repeatedly until all soil aggregations were reduced to individual particle size or were fine enough to pass through the No. 10 mesh sieve.

Unconfined Compressive Strength

The unconfined compressive strengths of the soils treated with various amounts of lime and cement were determined to evaluate the effectiveness of the additives on the hardening of the mixtures.

Test specimens were prepared from batches mixed in a Hobart C-100 kitchen mixer at the lower speed. The required amount of air-dry soil and additives were first machine mixed for 1 min. Then water in the amount required to bring the mixture to a moisture content close to the optimum moisture content determined by the standard Proctor test was added and machine mixed for 1 min. Next, the sides of the bowl were scraped, and the materials were mixed again for 1 min.

From each batch, nine specimens were molded 2 in. in diameter by 2 in. high. The specimens were compacted in an apparatus developed in the laboratory (9). In this method the desired amount of mixture is placed in a 2-in. diameter mold and receives five blows on each end from a hand-operated 5-lb drop hammer falling 12 in. This procedure was found to give a density near standard Proctor density.

The specimens were weighed to the nearest 0.1 g, and the height was determined to the nearest 0.001 in. All specimens tested were 2 ± 0.050 in. high. Then to prevent loss of moisture the specimens were wrapped in waxed paper, sealed with cellulose tape, and stored in a curing room where the relative humidity was maintained at 95 ± 5 percent and the temperature was kept at 70 ± 5 F. At the end of a predetermined curing period, the specimens were immersed in distilled water for 24 hr and then were immediately tested for unconfined compressive strength. Three different curing periods were used. At the end of each period three specimens were tested.

Prolonged Mixing Studies

The reduction of density with the time elapsed between mixing and compaction of the specimens and the minimizing effect of lime on this reduction were noticed during performance of the unconfined compression tests. Because density changes are known to introduce strength variations, and inasmuch as in field practice there is a time lag between mixing and compacting, a laboratory study of the effect of prolonged mixing was undertaken.

The study was conducted on two soils, the friable loess and the plastic loess, using two mixtures for each: with friable loess one was 10 percent cement, and the other was 10 percent cement plus 2 percent lime; with the plastic loess one was 10 percent cement, the other was 8 percent cement plus 2 percent lime. These amounts were chosen because they were expected to give approximately the same 7-day strengths.

The procedure of test was programed, taking into consideration results of earlier research (6) which brought out that the effect of time is more pronounced when the mixture is left undisturbed than when it is intermittently mixed. Intermittent mixing, which is more representative of the field condition, was adopted.

It has also been verified that the optimum moisture content varies when a certain time elapses between mixing and compacting (6). To take this effect into account, various amounts of water were used in the investigation.

For each mixture, the following procedure was used: the desired amounts of soil, additives, and water were mixed as for the unconfined compression tests. Immediately after mixing a set of three 2- by 2-in. specimens were molded, their densities were determined, and they were wrapped and stored in the curing room. The remaining mixture was kept in the bowl and covered with a cloth, which was kept damp throughout the process. Every 20 min the mixture was hand mixed for about 1 min. Two hours after the addition of water, a second set of three specimens was molded and processed. The intermittent mixing proceeded and after 2 more hours the procedure was repeated for a third set of three specimens. During the molding of each 3 specimen set, a sample of the mixture was collected for determination of the moisture content.

Flocculation Studies

The effect of lime and cement on flocculation of soils was studied qualitatively by sedimentation tests. Preliminary tests showed that when a flocculated soil dispersed in water settles down, a sharp line of demarcation is observed between the supernatant water and the suspension. The rate of sedimentation is practically constant from the beginning of the process until the piling up of particles on the bottom changes the density and the viscosity of the suspension and decreases the settling velocity. This result leads to the deduction that the process in this case is analogous to that observed in sedimentation of suspensions of uniform spheres (2). Although particles larger than the flocs are present in the soil, they settle faster, so that the over-all settling process resembles closely that of uniform flocs.

The initial rate of sedimentation is a function of the following factors (2):

1. Size of the flocs.
2. Shape of the flocs.
3. Concentration of the suspension or its density.
4. Specific gravity of the soil plus additive mixture.
5. Density of the fluid.
6. Viscosity of the fluid.
7. Temperature.
8. Shape of the glass in which the sedimentation takes place.

With the shape of the flocs corrected by a factor and the effect of the concentration of the suspension determined, the size of the flocs or their relative diameter could be determined by Stokes' law. However, no attempt was made with this purpose in the present investigation; only qualitative results were looked for. The last five of the factors indicated were kept constant or considered to be nearly constant; therefore, their relative effect was disregarded.

The concentration of the suspension was varied for each of the three soils studied. The amounts of mixture to be dispersed in 1,000 ml of water were selected from preliminary tests so that approximately the same rate of settling would be obtained when 8 percent of lime was added to each soil, 8 percent being considered the amount of lime above which no further flocculation would take place. The preliminary tests suggested the use of 155 g of mixture for the friable loess, 85 g for the leached Kansan till, and 65 g for the gumbotil. Such amounts would give approximately the same density of slurry if the suspension was made up only of particles smaller than 0.012 mm.

The procedure for running the sedimentation test was as follows: the desired amounts of air-dry soil and lime or cement were placed in a 1,000-ml hydrometer jar and mixed for 1 min. Then about 150 ml of distilled water was added, and the mixture was stirred thoroughly with a glass tube. More water was added to make a slurry of approximately 250 ml. Next the mixture was dispersed by the air-jet apparatus (3) for 10 min at a pressure of 15 psi. The hydrometer jar with a rubber stopper in the open end was then successively turned upside down and back for a period of 1 min to make the suspension uniform, and finally the jar was set on a horizontal surface.

The time was recorded when the suspension reached a 2.5-cm mark (T_1) and again when it reached a 12.5-cm mark (T_2) below the 1,000-ml line of the hydrometer jar. These data were used to calculate the average rate of sedimentation by

$$V = \frac{10 \text{ (cm)}}{T_2 - T_1 \text{ (sec)}} \tag{1}$$

Initial Hardening Studies

The results of the prolonged mixing studies showed that lime plays an important part in the initial hardening of soil cement. This effect was studied by means of unimmersed unconfined compressive strengths determined within a period of 24 hr after mixing.

Two-inch diameter by 2-in. high specimens were prepared, cured, and tested by the same procedure used for immersed unconfined compression tests. However, the study required that only one specimen be molded from each batch. Small amounts were used for each mixture and hand mixing had to be used instead of machine mixing.

The unimmersed unconfined compressive strengths were determined 5, 15, and 30 min, and also 1, 2, 4, 8, 16, and 24 hr after the addition of water to the mixture. Preparation of the specimens took approximately 3 min. All specimens were tested at their molding moisture content, which was close to the optimum moisture content for maximum density.

RESULTS

Unconfined Compressive Strength

The general pattern of the results of unconfined compressive strength tests (Figs. 1, 2, 3, 4, and 5) indicates that the beneficial effect of lime does not depend on the cement

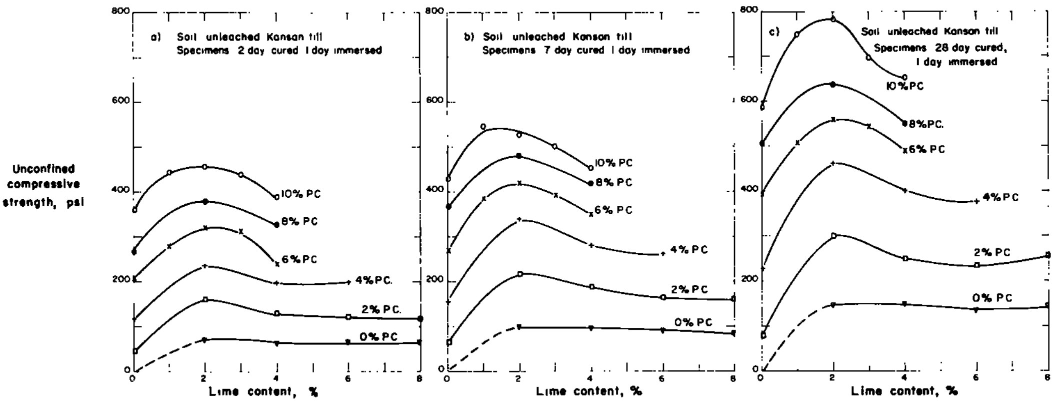

Figure 1. Unconfined compressive strengths of unleached Kansan till with varying lime and cement contents, after various curing periods.

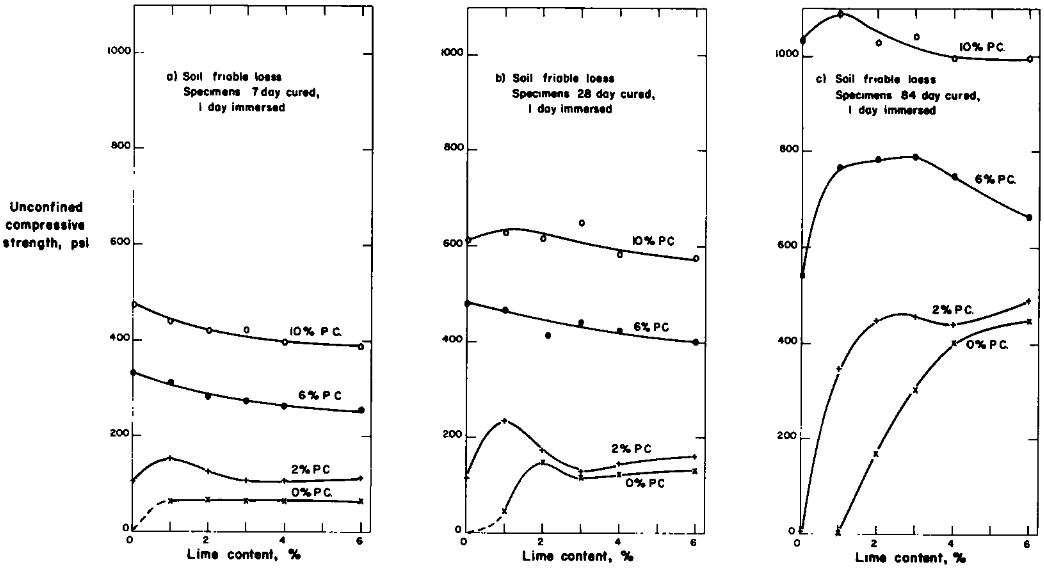

Figure 2. Unconfined compressive strengths of friable loess with varying lime and cement contents, after various curing periods.

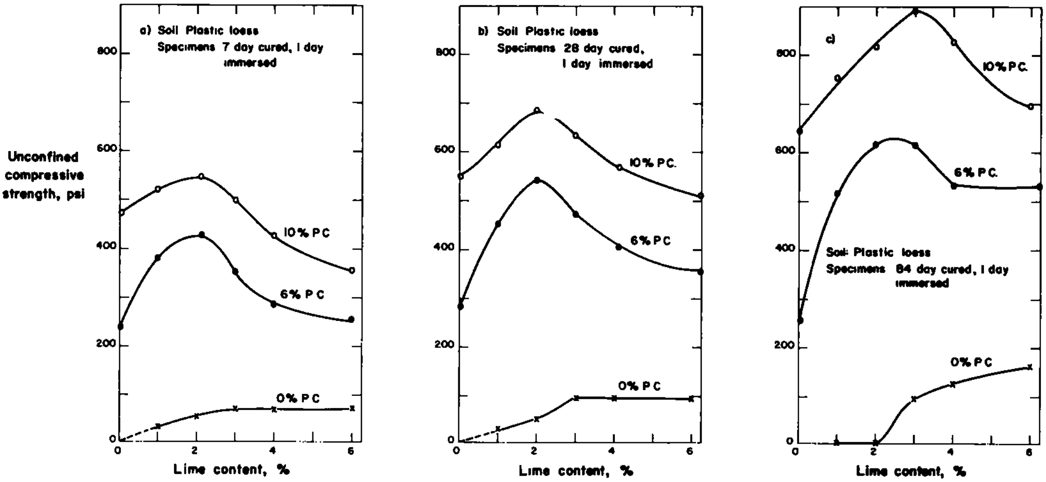

Figure 3. Unconfined compressive strengths of plastic loess with varying lime and cement contents, after various curing periods.

content in soil-lime-cement mixtures. In tests where lime proved beneficial, and this happened with four of the five soils studied, the compressive strength increased with addition of lime; however, above a certain amount of lime, further addition caused reduction of strength. The amount of lime for which a maximum strength occurred will be hereafter referred to as the "optimum amount of lime."

For the unleached Kansan till the optimum amount of lime was found to be 2 percent and remained constant for all cement contents and for all the curing periods—2, 7, and 28 days. These results suggested a modified program of testing for the other soils: concentration of work on only two cement contents and adoption of the 84-day curing instead of the 2-day.

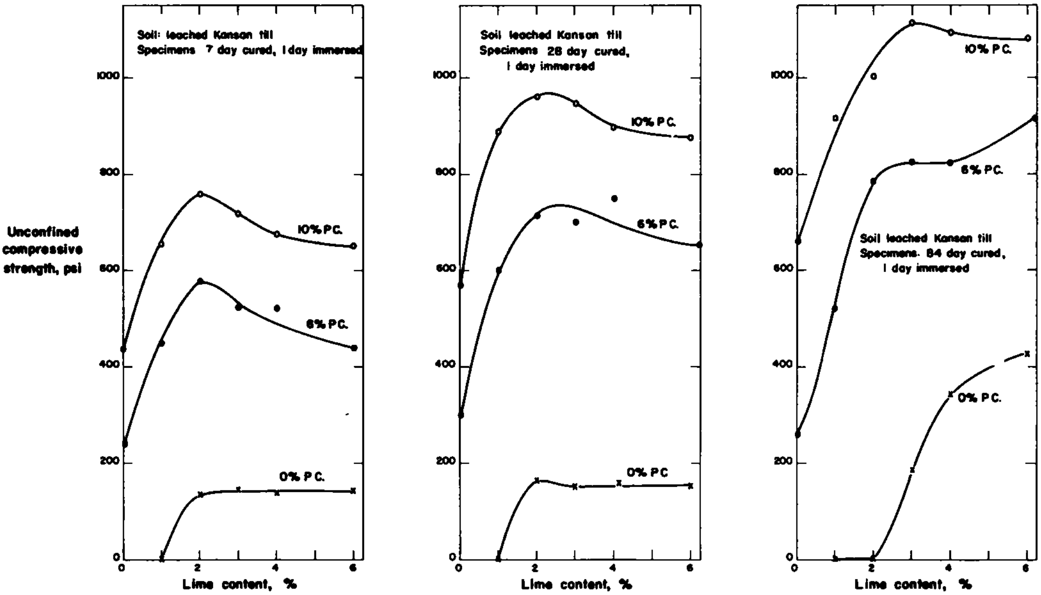

Figure 4. Unconfined compressive strengths of leached Kansan till with varying till and cement contents, after various curing periods.

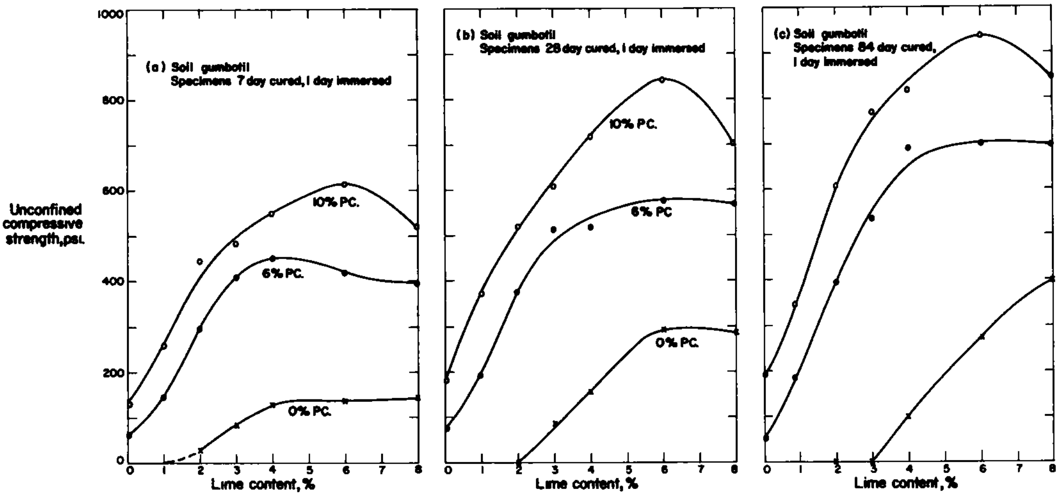

Figure 5. Unconfined compressive strengths of gumbotil with varying lime and cement contents, after various curing periods.

For the friable loess, addition of lime to mixtures with 6 and 10 percent cement did not have any beneficial effect within the 28-day curing period; in fact, it caused a gradual but small reduction of strength. In mixtures with 2 percent cement, however, addition of lime caused an increase of strength. When only lime was added to the soil, the strength of 7-day cured specimens was practically constant for all amounts of lime. At the end of the 28-day curing period, the 1 percent lime mixture showed a decrease of strength. The strength of mixtures containing larger amounts of lime, although higher than at 7 days, did not vary with the lime content. The 84-day strength indicated an interesting change of pattern in that the specimens with 1 percent lime failed completely when immersed in water, though the other mixtures gave relatively high strengths, which increased with lime content and reached 400 psi for 4 percent lime. No explanation was found for the decrease of strength for low amounts of lime in the mixtures. However, the results suggest that, in a second reaction of the system, the initially formed gel is destroyed; when lime is present in larger amounts the second reaction produces a stronger cementation, but for low amounts this cementation is not strong enough to withstand immersion in water. The same phenomenon occurred with admixtures of small amounts of lime and other soils.

The notable increase of strength after 28 days for mixtures with 2 or more percent lime indicates the effect of pozzolanic reactions occurring between 28 and 84 days. These reactions are reflected, although on a reduced scale, in the 84-day strength of friable loess-lime-cement mixtures.

Results of unconfined compressive strength tests for the plastic loess mixtures show a displacement of the optimum amount of lime from 2 percent at 7- and 28-day curing periods to 2.5 or 3 percent for 84-day cured specimens. The pozzolanic reactions, which occur mainly when the lime content is increased from 2 percent to 3 percent, may explain the displacement of the optimum amount of lime.

The test results for the leached Kansan till, indicate an optimum amount of lime of 2 percent for 7-day curing, 2.5 percent for 28-day curing, and 3 percent for 84-day curing. Exception to this is the 84-day strength of the 6 percent cement plus 6 percent lime mixture, which was higher than the strength of the 6 percent cement plus 3 percent lime mixture.

The results for the gumbotil suggest an optimum amount of lime of 6 percent, with only one exception: the maximum 7-day strength for the 6 percent cement occurred with 4 percent lime. Results of mixtures with lime only suggest that pozzolanic reactions did take place but did not change the position of the optimum amount of lime for different curing periods.

The increasing of strength with addition of lime indicates that while the increase of strength for the plastic loess and the leached Kansan till is higher for 6 percent than for 10 percent cement, for the gumbotil it is higher for 10 percent cement than for 6 percent (Table 4). Strength increase for unleached Kansan till evidently does not depend on cement content. In terms of percentage, however, the increase of strength with addition of lime always decreases for higher amounts of cement (Table 5).

TABLE 4

INCREASE OF THE UNCONFINED COMPRESSIVE STRENGTH WITH ADDITION OF OPTIMUM AMOUNT OF LIME, FOR VARIOUS CURING TIMES

Soil	Opt. Amt. of Lime (%)	Cement Content (%)	Diff. Betw. Strengths of Soil-Opt. Amt. of Lime-Cement Mix. and Soil-Cement Mix. (psi)			
			2-Day Cured[a]	7-Day Cured[a]	28-Day Cured[a]	84-Day Cured[a]
Unleached Kansan till	2	2	112	145	220	
		4	120	186	234	
		6	113	150	168	
		8	113	115	235	
		10	95	101	295	
Plastic loess	2	6		178	263	358
		10		75	135	175
Leached Kansan till	2	6		337	420	527
		10		225	395	340
Gumbotil	2	6		390	475	651
		10		485	662	747

[a] Plus 1-day immersed.

TABLE 5

PERCENTAGE INCREASE OF UNCONFINED COMPRESSIVE STRENGTH WITH ADDITION OF OPTIMUM AMOUNT OF LIME, FOR VARIOUS CURING TIMES

Soil	Opt. Amt of Lime (%)	Cement Content (%)	Ratio of Diff. Betw. Strs. of Soil-Opt. Amt. of Lime-Cement Mix and Soil-Cement Mix. to Soil-Cement Str. (%)			
			2-Day Cured[a]	7-Day Cured[a]	28-Day Cured[a]	84-Day Cured[a]
Unleached Kansan till	2	2	255	230	275	
		4	104	122	99	
		6	55	56	43	
		8	42	32	58	
		10	26	24	61	
Plastic loess	2	6		74	94	139
		10		16	25	27
Leached Kansan till	2	6		140	140	204
		10		52	69	54
Gumbotil	6	6		650	633	1,300
		10		374	368	388

[a] Plus 1-day immersed

Another point of interest in the results is the increase of strength with time. The compressive strengths for mixtures without lime and for those with the optimum amount of lime at different curing times are compared with those of 7-day cured specimens (Table 6). For the glacial till soils no significant difference in the rate of strength increase is obtained with the addition of lime. However, the addition of lime to the loessial soils caused a more pronounced gain of strength after the first 7 days of cure.

Also, the mixtures of plastic loess, leached Kansan till, and gumbotil with 6 percent of cement showed a reduction of strength after 28 days of curing. Although it is questionable that these mixtures would satisfy the durability criteria based on soil-cement losses during 12 cycles of either the wet-dry test or freeze-thaw test, they certainly do not satisfy the strength criteria which stipulates that compressive strength should increase both with age and cement content (12).

The maximum strengths attained by the five soils with 6 and 10 percent cement, and the optimum amounts of lime show that all the soils studied can be successfully stabilized with lime and cement on an economical basis (Table 7).

Prolonged Mixing Studies

Results from the study of the effects of prolonged mixing (i. e., the time elapsed between addition of water and compaction) on strength and density are shown in Figures 6 and 7.

The dry density of the mixtures decreased with prolonged mixing for both the plastic loess and the friable loess, whether any lime was in the mixture or not. The moisture-density curves of aged mixtures with friable loess show an increase of optimum moisture content with a decrease of maximum density. However, aged mixtures with plastic loess gave irregular moisture-density curves, resembling the result of a compaction test with sand.

The difference between the densities of specimens molded right after mixing and those molded after a definite interval of time, as outlined in the methods of investigation, was practically constant for all moisture contents below optimum moisture content. This difference, which was less beyond the optimum moisture content, tended to disappear as the densities approached the saturation line.

TABLE 6

UNCONFINED COMPRESSIVE STRENGTHS AT VARIOUS AGES OF MIXTURES WITHOUT AND WITH 7-DAY OPTIMUM AMOUNT OF LIME, EXPRESSED AS PERCENTAGE OF 7-DAY UNCONFINED COMPRESSIVE STRENGTH

Soil	Opt. Amt. of Lime (%)	Cement Content (%)	Ratio of Unconfined Compress. Str. to 7-Day Str. (%)					
			7-Day Cured[a]		28-Day Cured[a]		84-Day Cured[a]	
			Without Lime	With Opt. Amt. Lime	Without Lime	With Opt. Amt. Lime	Without Lime	With Opt. Amt. Lime
Unleached Kansan till	2	2	100	100	127	138		
		4	100	100	149	136		
		6	100	100	145	133		
		8	100	100	111	133		
		10	100	100	114	148		
Friable loess	1	2	100	100	110	136	0	356
		6	100	100	145	149	164	245
		10	100	100	126	143	217	247
Plastic loess	2	6	100	100	117	127	107	144
		10	100	100	117	125	137	150
Leached Kansan till	2	6	100	100	125	125	108	136
		10	100	100	131	146	152	152
Gumbotil	6	6	100	100	125	122	83	155
		10	100	100	138	137	148	153

[a] Plus 1-day immersed.

The beneficial effect of lime occurred during the first 2 hr. The lime in the mixture lessened the decrease of density due to prolonged mixing (Figs. 8a-c and 9a-c).

The unconfined compressive strengths of specimens compacted right after the initial mixing showed a dependence on density, with the maximum strength occurring at the optimum moisture content for maximum density. Noticeable, however, is the decrease of strength when the moisture content was near to or higher than 2 percent above the optimum moisture content.

TABLE 7

MAXIMUM UNCONFINED COMPRESSIVE STRENGTHS OF MIXTURES WITH 6 AND 10 PERCENT CEMENT PLUS LIME AFTER 7, 28, AND 84 DAYS OF CURING

Soil	Maximum Unconfined Compressive Strength (psi)					
	7-Day Cured[a]		28-Day Cured[a]		84-Day Cured[a]	
	6% Cement	10% Cement	6% Cement	10% Cement	6% Cement	10% Cement
Friable loess	330	475	479	649	789	1,089
Unleached Kansan till	419	527	559	782		
Plastic loess	427	546	543	685	617	892
Gumbotil	451	615	569	843	697	940
Leached Kansan till	578	760	717	961	828	1,116

[a] Plus 1-day immersed.

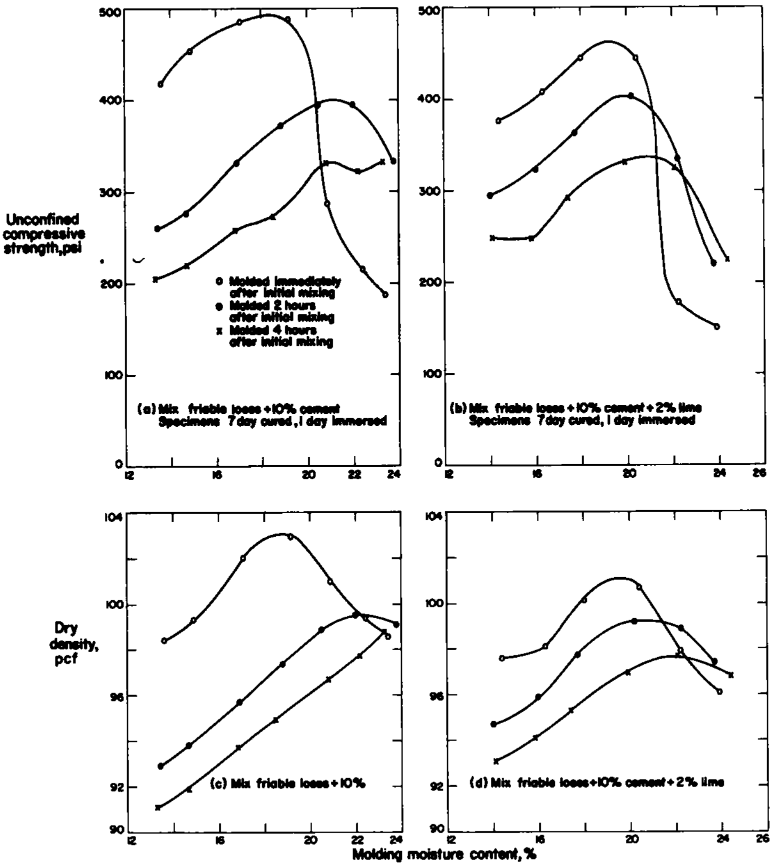

Figure 6. Unconfined compressive strengths and dry densities of friable loess with 10 percent cement, and with 10 percent cement plus 2 percent lime, at varying molding moisture contents; specimens being molded immediately after initial mixing, and after 2 and 4 hr of prolonged mixing.

The compressive strength of specimens molded after 2 hr of intermittent mixing did not show the same pattern for all mixtures. Though the compressive strength for the plastic loess-cement was continuously increasing with moisture content, the moisture-strength curve for the plastic loess-lime-cement and for the two mixtures with friable loess was similar to the density-moisture curve. However, with high moisture contents in all, this strength became greater than that of specimens molded right after mixing.

The compressive strengths of specimens molded after 4 hr of intermittent mixing, although lower in value, followed the same pattern as those molded after 2 hr mixing.

As it was with density, the compressive strength decreased whether lime was in the mixture or not, but here again, lime minimized the reduction. Figures 8d-f and 9d-f show that when compaction was done 2 hr after the initial mixing, the decrease in strength of soil-lime-cement mixtures was about one-half that with soil-cement mixtures. Similarly, when an increase of strength was observed, the increase was more pronounced for mixtures containing lime. The differences between the strengths of specimens molded after 4 hr of mixing was still the same, indicating that the effect of lime took place during the first 2-hr period.

In an attempt to explain the increase of strength with mixing time, for high moisture contents, X-ray diffraction patterns of mixtures were determined immediately after

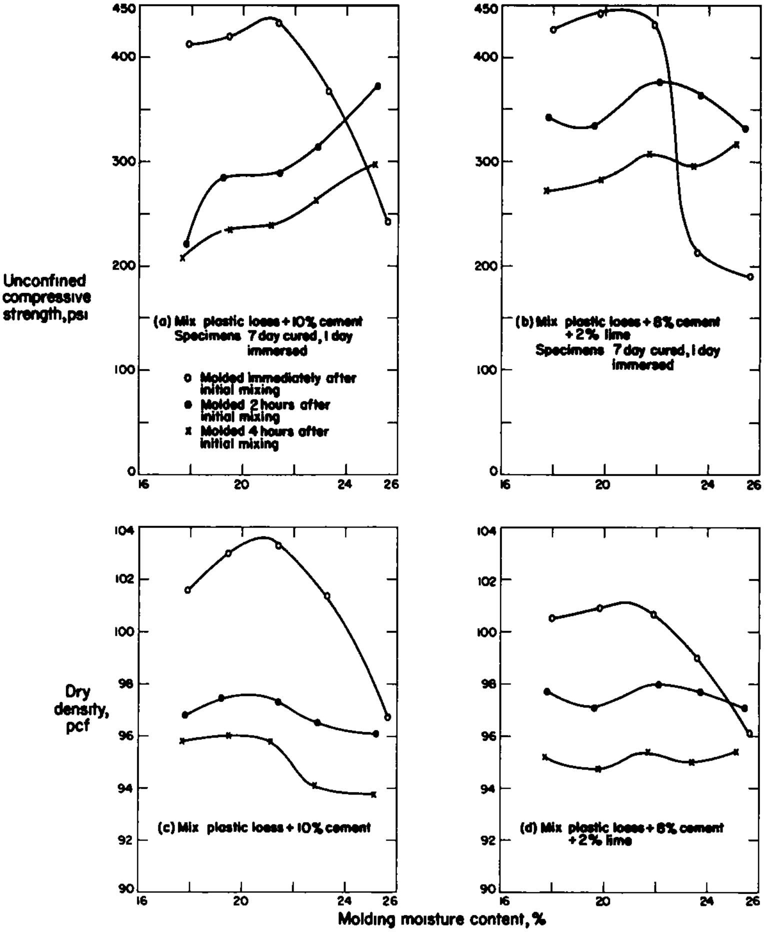

Figure 7. Unconfined compressive strengths and dry densities of plastic loess with 10 percent cement and with 8 percent cement plus 2 percent lime, at varying molding moisture contents; specimens being molded immediately after initial mixing, and after 2 and 4 hr of prolonged mixing.

mixing and then after several intervals of time within a 2-hr period. These patterns showed that the basal distance d of the clay remained constant during this period, being approximately 19.5 Å. This observation eliminates the hypothesis of the adsorption of additional water by the clay mineral with time; therefore, it cannot be said that the water-cement ratio decreases with attendant increase in strength.

Following are some visual observations during the performance of the tests and their explanations relative to the increase of strength.

When mixtures with moisture contents above the optimum were compacted immediately after mixing, horizontal cracks appeared in the specimens as a result of the high compactive energy for that particular moisture condition. Molding of specimens in transparent plastic molds showed that these cracks occur during compaction and not during the removal of the specimens from the mold. These local shear failures could be responsible for the low strengths of these specimens. After 2 hr of intermittent mixing, the reactions of the soil-lime-cement-water system caused an apparent drying out of the mixture; and crack-free specimens were obtained, although the moisture content did not change. This explanation seems satisfactory for both mixtures of the friable loess and also for the plastic loess-lime-cement mixture,

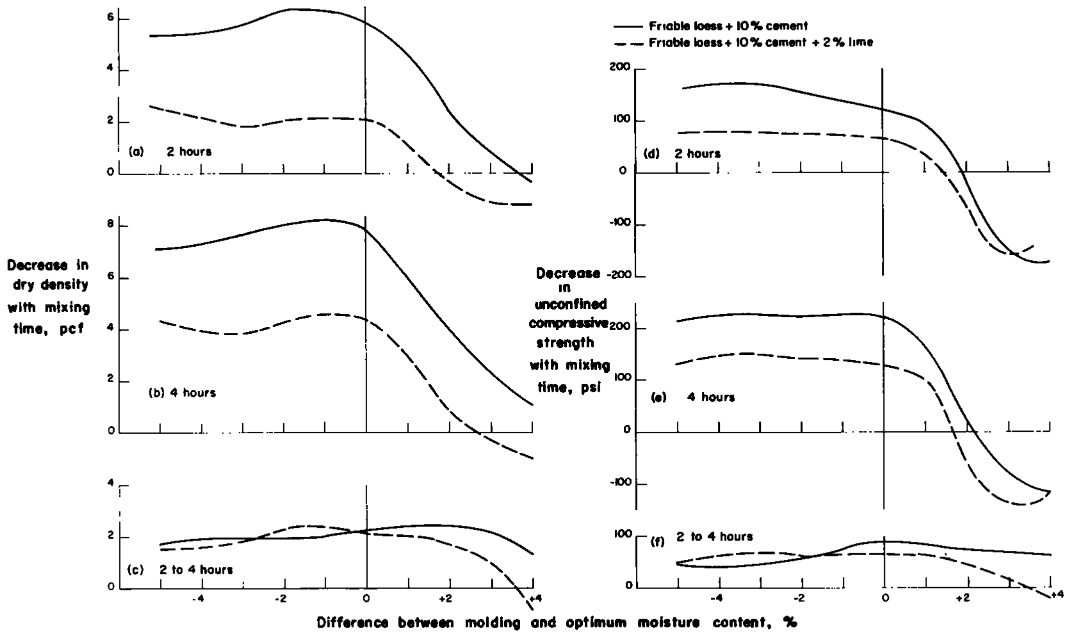

Figure 8. Decrease of unconfined compressive strength and dry density of specimens of friable loess with 10 percent cement and with 10 percent cement plus 2 percent lime, at varying molding moisture contents, during first 2 hr and first 4 hr of intermittent mixing, and during the period from 2 to 4 hr after initial mixing.

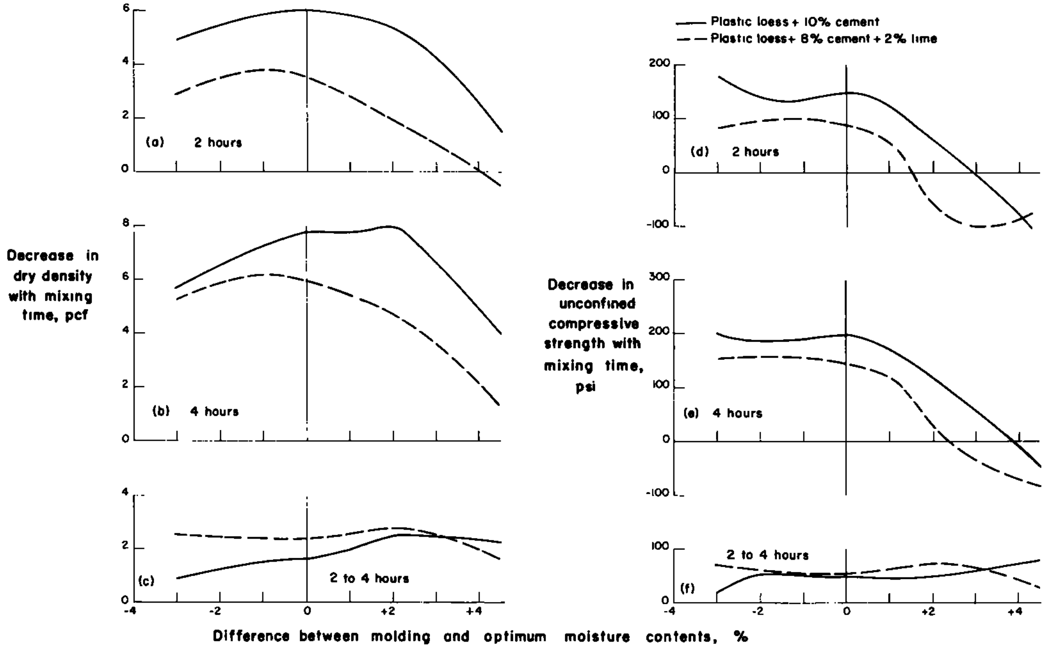

Figure 9. Decrease of unconfined compressive strength and dry density of specimens of plastic loess with 10 percent cement and with 8 percent cement plus 2 percent lime at varying molding moisture contents, during first 2 hr and first 4 hr of intermittent mixing, and during period from 2 to 4 hr after initial mixing.

because the compressive strengths for all of these mixtures after 2 and 4 hr of mixing were approximately proportional to the densities.

Further, in the plastic loess-cement it was observed during the intermittent mixing that small aggregates were formed and became hard after a time. Compaction of this mixture did not completely deform these aggregates, and small air voids in the specimens were visible. Strong aggregation did not take place in all mixtures of friable loess or when lime was added to the plastic loess, but formation of aggregates was observed in all clayey soils with cement. The hardness of the aggregates decreased with increase of moisture content; thus, for high moisture contents the aggregates were deformed much more during compaction and therefore a larger area of contact was attained, giving rise to higher strengths.

Flocculation Studies

When the square root of the rate of sedimentation, which is proportional to the floc diameter, is plotted vs the amount of lime or cement additive (Fig. 10), the rate of sedimentation shows an increase with increase in lime content for all soils, but tends to flatten out above a certain lime content. The leveling of the curves is gradual; therefore, the optimum point is not very distinct. However, this lime content can be roughly estimated to be 1 percent for the friable loess, 2 percent for the leached Kansan till, and 3.5 percent for the gumbotil.

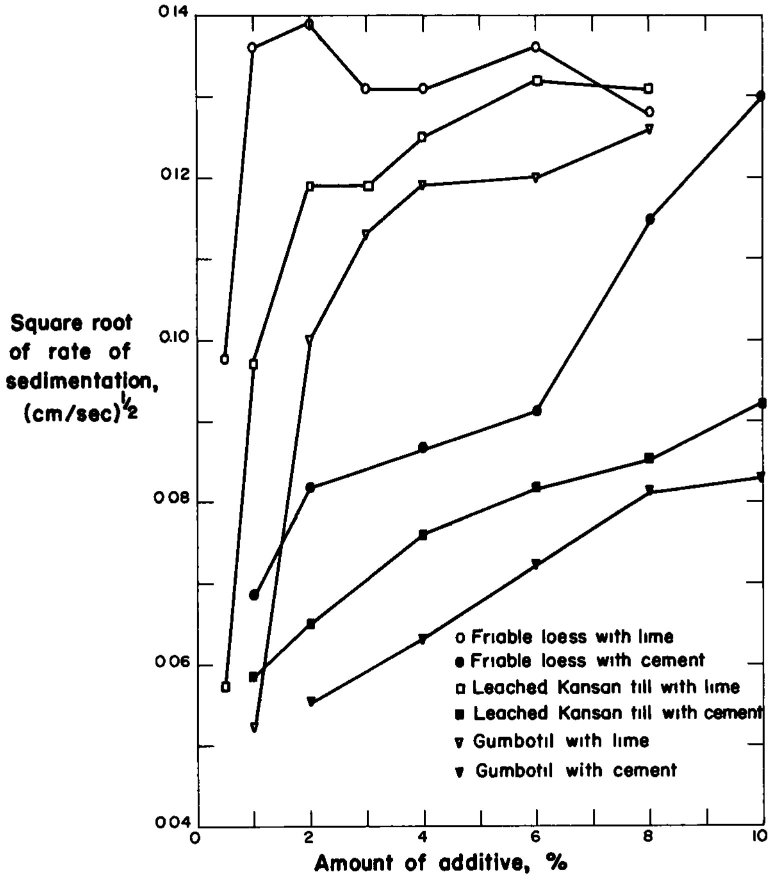

Figure 10. Square root of rate of sedimentation of friable loess, leached Kansan till, and gumbotil flocculated by addition of lime or cement, at varying additive contents.

The effect of lime is similar for both the sedimentation tests and the plastic limit tests. In both, the addition of lime produces a gradual change in results until an optimum lime content is reached beyond which further addition of lime does not seem to produce a further influence. The results of sedimentation tests agree both qualitatively and quantitatively with some recent studies on the plastic limit (9). If the rate of sedimentation is a measure of the degree of flocculation of clays, the increase of plastic limit with addition of lime is due, in fact, to the flocculation of the clay.

Flocculation of soil by lime has been stated as being due either to a replacement of the calcium ions for the cations naturally adsorbed on the soil clay or to a crowding of additional cations onto the clay (5). The relative importance of the second mechanism can be brought out by the following calculations:

The gumbotil has a cation exchange capacity of 37.54 meq per 100 g of oven-dry soil. This means that 100 g of gumbotil can adsorb $37.54 \times \frac{40.08}{2 \times 1,000} = 0.75$ g of Ca^{++}.

In this equation 40.08 is the atomic weight of Ca^{++} and 2 its valence.

On the other hand, the amount of Ca^{++} in 100 g of $Ca(OH)_2$ is $\frac{40.08}{74.10} = 54.1$ g, in which 74.10 is the molecular weight of $Ca(OH)_2$.

Assuming that no Ca^{++} is adsorbed on the clay in the natural gumbotil, the amount of $Ca(OH)_2$ for a complete replacement of adsorbed ions, per 100 g of the soil, would be $\frac{0.75 \times 100}{54.1} = 1.38$ g.

Because the commercial lime used in this investigation had 97.5 percent of $Ca(OH)_2$ as calculated from its chemical analysis considering the molecular weights, it is calculated that $\frac{1.38 \times 100}{97.5} = 1.4$ percent would be the required amount of lime for a complete saturation of the gumbotil with adsorbed Ca^{++} cations. Actually this value must be even smaller, because some Ca^{++} is probably present among the adsorbed cations of the soil. However, this soil shows a complete flocculation, determined by the sedimentation test, or a constant increase of plastic limit (9) only for amounts of lime in excess of 3.5 percent. This indicates that of the two mechanisms suggested, the crowding of additional calcium cations onto the clay requires a larger amount of lime than the amount required by the exchange reactions. The latter, however, probably takes place before the former does.

Cement and lime cause a similar flocculation of soil; the Ca^{++} cations are furnished mainly by the $Ca(OH)_2$ resulting from the hydrolysis of the tricalcium aluminate. However, as the percent of cement added is increased, it produces a much slower increase of the rate of sedimentation for the friable loess equivalent to that achieved with 1 percent of lime (Fig. 10). A similar difference in the effectiveness of lime and cement on the plastic limit has been reported (13).

Initial Hardening Studies

Results of unimmersed unconfined compressive strength determinations within the interval of 24 hr after the addition of water to the mixture are presented as a function of curing time (Fig. 11 a-c). Results for different mixtures should not be quantitatively compared because the test specimens of each mixture were tested at their molding moisture contents which were near the optimum moisture content, and the strength of unimmersed specimens is highly dependent on moisture content.

The most interesting aspect of this study was the results obtained within the first two hr, as shown in Figure 11 d-f. The effect of lime in delaying the hardening of the mixture was noticeable in all soils. This effect took place mainly during the first 30 min, after which, mixtures with the optimum amount of lime showed a more pronounced gain of strength than those without lime or with an excess of lime.

Final Remarks

The effect of lime on the flocculation of clays has been demonstrated by the sedimentation tests. Flocculation resulted in easier workability of the mixtures containing

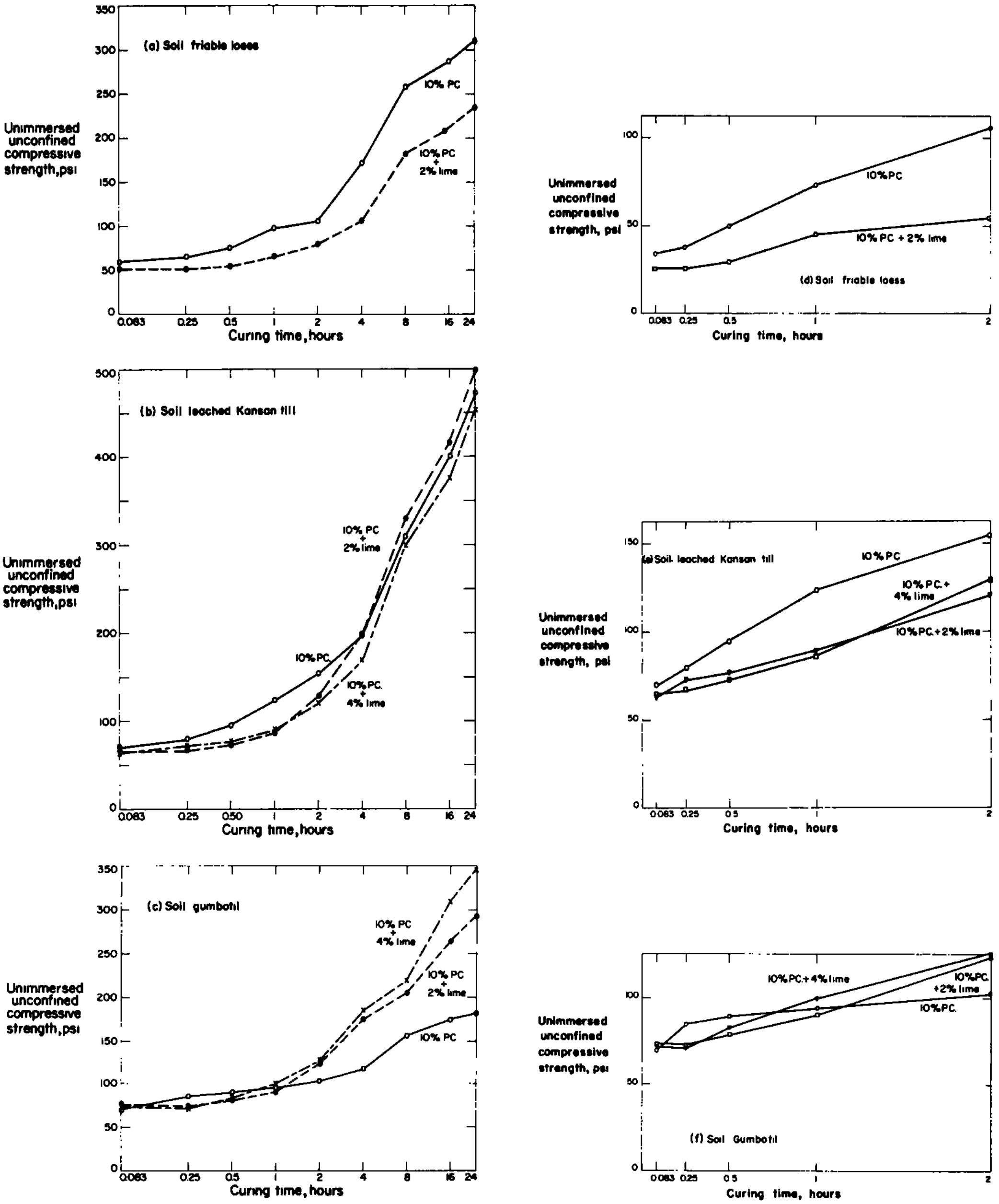

Figure 11. Unimmersed unconfined compressive strengths of friable loess, leached Kansan till, and gumbotil with cement and with cement plus lime, after varying periods of moist cure.

lime, which could be observed during the preparation of specimens from plastic soils mixtures. Though mixtures with lime had a rather "soft" texture, mixtures with only cement formed aggregates and tended to stick to the mixing apparatus. This was even more pronounced for mixtures tested at moisture contents above the optimum moisture content for maximum density.

The effect of lime in facilitating mixing should be even more noticeable in field construction of soil-cement than in the laboratory, where a uniform distribution of the cement within the already pulverized soil is obtained by the initial dry mixing. In the field, addition of lime before the cement should facilitate the achievement of the specified degree of pulverization, often one of the most difficult and costly phases of construction.

The amount of lime required for complete flocculation varies with the amount of clay in the soil and probably with other soil characteristics, such as the amount of carbonates present. Results of the sedimentation tests indicate that complete flocculation of montmorillonitic clay soils is achieved with the addition of approximately 0.6 percent of lime for each 10 percent of clay content below 0.002 mm in size. Observations during the preparation of laboratory specimens, however, indicated that equally workable mixtures are obtained with amounts of lime less than those indicated by this relationship; soils that were completely flocculated with 2 percent lime showed practically the same ease of mixing when either 1 or 2 percent of lime had been added.

The optimum amount of lime for maximum strength is well correlated with the clay content—the higher the clay content, the higher the optimum amount of lime. However, the optimum amount of lime for strength in mixtures of soil-lime-cement is equal neither to the required amount of lime for complete flocculation nor to the lime fixation capacity to which the amount of lime required for flocculation is very close. Although lime was not beneficial for the friable loess, which flocculates with 1 percent of lime, the optimum amount of lime for strength for the gumbotil, which completely flocculates with 3.5 percent of lime, was found to be 6 percent. Soils containing 30 to 40 percent of clay finer than 0.002 mm have an optimum amount of lime that is about equal to the amount causing complete flocculation, both amounts being close to 2 percent.

The reactions of either portland cement or lime with moist soils are complex and not yet completely understood; the simultaneous addition of lime and cement to soils makes the problem even more complex. However, an attempt is made to clarify some phases of the reactions on the basis of the results of this investigation and known hydration reactions of portland cement.

When portland cement comes in contact with water, the cement components undergo a hydrolysis reactions: a supersaturated solution is formed from which the excess solids precipitate as a complex combination of gels and crystalline hydrates. The major hydration products are hydrated basic calcium silicate, calcium aluminate, and calcium hydroxide, the first two being responsible for most of the strength-gaining properties of the portland cement. The calcium hydroxide, which has a low solubility, provides the saturated solution required for the formation of alumina, and of silicate gels with high Ca:Si ratios. Excess of lime is deposited as a separate crystalline solid phase (1).

In the cementation that takes place in a soil-cement mixture, the minerals are not only mechanically bonded to the cement but also react chemically with it (8). The finer the soil is, the more surface area that is available, and the more extensive the chemical reaction may become.

One of the reactions of soils with the cement compounds is an adsorption of ions to satisfy bonding energies unshielded by either polarization of surface ions or a deeper structural screening (8). The bonds developed between the mineral surface and the gel reduce the unbalance of forces at the mineral surface, with a resultant improvement of the chemical bonding. Although hydroxyl ions for this reaction are available since the beginning of the process, the increase of strength is gradual over a period of time. It has been suggested that some silicates, such as quartz and feldspars, mainly the first, respond to this chemisorption more effectively than clay minerals which are flakes too thin to present satisfactory structural screening. However, the clay minerals, by their affinity for adsorbing cations, influence the hydroxyl concentration of the solution during cement hydrolysis, and this influence is felt almost immediately. The sedimentation tests in this investigation were performed approximately 15 min after the addition of cement. Flocculation of the clay had occurred during this interval, as a result of the exchange of adsorbed cations and the concentration of Ca^{++} in the clay micelle.

The attraction of Ca^{++} cations by the clay minerals present in soil-cement mixtures causes a modification of the normal process of hydrolysis of portland cement in water, a modification probably similar to that caused by organic matter. One of the consequences may be that the solution is no longer saturated with $Ca(OH)_2$, a condition under which the precipitation of the calcium aluminates and silicates normally takes place. The chemical interference of clay in the hydration of cement may account, at least partially, for the low strength attained by high-clay soils when mixed with portland cement in amounts commonly used in soil-cement.

Small amounts of lime flocculate clay more effectively than cement. In a calcium-satisfied clay, the hydration of the cement can proceed normally. This is shown by the beneficial effect of adding small amounts of lime to clayey soil-cement, and it is helpful in explaining the increase of the optimum amount of lime with clay content. Results of the initial hardening studies evidence the effect of lime on the hydrolysis of the cement.

An excess of lime is known to be harmful to the hydration of concrete. The results obtained show the same to be true for soil-cement mixtures, although to a lesser degree. The particles of soil can not be completely surrounded by particles of cement, and there will always be an independent field of action for lime, with the pozzolanic reactions of the lime and soil particles mainly responsible for an increase of strength. The optimum amounts of lime for strength and flocculation do not coincide because of this additional gain of strength. Also, the increase of the optimum amount of lime with time is due to the slow rate of the pozzolanic reactions taking place.

The lack of improvement of friable soil-cement, the friable loess as an example, with even a small addition of lime may be because the amount of Ca^{++} required by the clay mineral is very small and can be supplied by the cement without much interference of hydration.

Another point to be considered in the use of friable loess is the presence of calcium carbonate, a potential source of calcium for the reactions. Although calcite is not soluble enough to supply calcium for the flocculation of the clay, it must have an overall beneficial influence on the reactions, as is indicated by the results obtained with the two Kansan tills. Both had approximately the same gradation and optimum amount of lime. However, the addition of this amount of lime increased the strength of the leached Kansan till-cement mixtures about twice as much as it did the unleached Kansan till-cement mixtures.

The effect of lime in retarding the hydration of the cement, or perhaps better stated, in opposing the accelerated hydration caused by the clay, may explain the results obtained in the study of prolonged mixing. The strength decreased with mixing time, primarily because of the reduction in density, due to the hardening of the aggregates. The addition of lime, by reducing the formation of aggregates and retarding their hardening, decreased the reduction of strength.

Economics of Lime-Cement Treatment

The addition of lime to soil-cement mixtures presents one possible disadvantage and three advantages. The disadvantage is the possible increase of operational cost of incorporating two additives instead of one.

The first advantage is the improved workability of the mixture, with the reduced depreciation of equipment and time required to achieve the specified degree of pulverization.

The second advantage stems from the effect of lime on the mixing period. The strength of the mixture compacted after the field mixing, during which some time is always taken up, is what determines the quality of a base course. Because lime minimizes the loss of strength during this period, either a reduction in the amounts of additives or a prolongation of the mixing time can be allowed when lime is added.

The third advantage concerns the cost of the additives. Because the cost of lime and of cement are practically equal, the total cost of the additives is in proportion to the total amount of additives. Although the mix designs are often based on results of wet-dry and freeze-thaw tests, the following comparisons are made in terms of the compressive strengths on the assumption that strength denotes quality:

1. Addition of lime to friable loess is uneconomical.
2. The plastic loess showed a 7-day strength of 420 psi with either 9 percent cement or 2 percent lime plus 6 percent cement. For the same strength the soil-lime-cement mixture gives a saving of 1 percent of additive material.
3. A strength of 360 psi for the unleached Kansan till after 7-day curing was obtained with 8 percent cement or with 2 percent lime plus 5 percent cement. Again, 1 percent of additive material is saved by the soil-lime-cement mixture.
4. A 7-day strength of 435 psi for the leached Kansan till obtained with 10 percent cement was also obtained with 2 percent lime plus 5 percent cement. Thus, for the same strength a saving of 3 percent additive material is given by the soil-lime-cement mixture.
5. The economic advantage of the simultaneous use of lime and cement for the gumbotil is paramount. Although 10 percent of cement gave a 7-day strength of 130 psi, and 8 percent lime produced a 7-day strength of 140 psi, an admixture of 4 percent lime plus 6 percent cement raised the 7-day strength to a value of 450 psi. These results leave no doubt that the gumbotil, which does not respond to stabilization easily, can be stabilized economically with a well-designed mixture of lime and cement.

CONCLUSIONS

The results of this investigation show that beneficial effects are obtained by the addition of small amounts of lime to soil-cement mixtures, and lead to the following conclusions:

1. Addition of lime increases the compressive strength of soil-cement mixtures containing montmorillonitic clay soils.
2. For each soil-cement mixture there is an optimum amount of lime which gives a maximum compressive strength.
3. The optimum amount of lime is independent of the cement content of the mixture, and increases with increase of clay content in the soil.
4. The increase of strength with addition of lime is, in terms of percentage, higher for soils with high clay content and for soil-cement mixtures with lower cement content.
5. Addition of lime produces a higher increase of strength for noncalcareous soils than for calcareous soils.
6. The pozzolanic reactions, noticeable in some soils, cause an increase in the lime requirements for optimum conditions and a higher gain of strength with prolonged curing periods.
7. Addition of lime minimizes the decrease of both density and compressive strength with the time elapsed between addition of water to the mixture and compaction.
8. Addition of small amounts of lime to clayey soils causes flocculation of the clay and facilitates mixing of cement with such soils.
9. Sedimentation tests show that there is for each soil an amount of lime above which further increase of lime does not cause corresponding increase of flocculation. This amount is nearly the same as the lime fixation capacity of the soil.
10. Soil-cement mixtures harden much faster during the first 30 min after compaction than soil-lime-cement mixtures.

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Strength-Maturity Relations of Soil-Cement Mixtures

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Soil-cement was observed to increase in compressive strength with time of curing with a better than random correlation in both a semi-logarithmic and logarithmic manner. Statistical analysis of 417 sets of data representing ages up to 5 years indicates that for granular soil-cement (A-1, A-2, and A-3 soils) the best relationship is semi-logarithmic. This is similar to the relationship observed in concrete, whose constituents are similar to granular soil-cement. Silty or clayey soil-cement (A-4, A-5, A-6, and A-7 soils), on the other hand, exhibits the closest relationship logarithmically. However, high correlations were observed in both a logarithmic and a semi-logarithmic manner in all soils. These correlations were found to exist, independently of changes in cement content, time of curing, temperature of curing, and immersion of test specimen before testing.

These relationships can be used to predict the compressive strength of soil-cement at a future time of curing. The accurate prediction of future compressive strengths can be useful in decreasing the cement content of certain soil-cements, and as a basis for periodically increasing maximum soil-cement road or airfield load capacities. The strength-age relationship of a soil-cement can be determined from data obtained from standard laboratory tests. Statistical analysis of 120 sets of data indicates that a semi-logarithmic plot tends, on the whole, to predict future compressive strengths best for all soil types. Compressive strength can be evaluated both graphically and by equation.

The slope of this strength-age relationship was found to be affected by the physical and chemical properties of the soil, the cement content, and certain chemical additives. Thus, it is evident that the slope of this strength-age relationship is an excellent indicator of the quality of a particular soil-cement mixture. The effect of additives can also be better evaluated by using this relationship. An additive which raises the slope of this strength-age relationship will, in the long run, produce much higher compressive strengths than an additive that merely raises the relationship in a parallel manner.

It is believed that further investigation would indicate a correlation between the slope of the strength-age relationship and the durability of soil-cement mixtures.

•THE ADDITION of portland cement to soil changes the properties and structure of the soil. Probably all soils can be adequately stabilized with cement; however, such stabilization is not always feasible. The compressive strength of soil-cement mixtures will increase with time of curing as does that of concrete. Soils with similar physical properties but different chemical properties may vary widely in compressive strength when stabilized with equal amounts of cement. Thus the interaction of cement with the chemical constituents of the soil can have a major affect on the compressive strength of soil-cement mixtures.

There is no simple way to determine the amount of this interaction or the resulting compressive strength without laboratory tests. Compressive strength and durability tests are used to determine the quality of a soil-cement mixture for a particular construction project. A more complete knowledge of the interaction of soil and cement would help to understand the mechanism of strength development in soil-cement mixtures better.

Although soil-cement is designed on the basis of durability or strength after 7 days of curing, it is recognized that the strength increases with age, probably providing a safety factor in the design. Some soil-cement mixtures, however, develop a strength of over three times the original 7-day strength after an additional interval of curing. If this additional strength could be predicted accurately, more economic mix designs would be forthcoming. Besides benefiting economy, a lower cement content may decrease shrinkage and the resulting cracking of soil-cement pavements. A knowledge of the rate of increase in strength of soil-cement in a road or airfield pavement also might serve as the basis for periodically increasing the maximum load on the pavement.

The objectives of this investigation were threefold: (a) to observe any correlation between the compressive strength of soil-cement mixtures and the time of curing; (b) to determine whether soil-cement mixtures gain strength predictably with time of curing; and (c) to gain an insight into the factors that influence the gain in strength with time of curing of soil-cement mixtures.

Because soil-cement and concrete are basically similar in nature and composition, a review of the strength-maturity relationships found for concrete was made. One extensive investigation indicates that there is a semi-logarithmic relationship between the compressive strength and the maturity of the concrete (9). The maturity is a function of the age and the curing temperature of the concrete.

The strength-maturity relationships of soil-cement were investigated for a curing period up to twelve weeks, using mixtures prepared with five soils (8). These soils ranged from sandy gravel to silty clay and were stabilized with various amounts of portland cement. It was found that the unconfined compressive strengths of the stabilized soils are linear with respect to the logarithm of the curing time. This relationship for soil-cement is similar to that obtained for concrete.

STRENGTH-AGE RELATIONSHIP

To investigate soil-cement for long-term strength prediction, it is first necessary to show that the compressive strength can be related to the curing period in a linear manner. More specifically, by knowing the initial characteristics of the strength-age relationship of the soil-cement, one may be able to determine what the future relationship will be.

The method of plotting strength against the logarithm of time was first investigated with long-term strengths. Soil-cement samples cured for a period of 5 years by the Portland Cement Association (11) were used (Fig. 1). The curves represent various soil types treated with portland cement. It is seen that the A-1, A-2, and A-4 soil-cements produce an excellent linear relationship for the period of curing. The relationships for the A-6 and A-7 soil cements, however, show a marked divergence from a straight line at long curing intervals.

Further investigation to seek a linear relationship for the A-6 and A-7 soil-cements resulted in a plot of the logarithm of strength against the logarithm of time (Fig. 2). The A-4, A-6, and A-7 soil-cements produced a good linear relationship, whereas the A-1 and A-2 soil-cements showed a poor relationship.

It appears from this preliminary investigation that A-1 and A-2 (granular) soil-cement will produce a linear relationship on a semi-logarithmic scale. A-6 and A-7 (silty and clayey) soil-cement will produce a linear relationship on a logarithmic scale. A-4 soil-

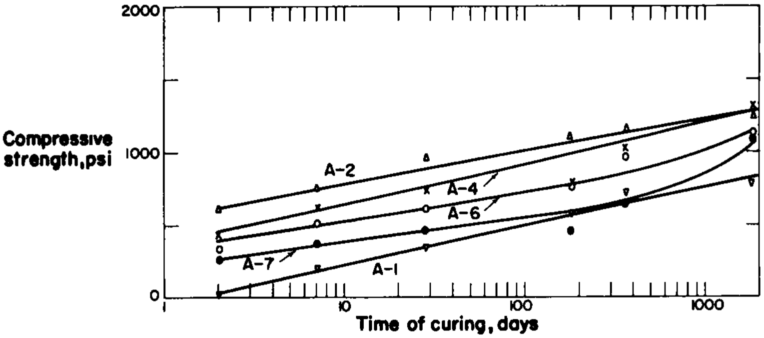

Figure 1. Effect of curing time on unconfined compressive strength of soil-cement mixtures.

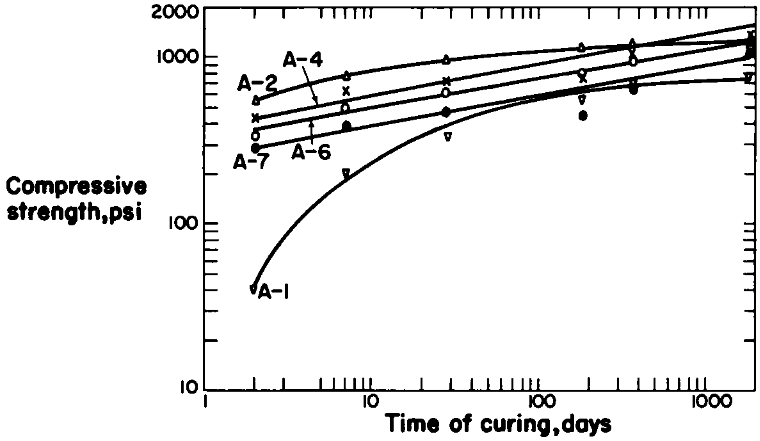

Figure 2. Effect of curing time on unconfined compressive strength of soil-cement mixtures.

cement, however, appears to behave in a transitional manner between granular soil-cement and fine-grained soil-cement.

The form for the linear relationship on the semi-logarithmic plot is

$$S = A + B \log T \qquad (1)$$

in which

S = strength;
T = time of curing; and
A, B = constants.

The form for the logarithmic plot is:

$$\log S = \log A + B \log T \qquad (2)$$

With a working hypothesis from this preliminary investigation, it seemed logical to continue by seeing if this hypothesis could be considered equally applicable to all soils. The best way to do this would be by a methodical statistical analysis with various amounts of cement tested at various curing intervals.

An equation incorporating both plots was applied, to find out whether the semi-logarithmic, the logarithmic, or a combination of the two plots would be best for a set of data:

$$\log T = A_1 + B_1 \left[pS + (1-p) \log S\right] \tag{3}$$

in which p is a parameter that can take on any value between 0 and 1. It is evident that when p = 0, the equation is of the logarithmic form; when p = 1, the equation is semi-logarithmic. Intermediate values of p would indicate a specific combination of the two equations. It was found that either logarithmic, semi-logarithmic or a combination of the two will produce a good linear relationship. However, the best relationship occurs either at p = 0 (logarithmic) or p = 1 (semi-logarithmic) (1).

The next set of computations was used to determine whether the logarithmic or semi-logarithmic equation would produce the best correlation for a particular soil type. The results agreed with the initial graphical conclusions. A relative comparison of the average correlation values offered further evidence for the validity of the graphical conclusions (1).

From the foregoing graphical and statistical investigation it appears that the strength of granular soil-cement correlates best with time of curing in a semi-logarithmic manner, but the strength of silty or clayey soil-cement correlates best with time of curing in a logarithmic manner. Although data for A-5 soil-cements were not available, it seems logical to assume that the increase in strength with time of curing would also be logarithmic.

An over-all evaluation of the degree of correlation observed for semi-logarithmic and logarithmic relationships for 417 sets of data indicates that high correlations exist in both a semi-logarithmic and logarithmic manner for all soil-cements (1).

PREDICTION OF UNCONFINED COMPRESSIVE STRENGTH

Because a definite correlation for the increase in strength with time of curing for soil-cement has been established, a method of prediction should be easily adaptable. Logically, the type of prediction equations should follow the previous conclusions; namely, that granular soil-cement would be semi-logarithmic, and fine-grained soil-cement would be logarithmic. However, due to the high correlations found both logarithmically and semi-logarithmically for all soil types, this should be further investigated.

A reasonable test for predictability which is in keeping with the manner in which prediction would be carried out in practice is as follows:

1. The equation for strength is determined by using the first few points (strength values for brief curing intervals) of a set of data.
2. The strength at a future time of curing is predicted with the use of this equation.
3. The predicted strength is compared with the actual strength at that given time.

The points used to determine the equation of each set of data were the strength values at the three curing times corresponding most closely to 2, 7, and 28 days. These are the curing times used by the Portland Cement Association (10) in standard tests on soil-cement mixtures. By the use of these equations, the last two points (strength values for long curing intervals) in each set of data were computed and compared with the actual strengths at this time of curing. The equation, whether semi-logarithmic or logarithmic, that gave the closest prediction was considered best for that particular soil.

The results of this investigation indicated that most predicted strengths are close to the actual strengths, even up to prediction periods of 5 years (Table 1). This was accomplished from equations derived, for the most part, from the 2-, 7-, and 28-day strengths of each set of data. Inasmuch as both logarithmic and semi-logarithmic equations seem to give similar results, a detailed analysis of this was considered to determine which type of equation would best predict future compressive strengths.

TABLE 1

COMPARISON OF ACTUAL AND PREDICTED COMPRESSIVE STRENGTHS OF SOIL-CEMENT BY LOGARITHMIC AND SEMI-LOGARITHMIC EQUATIONS

Soil Type	Time of Curing (days)	Actual Strength (psi)	Predicted Strength (psi)	
			Log	Semi-Log
A-1	365	1,160	1,802	1,375
	1,825	1,380	2,657	1,643
	365	2,525	3,609	2,615
	1,825	2,950	5,501	3,135
	365	150	125	89
	1,825	170	215	111
	365	380	7,030	919
	1,825	520	45,234	1,219
	365	710	5,767	672
	1,825	770	28,349	870
A-2	28	300	319	217
	56	350	544	259
	28	450	510	411
	56	500	713	475
	28	475	477	419
	56	625	603	475
	28	615	687	614
	56	800	867	696
	120	560	792	525
	365	620	1,323	637
	120	1,775	3,329	2,014
	365	1,920	5,964	2,460
	120	2,650	5,410	3,501
	365	3,550	9,176	4,247
	120	3,400	6,034	4,045
	365	3,750	9,922	4,884
	120	4,000	660	4,734
	365	4,150	1,019	5,655
	120	4,050	7,647	5,263
	365	4,650	12,352	6,345
	365	1,540	1,933	1,453
	1,825	1,650	2,908	1,746
	365	1,140	1,452	1,272
	1,825	1,230	1,893	1,477
	365	1,840	2,303	1,991
	1,825	2,470	3,044	2,320
A-3	60	483	944	662
	90	507	1,344	761
	60	774	1,286	889
	90	965	1,816	1,021
	60	881	1,087	996
	90	1,066	1,256	1,089
A-4	365	490	479	432
	1,825	630	589	488
	365	1,020	1,292	1,049
	1,825	1,300	1,800	1,237
	120	430	515	476
	120	725	906	723
	365	960	1,307	858
	120	1,100	885	853

TABLE 1 (continued)

Soil Type	Time of Curing (days)	Actual Strength (psi)	Predicted Strength (psi)	
			Log	Semi-Log
A-4 (continued)	365	1,600	1,030	949
	120	1,400	1,212	1,067
	365	2,000	1,569	1,231
	120	1,960	1,918	1,612
	365	2,850	2,611	1,886
	120	1,920	1,498	1,351
	365	2,100	1,895	1,549
	120	2,525	2,198	1,929
	365	3,100	2,907	2,247
	60	635	957	699
	300	1,049	2,119	962
	200	1,460	1,481	1,242
	600	1,910	2,323	1,556
	365	2,220	2,381	1,354
	600	2,540	5,315	1,774
	365	180	204	204
	1,825	425	211	209
	365	760	379	371
	1,825	1,060	414	400
A-6	365	450	568	503
	1,825	460	734	582
	365	960	1,114	875
	1,825	1,090	1,600	1,038
	365	1,215	1,804	1,353
	1,825	1,800	2,742	1,631
	28	425	573	528
A-6	56	500	692	591
	28	560	581	554
	56	625	666	607
	28	625	687	649
	56	750	803	717
	28	900	883	844
	56	1,050	1,017	927
	60	461	479	439
	90	486	530	467
	60	573	610	590
	90	694	652	619
	60	861	732	724
	90	905	771	752
	8	875	966	960
	14	900	1,091	1,056
A-7	28	285	282	274
	56	300	318	298
	28	340	346	328
	56	380	400	360
	28	525	499	471
	56	550	580	519
	120	330	345	312
	120	730	1,046	793
	365	925	1,630	958
	120	1,200	1,309	1,080

TABLE 1 (continued)

Soil Type	Time of Curing (days)	Actual Strength (psi)	Predicted Strength (psi)	
			Log	Semi-Log
A-7 (continued)	365	1,525	1,850	1,279
	120	1,375	1,696	1,355
	365	2,000	2,554	1,632
	120	1,475	1,820	1,434
	365	2,100	2,706	1,721
	120	1,725	2,110	1,676
	365	2,300	3,065	1,993
	120	1,825	2,346	1,713
	365	2,050	3,726	2,073
	365	640	772	654
	1,825	1,080	1,043	767
	60	254	326	293
	90	293	380	319
	60	422	404	394
A-7	90	434	433	415
	60	657	631	609
	90	699	686	647
	21	610	673	650
	28	700	728	688

A comparison of the actual and predicted soil-cement strengths for each of the 120 sets of data used indicated a definite preference for a semi-logarithmic plot for all soil types except A-4. The percent of error in strength prediction was lowest for the semi-logarithmic plot for all soil types. Also, the semi-logarithmic plot tended to predict strength on the safe side of (below) the actual strength, but the logarithmic plot predicted strengths on the unsafe side of (above) the actual strength (1).

Further evidence for favoring semi-logarithmic plotting was adduced as follows. A statistical test (7) of the hypothesis that the linear relationship for brief curing intervals if valid also for the longest available curing interval was performed. This test, conducted both for the semi-logarithmic and logarithmic relationships, showed the previous hypothesis to be more tenable in the semi-logarithmic case (1).

Thus, from the foregoing investigations, it is apparent that the best equation to use for predictability would be one of the semi-logarithmic type. Although there is some doubt about A-4 and A-5 soil-cements, because the soil types on both sides (granular and clayey) exhibit a definite preference for a semi-logarithmic equation, it should be safe to assume that they could also be predicted best semi-logarithmically.

Before practical use can be made of this method of prediction, a margin of error should be determined to predict safely compressive strengths. The use of a percent error was found not to be subject to any simple pattern. However, because the statistical test shows the plausibility of a single linear relation valid over all curing intervals, it is reasonable to explore the possibility that, for a given soil type, prediction error is proportional to the difference between the logarithm of the median of the curing times on the basis of which the prediction equation is computed (Fig. 3). This proportionality was found to apply. In addition, the proportionality factors for various soil types proved to be similar enough to suggest the use of the average proportionality factor, of approximately 80 psi for all soil types (1). An approximate expression for expected discrepancy is therefore given by

$$S_1 - S_2 = 80 \log \left(\frac{T}{7}\right) \qquad (4)$$

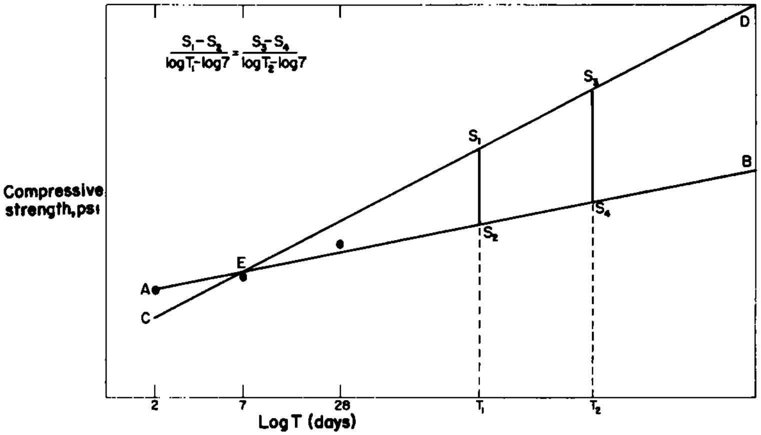

Figure 3. Determination of experimental error expected from use of semi-logarithmic prediction equation.

in which

$S_1 - S_2$ = difference between actual and predicted strength (psi); and

T = time of curing at which strength is to be predicted (days).

SUGGESTED PREDICTION PROCEDURE

The following procedure would be used to predict what the future strength of soil cement would be. The strength is plotted against the logarithm of the time of curing. At least two points must be plotted, although the more points plotted the more accurate will be the predicted strengths. The best straight line is drawn through these points and projected into the future time and strength of the graph. This line would be the strength-age line, from which strengths corresponding with future curing times can be evaluated. This line can be expressed by

$$S = A + B \log (T/7) \tag{5}$$

The constant A is evaluated as the strength at which the line begins, in this case taken as the 7-day strength. The constant B is the slope of the line. The time of curing, T, is the time at which the strength is to be predicted.

The expected discrepancy may be incorporated into the equation to determine the range wherein the actual strength would be expected to lie. Thus, the prediction equation would be

$$S = A + (B \pm 80) \log (T/7) \tag{6}$$

If the strength of a soil-cement mixture at 7 days is 250 psi and the slope from the graph is determined to be 400: $S = 250 + (400 \pm 80) \log (T/7)$. If the strength at 1 year (365 days) is to be evaluated, the solution would be $S = 250 + (400 \pm 80) \log (365/7)$; $S = 938 \pm 138$ psi. Therefore, the actual strength would probably lie somewhere between 800 and 1,076 psi. The data for this investigation were taken from the reports on several researches (2, 3, 4, 5, 11).

The relationships observed, in view of various experimental conditions inherent in the sources, should be generally applicable to all soil-cement mixtures. The various methods of testing indicate that the relationships are linear, independently of changes in (a) time of curing up to 5 years, (b) curing temperature, (c) cement content, (d) size of test specimen, (e) type of soil, or (f) immersion of test specimen before testing. Provided the curing conditions are kept constant, the unconfined compressive strength of a soil-cement mixture can be predicted for periods up to 5 years with reasonable and estimable precision.

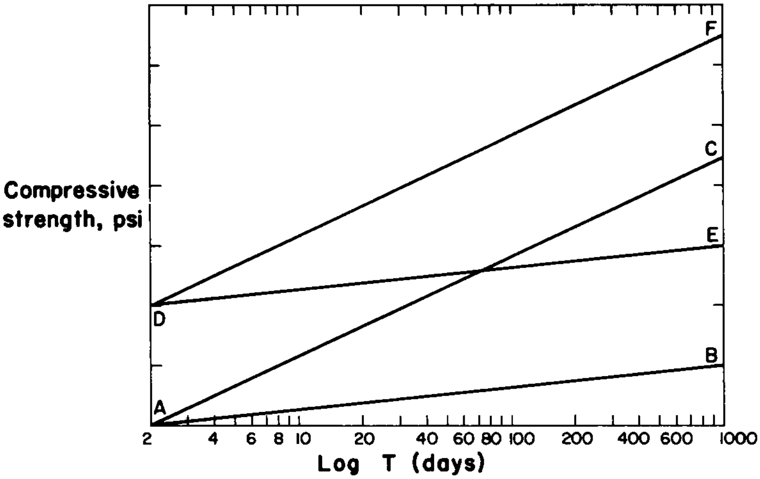

Figure 4. Effect of an additive on strength-age relationship of a soil-cement mixture.

FACTORS INFLUENCING SLOPE OF STRENGTH-AGE RELATIONSHIP

An experimental investigation of the observed strength-age relationships would be of value. If these relationships could be altered physically or chemically and if the causes of the relationships could be understood, possibly better soil-cement might be obtained at a significantly lower cost.

The most important single feature of the strength-age relationship is the slope of the line, which represents the rate of increase in the compressive strength of the soil-cement mixture with time of curing. An unstabilized soil would have a horizontal slope of zero, and the ultimate in a stabilized soil, academically speaking, would be a vertical slope of infinity.

Between these two extremes is a wide range of values which the slope of a soil-cement relationship can assume. The higher the rate of increase in strength of a soil-cement mixture, the higher the unconfined compressive strength will be. Thus, the slope would affect the 7-day compressive strength of a soil cement mixture, often used as a criterion of quality.

A physical treatment or chemical additive to soil-cement may alter the strength-age relationship in one of three ways (Fig. 4). AB represents the strength-age relationship of a normal soil-cement. The first change that could take place is represented by AC: the slope of the strength-age relationship could be increased, producing much higher compressive strengths than those of the normal soil-cement mixture as the curing interval is increased.

Second, as shown by the DE, the relationship may merely be raised parallel to that of the normal soil-cement mixture. This would be less desirable than the first change, because the compressive strength would be raised by the same constant amount throughout the curing interval. Third, a combination of the first two alterations might result, as shown by the DF. This would be the best achievement, resulting in extremely improved soil-cement mixtures.

Previous empirical testing of the effect of additives on soil-cement by the relative comparison of compressive strengths do not bring out the effect of the strength-age relationship in soil-cement. Indeed, erroneous conclusions may be reached by the comparison of compressive strengths. For example, an additive that produces the relationship of the second case (DE) might be considered much better than an additive that results in the first case (AC) on the basis of early compressive strengths. However, after a longer curing interval the latter soil-cement mixture may attain the highest compressive strengths.

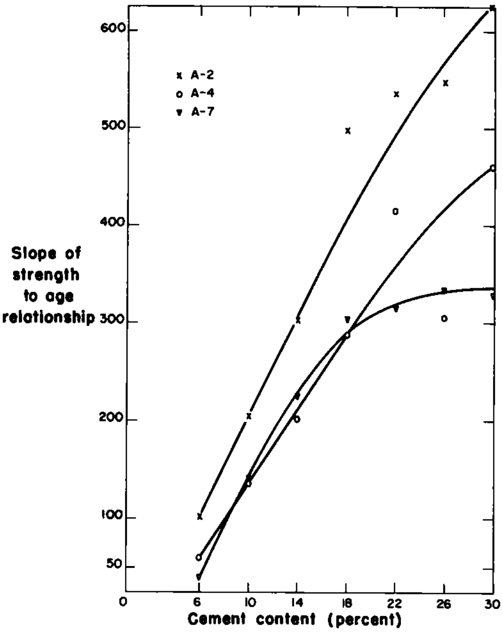

Figure 5. Effect of cement content on slope of strength-age relationship of soil-cement mixtures.

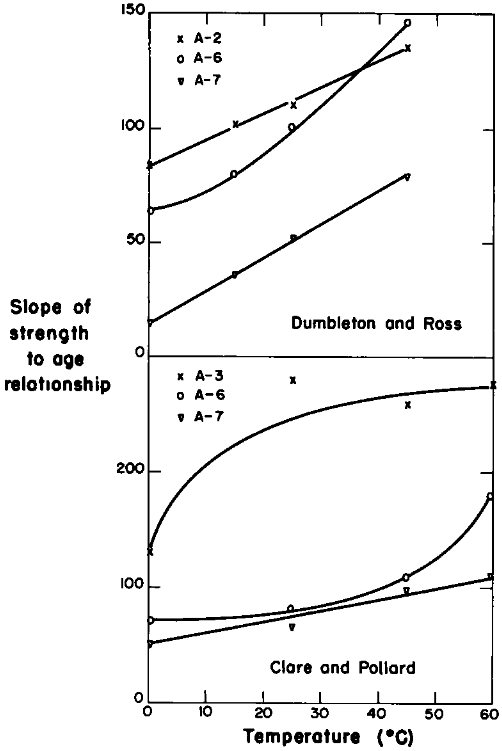

Figure 6. Effect of curing temperature on slope of strength-age relationship of soil-cement mixtures.

Thus it appears that the slope of the strength-age relationship is an excellent indicator of the quality of a soil-cement mixture, and of the beneficial effect of additives. Normally, the compressive strength of a soil-cement mixture can be increased if the cement content is increased. However, if it were possible to increase the slope of the strength-age relationship by the use of an additive or an alteration of the properties of a mixture, an equally high compressive strength might be realized with a lower cement content.

Physical or chemical properties of the soil-cement mixture that influence the slope of the strength-age relationship could be regulated during construction to produce maximum beneficial effects. This might be useful in obtaining a better quality of soil-cement. The following sections will be concerned with the factors that could influence the rate of increase in strength of soil-cement mixtures. The data of Winterkorn, Gibbs, and Fehrman (12), and Lambe and Moh (6) were used in this investigation along with the data from the sources mentioned previously.

Cement Content

Cement, considered the important constituent of soil-cement, is considered first. Plots of the slope of the strength-age relationship for various cement contents of each soil are shown in Figure 5. The slope is low at low cement contents and increases as the cement content is increased. In the range of cement contents feasible for soil-cement is a linear relationship between the amount of cement and the slope of the strength-age relationship. Therefore, as the cement content of a soil-cement mixture is increased, there is a corresponding increase in the rate of increase in strength.

Temperature of Curing

An increase in curing temperature will produce an increase in the compressive

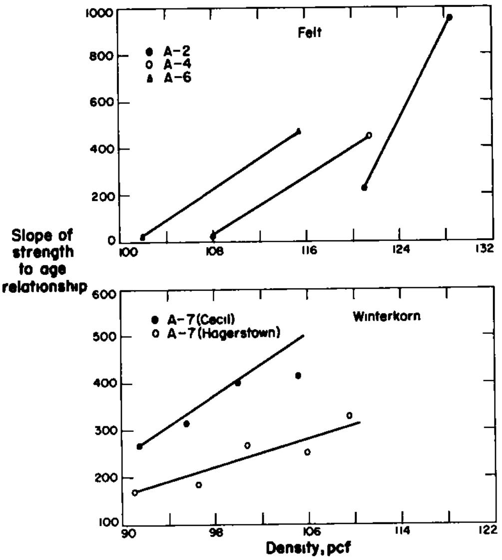

Figure 7. Effect of density on slope of strength-age relationship of soil-cement mixtures.

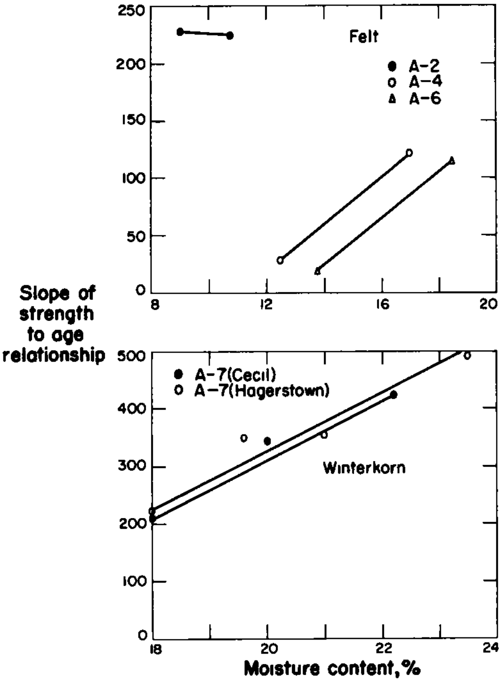

Figure 8. Effect of moisture content on slope of strength-age relationship of soil-cement mixtures.

strength of a soil-cement mixture. Figure 6 shows the effect of curing temperatures on the slope of the strength-age relationships. It can be observed that all data indicate a general increase in the slope of the relationship as the temperature of curing is increased, supporting the tentative conclusion that an increase in the curing temperature causes an increase in the slope of the strength-age relationship.

Density

The density of a compacted soil-cement mixture has an important effect on the compressive strength and durability of the cured soil-cement. The test specimens used in Felt's study were molded at AASHO-modified optimum moisture content to both standard Proctor and modified AASHO maximum density. The data of Winterkorn represents clay soil specimens molded at or near standard optimum moisture content to various densities. Standard Proctor densities were 95.5 pfc for the Cecil clay and 96.5 pcf for the Hagerstown clay.

Figure 7 shows the effect of density on the slope of the strength-age relationship. It can be seen that an increase in the density of a soil-cement mixture will produce an increase in the slope of the strength-age relationship. This is in accordance with the accepted fact that an increase in the density of a soil-cement mixture will generally increase the compressive strength of the mixture.

Moisture Content

The moisture content of a soil-cement mixture is important in obtaining the desired compacted density. Figure 8 shows the effect of moisture content during standard Proctor compaction on the slope of the strength-age relationship. It is apparent from the graph that an increase in the moisture content has a definite effect on the rate of strength increase in the case of the fine grained soils studied. The right points in the plots of Felt's data indicate the optimum moisture content. The standard Proctor optimum moisture contents were 20.5 percent for the Cecil clay and 19.6 percent for the Hagerstown clay. A small increase in the moisture content above optimum might increase the slope of the strength-age relationship for fine-grained soils, thereby increasing the resulting compressive strength of the soil-cement mixture.

Adsorbed Cations

Certain adsorbed cations on clay surfaces are known to benefit the compressive strength of a soil-cement mixture (12). Adsorbed cations may affect the slope of the strength-age relationship in various ways (Table 2). The slope for the Cecil clay increases or decreases depending on the cation. With the Hagerstown clay the slope decreases in all cases. These different effects could be due to the different chemical properties of the two natural soils.

From the foregoing it is evident that cations associated with the soil clay (either natural or artificially introduced) have a definite effect on the slope of the soil-cement strength-age relationship. Further investigation may indicate cations that can be added to certain soils to produce steeper slopes and correspondingly higher compressive strengths.

TABLE 2

EFFECT OF CATIONS ADSORBED BY SOIL CLAY ON THE RATE OF INCREASE IN COMPRESSIVE STRENGTH OF SOIL-CEMENT MIXTURES[a]

Cation	Slope of Strength-Age Relationship	
	Cecil Clay	Hagerstown Clay
Natural	208	349
H	200	100
Na	250	215
K	166	250
Mg	208	166
Ca	316	333
Al	200	133
Fe	316	100

[a] After Winterkorn et al. (12).

Chemical Additives

The effect of chemical additives on the slope of the strength-age relationship may indicate which additives best improve the quality of soil-cement mixtures. Knowing these effects may indicate why some natural soils do not respond well to chemical stabilization. Optimum amounts of chemical additives should be easier to evaluate using the slope of the strength-age relationship rather than making a relative comparison of the compressive strength data.

Table 3 shows the effect of chemical additives on the slope of the strength-age relationship. It can be observed that certain additives, such as Quadrafos, seem to greatly improve the slope of the strength-age relationship. Others, such as $Ca(OH)_2$, give no significant improvement. Thus, after equal curing intervals, Quadrafos additive would be expected to produce much higher compressive strengths than a $Ca(OH)_2$ additive.

TABLE 3

EFFECT OF CHEMICAL ADDITIVES ON RATE OF INCREASE IN COMPRESSIVE STRENGTH OF SOIL-CEMENT MIXTURES, NEW HAMPSHIRE SILT, 5 PERCENT CEMENT[a]

Additive		Slope of Strength-Age Relationship
Type	%	
None		50
$Ca(OH)_2$	0.5	66
	1.0	66
$CaCl_2$	0.5	83
	1.0	83
NaOH	0 5	100
	1.0	166
KCl	0.5	17
	1.0	233
Quadrafos	0.5	150
	1.0	224

[a] After Lambe and Moh (6).

Summary

This investigation was not conducted to produce any definite conclusions concerning the effect of any physical or chemical property on the slope of the strength-age relationship of a soil-cement mixture. Rather it was aimed at showing that the slope of the strength-age relationship is, indeed, influenced by the physical and chemical properties of soil-cement mixtures. Future research in the field of soil-cement stabilization may utilize this fact to good advantage. A more complete understanding of the effect of additives in soil-cement might be realized. Individual natural soils might be evaluated as to their value for soil-cement. Various physical and chemical properties of the soil-cement mixture could be better understood.

A reduction in the normal cement requirement of certain soils should be possible. The desired strength and durability might be obtained by controlling those factors that would produce the necessary strength-age relationship. This could lessen construction costs and widen the usage of cement stabilization.

Because the density and compressive strength of a soil-cement mixture influences the durability of the mixture, it is believed that further investigation will establish a correlation between the slope of the strength-age relationship and the durability of a soil-cement mixture. This might lead to the possibility of eliminating one or more of the durability tests, substituting the slope of the strength-age relationship as a criterion for durability. This slope might be used in conjunction with the 7-day strength to provide an over-all index of the quality of a soil-cement mixture.

CONCLUSIONS

Soil-cement was observed to increase in unconfined compressive strength with time of curing with a better than random correlation in both a semi-logarithmic and logarithmi manner. The best relationship for granular soil-cement is semi-logarithmic; silty and clayey soil-cement exhibit the best relationship logarithmically. These correlations were found to exist, independently of changes in (a) cement content, (b) time of curing up to 5 years, (c) curing temperature, (d) size of test specimen, (e) type of soil, and (f) immersion of test specimen before testing.

These relationships can be used to predict the compressive strength of soil-cement. The strength-age relationship can be determined from data obtained in standard laborator tests. A semi-logarithmic relationship tends best to predict future compressive strength for all soil types. Compressive strength can be predicted both graphically and by equation with a reasonable degree of accuracy.

The slope of the strength-age relationship was found to be affected by the physical and chemical properties of the soil, the cement content, and certain chemical additives. Thus, it is evident that the slope of the strength-age relationship can be used as an indicator of the quality of a soil-cement mixture. Also the effect of additives can be better evaluated by using this relationship.

ACKNOWLEDGMENTS

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Effect of Sulfates on Cement- and Lime-Stabilized Soils

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This paper describes the results of a laboratory investigation made to determine the effect of the presence of sulfate ions in soils on the durability of cement- and lime-stabilized soils. The method of investigation consisted of observing the behavior of specimens of stabilized soil when totally immersed in water. When tested in this way cement- or lime-stabilized clay mixtures containing calcium, magnesium, or sodium sulfates disintegrated within a few days of being immersed, whereas cement-stabilized sand mixtures containing the same proportions of sulfate were unaffected even after being immersed for one year. This suggested that the effect was related to the clay content of the soil, and differential thermal analysis and X-ray analysis showed that, at the high pH values encountered in cement and lime-stabilized soils, a reaction occurs between the clay fraction and any sulfates present.

It is proposed that sulfate attack on cement-stabilized soils is due principally to this reaction and that the relatively slow combination of the sulfates with the cement is of secondary importance. This explains why lime-stabilized clay soils are similarly affected by the presence of sulfates and also why the severity of the action of sulfates cannot be reduced by using sulfate-resistant cement in place of ordinary portland cement.

It is concluded that under certain conditions sulfates present in a cement- or lime-stabilized soil can lead to its disintegration. These conditions are most likely to occur when the soil contains an appreciable clay fraction and when the moisture content of the soil is liable to increase above that at which it was compacted. Because under other circumstances sulfates may appear to have a beneficial effect, it is suggested that it is inadvisable to make use of these beneficial effects unless all possibility of deleterious effects on the durability of the material can be excluded.

•ONE OF THE MAIN problems that arise from the use of cement-stabilized soil as a road base material is the presence of chemical compounds in the soil which can either inhibit the normal hardening of the cement or lead ultimately to a loss of durability of the hardened material.

The effects of organic compounds in preventing the hardening of soil-cement are now well understood but the effects of sulfate ions in the soil are still in some doubt. Experimental work carried out by Lambe and his co-workers (1) has shown that the strength of soil-cement can often be increased by the use of small proportions of metal sulfates as secondary additives. On the other hand, the action of sulfate ions in causing disintegration of hardened concrete would lead one to expect a similar affect with soil-cement. However, the two cases are not entirely analogous, because with concrete sulfate ions external to the concrete are reacting with hardened cement that has been mixed with a sulfate-free aggregate, whereas with soil-cement an additional problem arises from the presence of naturally occurring sulfates within the soil mass. Further, with soil-cement there is the added complication of the presence of clay.

Earlier work by the author (2) suggested that the presence of sulfates in cement-stabilized clay soils would result in a loss of durability under conditions where the soil-cement was subject to an increase of moisture content. Further research has now been carried out on this problem in which the investigation has been extended to soils having different clay contents and to soils stabilized with hydrated lime.

PREVIOUS WORK

Lambe, Michaels, and Moh (1) carried out extensive laboratory studies on the effect of small percentages (of the order of 1 percent) of alkali metal salts as secondary additives to cement-stabilized soils. In this investigation it was found that with many soils, particularly those containing organic matter, the addition of sodium sulfate resulted in a considerable increase in the strength of the stabilized soil, although with a few soils a reduction in strength was observed. Kozan (3) confirmed the ability of small proportions of sodium sulfate to increase the strength of some cement-stabilized soils.

Evidence for the detrimental effect of sulfate ions on the strength of cement-stabilized soil has been given by Mehra Chadda and Kapur (4), Cordon (5), and the author (2). These investigations showed that a loss in strength usually occurred when cement-stabilized soils were immersed in solutions containing sulfate ions or when cement-stabilized, sulfate-bearing soils were immersed in water. The sulfate attack on the soil-cement was more rapid than that on concrete, and Cordon suggested that the speed of reaction might be associated with the fact that soil-cement was less dense, so that diffusion of the sulfate solutions or water could occur more easily. Cordon found that stabilized soils made with coarse-grained soils, high cement contents, and sulfate-resistant cement produced the most resistant material.

Figure 1, which is taken from the author's previous paper (2), shows the effect of immersion in water on the strength of a cement-stabilized clay containing calcium sulfate. The presence of calcium sulfate had little effect on the compressive strength when the stabilized soil was cured at constant moisture content, but the same material suffered a considerable loss in strength when immersed in water. Substitution of sulfate-resistant cement for ordinary portland cement did not reduce the disintegrating effect of the calcium sulfate.

Other tests (Fig. 2), in which cement-stabilized clay, free of sulfate, was immersed in solutions of magnesium sulfate, showed that as little as 0.2 percent (as SO_3) of magnesium sulfate could reduce the strength by more than 50 percent.

An instance has been recorded in South Australia (6) where the presence of calcium sulfate in soil resulted in the disintegration of a soil-cement road.

SCOPE OF PRESENT INVESTIGATION

It is clear from the previous investigations of this problem that different effects can result from the action of sulfate ions on cement-stabilized soils and that the mechanism of the action depends on factors not fully understood. The investigations described in this paper have as their object the elucidation of this mechanism.

In view of the considerable knowledge of the reactions of sulfate ions with the cement in concrete it would be expected that similar reactions would take place with granular soils stabilized with cement and that with stabilized cohesive soils the clay

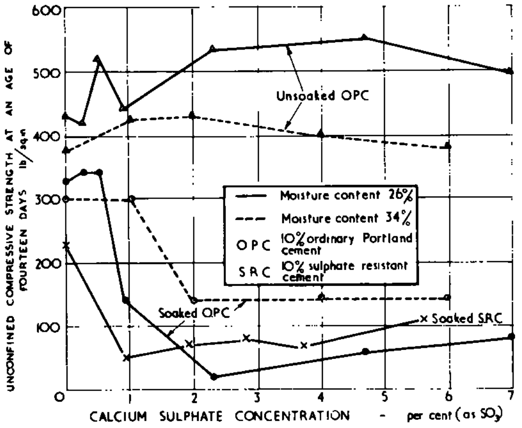

Figure 1. Effect of moisture content and calcium sulfate concentration on strength of clay stabilized with ordinary portland cement and sulfate-resistant cement.

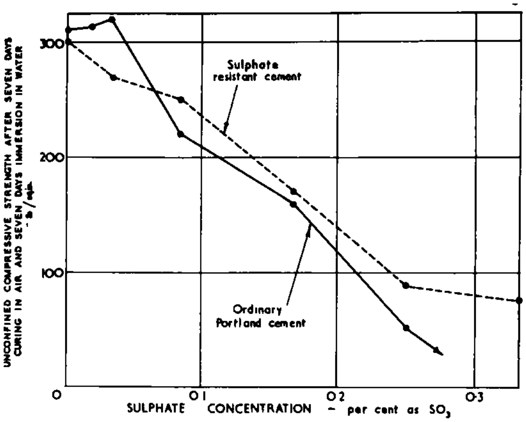

Figure 2. Effect of immersion in magnesium sulfate solution on strength of boulder clay stabilized with ordinary portland cement and sulfate-resistant cement.

fraction would be the principal factor introducing new features in the reactions involved. The primary object of the present investigation was therefore to explore the effect of the clay content of soil on the resistance of the stabilized soil to sulfate attack. Five artificial soils with different clay contents and having the particle-size distributions shown in Figure 3 were used for this study. They were prepared by mixing London clay (clay content 62 percent, liquid limit 80 percent, plastic limit 28 percent as measured by B. S. procedures (7)) and a pure silica sand in proportions (by weight) of 100:0, 75:25, 50:50, 25:75, 0:100.

The investigations involved tests an specimens of these soils, stabilized with cement, some of which contained calcium sulfate and some of which were free of sulfate but subject to external attack by sulfate ions, with the object of differentiating between the effects obtained in the two cases.

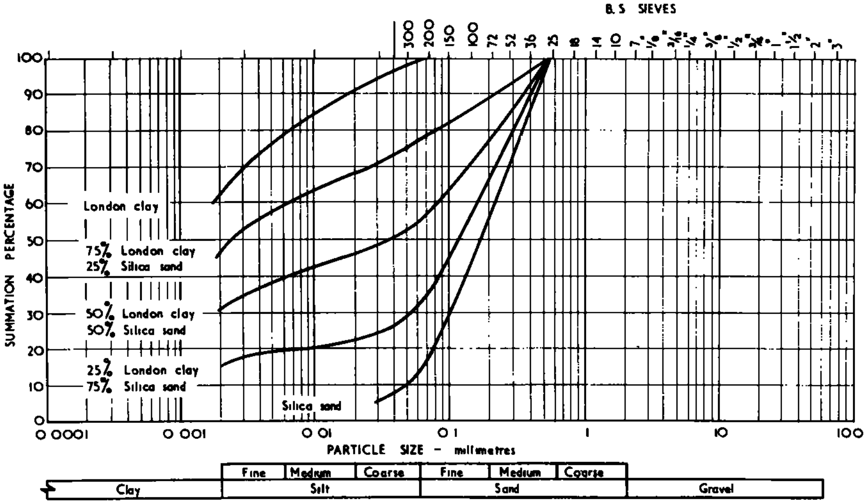

Figure 3. Particle size distributions of soil mixtures prepared from London clay and a silica sand.

Because in cement-stabilized soils chemical reactions take place at high pH values induced by the calcium hydroxide released during the hydration of portland cement, the investigations were subsequently extended to include a study of the resistance of lime-stabilized soil to sulfate attack in the hope that this would help to explain the reactions involved.

The ordinary portland cement used in the investigations had the chemical and physical properties given in Table 1. The hydrated lime that was used was of general reagent grade and contained 92 percent calcium hydroxide. The calcium, magnesium, and sodium sulfates that were used were all of analytical reagent grade.

TEST PROCEDURE

In the author's previous paper (2) it was shown that cement-stabilized soils containing sulfate ions were subject to a reduction in strength only when their moisture contents were increased by the immersion of specimens in water. In practice, however, there will be circumstances in which the stabilized soil does not increase in moisture content, as well as other circumstances in which an increase takes place. It is therefore

TABLE 1

PHYSICAL AND CHEMICAL PROPERTIES OF ORDINARY PORTLAND CEMENT USED

Property	Value (%)
Chemical analysis (%):	
CaO	66.08
SiO_2	22.54
Fe_2O_3	1.91
Al_2O_3	4.39
MgO	0.83
SO_3	2.01
Na_2O	0.17
K_2O	0.35
TiO_2	0.27
P_2O_5	0.24
Mn_2O_3	0.11
Loss on ignition (%):	
0 - 105 C	0.16
105 - 590 C	0.81
590 - 1,000 C	0.52
Calculated compound composition (%):	
$3CaO. SiO_2$	58.0
$2CaO. SiO_2$	20.0
$3CaO. Al_2O_3$	8.6
$4CaO. Al_2O_3. Fe_2O_3$	5.8
$CaSO_4$	3.4
CaO	1.4
Setting time (min):	
Initial	145
Final	275
Specific surface (sq cm/g)	3,790
Specific gravity	3.14
Strength tests on 2.78-in. mortar cubes (lb/sq in.):	
At 3 days	3,240
At 7 days	4,850

apparent that the laboratory test procedure should permit determinations to be made separately, both of the strength of the stabilized soil when cured at constant moisture content and of the loss of strength due to an increase of moisture content. Both these requirements are satisfactorily met by the B. S. resistance to immersion test (8). This test is a modification of the unconfined compressive strength test in which two identical sets of specimens are prepared, kept at constant moisture content by coating them with paraffin wax, and cured at a constant temperature of 25 C. One set of specimens is allowed to cure for 14 days; the other is allowed to cure for only 7 days, after which time the wax coatings are removed and the specimens are completely immersed in water for the next 7 days. The strengths of both sets of specimens are determined at the end of the 14-day period. The resistance to immersion of the stabilized soil (R_i) is given by

$$R_i = \frac{\text{Strength of specimens immersed in water}}{\text{Strength of control specimens}} \times 100$$

In addition to the presence of the sulfate ions, the resistance of cement-stabilized soil to immersion is affected by the texture of the soil; e.g., stabilized sandy soils are much less affected by immersion in water than stabilized clayey soils. It is possible to isolate this effect from the effect of the presence of the sulfate ions, by also determining the resistance to immersion of the stabilized soil in the absence of sulfates; a measure of the effect of the presence of sulfates on the durability of the material is given by

Resistance to sulfate attack =

$$\frac{\text{Resistance to immersion of stabilized soils containing sulfates}}{\text{Resistance to immersion of stabilized soil free from sulfates}} \times 100$$

Although the preceding procedure is suited to the study of the effects of sulfates when they occur in the soil, it has to be modified in studying the effect of immersing stabilized soil in solutions containing sulfate ions. In this case the equation is

Resistance to immersion in sulfate solutions =

$$\frac{\text{Strength of specimens immersed in sulfate solution}}{\text{Strength of specimens immersed in water}} \times 100$$

EXPERIMENTS WITH CEMENT-STABILIZED SOILS

Immersion in Magnesium Sulfate Solution

The previous work showed that sulfate attack was most severe when specimens of cement-stabilized soil were immersed in solutions containing sulfate ions (cf, Figs. 1 and 2) and as a preliminary investigation the effect on the strength of the cement-stabilized sand-clay mixtures of immersion in magnesium sulfate solution was studied.

Sets of specimens of each of the five soil mixtures, stabilized with 10 percent of ordinary portland cement, were prepared at moisture contents corresponding to a value of 2 percent below the plastic limit of each mixture and at densities corresponding to an air content of 5 percent. (Unconfined compressive strength specimens are normally prepared at the optimum moisture content and maximum dry density obtained by the B. S. compaction tests on the stabilized soil. In this instance insufficient quantities of the soil were available for compaction tests to be made; a moisture content of 2 percent below the plastic limit of the stabilized soil has been found to approximate to the optimum moisture content as found in the B. S. compaction test). All the specimens were cured at constant moisture content for 7 days and then immersed in solutions of magnesium sulfate for 7 days. At the end of this period they were removed and measurements made of their unconfined compressive strengths.

The resistance of the specimens to immersion in solutions containing sulfate ions was dependent on the clay content (Fig. 4). Although 0. 5 percent of magnesium sulfate was sufficient to cause the complete disintegration of the stabilized clays, the strength of the stabilized sands was unaffected. Figure 5 shows that the disintegration of the cement-stabilized clay specimen was due to cracking brought about by a large increase in volume; it also shows a white incrustation of magnesium hydroxide that formed on the cement-stabilized sand specimens. It was at first thought that this incrustation on the sand specimens formed an impenetrable skin which prevented ingress of magnesium sulfate; however, as will be seen later, this was not the case.

Effect of Presence of Calcium Sulfates In the Soil

Although it is possible that soil-cement might be attacked by water containing sulfate ions it is more likely that sulfate attack would originate from the presence of sulfates (usually calcium sulfate) in the soil itself. Different proportions of calcium sulfate were therefore added to each of the sand-clay mixtures and sets of

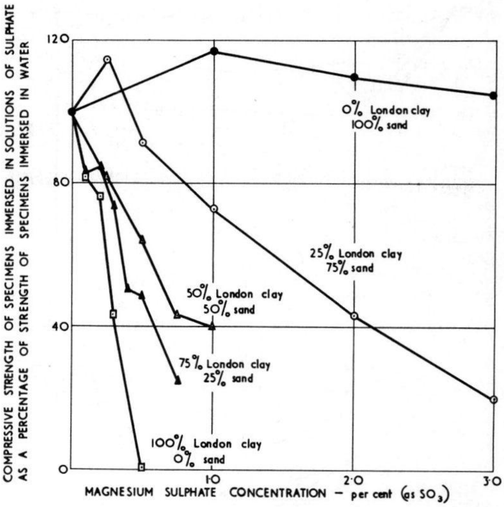

Figure 4. Effect of immersion in magnesium sulfate solution on strength of London clay-sand-cement mixtures stabilized with 10 percent of ordinary portland cement.

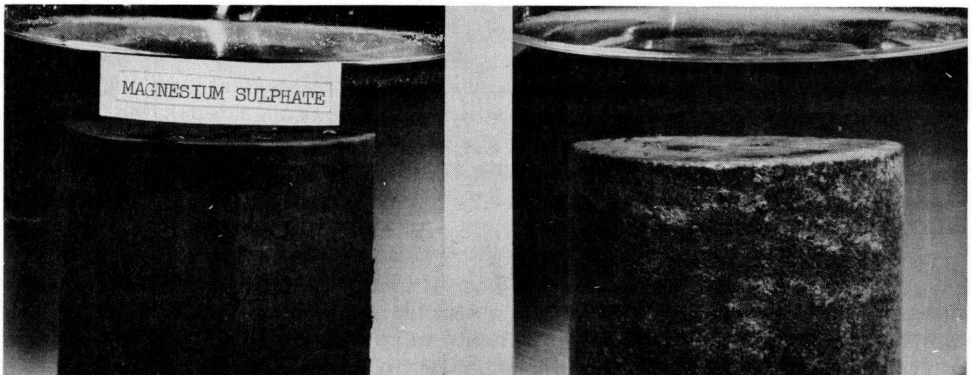

Figure 5. Specimen of cement-stabilized clay immersed in 2 percent solution of magnesium sulfate. Specimen of cement-stabilized sand immersed in 2 percent solution of magnesium sulfate. Large volume increases of clay specimen and white incrustation on sand specimen shown.

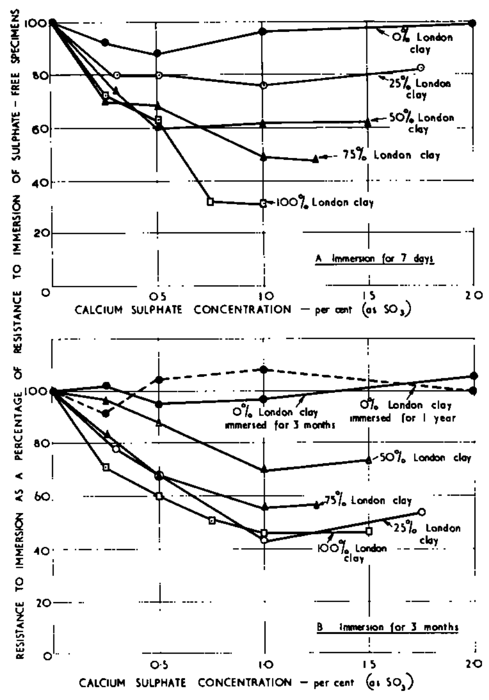

Figure 6. Effect of calcium sulfate on strength of cement-stabilized London clay-sand mixtures.

cement-stabilized specimens were prepared from each of these mixtures. One-half of each set of specimens was subjected to the B. S. resistance to immersion test, and the other half was tested by a modified form of the immersion test in which an immersion period of 3 months was used in place of the specified 7 days. The effect of immersion for both periods of time is shown in Figure 6, and the appearance after 7 days of immersion of clay specimens with and without calcium sulfate is shown in Figure 7.

The resistance to attack by calcium sulfate, as for magnesium sulfate, was found to be related to the clay content of the mixtures (Fig. 6). A comparison of Figure 6a with Figure 6b shows that the attack occurred within a short period after immersion in water and that except for the 75:25 sand-London clay mixture no further attack took place. The sand specimens containing no clay were not affected even after immersion for 1 year.

The rate of attack and its dependence on the clay content suggested that the main reason for the sulfate attack was not, as had been previously thought, due to combination of the sulfate ions with the cement as is the case with sulfate attack of concrete. The fact that soil stabilized with sulfate-resistant cement is equally liable to attack also suggests that combination of the sulfates with the cement is not the primary cause of sulfate attack on cement-stabilized soil. It has been suggested that in the presence of lime (or cement) calcium sulfate can combine directly with clay minerals and it seemed more likely that this was the cause of sulfate attack. Further experiments were therefore made to examine this possibility.

EXPERIMENTS WITH LIME-STABILIZED SOILS

If sulfate attack is due to a reaction between the clay fraction and the sulfate, then lime-stabilized soils should be affected in a similar manner to cement-stabilized soils. To find if this was the case experiments were made in which specimens of a heavy clay (London clay) stabilized with 10 percent of lime and cured at constant moisture content for one week were immersed in solutions of either sodium sulfate or magnesium sulfate, at concentrations ranging from 0 to 1. 5 percent (as SO_3). Figures 8 and 9 show the appearance of the two sets of specimens at the end of the immersion period. In all cases, cracking and swelling of the specimens immersed in the sulfate solutions had occurred, making them too weak for unconfined compressive strength tests to be performed.

Further experiments were made in which different proportions of calcium, sodium, and magnesium sulfates were incorporated in specimens of the heavy clay stabilized with 10 percent of lime. Two sets of specimens were made for each sulfate concentration, and were cured at constant moisture content. At the end of 7 days the wax coating was removed from one set and the specimens immersed in water. After a further 7 days the unconfined compressive strengths of both the immersed and control sets of specimens were determined. Figure 10 shows that provided the lime-stabilized clay remained at constant moisture contant calcium sulfate had a marked beneficial effect, whereas sodium sulfate and magnesium sulfate were without significant effect. However, a considerable loss of strength due to expansion and cracking occurred when the specimens were immersed in water.

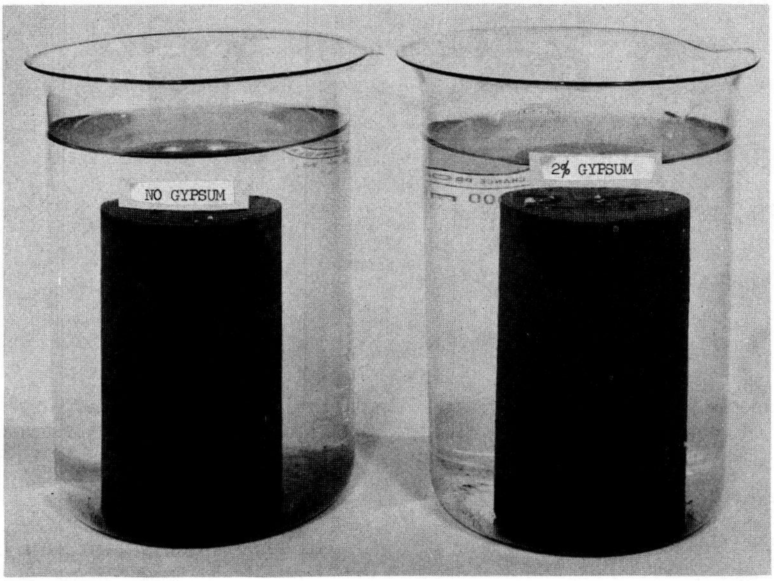

Figure 7. Effect of calcium sulfate (gypsum) on expansion of cement-stabilized clay.

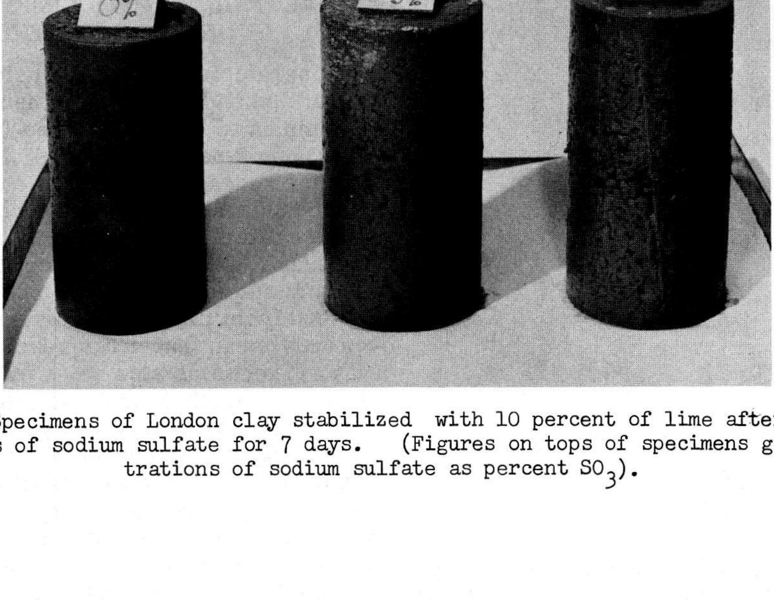

Figure 8. Specimens of London clay stabilized with 10 percent of lime after immersion in solutions of sodium sulfate for 7 days. (Figures on tops of specimens give concentrations of sodium sulfate as percent SO_3).

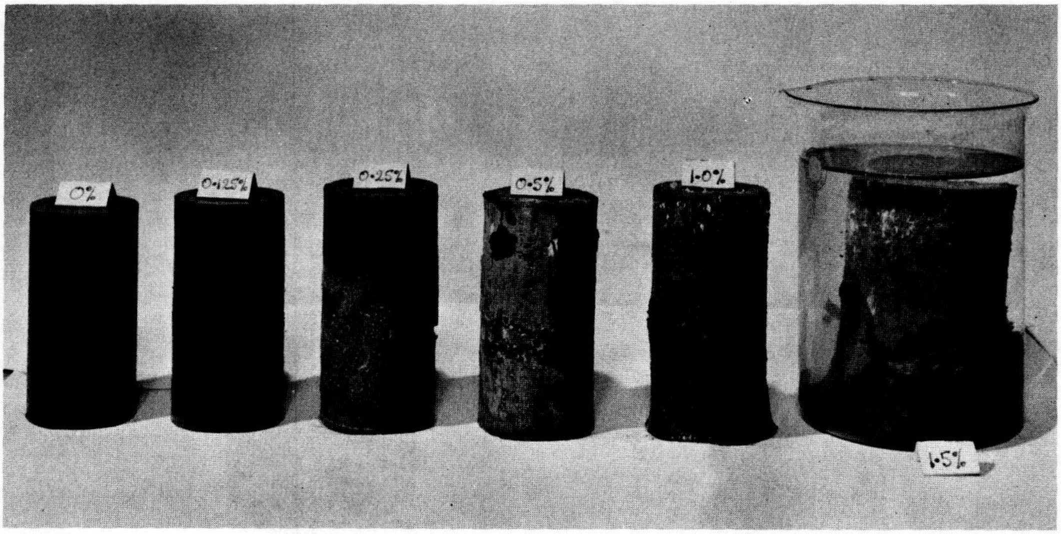

Figure 9. Specimens of London clay stabilized with 10 percent of lime after immersion in solutions of magnesium sulfate for 7 days. (Figures on tops of specimens give contrations of magnesium sulfate as percent SO_3).

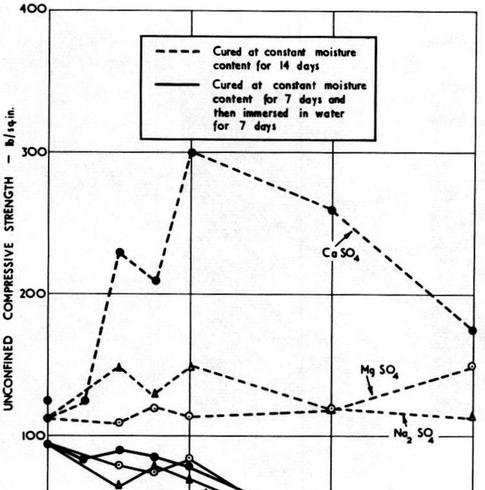

Figure 10. Effect of sulfates on strength of London clay stabilized with 10 percent of hydrated lime.

The experiments with lime clearly show that lime-stabilized soils are affected by sulfates in a similar manner to cement-stabilized soils. In the case of lime there can be no question of it reacting directly with calcium sulfate and the sulfate attack must be a result of a reaction between the sulfate and the clay fraction.

INVESTIGATION OF LIME-CLAY-SULFATE- REACTION

Mineralogical analyses, using X-ray diffraction and differential thermal methods of samples of lime-stabilized London clay containing 2 percent (as SO_3) of calcium sulfate, revealed that the calcium sulfate had disappeared from the specimens that had been immersed in water. As there was insufficient water present for the calcium sulfate to have been completely leached from the specimen sampled, this can only mean that it had reacted with the clay. There was also some evidence that ettringite (calcium sulfo-aluminate) had been formed as a result of this reaction.

Ettringite is the product formed when sulfate ions attack concrete; it occupies a greater volume than the reactants from which it is formed and this results in expansion of the concrete. It seems probable that the formation of ettringite is also responsible for the disintegration of cement-and lime-stabilized soils except that in these cases the ettringite is derived, primarily in the case of cement and exclusively in the case of lime, from a reaction between calcium sulfate and the clay. Further experiments are to be made to confirm this finding.

RESULTS

It is evident from this work that the disintegration of soil-cement due to sulfate ions is brought about, primarily, by a chemical reaction that is different from that involved in sulfate attack of concrete. The degree of disintegration was proportional to the amount of clay present in the soil and this is attributed to a reaction that occurs between clay and sulfate ions in the presence of lime and excess water. Although the sand-cement specimens were more permeable than those containing clay they were not affected by sulfate ions in the period of test, and the fact that soil-cement is more rapidly attacked than concrete by sulfate ions cannot therefore be attributed to its greater permeability. The main reason for the disintegration of soil-cement by sulfate ions is thus the clay-sulfate reaction which is clearly much more rapid than the cement-sulfate reaction that brings about the sulfate attack of concrete. Cement-stabilized sands cannot be affected by the sulfate-clay reaction; this explains why they have been found to be much less susceptible to sulfate attack than cement-stabilized clay soils. So far, tests have not been carried out for sufficiently long to enable any conclusions to be drawn about the long-term susceptibility of cement-stabilized sands to sulfate attack but there is no reason to suppose that it would be very different from that of concrete.

The sulfate-clay reaction helps to explain reported effects of sulfates on cement-stabilized soils which appeared to be anomalous. In their studies with sodium sulfate, Lambe and his co-workers (1) found that it had beneficial effects on some soils and detrimental effects on others, and it is significant that the detrimental effects were obtained with the soils that had the highest clay contents. The fact that the use of sulfate-resistant cement does not lessen the degree of sulfate attack (Figs. 1 and 2) is also readily explained.

ACKNOWLEDGMENT

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Fly Ash and Sodium Carbonate as Additives To Soil-Cement Mixtures

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Although the use of fly ash in mass concrete and in lime stabilization has been extensively investigated, there has been relatively little work done on its use in soil-cement stabilization. Apparently there is no published material available on the use of sodium carbonate in soil-cement-fly ash mixtures.

In this investigation, three soils (a dune sand, a friable loess, and an artificial sand-loess mixture) were studied. Three fly ashes from three different sources were used. The cement was Type I and the sodium carbonate was reagent grade.

It was found that fly ash could be used as an additive to, or as a replacement for, cement in friable soil-cement mixtures. The smaller the loss on ignition and the finer the particle size of the fly ash, the more useful it is as an additive or replacement; however these criteria are not in themselves sufficient to fully differentiate between the varying qualities of the fly ashes. For each fly ash there appears to be an optimum ratio of cement to fly ash. The advantages of fly ash are mainly reflected in long-term strengths. The addition of fly ash tends to retard the setting-up of soil-cement mixtures, thus allowing more time for mixing and compacting. The beneficial effects of the addition of sodium carbonate are most noticeable after short curing periods. Sodium carbonate can be detrimental over a long period of time to soil-cement and soil-cement-fly ash mixtures containing low cement contents.

•SOIL-CEMENT, where properly designed and constructed, has an enviable record as a reliable base course material. Nevertheless, as with all materials of construction, there is still some room for improvement in its properties and in its economy of use. A reduction in the cement requirement, without sacrificing needed strength and durability, is desirable from many view points.

The main objective, therefore, was to explore the possibility of benefiting soil-cement stabilization by using fly ash as an additive to, or as a replacement for, the cement. In addition, selected mixes were treated with a trace chemical (sodium carbonate) to determine its effect on the strength properties of soil-cement and soil-cement-fly ash.

BACKGROUND

Fly ash is the fine, gray, dust-like ash collected in power plants that burn powdered coal. As the finely divided pulverized coal is burned, particles of the ash are suspended in the gas stream and collected on the plates of electrostatic or on mechanical precipitators. Fly ash is available in vast quantities at some industrial centers, and because it is a waste product, it is low in cost.

Because of its low cost and its known pozzolanic behavior, fly ash has been widely used in concrete construction as a partial replacement for the more expensive cement (1). A pozzolanic material is one that, in the presence of moisture, will react with lime to form a cementitious compound (2). Calcium hydroxide (lime) is liberated by hydrating portland cement. This compound is believed to contribute little to the strength of the cementing action in concrete and it may in fact be leached out by percolating waters. It has been suggested, therefore, that if fly ash is present when cement is hydrating, then the fly ash will react with the liberated lime to produce supplemental cementitious compounds (1).

Although the use of fly ash in mass concrete (1, 3, 4, 5) and in lime stabilization (6, 7) has been extensively investigated, there is little published material available on its use in soil-cement stabilization. Brennan (8), working with a fine-grained soil and one fly ash, reported that a mixture containing 3 percent cement and 3 percent fly ash gave good strength improvement. Lilley (9) reported that a fly ash did not contribute to the strengths of cement stabilized cohesive soils. Davidson et al. (10) concluded that the addition of a fly ash appeared to be most beneficial with coarse-grained, poorly graded soils where strength improvements were due to improved soil gradation—resulting in more surface contact areas—and to complementary cementation from pozzolanic reaction products. The fly ash was observed to reduce shrinkage cracking during curing of a cement stabilized clay soil. Mateos and Davidson (11) found that the addition of a fly ash to mixtures of silt and cement increased strengths to a minor extent. Wright and Ray (12) added a fly ash to mixtures of three different soils and cement and concluded that fly ash can replace some of the cement.

Sodium carbonate has been found to accelerate the hardening of soil-lime-fly ash mixtures (6, 7, 11, 13, 14) and soil-cement mixtures (11, 15). Hence, it is not unreasonable to believe that it should therefore benefit the strengths of soil-cement-fly ash mixtures.

MATERIALS AND PROCEDURES

Soils

Two soils were extensively used in this investigation. One was a well-graded, high density sand-loess mix which was sampled from the blended material used in the soil-cement base of Iowa 117, north of Colfax. Because this soil was quite well graded, it was felt that any additional fly ash would not be considered as a filler material, but would instead displace soil particles. Hence, any strength increases that might occur could possibly be attributed to the supposed pozzolanic reaction. The other extensively investigated soil was a more poorly graded friable loess which is typical of that found in eastern Iowa.

In addition, a typical eastern Iowa dune sand was used in a preliminary evaluation of the effects of fly ash and sodium carbonate in soil-cement stabilization. A field description and some physical and chemical properties of each soil are given in Table 1.

Cement and Fly Ashes

Type I portland cement, which is the type commonly used in soil-cement construction, was used in this investigation. According to Davis (3), a larger replacement of cement by a pozzolan can be made when Type I or Type II cement is used instead of Type IV. The tricalcium silicate content of a cement is believed to be directly related to the amount of lime liberated during the cement hydration, and Types I and II cements contain more tricalcium silicate than Type IV. The analysis of the portland cement is given in Table 2.

Because fly ashes differ greatly with respect to their physical and pozzolanic properties, three different types of fly ash from three different sources were selected for this study. Their properties and descriptions are given in Table 3. The choice of the fly ashes was made on the basis of a previous study on the reactivity of different fly

TABLE 1

PHYSICAL AND CHEMICAL PROPERTIES OF TEST SOILS

Property	Sand-Loess (Colfax mix)	Friable Loess (100-8)	Dune Sand (S-6-2)
Geological description	Mix of approximately 82% waste sand from hydraulic gravel dredging operations and 18% Wisconsin-age loess, oxidized, calcareous, medium plastic	Wisconsin-age loess oxidized, calcareous, friable	Wisconsin-age eolian sand, fine-grained, oxidized
Source (Iowa)	Jasper Co.	Scott Co.	Benton Co.
Soil series	Tama (loess)	Fayette	Carrington
Horizon	C (loess)	C	C
Sampling depth (ft)	Stock pile (sand), borrow pit (loess)	25-25 1/2	6-11
Textural composition (%):			
Gravel (<2mm)	0.0	0.0	0.0
Sand (2.0-0.074 mm)	70.7	2.8	94.4
Silt (0.074-0.005 mm)	22.3	85.2	1.6
Clay (<0.005mm)	7.0	12.0	4.0
Colloids (<0.0001 mm)	6.0	8.9	3.5
Predominant clay mineral	Montmorillonite, vermiculite	Montmorillonite	Montmorillonite, illite
Chemical properties:			
Cation exchange cap.[a] (meq/100 g)	11.0	3.8	--
Carbonates (%)	11.6	20.0	0
pH[b]	8.0	--	6.5
Organic matter (%)	0.2	0.2	0.1
Physical properties:			
Liquid limit (%)	19	27	N.P.
Plastic limit (%)	16	20	N.P.
Plasticity index	3	7	N.P.
Classification:			
Textural[c]	Sandy loam	Silty loam	Sand
Engineering (AASHO)	A-2-4	A-4(8)	A-3(0)

[a] For fraction passing No. 40 sieve.

[b] For fraction passing No. 10 sieve.

[c] Textural classification is from triangular chart developed by U.S. Bureau of Public Roads. Sand and silt sizes are separated on No. 200 sieve (0.074 mm).

TABLE 2

PROPERTIES OF TYPE I PORTLAND CEMENT

Property	Value
Chemical composition (% by wt.):	
Silicon dioxide	21.62
Aluminum oxide	5.05
Ferric oxide	2.97
Calcium oxide	64.05
Magnesium oxide	2.90
Sulfur trioxide	2.26
Loss on ignition	0.58
Insoluble residue	0.16
Physical properties:	
Specific surface (Wagner) (sq cm/g)	1,855
Air Permeability (Blaine)(sq cm/g)	3,395
Setting time (Gillmore test) (hr):	
Initial	3.15
Final	5.15
Autoclave expansion (%)	0.120
Compressive strength (1:2.75 G.O.S.)(psi):	
At 3 days	2,269
At 7 days	3,721
At 28 days	5,625

TABLE 3

ANALYSIS OF FLY ASHES

Property	Fly Ash 1	Fly Ash 3	Fly Ash 4
Source	St. Clair, Mich.	Louisville, Ky.	C. Rapids, Iowa
Loss on ignition (%)	3.9	2.6	18.6
Specific surface, Blaine (sq cm/g)	2,820	3,226	4,550
Specific gravity	2.58	2.60	2.37
Fineness (% passing No. 325 sieve)	91.8	86.1	54.9
Chemical composition (% by wt.):			
Silicon dioxide (SiO_2)	43.5	42.5	36.2
Magnesium oxide (MgO)	0.2	0.8	0.9
Calcium oxide (CaO)	2.9	5.7	8.3
Aluminum oxide (Al_2O3)	23.2	23.4	15.8
Iron oxide (Fe_2O3)	24.8	20.0	16.7
Sulfur trioxide (SO_3)	0.8	2.3	1.5

ashes with lime (6). In that study fly ash 3 proved to be of very good quality, fly ash 1 of good, and fly ash 4 of poor quality when used in lime and fly ash mixtures.

Often used criteria for judging the quality of a fly ash are the loss on ignition (which is approximately equal to the carbon content) and the fineness of the fly ash as measured by the amount passing the No. 325 sieve. Using these criteria, fly ashes 3 and 1 could be considered to be of good quality and 4 to be of rather poor quality.

Trace Chemical

Although a number of chemicals are known to act as accelerators for pozzolanic reactions, sodium carbonate is believed to be one of the better, safer and more economical to use (14, 15). Sodium carbonate, reagent grade, was used in this investigation.

Mixture Proportions

In this investigation the percentages of cement, fly ash, and soil are expressed as percentages of the total dry weight of the soil, cement, and fly ash in the mix. The amount of sodium carbonate is expressed as a percentage of the total dry weight of soil, cement, and fly ash in the mixture under consideration.

In a preliminary study, specimens were first prepared which contained dune sand, 11 percent cement, and 0, 3, 6 and 9 percent fly ash 1. Specimens were also prepared that contained varying amounts of cement and fly ash but in all of these additional specimens, the total cement and fly ash content was 11 percent. All specimens were then duplicated except that ½ percent sodium carbonate was added.

Three cement contents (5, 8, and 11 percent) were used in the main studies. It was felt that these percentages would well enclose the cement requirement to meet the criteria for soil-cement. Soil-cement mixtures were first prepared without fly ash and then with fly ash contents of 3, 6, and 9 percent of each of the different fly ashes. Extra mixtures were then prepared in which ½ percent sodium carbonate was added in powder form to all soil, cement and fly ash combinations containing 5 percent cement.

Mixing and Molding

All soils were air dried, and ground with mortar and pestle to pass through the No. 10 sieve before any mixing took place. Mixing of batches was done in a Hobart kitchen mixer, model C-100, at low speed in the following sequence of operation: the dry ingredients were mixed for 30 sec, the required amount of distilled water was then added, and the batch was then mixed for 1 min. The mixture was then hand mixed for about 30 sec so as to clean the side of the mixing bowl and the paddle, after which the mixture was machine mixed for another minute. After mixing was completed, a damp cloth was placed over the bowl so as to prevent drying of the mixture during molding.

Molding of test specimens was started as soon as possible after completion of mixing. Test specimens, 2 in. in diameter by 2 ± 0.05 in. high, were compacted with the Iowa State compaction apparatus (6, 7, 10, 11, 13, 14, 16). Maximum densities obtained using this apparatus have been very closely correlated with those maximum densities obtained using the standard Proctor apparatus (16).

Numbers of Specimens

The average of three specimens was used to obtain each point on the moisture-density and moisture-strength relationship curves. When the optimum moisture content for maximum density was obtained for each soil-cement-fly ash combination, four specimens were molded at this moisture content for each curing condition. The average of these four strength values is reported.

Curing

Specimens for the main study were cured for 7, 28, and 90 days and for 7, 28, and 120 days for the preliminary investigation. The curing room was maintained at a temperature of 71 ± 3 F and a relative humidity of greater than 90 percent. To maintain the moisture in the specimens better and to reduce absorption of carbon dioxide from the air, they were wrapped in wax paper and sealed with cellophane tape before being placed in the humid room.

Strength Testing

At the end of a specified curing period, the specimens were unwrapped, immersed in distilled water for 24 ± 3 hr and then tested for unconfined compressive strength

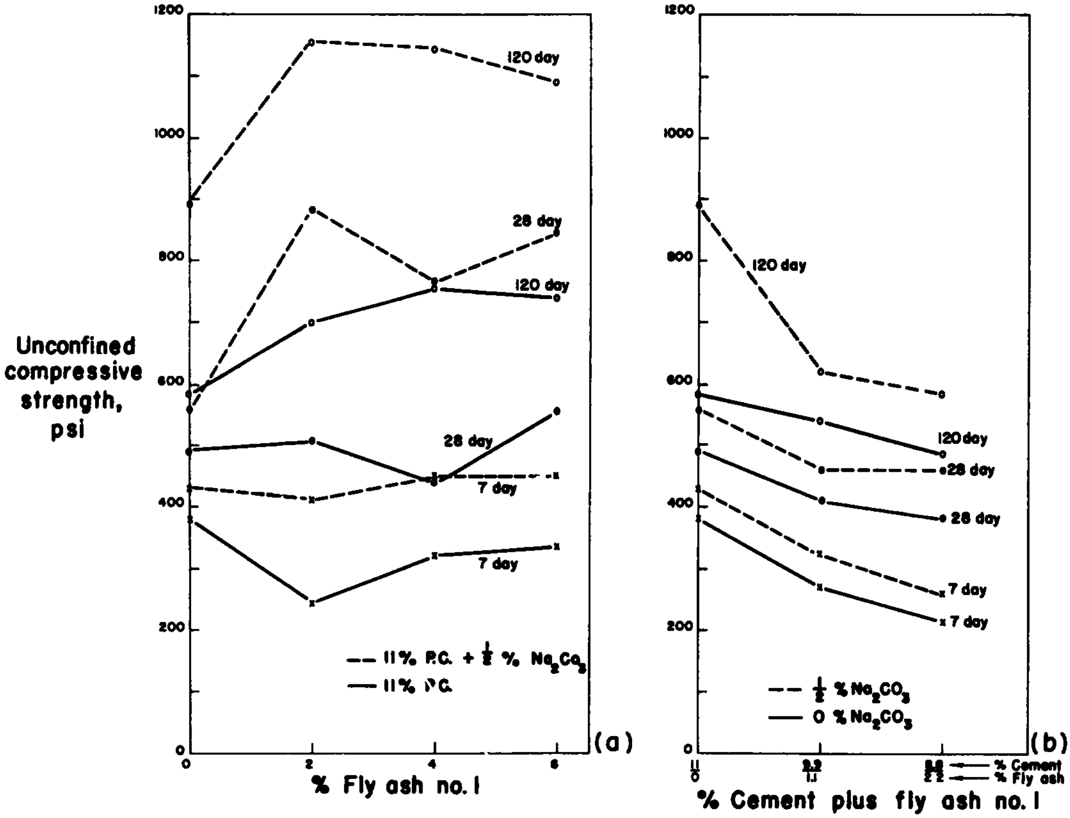

Figure 1. (a) Fly ash 1 without cement; (b) fly ash 1 with cement.

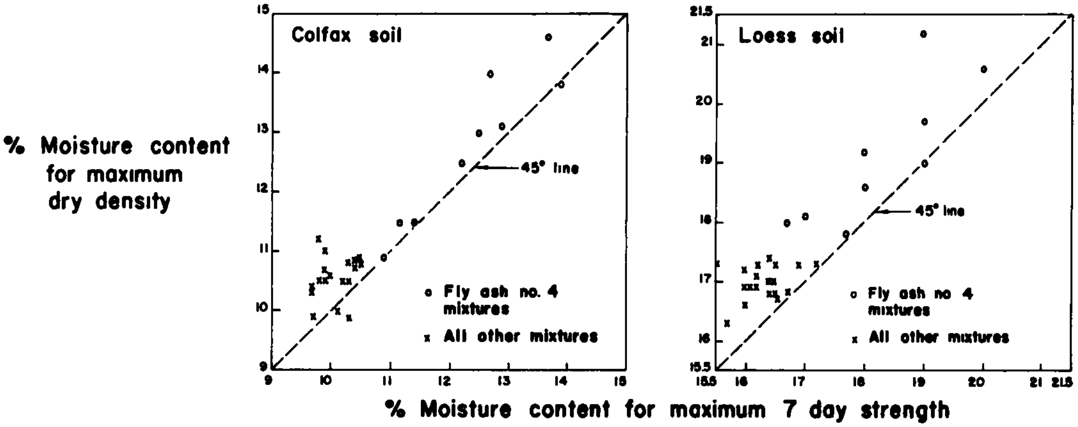

Figure 2. Comparison of optimum moisture contents for maximum densities with optimum moisture contents for maximum strengths.

using a load travel rate of 0.1 in. per min. Strength values were evaluated for reliability in a statistical manner (17) and any values which were then disqualified were not considered in determining average compressive strengths.

PRELIMINARY INVESTIGATION

Before the main study, the effect of fly ash and sodium carbonate on poorly graded dune sand-cement mixtures was investigated. In testing for the effect of fly ash as an additive, only one cement content, 11 percent, and fly ash 1 were used. Mixtures were also prepared at two lower cement contents and fly ash added until the total cement and fly ash content totaled 11 percent. All of these mixtures were then duplicated except that 0.5 percent sodium carbonate was added to each before dry mixing. Specimens were cured for 7, 28, and 120 days before testing for unconfined compressive strength.

The results obtained are shown in Figure 1 . The addition of fly ash to 11 percent cement causes decreasing strengths at 7 days (Fig 1a). At 28 days, there is perhaps a slight strength increase. After 120 days the addition of fly ash is seen to be most beneficial, the addition of 4 percent fly ash causing a 25 percent increase in strength. When 0.5 percent sodium carbonate was added to these mixtures, great increases in strength were noted. For the soil-cement mixes, increases amounted to 16 percent at 7 and 28 days and 33 percent at 120 days. Best results were obtained with 11 percent cement, 2 percent fly ash, and 0.5 percent sodium carbonate. At 7 days there was a negligible gain in strength over those mixtures containing cement alone. However, after 28 and 120 days these mixtures gave strengths that were 82 percent and 95 percent, respectively, greater than the soil-cement strengths.

Figure 1b shows quite clearly that the substitution of fly ash 1 for an equal amount of cement is not beneficial. However, this limited study indicated that there might be some ratio of cement to fly ash, with or without sodium carbonate, that might allow some economical substitution.

MAIN STUDY RESULTS

A moisture-density relationship was run for each combination of soil, cement, and fly ash. Five points were obtained on each curve, each point being the average of three specimens. After tabulation of the moisture-density data, the specimens were placed in the humid room, cured for 7 days and then tested under unconfined compression. Moisture-strength curves were then drawn for each of the soil-cement-fly ash combinations.

The optimum moisture contents for both density and strength increased as the amount of fly ash in a particular mix increased. The effect of the different types of fly ash on these optimum moisture contents is noteworthy. For a particular soil, the mixes containing fly ashes 1 and 3 had their optimum moisture contents all contained within 1.3 percentage points and this spread contained these required for the cement and soil alone (Fig. 2). On the other hand, the mixtures containing the coarsest and highest carbon content fly ash (No. 4) had a spread of 4.5 percentage points.

The optimum moisture contents for maximum 7-day strengths were consistently on the dry side of the optimum moisture content for maximum density for both the Colfax and loess soils. The reason for this is not exactly known. An interesting conjecture reasons that the difference is related to soil texture. The surfaces of such sands and silts are relatively inactive and hence little of the lubricating water is adsorbed onto the soil particles. As a result most of the water that has been added to ensure maximum density is also available for cement hydration purposes. This amount of free water may be over and above the amount required for cement hydration. In concrete work, the lower the water to cement ratio, the higher the strength. Therefore, it may be that the optimum moisture content for obtaining maximum density for a soil-cement-fly ash or soil-cement mixture is greater than the water requirements to satisfy the cementitious reaction. As a result, the optimum moisture content for maximum strength will be on the dry side of the optimum moisture content for maximum density.

In general, densities decreased as the amount of fly ash in a mixture increased. Again the different fly ashes had different effects on densities. For fly ashes 1 and 3, density decreases ranged from 1 to 4 pcf depending on the amount of fly ash used. With fly ash 4 density decreases were as much as 12 pcf. This is due to the low unit weight of this fly ash.

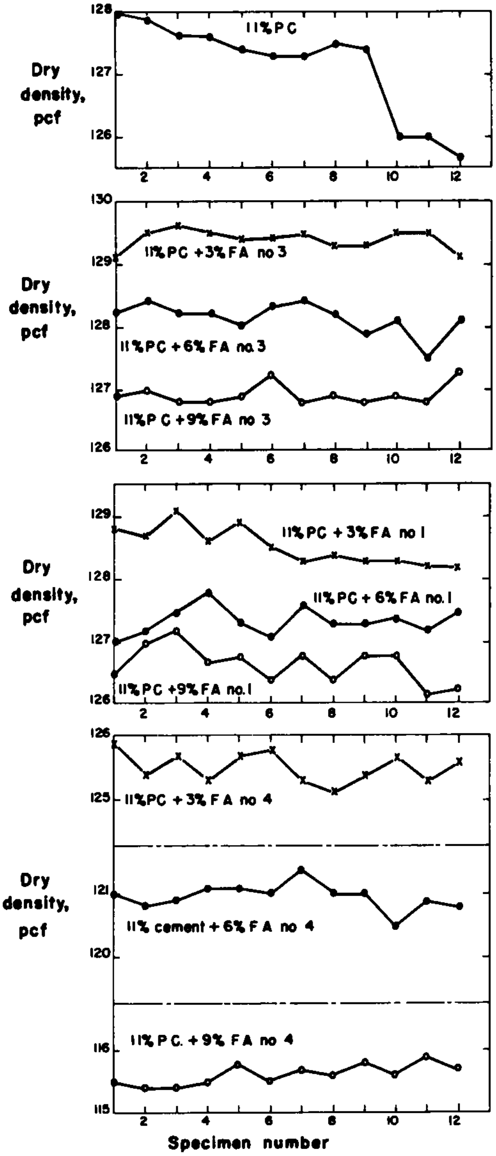

Figure 3. Effect of fly ash as a retarding agent on setting up of Colfax soil-cement mixtures—typical data.

After the optimum moisture content for maximum density was obtained for a particular mix, 12 specimens were molded at that moisture content. The strength results obtained from these specimens are discussed later. However, the effect of time on the specimen densities within a particular batch (Fig. 3) is noteworthy. The time from when moisture was first added to the dry mix until the end of the compacting of the last specimen varied from 17 to 24 min. The time from the beginning of compaction of the first specimen until the end of the last varied from 14 to 21 min. Soil mixtures containing only cement and sodium carbonate or cement alone showed definite decreasing density trends as compaction time progressed. On the other hand, the addition of fly ashes 3 and 4 apparently had the effect of retarding the setting-up of the soil-cement. The medium type fly ash (No. 1) had less of a retarding effect.

UNCONFINED COMPRESSIVE STRENGTH RESULTS

Fly Ash as Additive to Soil Cement

As mentioned earlier, 12 specimens were molded at the optimum moisture content for maximum density for each combination of cement, soil, and fly ash. After being cured in the humid room, 4 specimens were tested after 7 days, 4 after 28 days, and the remaining 4 after 90 days. The 4 specimens tested at a particular time were selected statistically to minimize differences in the time and space. The average strength results obtained are shown in Figures 4 and 5.

As expected, both the soil-cement and the soil-cement-fly ash mixtures increased in strength as curing time progressed. However, the manner in which the different fly ashes affected these strength increases was very significant.

Fly Ash 3.—This fly ash appears to be the most beneficial of the three. The 7-day strengths show the addition of the fly ash to have relatively little effect on both soils. However at 28 days, fly ash is seen to be most beneficial. Best results are obtained with the Colfax soil where the addition of 9 percent fly ash to 11 percent cement shows a strength increase of 46 percent at 28 days and of 37 percent after 90 days. With the more poorly graded loess soil, the results were not so striking, the maximum strength gains being in the range of 25 to 34 percent.

Fly Ash 1.—Again the 7-day strengths are relatively nonsignificant. Actually, for the loess soil there is a tendency towards a slight strength loss. Again at 28 days, the loess soil-cement mixtures tend to be slightly hurt by the addition of this fly ash. However, at 90 days slight strength increases of up to 15 percent are recorded as fly ash

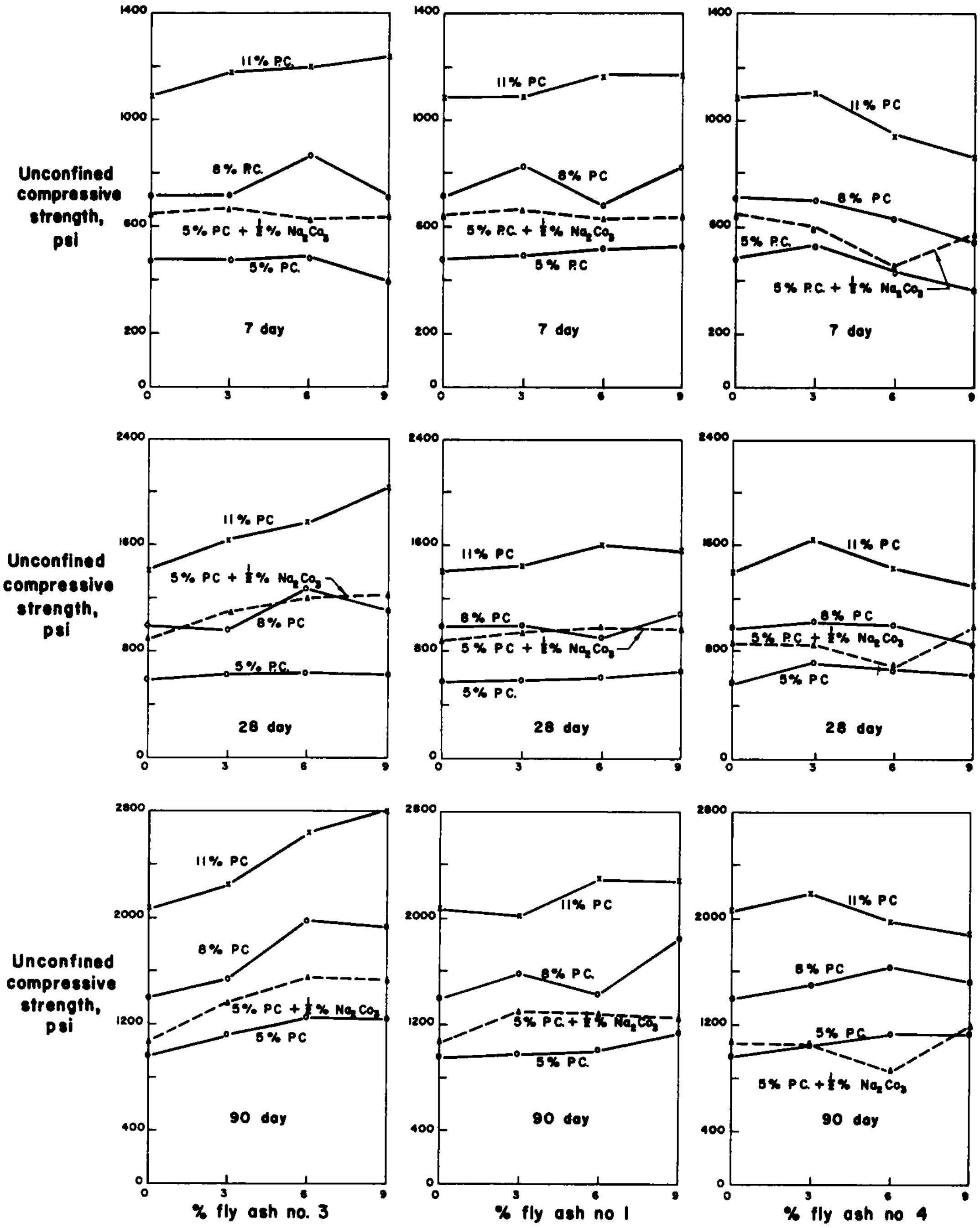

Figure 4. Strengths obtained by Colfax soil-cement mixtures containing fly ash and sodium carbonate after varying curing periods.

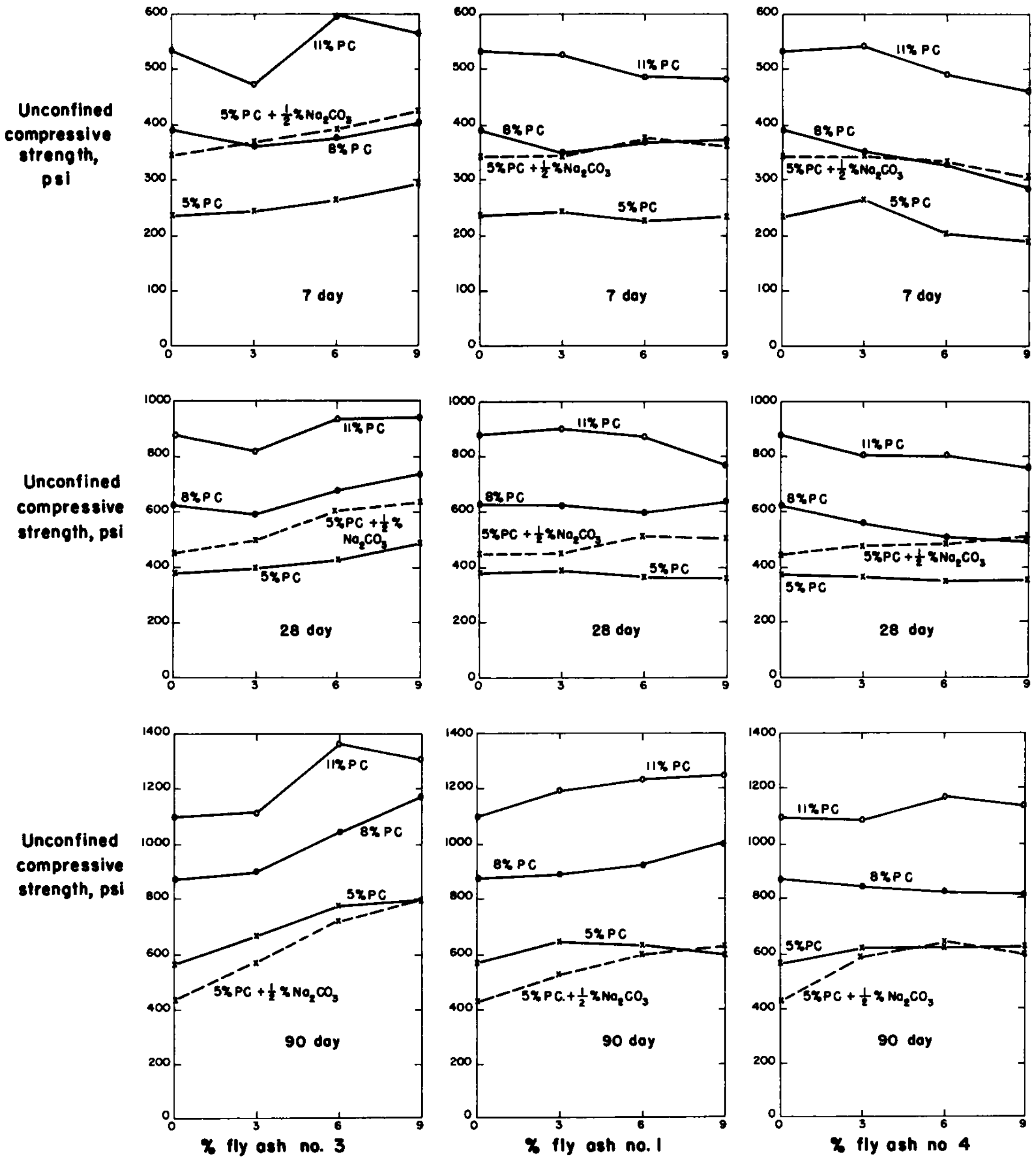

Figure 5. Strengths obtained by loess soil-cement mixtures containing fly ash and sodium carbonate after varying curing periods.

is added to the loess-cement. The Colfax mixtures show minor strength increases after 28 days. The beneficial effects of the fly ash are most noticeable after 90 days where the addition of 9 percent fly ash to 8 percent cement caused a strength increase of 31 percent. Also, fly ash 1 was sampled from the same source as that used by Davidson et al. (10) in their investigation.

Fly Ash 4.—The effects of fly ash 4 on soil-cement mixtures are very clear. Strengths at 7 days indicate quite strongly that the addition of more than 3 percent fly ash to either of the soils causes a definite decrease in strength. The 28- and 90-day strength results

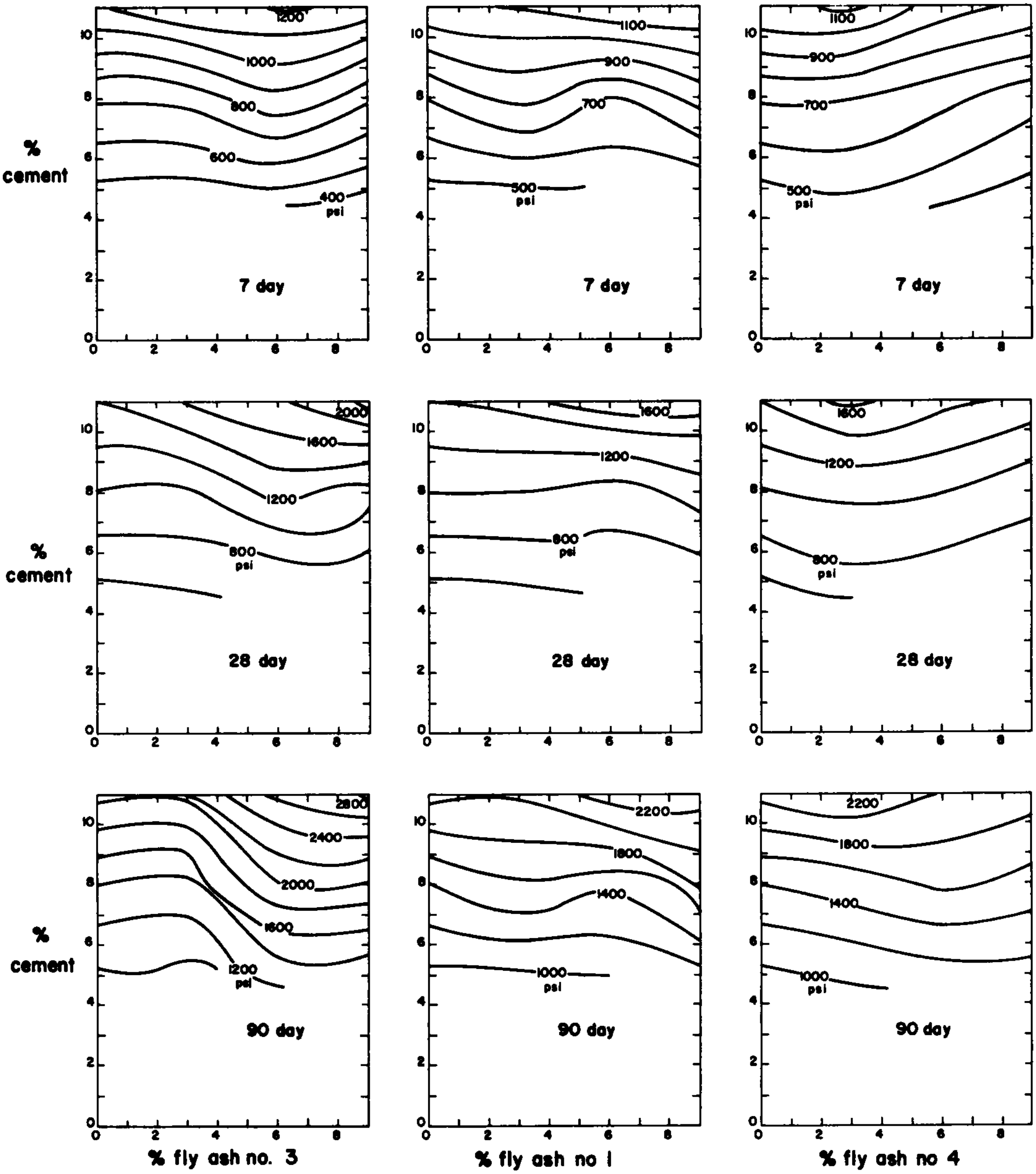

Figure 6. Iso-strength contour graphs for Colfax soil-cement mixtures containing fly ash after varying curing periods.

indicate that the addition of this fly ash to the loess-cement mixtures can be a disadvantage. With the Colfax soil, 3 to 6 percent would seem to be about optimum for beneficial results.

In summary, the addition of fly ash can be beneficial to soil-cement mixes. However, its helpfulness depends on the type of fly ash used. The smaller the loss on ignition and the finer the particle size of the fly ash, the more useful it is as an additive. Poor quality fly ash can be a detriment in soil-cement mixtures. The manner in which the different fly ashes react would seem to support the liberated lime-pozzolan reaction theory (1) because it is known that the pozzolanic activity of fly ash is dependent on its particle size and the loss on ignition.

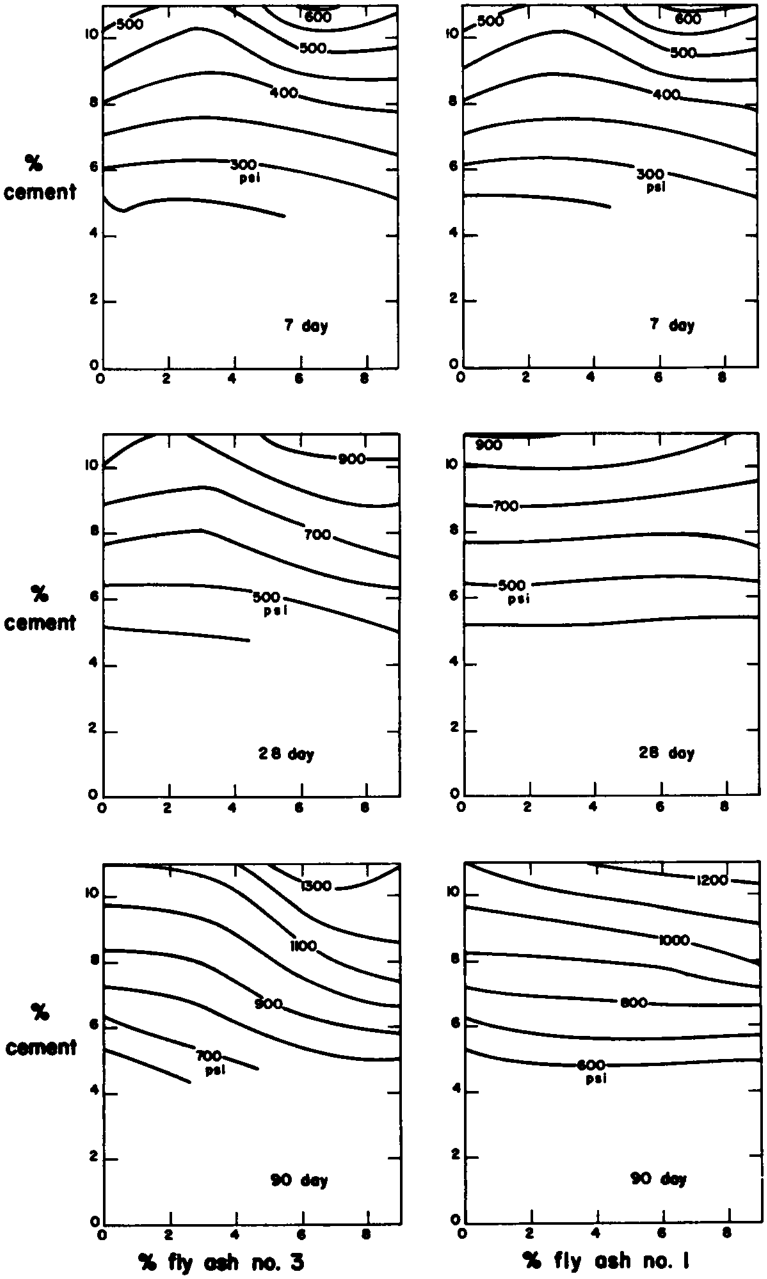

Figure 7. Iso-strength contour graphs for loess soil-cement mixtures containing fly ash after varying curing periods.

Fly Ash as Replacement for Cement

Contour graphs for the different soil, cement, and fly ash mixtures, based on the preceding strength values, are shown in Figures 6 and 7. An examination of these indicates some interesting trends.

Fly Ash 3.—For both soils a ratio of cement to fly ash of about 3:2 is optimum at 7 days. After 28 days and 90 days, and again for both soils, the best ratio is about 11 cement to 9 fly ash.

Fly Ash 1.—With the Colfax mixtures a ratio of 11 parts cement to 9 parts fly ash might again be taken as optimum for 7, 28, and 90 days. For the loess soil mixtures the results are rather inconclusive. At 7 days, a ratio of 3 parts cement to 2 parts fly ash appears best. At 28 days, a ratio of 11 parts cement to 1 part fly ash would seem to be about optimum, but the ratio again changes to 11:9 after 90 days.

Fly Ash 4.—For 7-day strengths a ratio of 4 parts cement to 1 part fly ash might be deemed optimum for the Colfax mixtures. After 28 and 90 days, the effect of the fly ash is more noticeable giving an optimum ratio of 3 parts cement to 1 part fly ash for both curing periods. Contours for the loess-cement mixtures containing this fly ash are not shown here as it is apparent from Figure 5 that the addition of this fly ash generally has a detrimental effect.

In summary, fly ash can be used to replace cement in soil-cement mixtures. Table 4 gives the amounts of cement and a particular fly ash that can be used to attain the same strength at a given time as that obtained by, for instance, 11 percent cement alone. The amount of fly ash required to replace cement to get a certain strength at a given time is variable, depending mainly on the type of fly ash, and on the soil.

Effect of Sodium Carbonate

As already indicated, optimum moisture contents for maximum density were obtained for all combinations of cement, soil, and fly ash. To all the mixtures that contained 5 percent cement and to the mixtures containing 8 and 11 percent cement but without any fly ash, 0.5 percent of sodium carbonate was added in powder form. The mixtures were then compacted at the same moisture contents as those mixtures without sodium carbonate. Unconfined compressive strengths obtained from the mixtures containing only soil, cement, and sodium carbonate are given in Table 5, and those containing soil, 5 percent cement, fly ash, and sodium carbonate are shown in Figures 4 and 5.

As can be clearly seen, the addition of sodium carbonate has varying effects on the strengths of the soil-cement and soil-cement-fly ash mixtures.

Colfax mixtures containing only cement show increases in strength of up to 32 percent after 7 days, 65 percent after 28 days, and 10 percent after 90 days when sodium carbonate is added. The loess soil-cement mixtures show increases of 44 percent after 7 days and 18.5 percent after 28 days and a decrease of about 30 percent after 90 days.

The effectiveness of the addition of sodium carbonate to soil-cement-fly ash mixtures is dependent on the type of fly ash used. The best results are obtained with fly ash 3. For instance, with the 5 percent cement-Colfax soil mixtures, the addition of 9 percent of this fly ash and 0.5 percent sodium carbonate gives a 28-day strength that is 38 percent greater than that obtained by adding only 0.5 percent sodium carbonate and 133 percent greater than with just 5 percent cement. The corresponding 90-day values are 17 and 30 percent. With the loess soil mixtures (Fig. 5) it is clearly seen that the more fly ash in the mix, the more effective is the addition of sodium carbonate. For example, when 0.5 percent is added to the mixture containing 5 percent cement and 9 percent fly ash 3, the 28-day strength is increased by 17 percent over that obtained by adding sodium carbonate to the mixture containing 5 percent cement and no fly ash, and by 68 percent over that obtained when the mixture contains no fly ash and no sodium carbonate. Again at 90 days, sodium carbonate is most effective with the mixtures containing high fly ash contents. For instance, the mixture of loess, cement, 9 percent fly ash 3, and sodium carbonate gives a strength that is 54 percent greater than that obtained by omitting the fly ash entirely. However, with this soil the addition of sodium carbonate appears to be detrimental to long-term strengths. Figure 5 shows the loess mixtures containing sodium carbonate tend to give lower strengths after 90 days than the mixtures without any sodium carbonate. The reason for this is not fully understood.

TABLE 4

AMOUNTS OF CEMENT AND FLY ASH GIVING SAME STRENGTHS AS 11 PERCENT CEMENT AFTER A GIVEN CURING PERIOD

Soil	Length of Curing (days)	Type of Fly Ash (no.)	Ratio of Cement to Fly Ash	Amount of Cement and Fly Ash to Replace 11 Percent Cement	
				% Cement	% Fly Ash
Colfax	7	3	3:2	9.9	6.6
	28		11:9	8.8	7.2
	90		11:9	8.2	6.8
	7	1	11:9	10.2	8.3
	28		11:9	9.8	8.0
	90		11:9	9.6	7.9
	7	4	4:1	10:6	2.6
	28		3:1	9.8	3.3
	90		3:1	9.6	3.5
Loess	7	3	3:2	10.2	6.8
	28		11:9	9.9	8.1
	90		11:9	8.2	6.8
	7	1	3:2	10.2	6.8
	28		11:1	10.9	0.9
	90		11:9	9.4	7.6
	7	4	--	--	--
	28		--	--	--
	90		5:3	10.4	6.2

TABLE 5

UNCONFINED COMPRESSIVE STRENGTHS OBTAINED FOR SOIL-CEMENT AND SOIL-CEMENT-SODIUM CARBONATE MIXTURES

Soil	Additive		Unconfined Compressive Strength (psi)		
	% Cement	% Sodium Carbonate	7-Day	28-Day	90-Day
Colfax	5	0	480	580	964
		1/2	645	885	1,077
	8	0	714	996	1,400
		1/2	765	1,064	1,447
	11	0	1,085	1,408	2,072
		1/2	1,262	1,857	2,465
Loess	5	0	239	381	572
		1/2	344	447	431
	8	0	396	627	879
		1/2	481	708	893
	11	0	538	880	1,097
		1/2	654	927	1,213

In summary, the addition of 0.5 percent sodium carbonate can cause great strength increases or decreases in soil-cement and soil-cement-fly ash mixtures. In certain situations, the cement requirement for soil-cement can perhaps be lowered by the substitution of a small amount of sodium carbonate. The cement requirement can be further reduced by the addition of fly ash. The type of fly ash to be used is most important. Of course, for these substitutions to be made, special attention should be paid to the design criteria, especially the length of curing time in which the desired strength must be attained.

Cost Data

If fly ash and/or sodium carbonate are to be used to the greatest advantage, it must be on a sound economical basis. Cement costs approximately $20 per ton on the site. Fly ash, being a waste product, is very low in cost, approximately $1 per ton at source. Cost of transportation of fly ash varies with job location but an average value of $4 to $5 per ton might be used. Cost of spreading and mixing the fly ash with the cement and soil will vary, depending on the amount of fly ash to be used and the equipment on the job site. Because the cost of transportation is perhaps the most prohibitive, it would seem that fly ash could be most economically used in locales near the fly ash source.

Sodium carbonate costs about $35 to $65 per ton at its source. Although this cost is relatively high, it should be kept in mind that very small amounts are required. For this reason, the cost of transportation, spreading and mixing should be very small. Sodium carbonate would seem to be used to its greatest advantage when high early strengths are required.

CONCLUSIONS

Fly ash can be used either as an additive to, or as a replacement for, cement in friable soil-cement mixtures. The smaller the loss on ignition and the finer the particle size of the fly ash, the more useful it is as an additive or replacement; however, these criteria are not sufficient in themselves to differentiate fully between the varying qualities of fly ashes. For each fly ash there appears to be an optimum ratio of cement to fly ash. The advantages of fly ash are mainly reflected in long-term strengths. The addition of fly ash tends to retard the setting-up of soil-cement mixtures, thus allowing more time for mixing and compacting. Moisture contents for maximum densities tend to the wet side of moisture contents for maximum 7-day strengths for all combinations of mixtures. The manner in which the different fly ashes reacted would seem to support the liberated lime-pozzolan reaction theory. The beneficial effects of the addition of 0.5 percent sodium-carbonate are most noticeable after short curing periods. Sodium carbonate can be detrimental over a long period of time to soil-cement and soil-cement-fly ash mixtures containing low cement contents. In certain cases, the cement requirement for soil-cement can be reduced by the addition of small amounts of the trace chemical.

The usefulness of fly ash and/or sodium carbonate as an additive to, or as a replacement for, cement in soil-cement is dependent on the available curing time in which the design strength is to be attained and on the availability and cost of the materials at the site.

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