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Bulletin 69

Soil Stabilization

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HIGHWAY RESEARCH BOARD

Bulletin 69

Soil Stabilization

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Contents

Properties of Lime-Flyash-Soil Compositions Employed in Road Construction

L. JOHN MINNICK, Chief Chemist, and
W. F. MEYERS, Chemical Engineer;
G. and W. H. Corson, Inc.

AN evaluation of field projects in which lime and flyash are used for the stabilization of several types of soil indicates that the resulting compositions are very satisfactory as road bases. The evaluation includes laboratory tests for unconfined compressive strength, wetting and drying, freezing and thawing, and pulse group velocity. The equipment used to measure pulse velocity is described in some detail. The velocity measurements are found to be beneficial in evaluating the strength and durability of the compositions and good correlation is found to exist between the different test series. The construction work utilizes the mixed-in-place method. Several types of surface treatment have been applied to the stabilized bases.

● THE use of lime and flyash for the stabilization of soil and in the preparation of lime-flyash-aggregate mixtures has received considerable attention in recent years (1, 2). In the evaluation of these compositions the earlier work showed that the lime and flyash develop, through pozzolanic action, a cementitious matrix which acts to bond the material together into a coherent mass. For fine-grained and plastic soils the addition of lime and flyash also produces a substantial improvement in the engineering properties of the soil immediately after preparation of the mixtures. The method has given good results with natural soils and with aggregate materials such as crushed stone, boiler slags, and cinders. These compositions are currently being used in road construction with very satisfactory results. A few of these projects are described in this paper.

In order to adequately evaluate the field compositions, a laboratory investigation has been carried out on the same soils which were used in the field projects. As part of this study it has been necessary to give consideration to test methods which are applicable to compositions of this type and which are of use in developing relationships between laboratory tests and field performance. These methods include compressive-strength tests and a study of the effects produced by wetting and drying and by freezing and thawing. Since results obtained from sonic test methods (2, 3) have indicated that measurements of velocity of sound through the composition are of value in considering the durability and other physical characteristics of the test mixtures, this method has been included as an important part of the investigation.

The equipment employed in making group-velocity measurements is somewhat different from that reported by other investigators (3, 4, 5). A detailed description of the apparatus and the electric circuit is therefore included below. The equipment was found to give unusually stable readings with laboratory samples and also worked very well as a portable battery operated unit in the field. Longitudinal wave-front velocities, both compression and traction, have been measured and the results compared to other data such as water content, compressive strength, and durability measurements.

While most of the field projects will require more time before a complete evaluation can be made, sufficient in-

TABLE 1

SOIL CLASSIFICATION

Soil	Soil Designation	SCREEN ANALYSIS Weight Percent Passing Screen 2 in.	1½ in.	1 in.	¾ in.	⅜ in	No. 4	No. 10	No 40	No. 60	No. 200	Liquid Limit	Plasticity Index	Std. Proctor Density lb. per cu. ft.	Std. Proctor Optimum Moisture Percent
N J Turnpike New Brunswick Interchange 9	A-1-b Sandy Gravel	--	--	100	89 7	70.0	66 2	56 4	23.2	13 2	8 2	18.7	NP	133	8 5
N J Turnpike Hightstown Interchange 8	A-1-b Sandy Gravel	--	--	100	95	88	85	75	41	32	14.5	20.0	NP	126	9.0
N J. Turnpike Bordentown Interchange 7	A-1-b Sandy Gravel	--	--	100	93	86	73	56	36	19	11	17.0	3 5	125	10.0
N J Turnpike Burlington Interchange 5	A-1-b Sandy Gravel	--	--	100	96	87	80	66	48	35	17	22	NP	125	9 0
Marlboro By-Pass Marlboro, Md.	A-2-5 Silty Sand	--	--	--	--	--	--	100	99	93	19	28	NP	107	17.0
Navajo St. HiNella, N. J Contains Some Glauconite	A-5 Fine Sand	--	--	--	--	--	100	94	43	26	11	--	NP	124 5	8 0
Crestwood Ave Somerdale, N J Contains Glauconite	A-5 Fine Sand	--	--	--	--	--	100	86	40	26	9.6	--	NP	120 0	9
U S Avenue Lindenwald, N J	A-3 Fine Sand	--	--	--	100	89	86	83	65	52	1	--	NP	125 0	8 5
Mercer Rd. Barrington, N J. No. 1	A-3 Fine Sand	--	--	--	--	100	99	96	82	63	9	--	NP	118	10
Mercer Rd. Barrington, N J No. 2	A-3 Fine Sand	--	--	--	100	99	93	89	69	51	10	--	NP	122	10
A-3 Soils Whaleysville, Md	A-3 Fine Sand	--	--	--	--	--	--	100	79.5	55	2 8	--	NP	127	9
East Atlantic Ave , Oaklyn N J	A-1-a Sandy Gravel	100	92	89	81	72	64	49	27	19	4	--	NP	113 8	11.5
Underwood Hospital Woodbury, N J	A-1-b Sandy Gravel	--	--	100	88	74	66	49	24	18	16	--	NP	111 0	10 5
Laurel By-Pass Laurel, Md.	A-3	100	97	94	92	83	70	45	20	17	7	16 5	NP	124.3	9 5

formation is available to indicate the properties and the early performance of the stabilized compositions.

MATERIALS

The materials used in the investigation were selected from the sites where the field projects were to be carried out. The physical properties of the soils used in the study are given in Table 1. A few of the soils contained appreciable quantities of a bituminous fraction from previous surface treatments of the roads with asphaltic oil or tar and stone chips.

The chemical and physical characteristics of the hydrated lime and flyash used for practically all of the work are given in Table 2. In the field projects the flyash was supplied in moist condition usually containing from 10 to 20 percent water. The hydrated lime was supplied dry in paper bags.

TEST PROCEDURE

The previous papers (1, 2) described the methods employed to develop optimum proportions of lime and flyash for both fine- and coarse-grained materials.

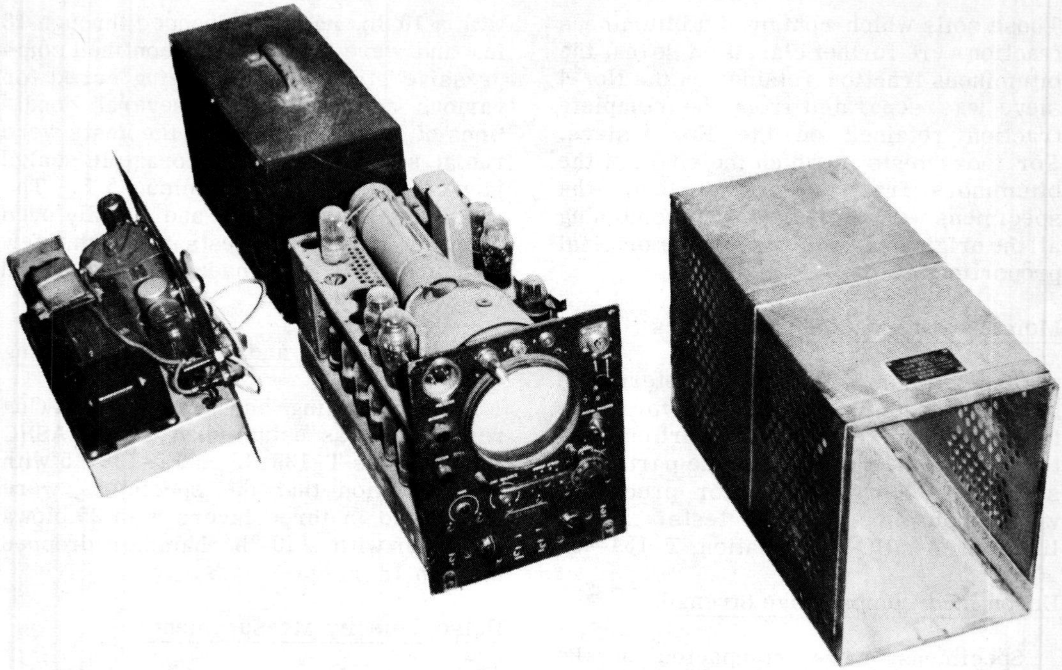


Figure 1. Pulse-velocity equipment with covers removed.

The same procedure was employed in the present investigation. In order to cut down on the amount of work, the durability studies were largely confined to the investigation of these optimum mixtures. The field projects also employed proportions as used in the laboratory tests.

The following procedures were used in the laboratory investigation.

Soil Classification

The soil samples were classified in accordance with standard AASHO methods.

TABLE 2

PROPERTIES OF HYDRATED LIME AND FLYASH

	Dolomitic Hydrated Lime	Flyash	
Chemical Analysis		Philadelphia Area	Baltimore Area
SiO_2	1.0	35.95	44.17
Fe_2O_3	0.4	22.67	17.28
FeO	0.0	-	-
Al_2O_3	0.2	23.13	28.65
CaO	47.8	7.06	2.79
MgO	33.8	1.01	0.89
Loss on Ignition	16.3	6.15	4.97
CO_2	0.8	-	-
H_2O	0.5	0.28	0.21
Sieve Analysis			
Sieve No.			
60 (Total % Ret.)	1.0	0.75	2.23
100 (Total % Ret.)	2.8	4.60	8.33
200 (Total % Ret.)	5.6	-	-
325 (Total % Ret.)	-	28.58	27.84
Specific Gravity	2.60	2.46	2.31
Dry Rodded Density (lb. per cu. ft.)	45	70	70

Those soils which contained a bituminous fraction were further classified so that the bituminous fraction retained on the No. 4 sieve was separated from the complete fraction retained on the No. 4 sieve. For those tests in which the effect of the bituminous fraction was studied, the specimens were prepared by recombining all the original ingredients in their original proportions.

Moisture-Density Characteristics

The optimum moisture was determined for samples of natural soil and for lime-flyash-soil mixtures of proportions determined to be suitable for the particular soil type. Standard Proctor procedure was employed for these tests, as outlined in AASHO Designation T-134-45.

Unconfined Compressive Strength

Specimens were compacted in the Proctor mold with 25 blows per layer with a 10-lb. hammer dropped through 18 in. and were tested for unconfined compressive strength after being cured for various ages and under several conditions of storage. Most of the tests were run at seven days with storage in sealed jars at 140 F., plus or minus 5 F. The specimens were capped and usually oven dried for most of the tests although a few measurements were made under saturated moisture conditions.

Wetting-Drying and Freezing-Thawing

Wetting-drying and freezing-thawing resistance was established using AASHO Designations T-135-45 and T-136-45 with the exception that the specimens were compacted in three layers with 25 blows per layer with a 10-lb. hammer dropped through 18 in.

Pulse-Velocity Measurements

The equipment used for the measure-

Figure 2. Measuring pulse velocity with portable field unit.

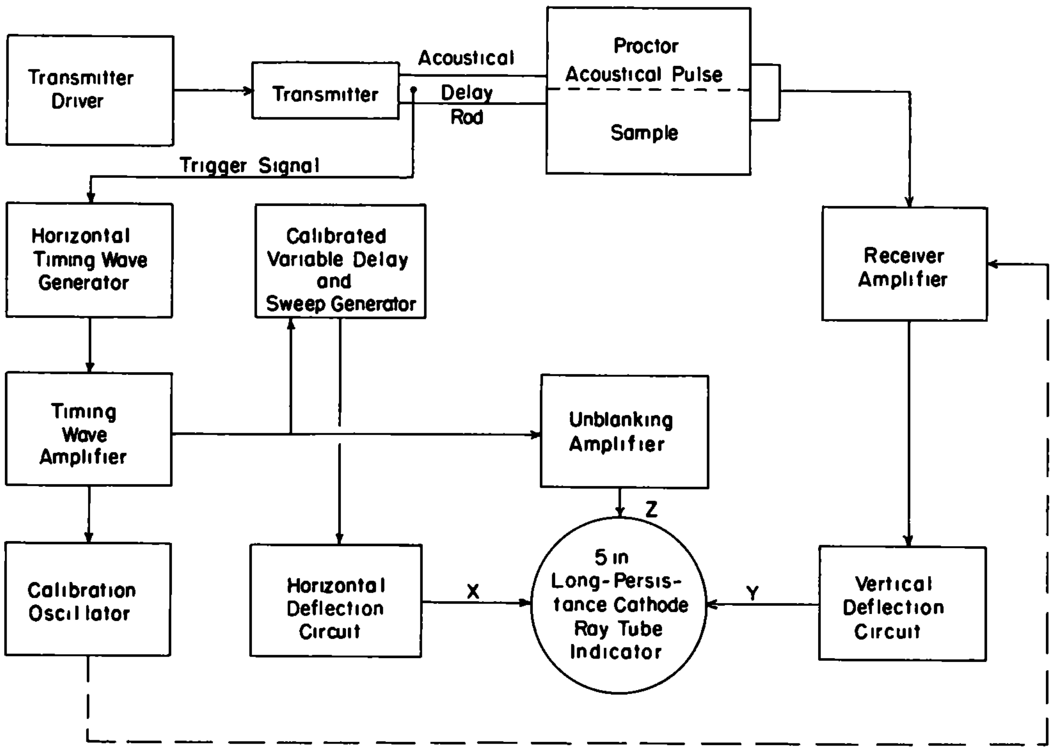


Figure 3. Block diagram for compressive wave-front-velocity measurement of a proctor sample A, transmitter driver, B, transmitter, C, acoustical-delay rod, D, proctor sample, E, receiver, F, receiver amplifier, G, vertical-deflection circuit, H, horizontal timing-wave generator, J, timing-wave amplifier; K, unblanking amplifier, L, calibrated variable-delay and sweep generator; M, horizontal-deflection circuit, and N, calibration oscillator.

ment of pulse group velocity is described in the next section in detail. A number of the laboratory samples prepared for the program were tested with this equipment. Specific efforts were made to obtain measurements for compression wave-front velocity during the wetting and drying, and freezing and thawing tests, and on samples which were tested for unconfined compression. Measurements were also made on specimens with different moisture contents to determine the effect of water on the test results.

In addition to the laboratory tests, velocity measurements were made in the field. Some of the tests were carried out using axial in-line placement of the transmitter, receiver, and sample, which resulted in the use of the compressional wave. In order to do this, sufficiently large depressions were cut in the road to allow placement of the transducers 3 in. below the surface. The transducers were then pressed laterally face to face against a significant length of road base, usually about 20 in. An oiled-clay mixture was placed between the road base and each transducer to assure good transfer of energy. The reading was made of the time required for the impact leaving the transmitter to reach the receiver.

In order to reduce the amount of disturbance to the road resulting from preparation of the base for the longitudinal wave-front tests, traction impact velocities were also measured at several field locations, and have been used in the majority of the recent tests. Traction

TABLE 3

Soil and Sample Designation	Remarks	Composition: Soil	FA	Lime	Moisture (dry basis)	Dry Density	Curing: Oven	Moist.	Comp Str	Velocity
		parts by weight			%	lb per cu ft.	Days	Months	psi.	ft. per sec
N J Turnpike A-1-b New Brunswick Interchange 9										
Field Sample		90	10	5	10.0	125 0	7	--	583	
" "		90	10	5	10 0	125 0	7	--	535	
" "		90	10	5	10.0	125 0	7	--	694	
" "		90	10	5	10 0	125.0	14	--	1050	
" "		90	10	5	10 5	125 0	7	2/3	704	
" "		90	10	5	11 5	125 0	7	2/3	800	4620
" "		90	10	5	11 5	125 0	7	2/3	810	4670
" "		90	10	5	9 0	125 0	7	2/3	621	3420
" "		90	10	5	12 0	123 0	7	3	920	4145
" "		90	10	5	10.0	126.0	7	3	398[f]	3430[f]
" "		90	10	5	10.0	126 5	7	3	706[f]	4880[f]
" "		90	10	5	11 5	123.5	7	3	850[f]	4170[f]
" "		90	10	5	10.0	126.0	7	2½	865[a]	5100[a]
" "		90	10	5	11 5	124.0	7	2½	780[b]	3050[b]
N J Turnpike A-1-b Hightstown Interchange 8										
Lab Sample	(Bituminous	90	10	5	10 0	125 0	7	--	400	
" "	(fraction	90	10	5	9 0	124.0	7	--	430	
" "	(omitted	90	10	5	9 0	124 0	7	2/3	1191	
" "	(	90	10	5	9 0	124.0		3	1225	
" "	(Bituminous	90	10	5	10 0	125 0	7	--	375	
Field Sample	(fraction	90	10	5	9 5	123 5	7	5	871[a]	4330[a]
" "	(included	90	10	5	9 5	123 5	7	5	872[b]	3590[b]
N J Turnpike A-1-b Bordentown Interchange 7										
Lab Sample	(Bituminous	90	10	5	10 0	122 0	5	--	655	
" "	(fraction	90	10	5	10 0	122 0	7	--	750	
Field Sample	(omitted	90	10	5	11.5	123 5	7	--	708	
Lab Sample	(Bituminous	90	10	5	10 0	122.0	5	--	485	
" "	(fraction	90	10	5	10 0	122 0	7	--	735	
Field Sample	(included	90	10	5	10 0	122 0	7	2½	280[a]	2130[a]
N J Turnpike A-1-b Burlington Interchange 5										
Field Sample		90	10	5	9 0	125 0	7	--	375	
" "		90	10	5	7 9	120 2	28	--	960	
" "		90	10	5	7 9	120 2	28	2	732	
Marlboro By-Pass A-2-5 Marlboro, Md.										
Field Sample		90	10	5	9 2	121 0	7	--	467	
" "		90	10	5	9 0	121 0	7	2/3	896	6250
" "		90	10	5	9.0	121 0	7	2/3	912	6650
" "		90	10	5	9 5	121 0	7	2½	910[b]	6170[b]
" "		90	10	5	9 5	121 0	7	2½	1280[a]	6190[a]
HiNella, N J A-5										
Lab Sample		90	10	4	8 0	125 0	5	--	350	
" "		90	10	4	9 0	124.0	5	--	360	
Field Sample		90	10	4	8 0	125 0	7	--	1010	
" "		90	10	4	8 0	125 0	7	2/3	1253	

TABLE 3 (continued)

Soil and Sample Designation	Remarks	Composition Soil	FA	Lime	Moisture (dry basis)	Dry Density	Curing Oven	Moist.	Comp. Str.	Velocity
		parts by weight			%	lb. per cu ft.	Days	Months	psi	ft. per sec.
Crestwood Ave										
A-5										
Somerdale, N. J.										
Lab Sample		90	10	4	8 0	124 5	7	--	450	
" "		90	10	6	9 0	120.0	7	--	390	
Field Sample		90	10	3	9 0	130.5	7	2/3	1605	6450
" "		90	10	3	9 0	132 0	7	--	1382	6450
" "		90	10	3	9 0	131 2	7	--	1185	6350
" "		90	10	3	9 0	130.2	7	--	1180	6550
" "		90	10	3	9 0	123 8	7	1 1/2	1660[b]	7010[b]
U S Avenue										
A-3										
Lindenwald, N. J										
Lab Sample		85	15	3	8.0	127 2	2	--	177	
" "		85	15	4	7.8	128.2	2	--	244	
" "		85	15	5	8.0	127.2	2	--	365	
Mercer Ave										
A-3										
Barrington, N J										
Lab Sample No	1	90	10	3	10 0	120 5	4	--	585	
" "		90	10	5	11 7	118.8	4	--	758	
Lab Sample No	2	90	10	3	9 0	117.6	1	--	242	
" "		90	10	5	9 0	117 8	1	--	258	
Field Sample		90	10	4	9.0	127 5	7	--	1345	5720
" "		90	10	4	9.0	128 5	7	--	1365	5790
" "		90	10	4	9.0	128.8	7	--	1680[a]	5410[a]
" "		90	10	4	9.0	128.0	7	--	1325[c]	5840[c]
" "		90	10	4	9 0	128.1	7	--	1610[d]	5320[d]
" "		90	10	4	9 0	128.2	7	--	1790[b]	4620[b]
" "		90	10	4	9 0	128.7	7	2/3	1459[e]	6440[e]
Whaleysville, Md.										
A-3										
Lab Sample		85	15	6 3	6 5	124.1	7	--	1420	8280
" "		80	20	5 3	5 2	124.8	7	--	984	5640
" "		75	25	3 0	5 3	124.8	7	--	1152	5720
E. Atlantic Ave										
A-1-a										
Oaklyn, N. J										
Lab Sample		92.2	7 8	3.0	10 2	117 2	7	--	374	3760
" "		92.2	7 8	3.0	10.2	114.2	7	--	371	3760
" "		90.25	9 75	3 25	9 5	116.0	7	--	383	
" "		90.25	9 75	3 25	9 5	116.0	7	--	328	
Woodbury, N J.										
A-1-b										
Lab Sample		90	10	4	9.0	121 0	7	--	390	
" "		90	10	4	9 0	121 0	7	--	365	
Field Sample		90	10	4	9 0	121 0	7	--	580	
" "		90	10	4	9 0	121.0	7	--	590	
Laurel By-Pass										
A-2-5										
Laurel, Md.										
Lab Sample		90	10	5	9 0	127.0	7	--	900	
" "		90	10	5	9 0	127.0	7	--	920	

[a] After 12 cycles of wet-dry

[b] After 12 cycles of freeze-thaw

[c] After 12 cycles of wet-dry, tested in saturated state.

[d] After 12 cycles of freeze-thaw, tested in saturated state

[e] Tested in the saturated state

[f] Tested in the 100 percent relative humidity state

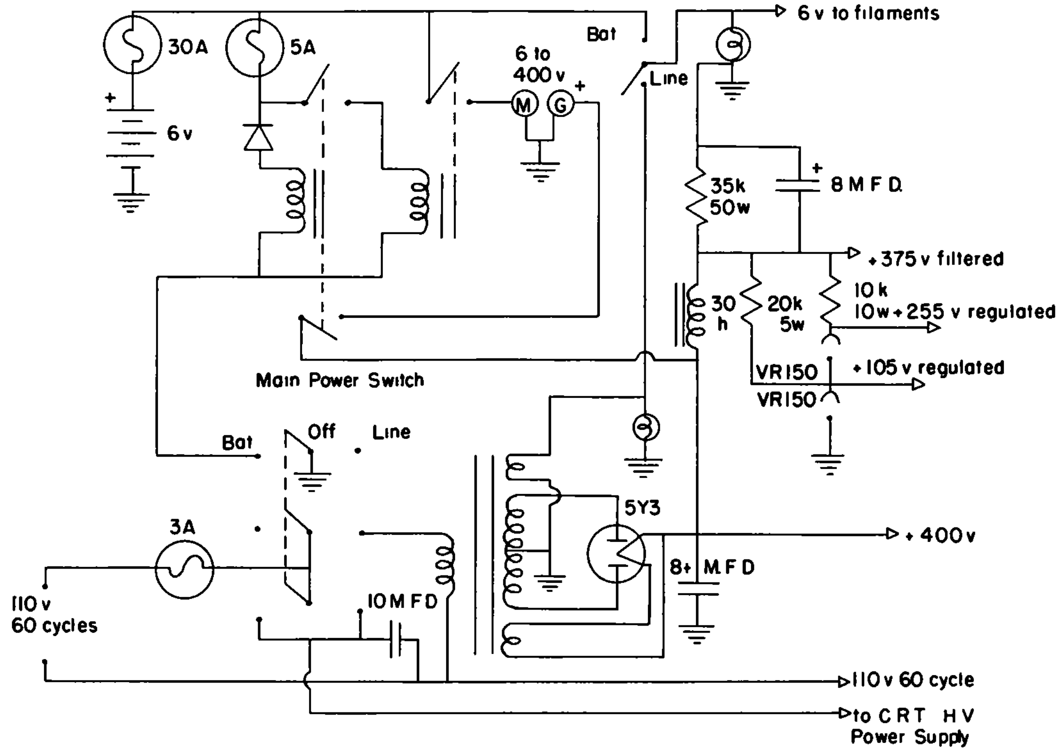


Figure 4. Battery-line-regulated power supply.

impacts were generated by inserting the transmitter delay rod vertically into a 1-in.-diameter hole cored two thirds of the way into the road base. For this type of measurement the pick-up was placed at a number of points on a 3-in. circle around the transmitter and was applied face down to the surface of the road. Another procedure which is currently being investigated involves the measurement obtained by placing both the transmitter and receiver vertically downward upon the surface of this base material. In some cases cuts were made in the material between the transmitter and receiver to determine the

TABLE 4

Typical Example of Five Soils Measured for Compression Wave-front Velocity in the Saturated and Dried States of Wetting-Drying Cycle Tests on Cured, Optimum Range, Lime-Flyash-Stabilized Soil Compositions

Woodbury, New Jersey

Cycle	Moisture Content: Saturated %	Moisture Content: Dried %	Velocity: Saturated ft. per sec	Velocity: Dried ft. per sec
0	15.2	9.0[a]	4,180	4,590[a]
1	14.6	1.5	4,190	4,200
2	14.1	0.9	4,010	4,280
3	14.2	0.7	3,780	3,770
4	14.6	0.6	4,120	3,870
5	13.8	0.5	3,990	3,910
6	13.8	0.4	4,220	4,030
7	14.0	0.8	4,260	4,260
8	13.7	0.8	3,950	3,710
9	13.5	1.1	4,040	4,460
10	13.4	0.9	3,830	4,310

[a]Sample as cured at 100 percent relative humidity

depth to which some of the waves penetrated. In making the measurements on the road base, the condition of the base was carefully noted, particularly with respect to moisture content. In general, the tests were run at times when the road base was either quite dry or in a wet state rather than at times when the material was in a partially dried out condition.

SONIC TEST EQUIPMENT

Leslie and Cheesman have found the velocity of sound as measured by the soniscope (4) to be a useful method for investigating the characteristics of concrete. The soniscope has been used by Whitehurst in the evaluation of concrete (5) and also by Whitehurst and Yoder in the study of lime-soil stabilization (3). The authors wish to express appreciation to the above mentioned investigators for their very helpful assistance and advice during the construction of the instrument used in the previous study (2) and as modified for the present investigation. This apparatus represents a consider-

TABLE 5

Field inspection data from three lime-flyash-stabilized soil shoulder installations in the New Jersey Turnpike
Composition, parts by weight. Lime, 5, flyash, 10, soil 90

Item No	Curing-Days Field	Type of Wavefront Measured	Details Concerning Velocity Determination	Velocity Determination Ft. per Sec
Location New Brunswick Interchange No. 9 Ramp to south on Route S28 Station 0 + 20 situated under an overpass Material at this location was generally wet, cool, and subject to little traffic				
1	0	Direct Compression	Received signal weak and wavefront degenerated	1,710
2	0	" "	" " " " " "	2,050
3	0	Traction	Dial readings 253, 266, 295, and 300	1,260
4	87	"	" " 140, 141, 137, and 144	3,100
Drilling was fairly difficult, material compact, frequent visible particles of lime one-quarter inch and smaller evident from test hole corings.				
5	87	Traction	Dial readings 143, 135, 141, and 131	3,200
6	"	"	" " 141, 140, 153, and 140	3,000
7	88	Compression	On sample removed from road	4,030
8	"	"	" " " " "	4,180
Location New Brunswick Interchange No. 9 Ramp to south on Turnpike Station 2 + 80. Material at this location bearing steady traffic and the majority of heavy trucks rising and accelerating.				
9	87	Traction	Dial readings 106, 110, 107, 103	4,500
Drilling was extremely difficult using star drill and hand sledge Some visible particles of lime one-quarter inch and smaller were evident from test hole corings The appearance of the shoulders was excellent and no distinction between shoulders and the pavement was evident from performance under traffic				
Location. Hightstown Interchange No 8 Northbound Exit Ramp, right-hand shoulder Station 12 + 15. The traffic load observed was heavy.				
10	9	Direct Compression	Signal path, 3 12 ft.	3,520
11	"	" "	" " 1.33 ft.	3,550
12	"	Traction	Dial Readings 175, 183, 174, 202	2,135
These readings were taken at the center of the shoulder				
13	95	Traction	Dial Readings 132, 140, 142	3,300
These readings were taken one foot from the outer edge of the pavement.				
14	95	Traction	Dial Readings· 195, 160, 152	2,400
These readings were taken one foot from the outer edge of the shoulder				
15	95	Traction	Dial Readings. 151, 150, 155	2,800
Location. Bordentown Interchange No 5 Ramp "B" Station 333 + 00 This location is on a "J" Loop before the southbound acceleration lane Traffic was light, but generally rode on the shoulder No visible distinction between pavement and shoulder was evident from appearance				
16	180	Traction	Horizontal distance between transducers, 1½ in.	3,870
17	"	"	" " " " 3 in.	4,250
18	"	"	" " " " 4 in.	4,060
19	"	"	" " " " 5 in.	3,960
The initial wave front intensity decreased steadily as the distance between transducers was increased. The received signal showed a second wavefront of lower velocity increasing with receiver distance				

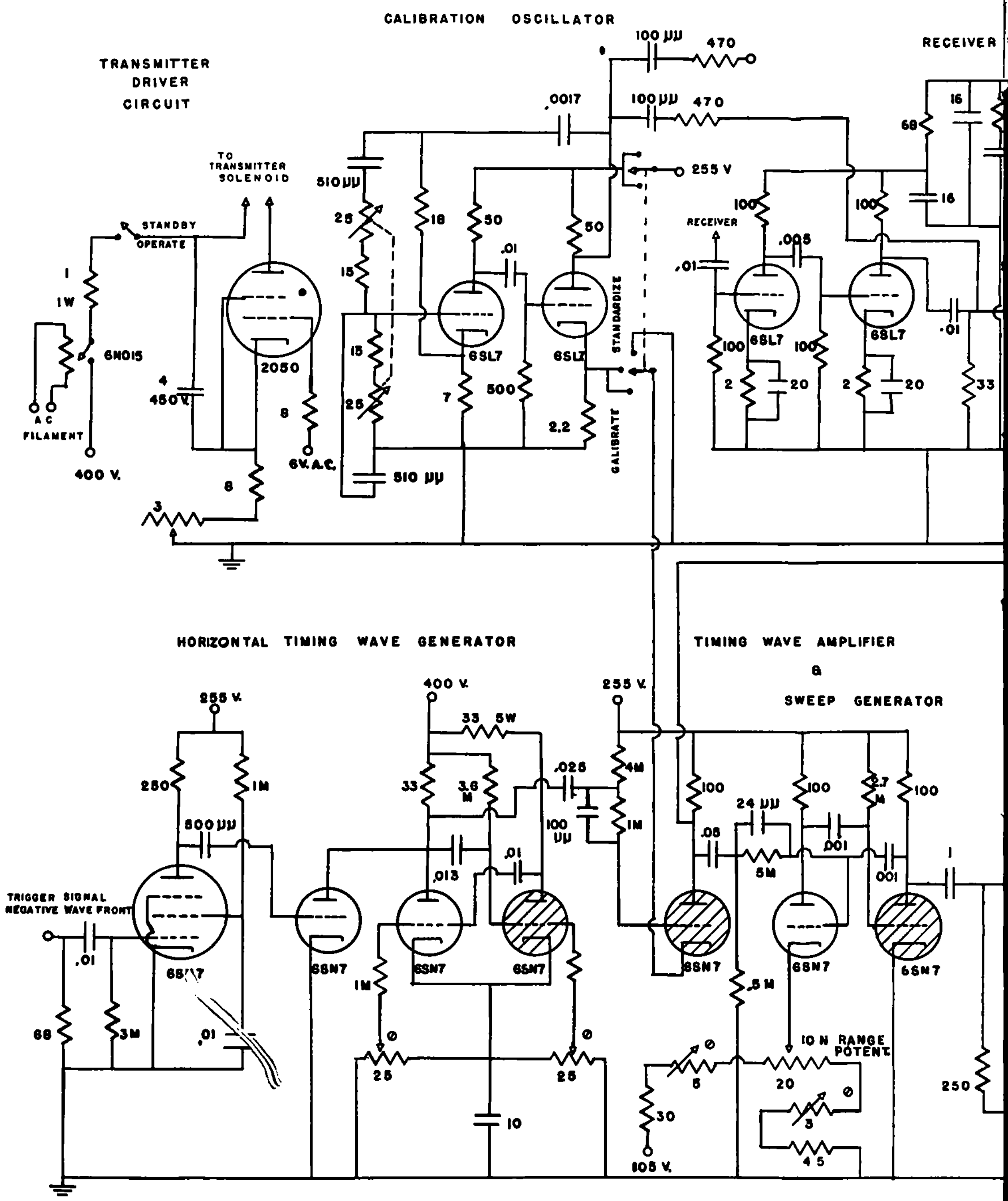
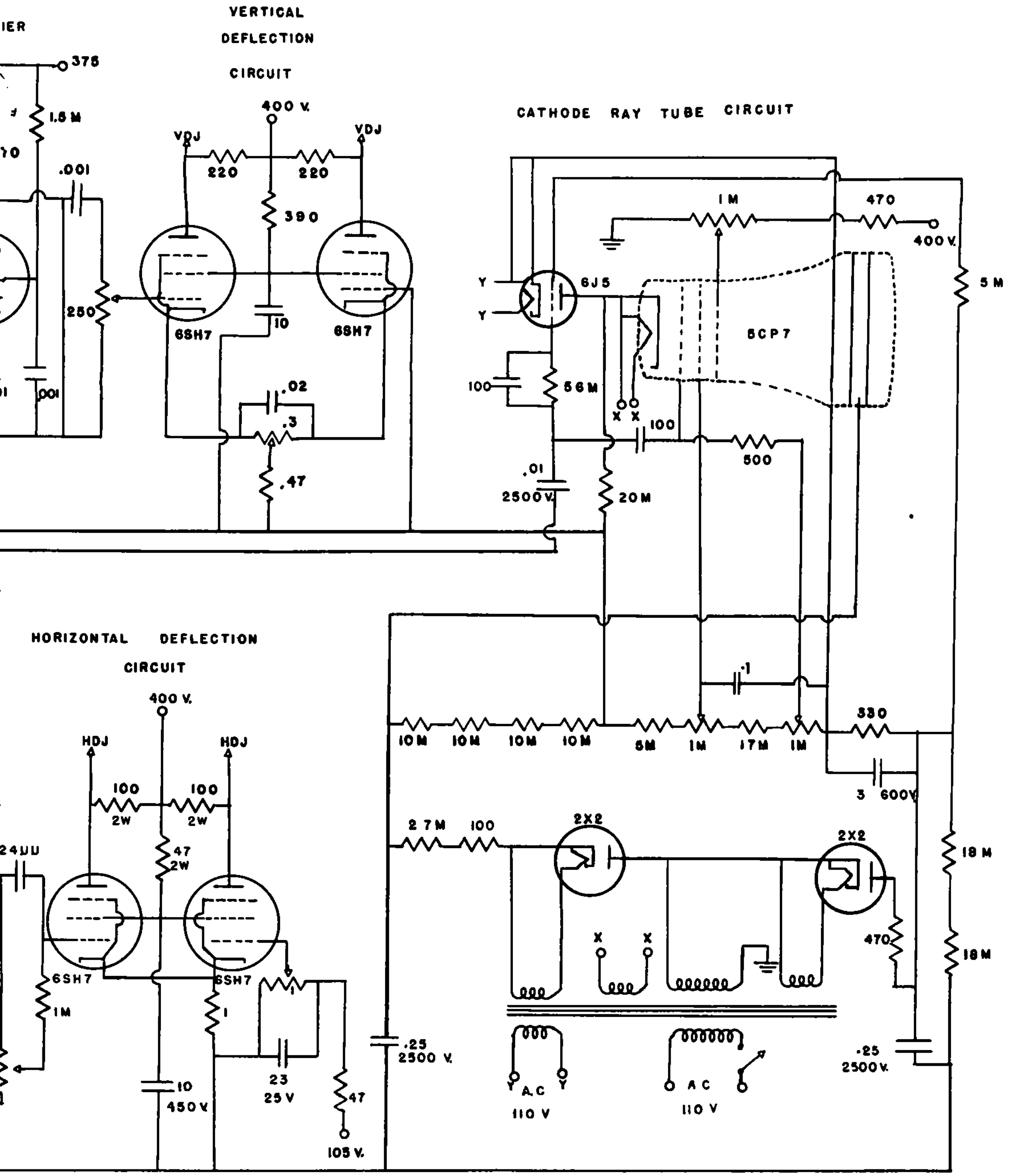


Figure 5. General schematic: Resistance in kilohms



acitance in microfarads unless otherwise stated.

Figure 6. New Jersey Turnpike project: Condition of shoulder prior to treatment.

Figure 7. Preparation of shoulder for treatment. Blade is removing loose bituminous material. This operation is followed by scarifying to desired depth with the motor grader, N. J. Turnpike project.

Figure 8. Bulk flyash being spread by motor grader, N. J. Turnpike project.

able departure in design from that described by the above investigators, although measurements made on both type instruments give substantially identical results on the same laboratory specimens. Recently Meyer (6) has described a modified form of soniscope which also employs a different transmitter system to that described in the early investigations.

Calibration has been carried out by the use of standard frequencies and by measurements of materials for which the velocity values are available in the literature. It is felt that the instrument described in this paper not only measures pulse velocity but is adaptable to other quantitative measurements of particular use in soil stabilization work. This instrument has given stable readings upon numerous materials in the field and laboratory. The unit may be operated from 100- to 130-volt, 60-cycle, A. C. lines or by a conventional 6-volt lead-acid storage battery. A 120-amp.-hr. battery provides 4 hr. of continuous service from a full charge. The instrument was built up from both new and surplus components, and the cost of all parts (some of which are not now in use) was approximately $500. There has been practically no maintenance required on the equipment, even though it has been subjected to rather rough usage on numerous field trips.

Views of this instrument are shown in Figures 1 and 2. The unit is divided into two-handle-equipped major units of approximately equal weight for convenience in transportation between the laboratory and field locations. Storage batteries are maintained as separate units as well as a 65-watt inverter which it is planned to eliminate by circuit revision.

The internal operation of the apparatus is outlined by Figure 3, a block diagram. An acoustical pulse is generated by a thyratron-driven electric hammer which strikes a target at a rate of approximately three times a second. The leading edge of this impact triggers the horizontal and unblanking systems for a long-persistence-screen cathode-ray

Figure 9. Mix-in-place Pulvimixer incorporates the lime and flyash into the existing shoulder. Lime was supplied in 50-lb. sacks and dropped on shoulder as shown, N. J. Turnpike project.

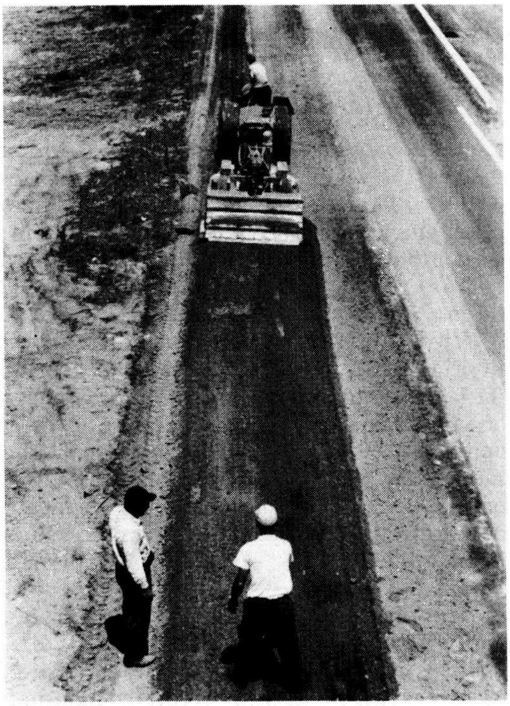


Figure 10. After several passes with Pulvimixer a homogeneous composition results which is ready for moisture check and rolling with rubber-tired and flat-wheeled rollers, N. J. Turnpike project.

tube. The pulse is conducted through a steel delay rod 1 ft. long at the end of the transmitter to the sample where transfer is effected through an oil clay mixture. The impact passing through a sample path of known length is registered by the receiver pickup. The polarity (compression or traction) and modulation of the wave front as well as the transit time required in the sample are indicated by vertical displacement of the trace. Measurement of the transit time is effected by the introduction of an equivalent calibrated

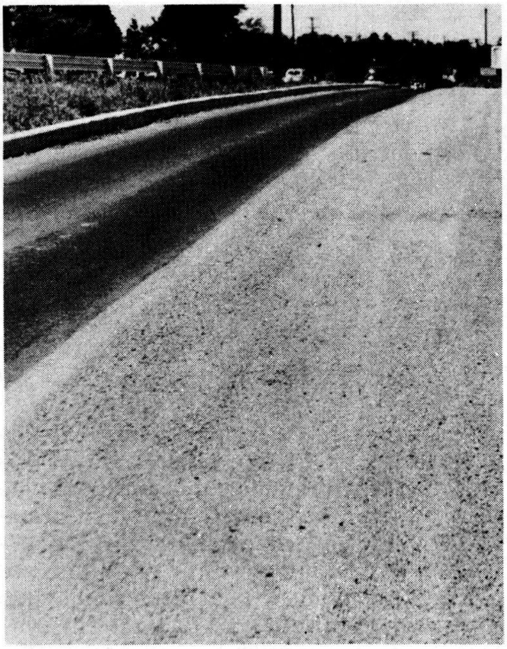


Figure 11. Completed shoulder after application of oil and stone chips to stabilized base, N. J. Turnpike project.

delay in the horizontal sweep; this results in the leading edge of the wave-front trace being translated to zero. The first portion of the sweep is rapid which permits easy measurement. In addition, the latter portion of the sweep may be varied by the horizontal gain control to provide for convenient interpretation.

Figure 4 shows the 110-v. -A. C. -to-6-v. -D. C. -power-supply schematic drawing not including the inverter which at present provides low-wattage 110 volts of A. C. on battery operation for the cathode-ray-tube power supply. Figure

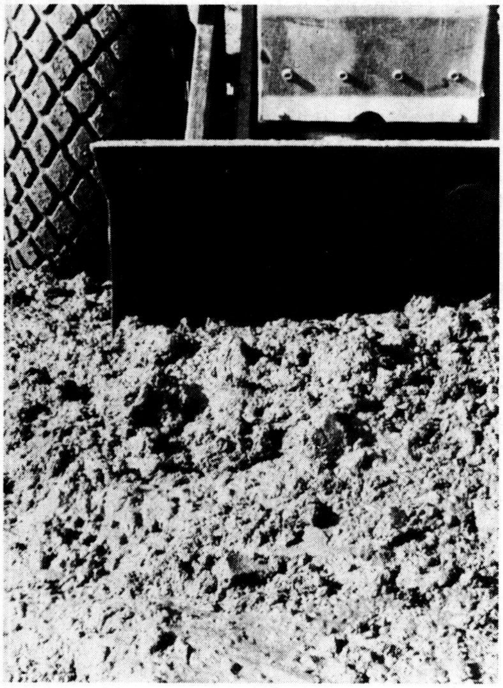


Figure 12. Soil at HiNella, N. J.: Project contained appreciable quantities of green sand marl.

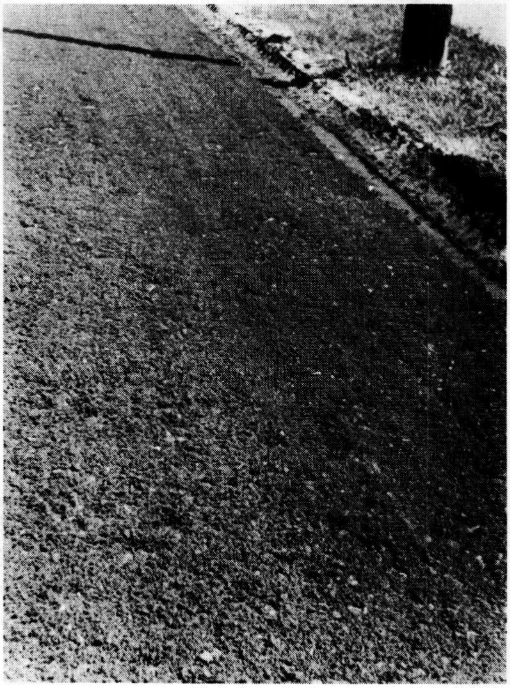


Figure 13. Condition of road base prior to compaction and after mixing lime and flyash into the soil, HiNella, N. J.

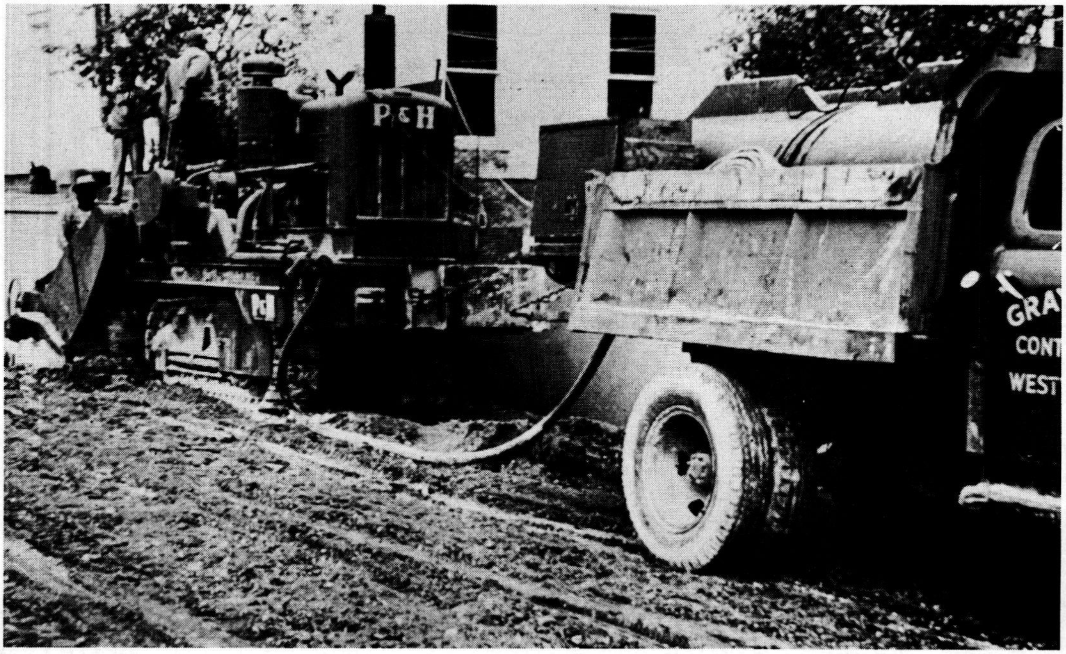


Figure 14. Stabilization project at Woodbury, N. J.,using P&H single pass unit for incorporating lime and flyash into the soil.

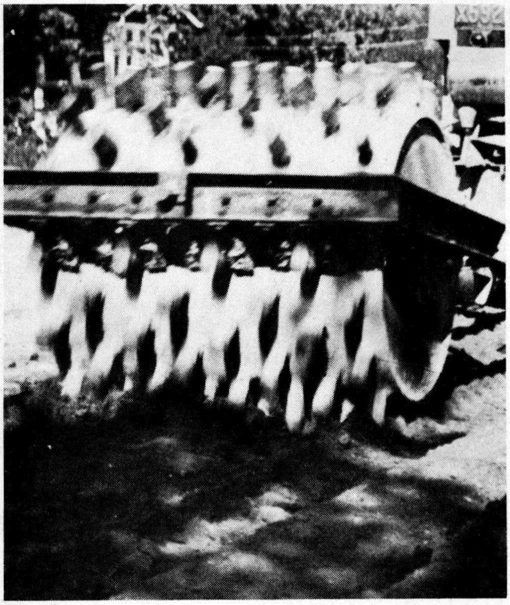


Figure 15. Compaction of stabilized shoulder using sheepsfoot roller, followed by rubber tire and flat steel rollers. Taken on shoulder at Somerdale N. J.

5 is a general schematic diagram of the instrument.

The signal generated contains an initial high-energy wave front which passes from the steel rod to the sample with a satisfactory acoustical impedance ratio for the materials giving good high-frequency transfer. Fast initial rise provided in the wave front by the high-frequency component results in sharp resolution in small samples. In addition, the attenuation effect of the samples as related to the wave form is felt to be useful in the further evaluation of the compositions. The peak force of the impact may be set to allow relatively low receiver-amplifier gain despite passage of the signal through such field specimens of high attenuation as stabilized soil. Good orientation of the transducers is also beneficial in such applications.

FIELD PROJECTS

The field projects reported in this paper represent mixed-in-place opera-

TABLE 6

Field inspection data from the lime-flyash-stabilized soil project at Woodbury, N. J.
Composition, parts by weight: Lime, 4; flyash, 10; soil, 90.

Item No.	Curing-Days Field	Type of Wavefront Measured	Station No.	Details Concerning Velocity Determination	Velocity Determination Ft. per Sec.
1	55	Traction	U-1	Dial readings: 114, 118, 120, and 113	5,420
	This material showed good mixing, compaction, and was difficult to drill. This station was observed to bear maximum traffic and to be exposed to weathering.				
2	55	Traction	U-2	Dial readings: 170, 185, 168, 162	2,450
	Test hole drilling was very easy and the corings indicated a variable composition. The job history showed that the machine mixer could not operate in this station. See item No. 7 below.				
3	55	Traction	U-3	Dial readings· 137, 142, 150, 148	3,130
	The location and properties of station U-3 were intermediate to U-1 and U-2.				
4	55	Traction	U-4	Dial readings: 130, 125, 126, 142	5,070
	This area was exposed to weathering. Test hole corings show particles of lime one-quarter inch and smaller.				
5	55	Traction	U-5	Dial readings: 117, 124, 142, 138	3,750
	This area is shaded by a large tree. Particles of lime one-quarter inch and smaller were present.				
6	55	Traction	U-6	Dial readings: 141, 151, 132, 141	3,140
	This station similar to U-5 though traffic load was greater.				
7	133	Traction	U-2	Dial readings: 102, 122, 149, 122	4,260
	Drilling was rather difficult. Compare with item No. 2 above.				
8	133	Traction	U-3	Dial readings: 147, --, 125, 147	3,080

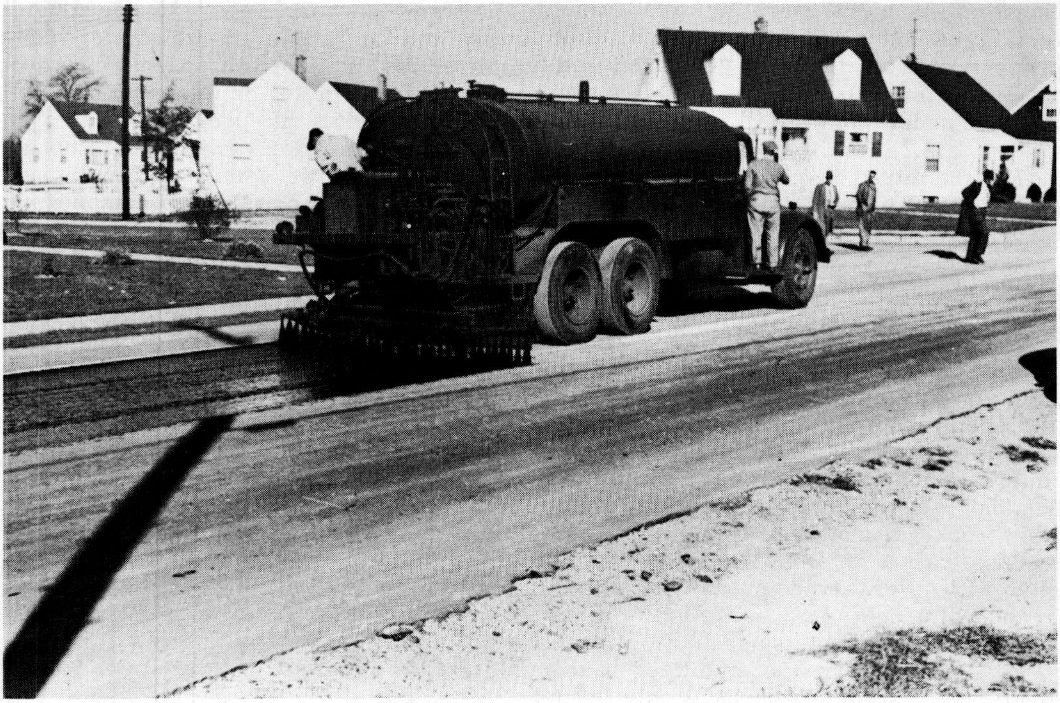


Figure 16. Application of MC-3 oil on stabilized base at project in Lindenwold, N. J.

Figure 17. Application of stone chips over shoulder project at US 301, Marlboro, Md.

tions in which the existing road material was subjected to the stabilization process. Projects using manufactured material imported to the job sites are likewise under way but will be reported at a later date.

For the mixed-in-place operation, the procedure in general followed that illustrated in Figures 6 to 11. The chief concern during construction was to establish good mixing and proper compaction. Constant check of moisture and density was carried out during the operation. Figures 12 to 19 show scenes taken from field projects constructed with the materials referred to in this paper.

A few tests were run using hydrated lime supplied in bulk form. While this has not been employed in the present field work, it has been established that the application of bulk lime is quite feasible with several types of sand spreaders in general use.

Several types of surface treatment have been placed on the stabilized base. Since most of the roads have only been constructed recently, it is too early to determine the relative merits of the various methods employed. The oldest application has been through three winters and is holding up quite well, showing no sign of deterioration. The use of both asphaltic oil and tar with applications of stone chips has been tried. In addition, the use of bituminous concrete in thickness of 1 to 2 in. has been employed and this treatment also appears to be performing satisfactorily.

There have been frequent inspection trips made to the field projects and a definite program of checking has been set up. Of particular interest are those points which were prepared under adverse weather conditions or which were subjected to unusual conditions of traffic during construction or were involved with such problems as poor drainage conditions, high water tables, poor subbase material, etc. Occasional specimens have been removed for unconfined compressive strength. The use of the portable equipment for making pulse-velocity measurements in the field has been in operation for about 6 mo. These measurements were usually made during the early stages on those roads recently constructed. Since this is the age period where considerable increase in strength is developed, the

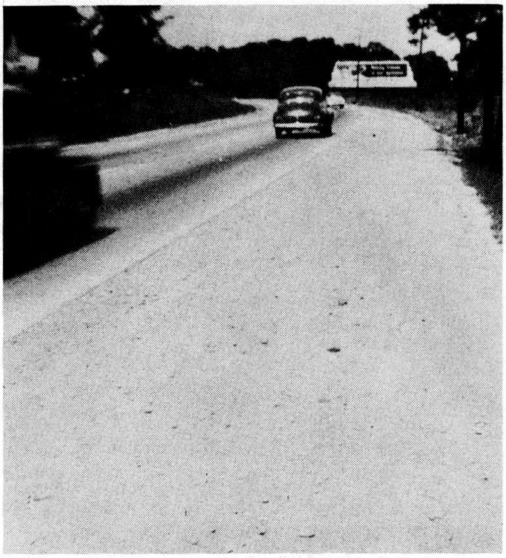


Figure 18. Condition of stabilized shoulder after one winter of service, no wearing course applied, US 1 Bypass, Laurel, Md.

TABLE 7

Field inspection data from the lime-flyash-stabilized shoulders of Navajo Road, Camden County, New Jersey. Composition, parts by weight: Lime, 4; flyash 10; soil, 90.

Item No.	Curing-Days Field	Type of Wavefront Measured	Station No.	Details Concerning Velocity Determination	Velocity Determination Ft. per Sec.
1	68	Traction	1	Dial readings: 116, 112, 123, 133	3,930
The native soil underlying this road and that used in the road, contained considerable glauconite.					
2	68	Surface Compression	4	Distance between transducer center axes, 3 in.	4,170
3	"	"	"	Distance, 6 in.	3,850
4	"	"	"	Distance, 9 in.	3,710

Figure 19. Condition of shoulder adjacent to stabilized section. US 1 Bypass, Laurel, Md.

results of these measurements are considered to be quite pertinent. Some of the observations made of the field tests are included in Tables 5, 6, and 7.

TEST RESULTS

The results of tests on optimum density-moisture requirements are included in Tables 1 and 3. In general, it has been noted that there is a slight drop in density with the lime-flyash mixtures as compared with those of the straight soil. Also there is a small change in the water requirement for the optimum compositions.

Unconfined Compressive Strength

Table 3 includes results for compressive strength tests for various soils. Most of these results are based on oven curing for 7 days, although it may be noted that the soil designated Mercer Avenue, A-3, was tested at 1 and 4 days, and the soils designated New Brunswick, New Jersey A-1-b and Marlboro, Maryland, A-2-5 were tested after additional 21 days curing at room temperature in a moist curing room. Also the soil designated Hightstown, New Jersey A-1-b was tested after aging three months by curing under moist conditions at room temperature. The soils marked HiNella, A-5 and Somerdale, A-5 both contained appreciable quantities of glauconite.

A number of compressive-strength values are given for specimens which were first subjected to freezing-and-thawing and wetting-and-drying tests. The strengths of these specimens were observed to be quite good.

It will be noted that the test results are included with the Bordentown, New Jersey, Soil A-1-b for those specimens which contained a bituminous fraction. Tests at both 5 and 7 days are presented. It is evident that the sample containing the bituminous fraction shows a significantly lower compressive strength value in 5 days, although the 7-day results are more nearly in agreement with the sample free of bituminous material.

Two of the soils, Mercer Avenue A-3 and New Brunswick A-1-b, were tested for compressive strength both in wet and in dry condition. From the results of the latter tests as well as other data available but not reported here, it is indicated that the samples which are in wet condition at the time they are tested give slightly lower compressive-strength values than

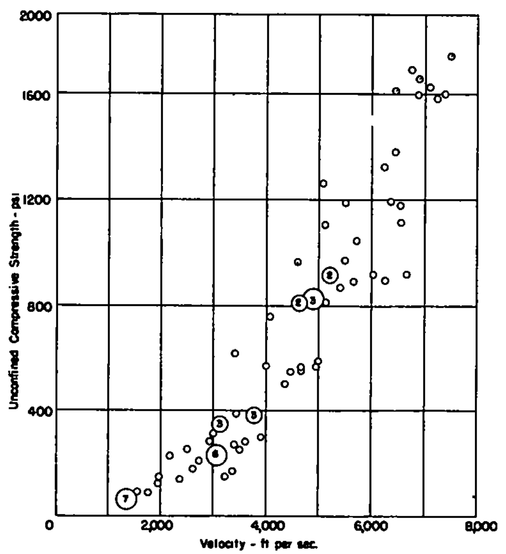


Figure 20. Unconfined compressive strength and velocity data for cured lime-flyash-stabilized soil samples in the oven dry state. These values represent both Proctor and 2-in. cubes of various soil classifications and curing conditions. The compositions include optimum and non-optimum proportions. Clusters of points are represented as circled numerals indicating the quantity of data contained in the area.

samples which have first been dried to constant weight. This would appear to conform to the results obtained from velocity tests described below.

All of those values which represent optimum proportions of lime and flyash are quite good, although it has been found that the laboratory results are usually somewhat lower than the results obtained on samples which were made from material mixed at the job site, prepared in the field, and then brought back to the laboratory. For example, in the soil designated Somerdale, New Jersey, A-5, the results from the field specimens were approximately 1,200 psi. as compared with the laboratory specimens which were 400 psi. The lime content in this field test was lower than that used in the laboratory mix. Similarly, the tests taken at HiNella showed nearly three times the strength in the field specimens. In addition to the above, samples removed from the road after being in service for periods of 6 mo. or longer usually averaged better than 2,000 psi. in unconfined-compression tests.

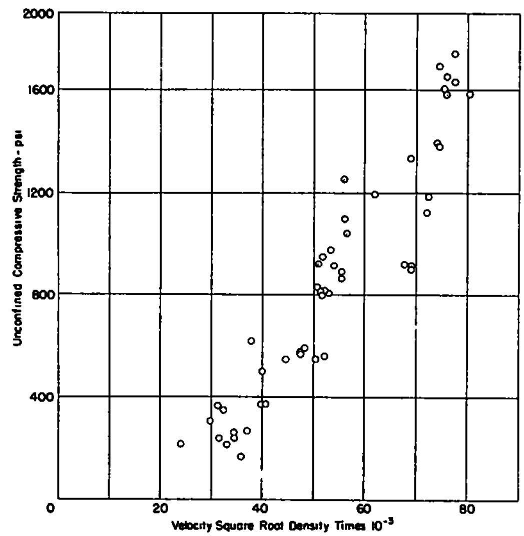


Figure 21. Unconfined compressive strength versus velocity-square-root density for optimum range lime-flyash-stabilized soil compositions. Proctor samples in the oven dry state.

Wetting and Drying

Wetting-and-drying specimens gave the same type results described in the previous papers (1, 2). The results of the wetting-and-drying tests are shown in Figures 23 to 27, and as will be noted, the specimens show good resistance to this treatment. In addition, the results are compared with the pulse-velocity tests. This is discussed more fully below.

Freezing and Thawing

Results of the standard test on the Proctor specimens showed that the materials with lime and flyash developed excellent resistance to freezing and thawing. The results of the test are given in Figures 23 to 27 inclusive. This method of determining freezing-and-thawing resistance is also compared with the pulse-velocity method as discussed in the next section.

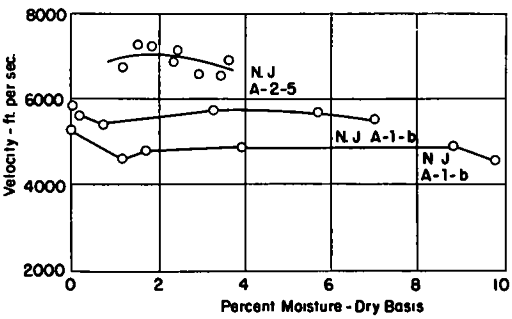


Figure 22. Moisture-velocity curves for cured lime-flyash-stabilized soil samples.

Pulse-Velocity Tests

Results of the pulse-velocity measurements are contained in Tables 4 to 7 and Figures 20 to 27 inclusive. A preliminary correlation between unconfined compressive strength and pulse velocity for cured lime-flyash-stabilized soil samples was reported in a previous paper (2) and included both optimum and nonoptimum compositions for an A-2-5 and an A-7 soil. Figure 20 gives an overall plot of unconfined compressive strength against pulse velocity measured on oven-dry samples prepared in the present investigation and also includes the data previously reported. Fourteen soils are represented including the following designations: A-1-a, A-1-b, A-2-5, A-3, and A-5. Also included are four points representing values for a lime-fly-ash-stabilized cinder mix which was available for use in this study. Those points which indicate a drift to the left at low values of compressive strength in Figure 20 represent the previous data on compositions made up with nonoptimum proportions while the companion compositions using optimum proportions fall into the general pattern.

Figure 21 is a plot of velocity compensated for the effect of density against unconfined compressive strength for optimum compositions measured in the oven dry state. The variation introduced by nonoptimum compositions is eliminated from this chart but very little change in overall correlation is indicated between the various soils by this operation.

The effect of the presence of water with several nonoptimum range compositions of A-7 soil was also previously studied and significant variations were reported. Figure 22 shows the effect of moisture content upon the pulse velocity of several optimum-range lime-flyash-stabilized soils from saturation to oven dryness. Table 4 shows group velocity data typical of five different stabilized soils from saturation to low moisture contents during wetting and drying cycles. This data indicates that velocity is not greatly affected by moisture in the case of optimum range compositions for the soils so far tested. Several sets of data taken during wetting-drying and freezing-thawing cycles showed that velocity is not greatly affected by temperature variations above 40 F. for cured optimum range lime-flyash-stabilized soil compositions. Mechanical stress, however, has been observed to decrease transit time by as much as 25 percent and the major portion of the data given in Figures 20 and 21 is known to have been affected by considerable variation in axial loading from transducer contact pressure.

In Figures 23 to 27 the cumulative weight loss, moisture content, and pulse velocity for a number of compositions during wetting-and-drying and freezing-and-thawing cycles is presented. For the materials tested the velocity values at the termination of the durability cycles showed a gradual decrease during the course of the test. Circled points at the end of the velocity curves of Figure 27 indicates that measurements taken on the core of the sample after the outside had been carefully removed are essentially equal to the measurements made on the complete sample. Several points which fall above the general pattern indicated in Figure 20 were obtained from unconfined compressive-strength tests made on samples which had been subjected to wetting-and-drying and freezing-and-thawing tests. It is indicated from these

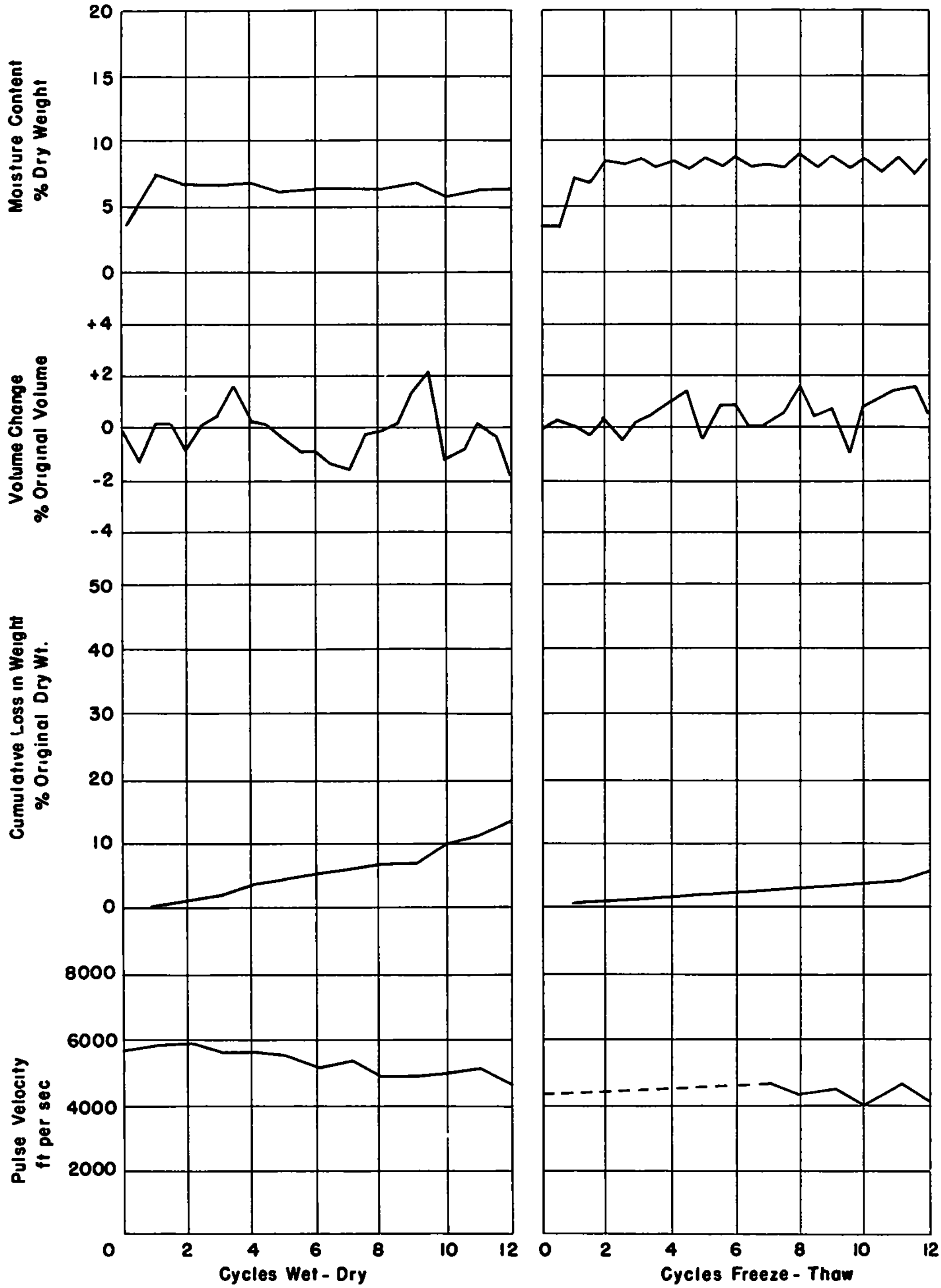


Figure 23. New Jersey Turnpike, New Brunswick Interchange.

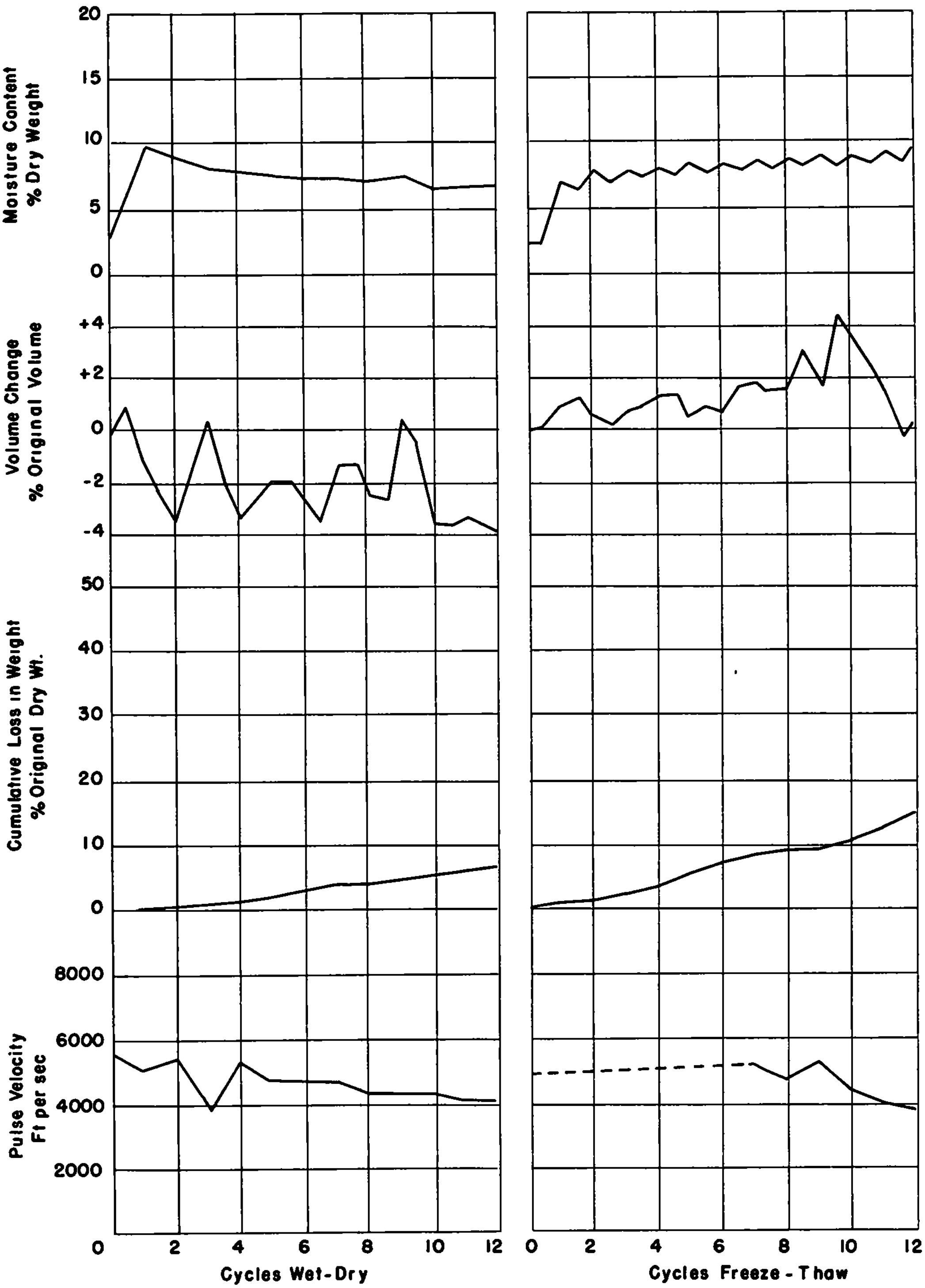


Figure 24. New Jersey Turnpike, Hightstown Interchange.

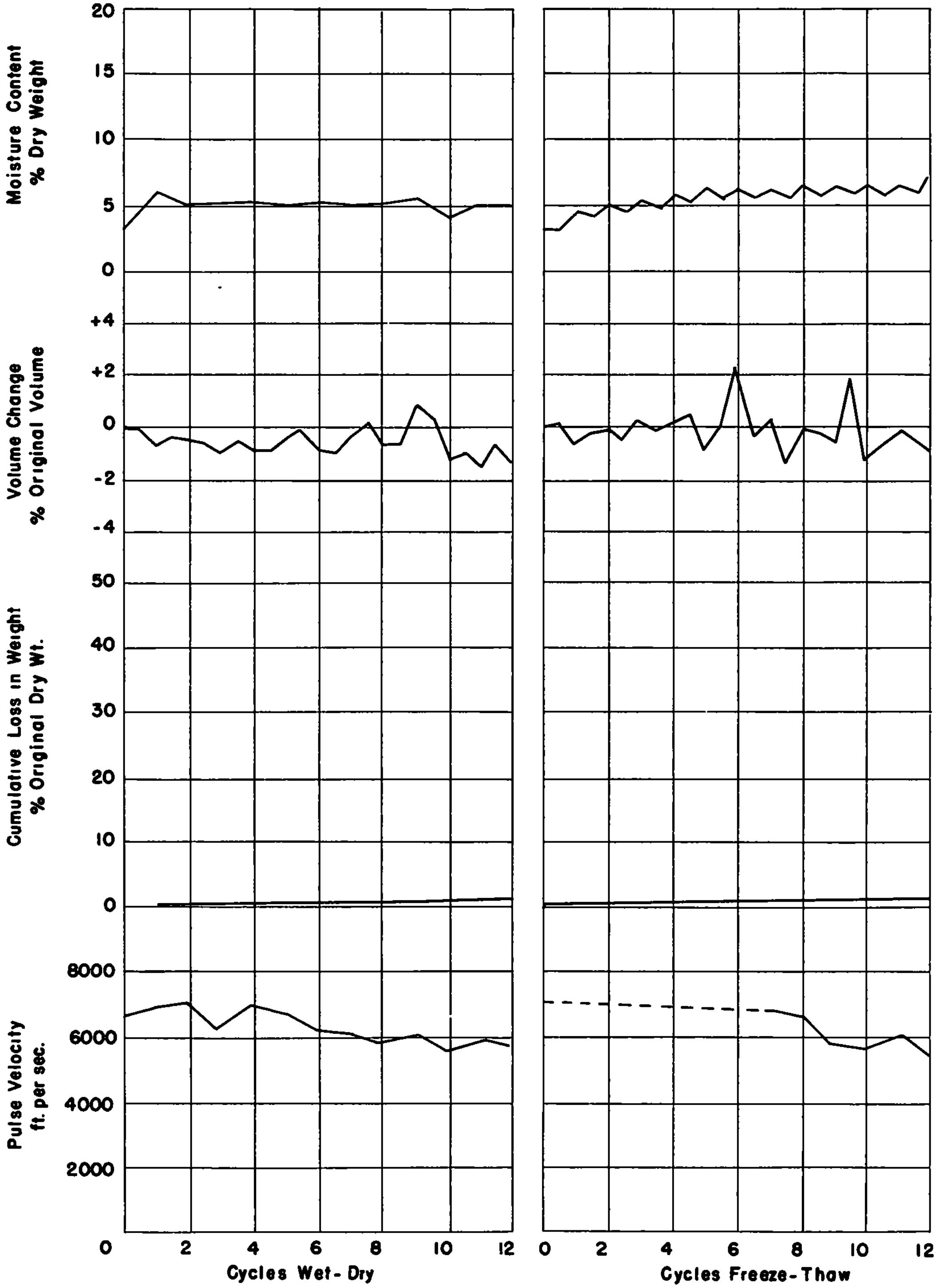


Figure 25. Marlboro, Md.

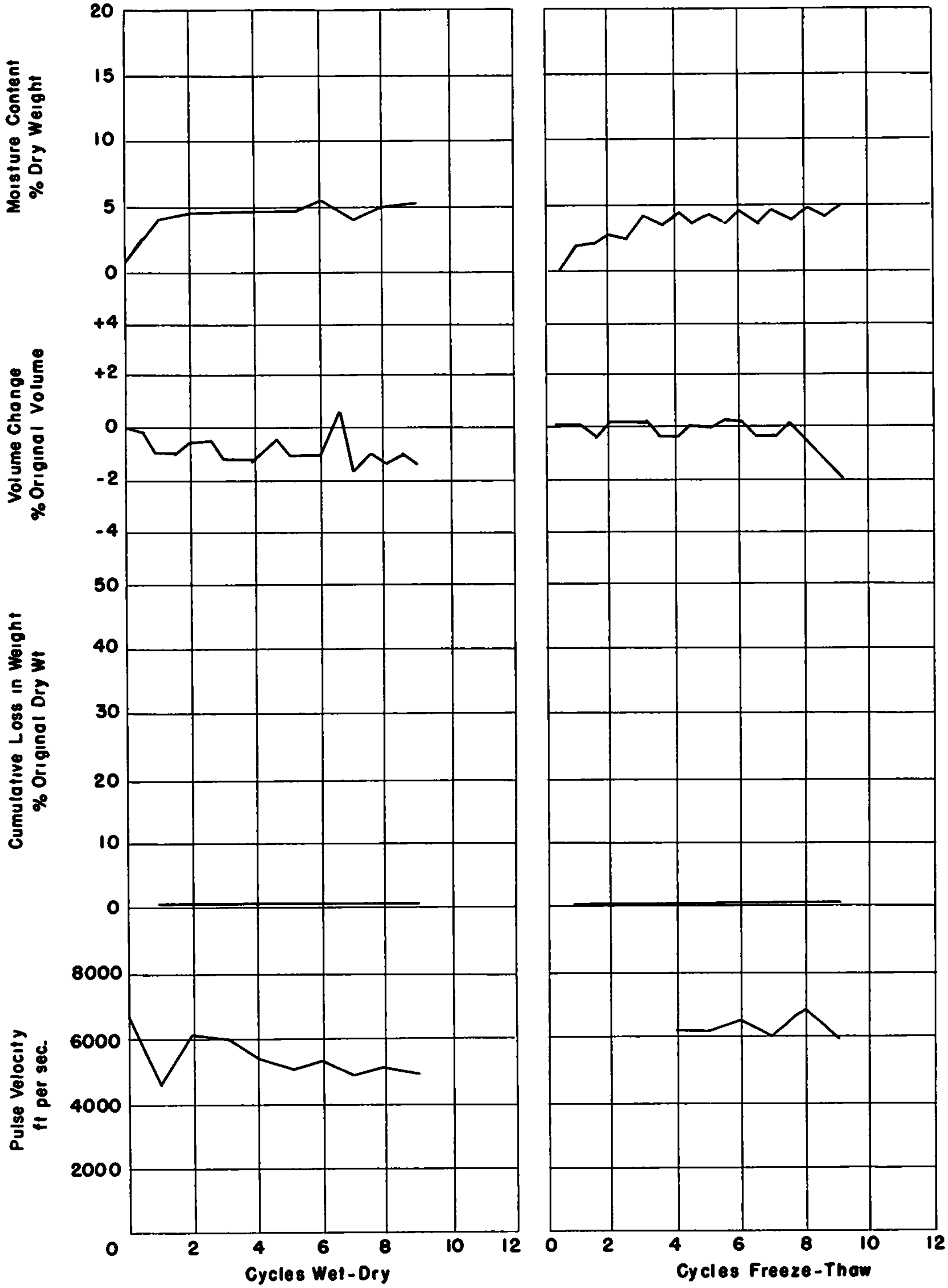


Figure 26. Mercer Avenue, Barrington, N. J.

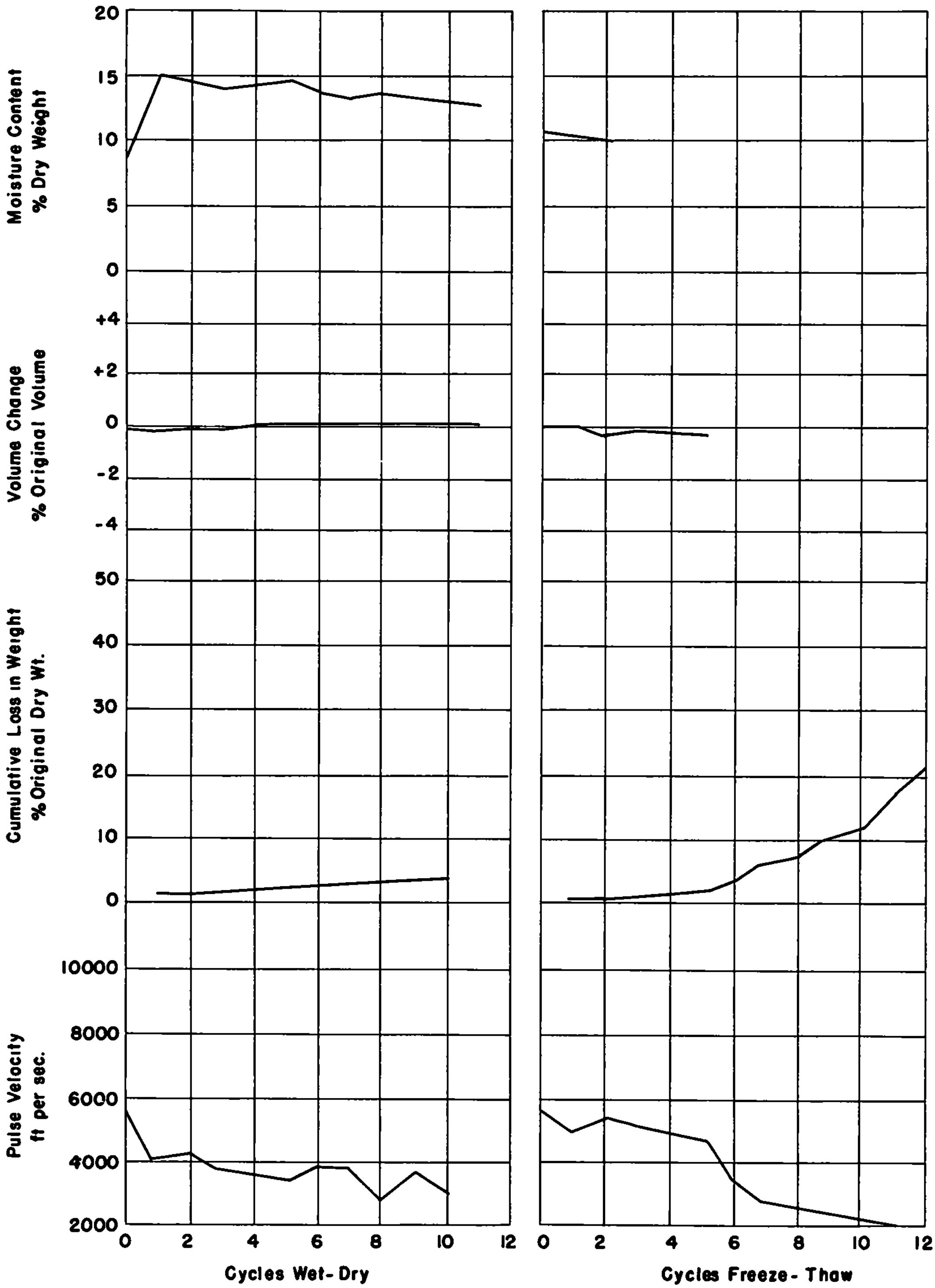


Figure 27. Woodbury, N. J.

values that the velocity measurements may be more effective in evaluation of the durability tests than unconfined compressive strength. Results of pulse-group-velocity measurements taken in the field are given in Tables 5, 6, and 7. It will be noted that initial compression-wave-front-velocity measurements taken with the transmitter and receiver vertically down upon the base material surface gave lower values than direct readings taken with axial in-line transducer orientation[1] under which conditions axial loading of the material is involved as in the case of the measurement of laboratory specimens.

Station numbers were determined to within 10 ft. Where traction readings were taken around a test hole, the instrument dial readings are given to indicate the velocity variation and the average used to calculate velocity from a calibration curve and the signal-path length.

CONCLUSIONS

While the study of lime-flyash-soil treatment has not extended to a point where full information is available, it is possible to draw several tentative conclusions which are indicated by the investigation up to the present time:

1. For the soils studied it is evident that the additions of small amounts of hydrated lime and flyash develop concrete-like compositions of high strength at relatively early ages. When compacted with optimum moisture, unconfined compressive strengths are developed of the order of 1,000 psi. in 28 days (ambient temperature condition). When cured at elevated temperature (140 F.) which accelerates the pozzolanic reaction of the lime and flyash, the compressive strengths obtained in 7 days are of the order of 350 to 1,400 psi. depending upon the soil type used.

2. The weathering resistance of the compositions appears to be exceptionally good since tests for wetting and drying and freezing and thawing show excellent resistance to deterioration after 12 cycles of treatment.

3. The use of pulse-velocity measurements indicates that this method is useful in evaluating the strength and weathering resistance of the compositions both in the laboratory and in the field. A relationship exists between unconfined-compressive-strength and pulse-group-velocity measurements. The presence of moisture in the specimens made with optimum proportions of lime and flyash is found to have little effect on the velocity readings. For nonoptimum mixtures the moisture content has been observed to effect the velocity reading.

It is indicated that the use of pulse-velocity methods are better suited to measuring durability of the compositions than are the standard wetting-and-drying and freezing-and-thawing tests. The laboratory tests are convenient to carry out and the field investigation is markedly simplified, since the measurements may be made on the same section of road at periodic intervals. It is felt that the use of fundamental transverse-resonant-frequency measurements can also serve to give adequate indications of performance of laboratory samples (2, 3). However, this method is involved with shape considerations and is, of course, not readily adaptable to use in the field.

4. The newly developed equipment for measuring pulse velocity is rugged, stable, and inexpensive to build. A portable unit has been produced which is well suited for use in the evaluation of stabilized road bases.

5. Field tests up to the present time show in all cases good performance for the compositions that have been prepared by the mixed-in-place operation. Several procedures are being tested to evaluate surface coverings for the stabilized base and these will be reported at a future date.

ACKNOWLEDGEMENT

The authors wish to express their appreciation of the interest that Havelin and Kahn have taken in this work. These gentlemen are responsible for the original discovery -- the benefit of adding a small amount of lime and flyash to aggregate materials. Also, the authors wish to express their appreciation to the Philadelphia Electric Company for its support of this work.

[1]By comparison to traction wavefront velocity measurements, Table 7, Item No 1 - 4

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Soil-Cement Test-Data Correlation in Determining Cement Factors for Sandy Soils

J. A. LEADABRAND, Manager, and
L. T. NORLING, Laboratory Chief;
Soil-Cement Bureau, Portland Cement Association

TO provide quick and simple procedures for determining cement factors for soil-cement construction and to relieve the pressure on laboratory personnel and facilities, the Portland Cement Association is correlating data obtained from testing more than 6,000 soils, representing many different soil types, textures, and mixtures.

This paper presents and discusses the results of a correlation of soil and soil-cement laboratory data obtained by testing 2,229 sandy soils following ASTM or AASHO standard test procedures. By use of the correlation, methods of quickly determining cement factors for most sandy soils encountered in soil-cement construction were developed. The 2,229 soils were placed into three groups, two of which are based on textural classification. The third group includes special or miscellaneous granular materials.

The methods involved are presented as step-by-step procedures and include the use of charts based on relationships between maximum density, combined silt, and clay content and the cement requirement for adequately hardening the soil. Minimum compressive strengths also are required.

The procedures require considerably less laboratory work and time than is needed for making complete ASTM or AASHO soil-cement tests, and in addition, smaller soil samples can be used.

The dependability of the test methods when checked against the sandy soils previously tested by the standard ASTM-AASHO tests is discussed. The step-by-step testing procedures provided reliable methods for establishing safe cement factors for 2,201 (or 98.7 percent) of the 2,229 soils. While the cement factors obtained were practical, they were not always the minimum or most economical that could be used to harden the soil.

The paper suggests adoption of the test methods developed. It further suggests that the charts be used in the form shown until local data and experience are obtained that will permit revision to conform more closely to local conditions.

● THE use of established ASTM - AASHO[1] test methods for control of cement content, moisture content, and density in the construction of many million square yards of satisfactory soil-cement pavement attests the dependability of these proven test methods. But invaluable as they are to engineers, standard ASTM procedures are time consuming. Approximately 6 weeks of time and 100 lb. of soil are usually needed for the tests.

To provide quick and simple procedures for determining cement factors for soil-cement construction, and to relieve the pressure on laboratory personnel, always in short supply, and to lighten the load on laboratory facilities as the use of soil-cement in highway airport, and other construction increases, the Portland Cement Association has undertaken to correlate the data obtained from testing more than 6,000 soils. These soils represent many different soil types, textures, and mixtures and were distributed over the whole of the United States and parts of Canada.

[1]ASTM Designations D558-44, D559-44, D560-44 or AASHO Standard T-134-45, T-135-45, T-136-45 For the sake of brevity these are referred to hereafter as ASTM tests

The results of the work complete to date cover 2,229 sandy soils and are presented in this paper. These soils were first placed into three groups, two based on textural classification and one consisting of special or miscellaneous granular materials.

It was found possible to correlate maximum density, combined silt and clay content and compressive strengths with the cement factor for most of the soils studied. Step-by-step testing procedures were then developed for sandy soils falling into the three groups. These involve only a few well-known simple laboratory tests and the use of charts. Small soil samples of approximately 25 to 40 lb. will usually suffice. One laboratory man can perform all the test work needed for most samples in one day although a time interval of one week is required to obtain 7-day compressive strengths.

There are limitations to the use of the procedures, but with proper application they should save considerable time and prove of great help to engineers doing soil-cement work. The procedures suggested should prove even more exact and helpful if the charts presented are checked and possibly revised by using local test data obtained from testing soils from more localized areas. The soils from a given area will have similar basic properties and react in the same general manner with cement.

Included in this paper are (1) discussion of present soil-cement test methods and criteria; (2) presentation of the suggested step-by-step procedures for determining cement factors; (3) discussion of correlation and results obtained in checking the suggested step-by-step test procedures; and (4) summary and suggestions.

PRESENT TEST METHODS AND CRITERIA

Three procedures which have been generally used for determining the required cement content of soil for soil-cement construction were adopted as standards by the ASTM in 1944. Identical procedures were adopted by the AASHO in 1945. These tests are: (1) "Method of Test for Moisture-Density Relations of Soil-Cement Mixtures" ASTM Designation: D558-44; AASHO Standard T-134-45; (2) "Method of Wetting-and-Drying Test of Compacted Soil-Cement Mixtures" ASTM Designation: D559-44; AASHO Standard T-135-45; and (3) "Method of Freezing and Thawing Test of Compacted Soil-Cement Mixtures" ASTM Designation: D560-44; AASHO Standard T-136-45.

Based on test data obtained from tests made in accordance with the above standards and from supplemental compressive strength tests, the following criteria have been suggested and used by the Portland Cement Association for determination of cement, moisture, and density factors required to produce a soil-cement of satisfactory hardness and durability (1). These criteria were based on data obtained from performance and condition surveys of soil-cement projects in service for many years throughout the United States and in Canada, from laboratory research data, and from information obtained from outdoor weathering studies (2).

1. Soil-cement losses during 12 cycles of either the wet-dry test or freeze-thaw test shall be within the following limits:

Soil Group A-1-a, A-1-b, A-3, A-2-4 and A-2-5, not over 14 percent.

Soil Group A-2-6, A-2-7, A-4 and A-5, not over 10 percent.

Soil Group A-6, A-7-5, and A-7-6, not over 7 percent.

2. Maximum volume change during either wet-dry or freeze-thaw tests shall not exceed the volume at time of molding by more than 2 percent.

3. Maximum moisture content during either wet-dry test or freeze-thaw test shall not exceed that quantity which will completely fill the voids of the specimen at time of molding.

4. Compressive strengths shall increase with age and cement content in the ranges of those producing results meeting Requirements 1, 2, and 3 above.

The criteria suggested above have not been considered irrevocable or final. However, many millions of square yards of soil-cement have been constructed involving control factors selected on the basis of these criteria. The outstanding performance of the projects in service indicates the complete adequacy of the control factors established in this manner. There are, in fact, some indications that the cement factors indicated by these tests and criteria for certain soils in some climatic situations may be more than required to provide soil-cement of adequate durability. As experience on soil-cement testing and construction is gained and tied into performance and condition surveys of local jobs, modifications of test procedures or criteria may be justifiably made to meet these soil and local climatic conditions.

In a few areas, test methods and criteria other than those discussed above have been developed and are used in establishing control factors for local soil-cement construction. In those areas for the particular soils and climate involved, these locally-developed test methods are apparently proving satisfactory.

In other instances a shortened or modified ASTM test procedure is used. In the Portland Cement Association's Soil-Cement Laboratory, for example, routine ASTM tests for establishing field construction factors have been made in reduced form for some time with satisfactory results except for unusual or special soils (3). The reduced procedure involves only one moisture-density test and the molding and testing at three cement contents of No. 2 freeze-thaw specimens and one No. 2 wet-dry specimen at the median cement content. No. 1 specimens for volume change or moisture content determination are not molded. Although a No. 2 wet-dry specimen has been included in the tests, losses incurred for it rarely are higher than for the freeze-thaw specimens. As a supplemental test, a total of four 2-in.-diameter-by-2-in.-height compressive-strength specimens have been molded and tested at two cement contents. In addition to making soil-cement tests, a grain-size analysis and liquid-limit, plastic-limit, and shrinkage-limit determinations are made for each soil. This reduced testing procedure lessens appreciably the amount of laboratory work but not the period of time required for making tests. The quantity of soil needed is also reduced.

The need for conducting soil-cement tests can be sharply reduced and even eliminated altogether for large soil areas by establishing cement requirements by horizon for soil types as classified for most areas of the United States by the Bureau of Plant Industry (Bureau of Chemistry and Soils)[2]. This system of soil classification or identification is based on the fact that soils having the same weather, topography, and the same drainage characteristics will grow the same type of vegetation and will be the same kind of soil. This is illustrated by the fact that the black wheat-belt soils of our Midwest are same as the black, wheat-belt soils of Russia, Argentina, and other countries. The system is important basically because a soil of a particular series, horizon, and grain size will perform the same as a highway material wherever it occurs, because such important factors as rainfall, freezing, groundwater table, capillarity of the soil, etc., are a part of the identification system.

This soil-classification system has proved helpful in soil-cement work. As test data were obtained by following the ASTM tests and the other test procedures outlined above, it was found that the cement requirements of a definite soil type and horizon are the same, regardless of where the soil is encountered, and no further soil-cement tests for this particular soil are, therefore, needed. A number of engineers are making use of this system to reduce their soil-cement testing work (4).

In many instances soil and soil mixtures are found in the roadway or street which cannot be definitely identified as to soil series, nor can the original location of the material be determined to permit checking it against previously obtained soil-cement test results. It is usually necessary in such cases to conduct complete tests to establish cement requirements. Similarly, it is sometimes difficult to identify the soil series of existing undisturbed soil material, and in some cases the soils have not been mapped. Further, while quite a few soil types have been tested for soil-cement, there are many remaining untested. In

[2]Now identified as U S. Bureau of Plant Industry

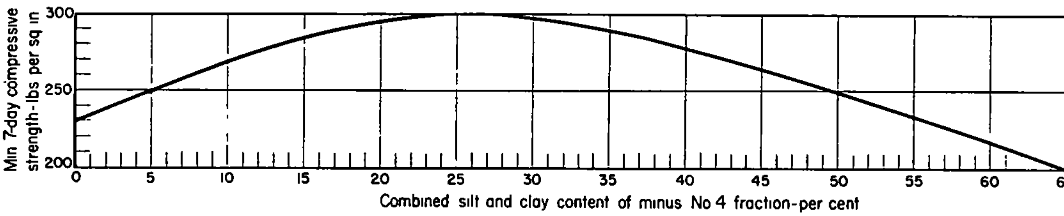


Figure 1. Minimum 7-day compressive strengths required for Group I soils at the indicated cement content obtained from Figure 2.

many cases, too, samples of soil are lifted and forwarded to laboratories without soil series or horizon identification being noted. Many of the soils included in the present study fall in the above categories.

TABLE 1
TEXTURAL CLASSIFICATION GROUPS[a]

SOILS CONTAINING LESS THAN 20 PERCENT CLAY AND 50 PERCENT SILT AND CLAY COMBINED

A Soils containing less than 15 percent silt and clay combined

SANDS AND GRAVELS

Gravel, 85 percent or more fine and coarse gravel combined
Gravel and sand, 50 to 85 percent fine and coarse gravel combined
Sand and gravel, 25 to 50 percent fine and coarse gravel combined
Coarse sand, 50 percent or more coarse sand and gravel combined, but less than 25 percent gravel
Sand, less than 50 percent fine sand and less than 50 percent coarse sand and gravel combined
Fine sand, 50 percent or more fine sand

B Soils containing from 15 to 20 percent silt and clay combined

LOAMY SANDS

Gravelly loamy sands, loamy sands listed below, having 25 percent or more fine and coarse gravel combined
Loamy coarse sand, 50 percent or more coarse sand and gravel combined
Loamy sand, less than 50 percent fine sand and less than 50 percent coarse sand and gravel combined
Loamy fine sand, 50 percent or more fine sand

C Soils containing from 20 to 50 percent silt and clay combined

SANDY LOAMS

Gravelly sandy loams, sandy loams listed below, having 25 percent or more fine and coarse gravel combined
Coarse sandy loam, 50 percent or more coarse sand and gravel combined
Sandy loam, less than 50 percent fine sand and less than 50 percent coarse sand and gravel combined
Fine sandy loam, 50 percent or more fine sand

SOIL SIZE LIMITS

Coarse gravel	76 2 mm to 4 76 mm (3-in. to No. 4 sieve)
Fine gravel	4 76 mm to 2 mm (No 4 sieve to No 10 sieve)
Coarse sand.	2 mm to 0 25 mm (No 10 sieve to No. 60 sieve)
Fine sand.	0 25 mm to 0 05 mm (No 60 sieve to No. 270 sieve)
Silt.	0 05 mm to 0 005 mm
Clay	0 005 mm to 0 00 mm

[a]Basic concept for these classifications is from the U S. Bureau of Chemistry and Soils

PROCEDURES FOR DETERMINING CEMENT FACTORS

Step-by-step procedures for establishing cement requirements of many sandy soils encountered in soil-cement construction and falling in one of three established groupings are listed below. The charts are based on correlations of test data obtained from making ASTM soil and soil-cement tests and compressive strength tests on 2, 229 soils containing less than 20 percent clay and less than 50 percent combined silt and clay. Dark-gray to black soils containing appreciable amounts of organic material were not included in this study, and the procedures, therefore, do not apply to such soils.

To apply the step-by-step procedures, it is first necessary to make a grain-size analysis of the soil to permit identifying it texturally and to see if it contains less than 20 percent clay and less than 50 percent of silt and clay combined (Table 1). If it meets these requirements, it is then placed in either Group I or II depending upon its texture, or in Group III, if it is a special type of material, such as shale, scoria, caliche, cinders.

The textural classifications, types of materials, and the step-by-step testing procedures for each group follows:

Group I Soils

Textural classifications included:
- Coarse sand, sand, fine sand
- Loamy coarse sand, loamy sand, loamy fine sand
- Gravelly sandy loam, coarse sandy loam, sandy loam, fine sandy loam

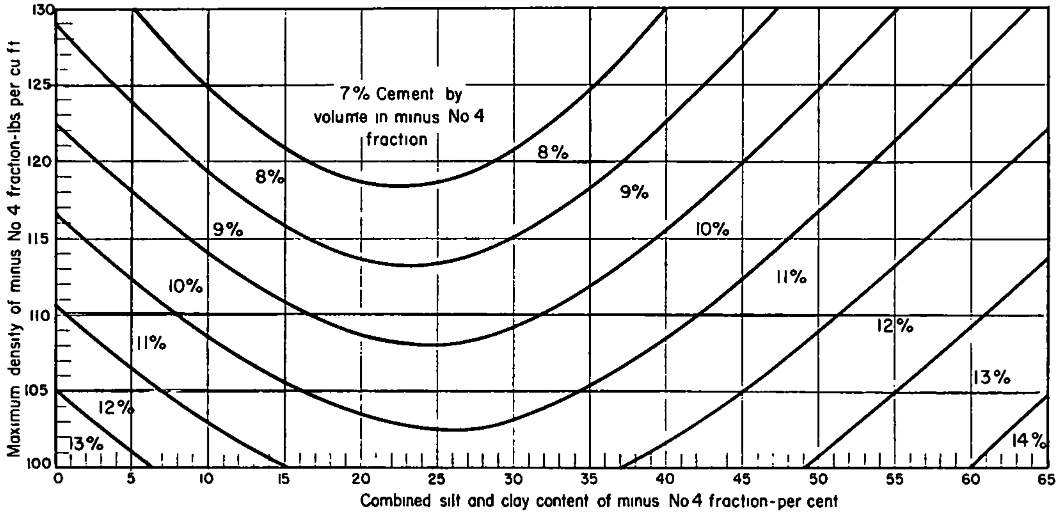


Figure 2. Indicated cement content required for adequate hardening of Group I soils.

Testing procedure:

Approximately 25 to 40 lb. of soil generally will suffice.

Step 1: Determine the maximum density optimum moisture content for a mixture of the soil and portland cement by making a moisture-density test in accordance with ASTM D558-44. The cement content to use for the Test may be determined by first obtaining from Figure 3[3] an approximation of the expected density and using this value to select the cement content from Figure 2.

Step 2: Using the maximum density obtained from the moisture-density test in Step 1 and percent of combined silt and clay content of the minus-No. -4 portion of the soil, determine from Figure 2 the percent of cement by volume of minus-No. -4 mixture.

Step 3: Mold compressive strength specimens[4] in triplicate using minus-No. -4 soil and the maximum density, optimum moisture and the cement factor obtained in Steps 1 and 2. Determine the average compressive strength of the specimens after 7 days of curing in a moist room.

Step 4: Plot on Figure 1 the compressive-strength value obtained by Step 3. If this value falls above the curve shown, then

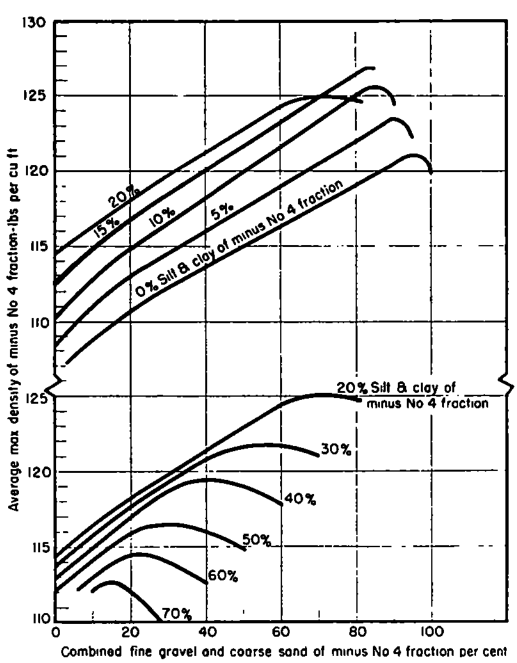


Figure 3. Average maximum densities of soil-cement mixtures.

[3]The curves in Figure 3 were obtained by first plotting densities for 2,020 soils (Groups I and II) on a triaxial chart according to (1) coarse sand including fine-gravel content, (2) fine-sand content and (3) combined silt-and-clay content. The average densities for various combined silt-and-clay contents were then plotted against the combined coarse-sand-fine-gravel content. From these points the curves were drawn This chart gives a reasonable estimate of maximum densities, but does not preclude making the moisture-density test to obtain the actual density of the soil-cement mixture

[4]Two-in dia by 2-in ht. specimens or 4-in dia. by 4 6-in. ht specimens may be molded using minus No 4 soil-cement material The 2-in specimens shall be submerged in water one hour before testing and the 4-in specimens four hours

the adequacy of the cement factor obtained in Step 2 for soil-cement construction is indicated. If the original soil sample contained material retained on a No. 4 sieve, it is necessary, for field construction, to convert the cement factor based on the minus-No. -4 mixture, obtained in Step 2, to the cement factor based on the total mixture. This is quickly done by using the appropriate curve of Figures 5a to 5h.

If the strength value from Step 3 falls below the curve of Figure 1, the cement factor obtained in Step 2 is indicated as too low, and further laboratory testing of the soil is needed to establish the cement requirement. The additional testing may consist only of the molding of two specimens, one at the cement content indicated in Step 2 and one 2 percentage points higher and testing as No. 2 specimens following ASTM freeze-thaw tests.

Group II Soils

Textural classifications included:
- Gravel and sand
- Sand and gravel
- Gravelly loamy sand

Testing procedure:

Approximately 25 to 40 lb. of soil generally will suffice.

Seven percent cement by volume based on the total mixture is the indicated cement factor for these soils for soil-cement construction. Verify this requirement by the following procedure:

Step 1: Determine the maximum density and optimum moisture content for a mixture of the soil and portland cement by making a moisture-density test in accordance with ASTM D558-44. The cement content to be used for the test may be determined by first obtaining an estimated density from Figure 3. Then, by using the appropriate curve of Figures 5a to 5h, the cement content of minus-No. -4 soil-cement mixture equivalent to 7 percent by volume of total soil-cement mixture is ascertained and used in making the moisture-density test.

Step 2: Mold compressive-strength specimens in triplicate using minus-No. -4 soil, and the maximum density and optimum moisture and the equivalent minus-4 cement factor determined in Step 1. Determine the average compressive strength of the specimens after 7 days of moist-room curing.

Step 3: Plot on Figure 4 the compressive-strength value obtained in Step 2. If this value falls above the curve, then the adequacy of 7 percent cement by volume of the total soil-cement mixture for soil-cement construction is indicated

If the strength value is below the curve of Figure 4, then 7 percent by volume of the total mixture is indicated as too low a factor and additional testing of the soil is needed to establish the cement requirement. The additional testing may consist of the molding of specimens at 7 and 9 percent cement by volume of total mixture and testing as No. 2 specimens in ASTM freeze-thaw tests.

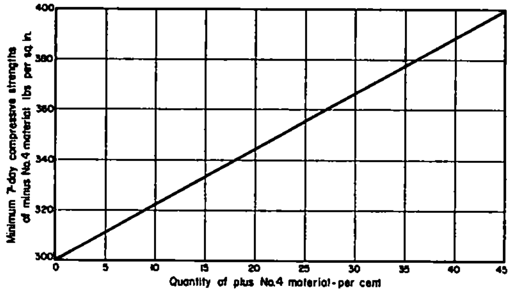


Figure 4. Minimum 7-day compressive strengths required for Group II soils at cement contents equivalent to 7 percent by volume of total mixture.

Group III Soils

Includes the miscellaneous or special soil materials falling in all textural classifications listed in Table I. This includes the following:

caliche	limestone screenings	shale
chat	marl	shell
chert	red dog	slag
cinders	scoria	others

Testing procedures:

Approximately 40 lb. of soil generally will suffice.

A tabulation of probable cement requirements based on limited test data for these special materials is given in Table 2 along with other pertinent information.

By use of the information in Table 2, a step-by-step procedure is given below that reduces the testing over that previously required.

All shell soils or limestone screenings may be placed according to textural classification in Group I or II and the respective testing procedure followed. For the remaining soils:

Step 1: Determine the maximum density and optimum moisture content for a mixture of the soil and portland cement by making a moisture-density test in accordance with ASTM D558-44. The cement content to be used for this test may be obtained from Column 1, Table 2. If the soil contains material retained on a No. 4 sieve, it is necessary to convert the cement content of Column 1 to that based on the minus-No. 4 soil-cement mixture as indicated in Step 1 for Group II soils.

Step 2: Mold two specimens at the cement contents shown in Column 2, Table 2, and test as No. 2 specimens in accordance with ASTM freeze-thaw tests.

TABLE 2
DATA FOR GROUP III SOILS

Type of Miscellaneous Material	Cement Content by Volume To be Investigated[a]		Probable Cement Requirement
	Moisture-Density Test	Freeze-Thaw Specimens	
Shell Soils	Use the procedure outlined for Group I or II soils depending upon the textural classification		
Limestone Screenings	Use the procedure outlined for Group I or II soils depending upon the textural classification		
Cinders, chert, chat, caliche[b], scoria[c], slag[d], red dog, brick waste, roofing granules	8	7 & 9	9
Marl, shale, slag[e], fire clay, iron ore base	10	9 & 11	11

[a]Cement content given is based on the total mixture. For soils containing material retained on the No. 4 sieve, the cement contents to be used in the moisture-density and compressive strength tests shall be the cement content in the minus-No. -4 mixture equivalent to the cement contents given.

[b]If the textural classification is included in Group II, a cement content as low as 7 percent by volume may be adequate

[c]If the scoria contains appreciable material retained on the No 4 sieve, it may require as much as 12 percent cement by volume In this case, 11 percent cement by volume freeze-thaw specimens should be included.

[d]Air-cooled slag

[e]Water-cooled slag

CORRELATION AND RESULTS OBTAINED IN CHECKING TEST PROCEDURES

Description of Soils

Of some 6,000 soils of all textures and types for which ASTM test data were available, 2,229 were sandy soils containing less than 20 percent clay and less than 50 percent combined silt and clay and were included in this study. Dark-gray to black sandy soils containing appreciable amounts of organic material were excluded, as preliminary studies indicated that such soils require separate study. The 2,229 soils were from many sections of the United States as shown in Figure 6. All states are represented except South Dakota.

Included were 209 miscellaneous materials, such as cinders, scoria, caliche, shale, which were treated in a separate group. The remaining 2,020 soils were classified texturally according to Table 1 as follows:

Textural Classification	No. of Soils
gravel and sand	222
sand and gravel	171
coarse sand	227
sand	94
fine sand	232
gravelly loamy sand	79
loamy coarse sand	44
loamy sand	43
loamy fine sand	66
gravelley sandy loam	231
coarse sandy loam	52
sandy loam	377
fine sandy loam	182

Maximum densities of minus-No. -4 soil-cement mixture using these soils ranged from 136 lb. per cu. ft. to 97 lb. per cu. ft. The miscellaneous materials such as cinders had maximum densities as low as 65 lb. per cu. ft. Plasticity indices ranged from non-plastic to a high of 34.

Method of Analysis

In starting the study, test data for 525 soils that were identified as to horizon,

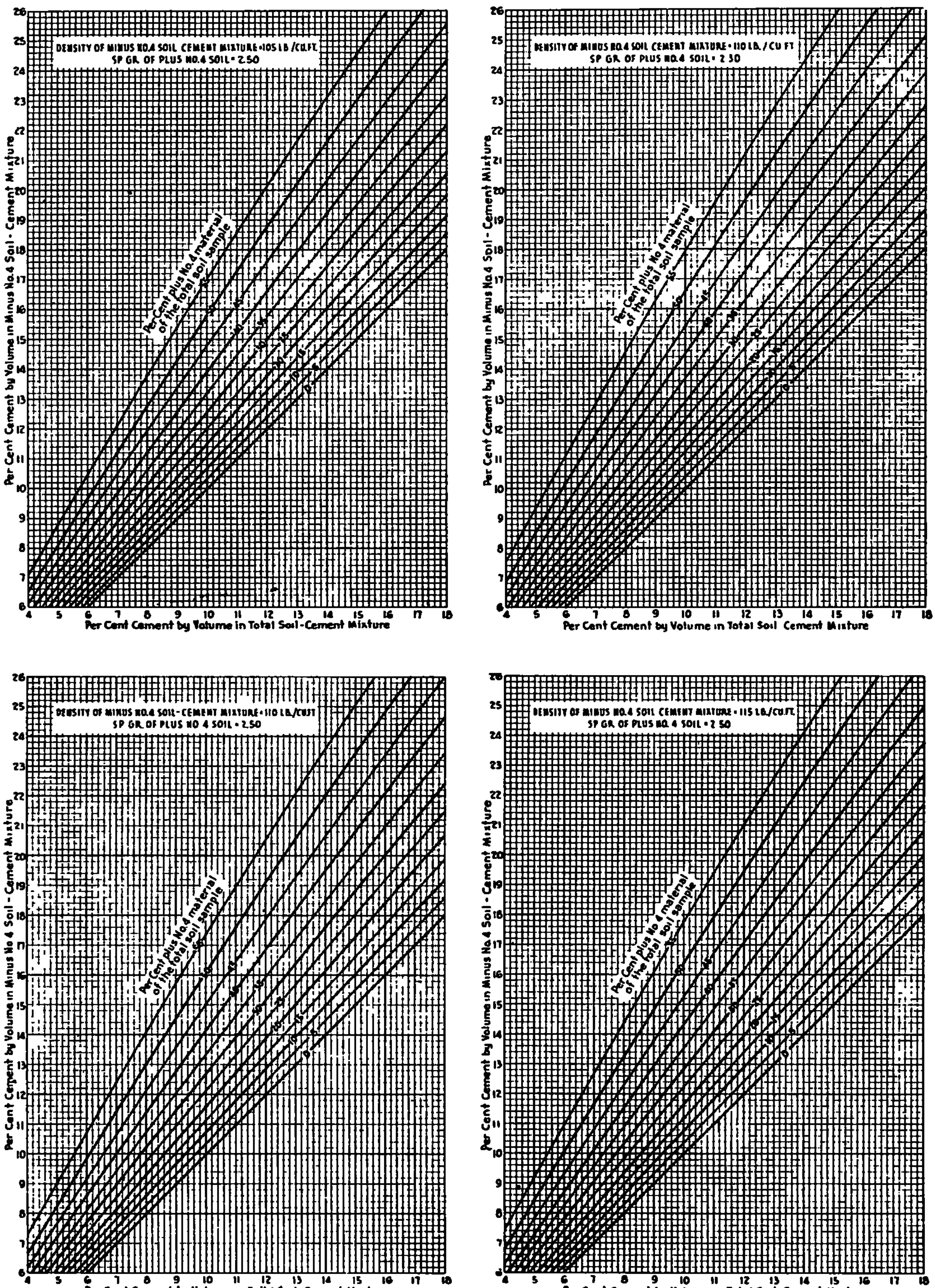
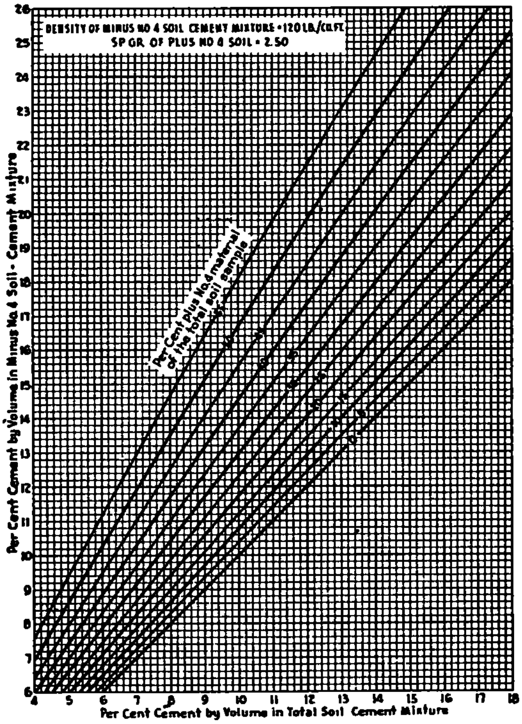
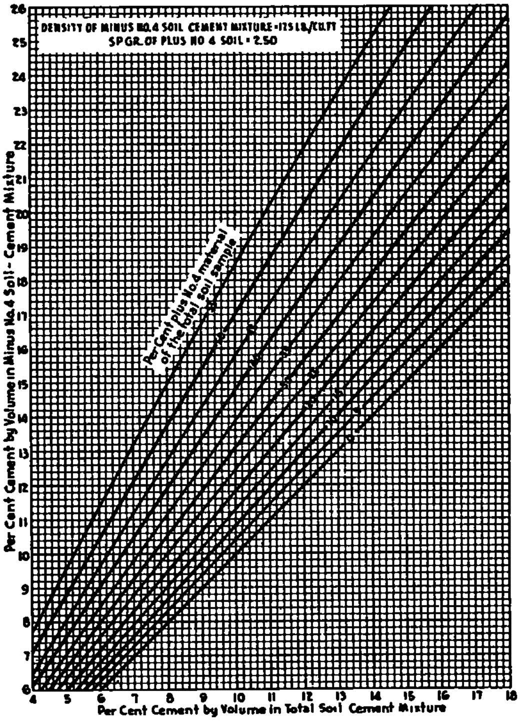
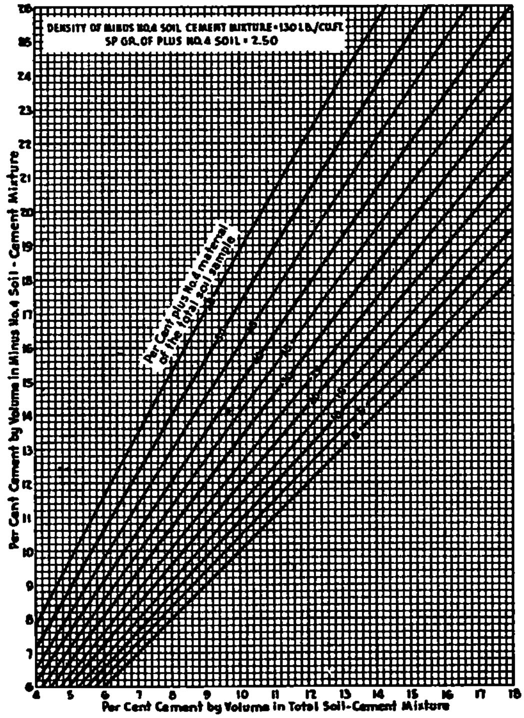
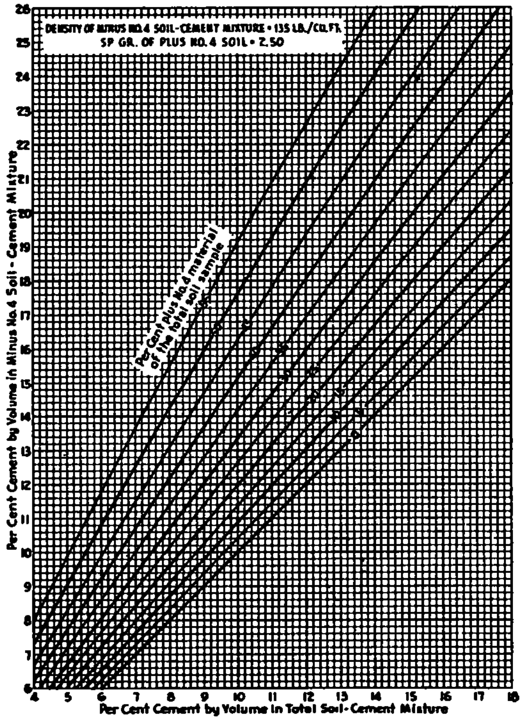


Figure 5. Relation of cement content by volume in the minus-No.-4 soil-cement









mixture to the cement content by volume in the total soil-cement mixture.

but not necessarily by soil series or type, were first investigated. Soils classified texturally as gravel and sand, sand and gravel, and gravelly loamy sand were not included for reasons given later. The 525 soils were subdivided according to: (1) horizon (A, B or C); (2) color (gray, brown, red and yellow); (3) great soil group such as chernozen, podzol, (based on U. S. Bureau of Plant Industry Classification system); and (4) texture (sand, loamy sand, sandy loam, etc.).

It was thought that a soil's reaction with cement might be correlated with these factors. Usable relationships based on these factors could not be established that would permit developing a procedure to indicate cement factors. This may be due to the predominant amount of inert sand and gravel-sized particles contained in these soils and the relatively small amount of clay and colloidal size material which is chemically the most significant. It is believed, however, that such factors as horizon, color, identification by Great Soil Group, may be significant in a correlation of data for the silty and clayey soils. As previously discussed, it has already been established that the cement requirements of a given soil identified by soil series, type, and horizon is the same wherever it is located.

Studies were also made of physical properties, such as maximum density, optimum moisture, liquid limit, plasticity index, group index, silt content, clay content, sand content, compressive strength, cement-void ratio, in respect to required cement factors for the soils of the several divisions above. The relationships of these factors were too general or broad to use as criteria for establishing cement factors and were quite similar for all the subdivisions. These comparisons checked those similarly made and reported previously (5).

A most-promising relationship, however, was obtained between combined silt and clay content, maximum density, and the required cement factor. This relationship was, therefore, investigated further and an additional 1,023 sandy soils identified only by texture were included in the group. The total of 1,548 soils were designated as Group I soils. The cement requirements for adequately hardening most of these soils ranged from 7 to 14 percent cement by volume

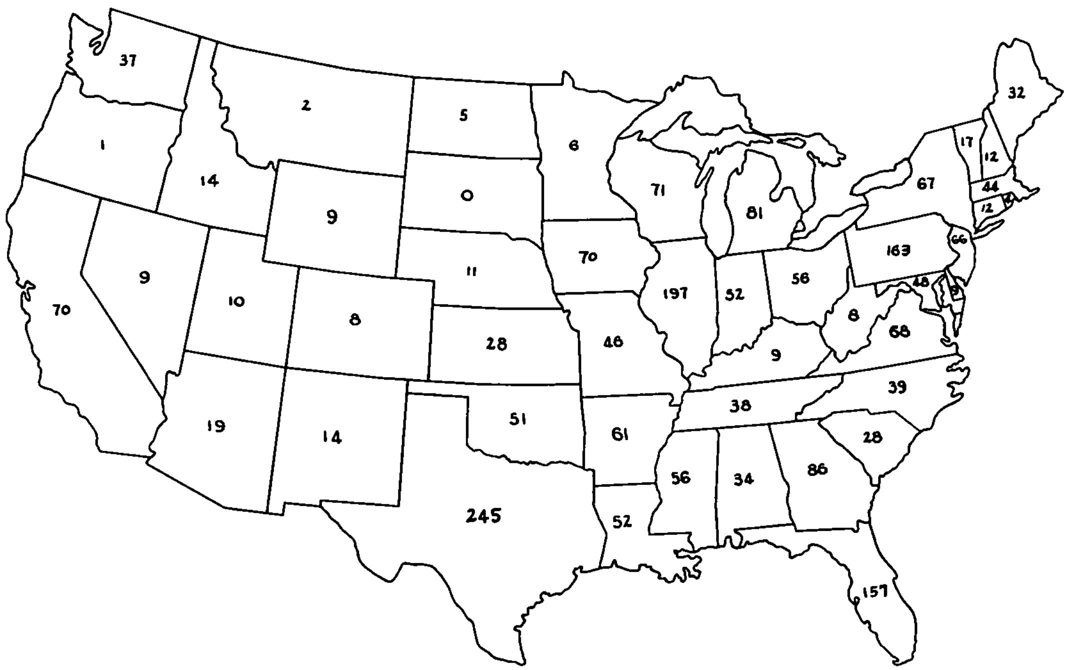


Figure 6. Geographic location of 2,229 soils included in the study.

according to ASTM tests and the criteria listed previously.

Group I Soils

The maximum density and percent of combined silt and clay were then plotted for the 1,548 soils according to cement content for adequate hardening. Typical the cement factor needed for normally reacting soils of similar gradation and density may be so delayed that there is no appreciable hardness at age 7 days, the usual period allowed for cement hydration. These soils are termed "slow-hardening," or "poorly reacting" sands. It is necessary, therefore, to conduct ASTM standard tests to determine the

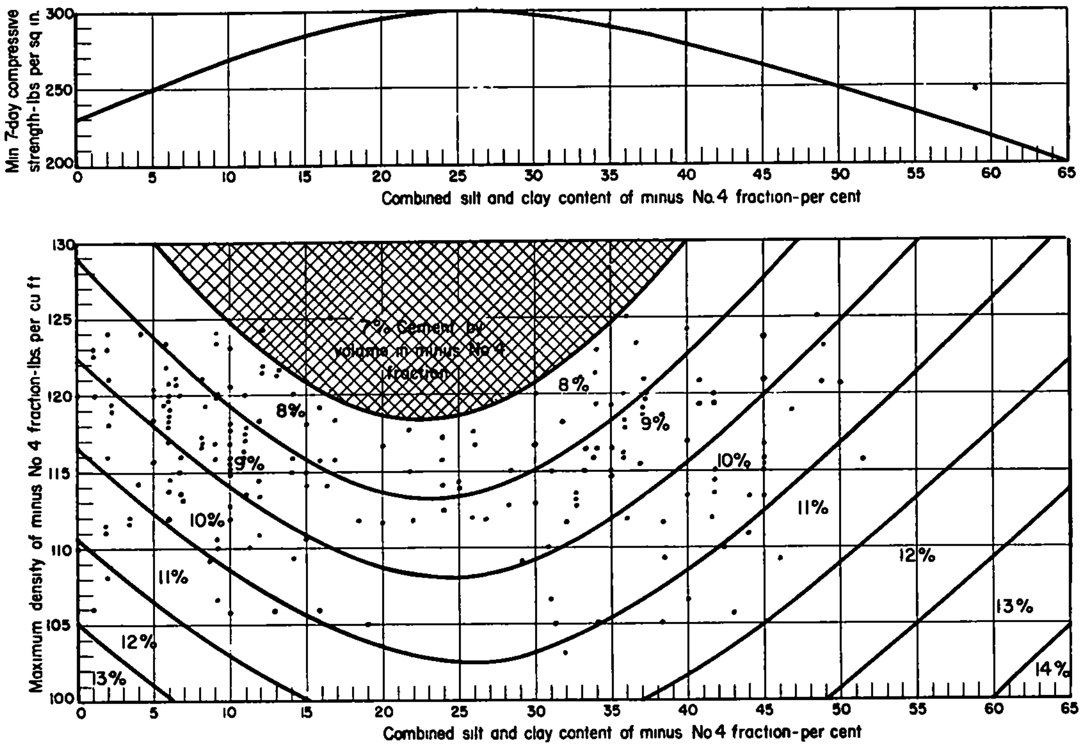


Figure 7. Soils for which 8 percent cement by volume is adequate based on ASTM tests and which have adequate compressive strength according to Figure 1.

plotted points for soils for which 8 percent cement by volume is adequate are shown in Figure 7. Those for 10 percent cement are shown in Figure 8. Curves similar to those in Figure 2 were then drawn to include as many points as possible within each band of similar cement factors.

In checking the curves of Figure 2 for possible use in determining cement factors, it became apparent that some certainty that cementing action or hardening was taking place normally was needed for use in conjunction with these curves. The rate of hardening of a few sandy soils at adequate cement factor which may be quite high or average, depending upon rate of hardening — particularly during the early period of testing[5]. Slow-hardening soils may be recognized by noting the hardness of 7-day-old specimens molded at cement content usually found adequate for similar normally reacting soils. Often this is done on inspection of moisture-density "tail-end" specimens (6). Logically, compressive - strength data could be used for indicating slow-hard-

[5]For a discussion of "poorly reacting" or "slow hardening" sandy soils, see also "Effect of Soil and Calcium Chloride Admixtures on Soil-Cement Mixtures", Proceedings, Highway Research Board, 1943

ening soils if adequate strength criteria were available.

In order to provide such criteria, 7-day compressive strengths[6] at the cement content producing soil-cement of adequate hardness as determined by ASTM tests were plotted against percent of combined silt and clay content (see Fig. 9). Compressive strengths lower than 100 psi. were not included as they were obviously abnormally low and in- dicative of poorly reacting or slow-hardening soils Curve A, Figure 9, represents the average of these plotted strength values.

The strengths for soils of Group I that were found by ASTM tests to require higher cement factors than indicated by Figure 2 were next plotted using the strengths obtained at the cement content indicated by Figure 2. Curve B, simulating Curve A, was then placed so that most of these points were below it (see Fig. 10), and the strengths for most of the normally reacting soils were above it (Fig. 9). Curve B, therefore, represents the minimum compressive strengths that were obtained for most normally reacting soils of Group I at the cement content needed for adequate hardening.

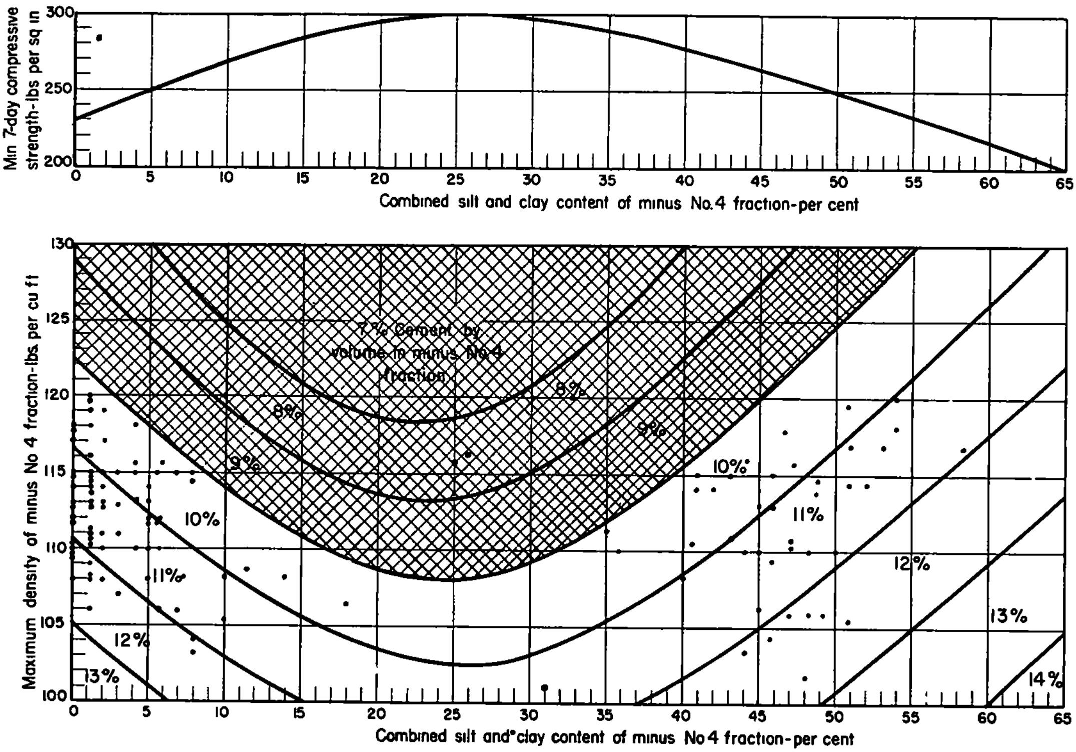


Figure 8. Soils for which 10 percent cement by volume is adequate based on ASTM tests and which have adequate compressive strength according to Figure 1.

The strengths at the adequate cement content may, however, be considerably above these minimum values as the many points above Curve B in Figure 9 shows. Compressive strengths falling below Curve B would, on the other hand, indicate that the cement contents producing these strengths are possibly too low for satisfactory hardening.

A final check of the curves of Figures 1 and 2 was then made using all 1,548 soils in Group I. A cement factor as indicated by Figure 2 was first selected.

[6]These strengths were determined from 2-in -dia ,2-in - high specimens molded using minus-No -4 soil-cement mixture and soaked 1 hr before testing Available test data shows that comparable results will be obtained using 4-in - dia., 4.6-in -high specimens, such as obtained from most moisture-density molds

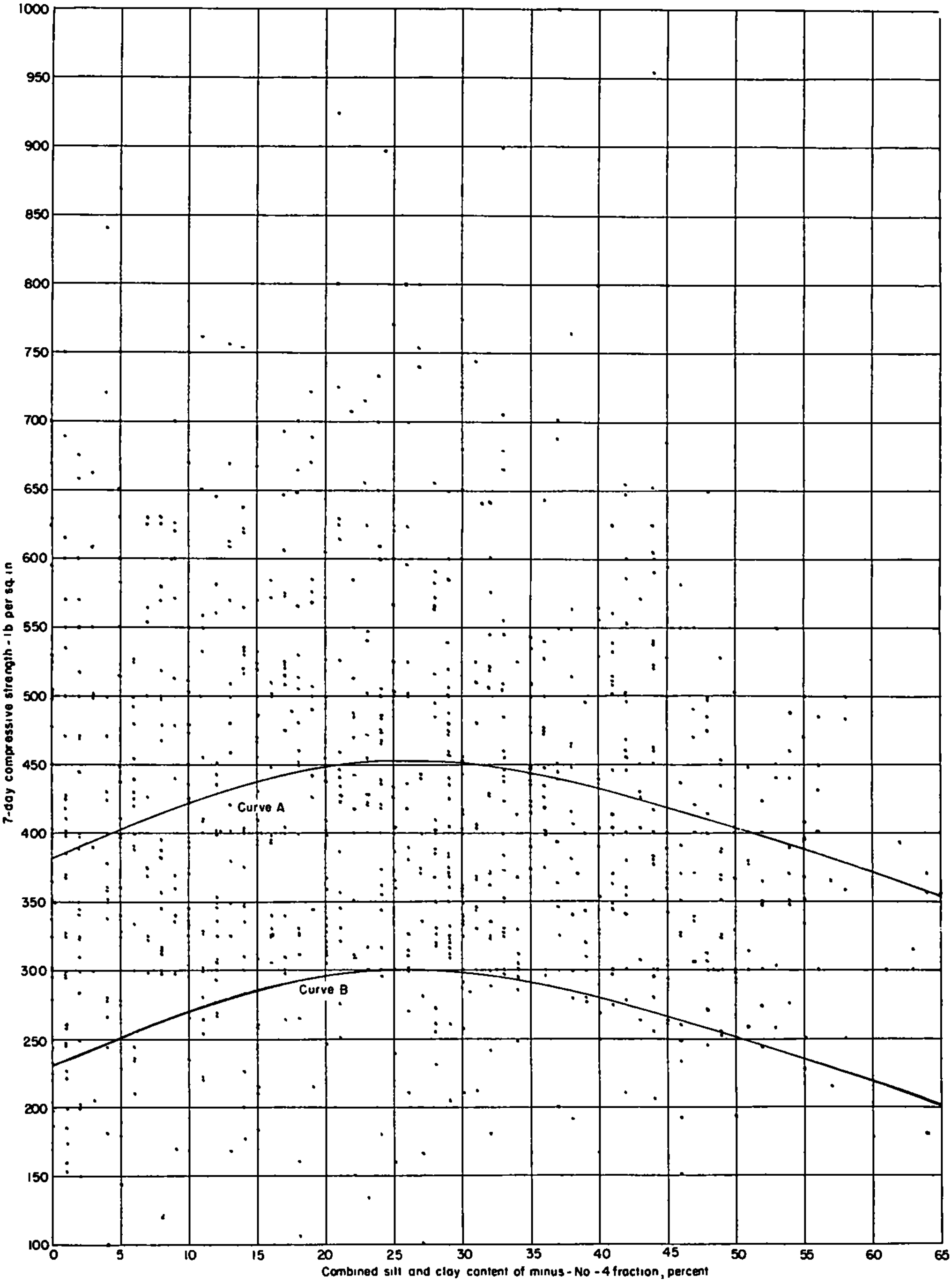


Figure 9. Seven-day compressive strengths of Group I soils at the cement factor shown adequate by ASTM tests.

This cement factor was considered verified if the compressive strength at this cement content fell above the minimum curve of Figure 1. If it fell below the curve, additional testing was considered necessary for determining the cement requirement. The cement factors indicated were then compared with those previously obtained from ASTM tests. The comparison produced the following results:

Adequate cement factors or the need for further testing were indicated for 98.2 percent or 1,520 of the 1,548 soils. For 28 or 1.8 percent of the soils, the procedure did not indicate adequate cement factors; nor did it indicate the need for additional testing, since the strengths for these soils at the indicated cement contents fell above the minimum strength Curve B as shown in Figure 10. The cement factors indicated for 17 of these 28 soils were just slightly less than adequate.

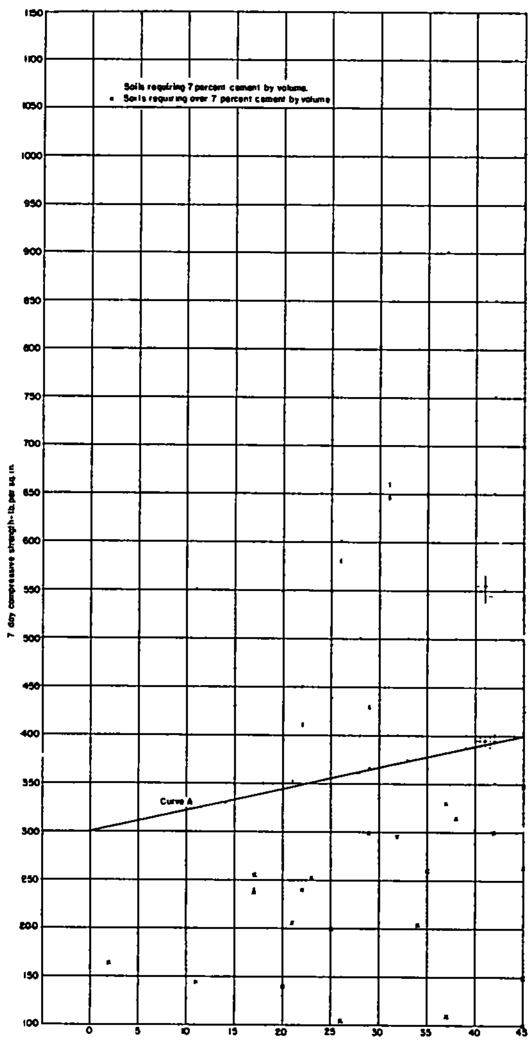


Figure 10. Seven-day compressive strengths of abnormally reacting soils in Group I.

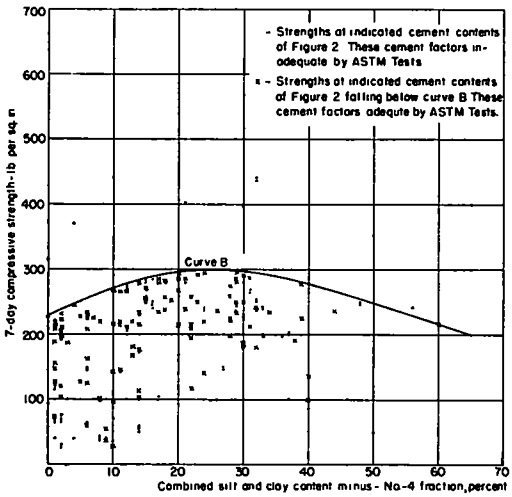


Figure 11. Seven-day compressive strengths of Group II soils at the cement content in the minus-No.-4 soil-cement mixture equivalent to 7 percent in the total mixture.

There were 377 of the 1,548 soils for which additional testing was indicated by the procedure. This was because the strength at the indicated cement content fell below the minimum strength curve shown in Figure 1. Most of these soils were actually poorly reacting or slow-hardening soils. This, of course, is the purpose of the minimum-strength curve: to permit recognition of soils that are not hardening normally. In some cases, even though 7-day strengths were below the minimum-strength curve, the rate of hardening was such that the cement factors indicated by the procedure were shown adequate by ASTM tests. In Figure 10, for example, the points shown below Curve B and identified by an X are in this category. The points below Curve B identified by dots represent those soils in the group of 377 for which ASTM test results showed that higher cement factors than indicated by Figure 2 were actually needed.

Group II Soils

Of the remaining soils under study, 472 were designated as Group II. These consisted of 222 soils classified texturally as gravel and sand, 171 classified as sand and gravel and 79 classified as gravelly loamy sand. According to ASTM tests 7 percent cement by volume of total mixture was adequate for 444 of these soils indicating a rather consistent cement factor for the group.[7]

These soils were handled in the same manner as Group I soils, except that no chart similar to Figure 2 is needed since 7 percent cement is the indicated cement factor. To develop a minimum-strength curve, 7-day compressive strengths of minus-4 material with a cement content equivalent to 7 percent in the total mixture were plotted (see Fig. 11). Line A was drawn to include above it most of the soils which actually required 7 percent cement and to include below it those requiring more than 7 percent cement. Line A, therefore, was considered as representing about the minimum compressive strengths generally obtained for normally reacting Group II soils with 7 percent cement by volume of total mixture. Compressive strengths below this line were considered to indicate that over 7 percent of cement by volume of total mixture may possibly be required and that ASTM testing is needed to definitely establish the cement factor. Line A of Figure 11 is the minimum-compressive-strength curve shown in Figure 4.

The following results were obtained for the 472 Group II soils using Line A of Figure 4 as a strength criterion.

The compressive strengths for 414 of the 472 soils at the cement content in the minus-No. -4 mixture equivalent to 7 percent in the total, were above the minimum specified in Figure 4 and the 7 percent cement content was thereby verified.

For 58 soils, the compressive strengths fell below the minimum curve, indicating that additional testing was needed to determine the cement factor. For 28 of the 58 soils, a cement factor of over 7 percent cement by volume of total mixture was obtained by ASTM tests, and 7 percent was found adequate for the remaining 30 soils.

There were no soils of this group that had strengths falling above the minimum-strength curve, which required a cement factor of more than 7 percent by volume of total mixture.

[7]Some of these soils probably could be adequately hardened with less cement, but cement contents below 7 percent by volume were generally not investigated.

Group III Soils

The remainder of the soils under study, 209 miscellaneous sandy materials such as scoria, shale, slag, were placed in Group III. In general, the data for each type material available for study were insufficient to permit correlations or the assignment of specific cement factors.

Probable cement requirements based on available data and experience are listed in Table 2, along with pertinent information that will serve as a guide and be useful in reducing laboratory work when these materials are to be tested. Since no correlation was made, ASTM tests in modified form are required to indicate cement factors. Generally, it will be necessary to mold and test only a few specimens, as indicated by the suggested step-by-step procedures for Group III soils.

The miscellaneous materials studied and the cement factors required as determined by ASTM tests were:

Shell soils (soils containing appreciable amounts of shells) - 48 included.

The step-by-step procedure suggested for Group I and II soils can be used to determine cement factors.

Limestone screenings - 20 included

The step-by-step procedure suggested for Group I and II soils can be used to determine cement factors.

Red dog - 8 included

Cement requirements ranged from 7 to 9 percent by volume; 4 required 9 percent.

Shale - 21 included

Cement requirements ranged from 7 to 16 percent by volume, the majority required 10 to 12 percent.

Caliche - 24 included

Cement requirements ranged from 7 to 9 percent by volume.

Cinders - 51 included
Cement requirements ranged from 7 to 16 percent by volume, the majority required 7 to 10 percent.

Chert - 3 included
Two required 8 percent by volume and one required 10 percent.

Chat - 11 included
Cement requirements ranged from 7 to 16 percent by volume, the majority required 7 percent.

Marl - 6 included
Cement requirements ranged from 9 to 12 percent by volume.

Scoria - 4 included
Cement requirements ranged from 8 to 14 percent by volume.

Slag - 8 included
Cement requirements for 6 air-cooled slag samples ranged from 7 to 13 percent by volume and for two samples of water-cooled slag cement requirements were 10 and 11 percent by volume.

Fire clay - 1 included
Cement requirement 11 percent by volume.

Iron Ore Base - 1 included
Cement requirement 11 percent by volume.

Brick waste - 1 included
Cement requirement 7 percent by volume.

Roofing granules - 2 included
One required 7 percent cement by volume and one 9 percent.

SUMMARY AND SUGGESTIONS

From this study a correlation of test data was made, making it possible to develop step-by-step procedures for quick determination of adequate cement factors of most sandy soils encountered in soil-cement construction. Soil-cement test data for 2,229 sandy soils from all over the United States and Canada previously tested in accordance with ASTM-AASHO standard test procedures were used in the study.

The step-by-step procedures are applicable only to sandy soils containing less than 20 percent clay and less than 50 percent combined silt and clay. Dark-gray to black soils containing appreciable organic materials are excluded. Only a few, well-known, simple laboratory tests and charts developed by the study are used in following the procedures. Time and work involved in establishing adequate cement factors following the procedures are greatly reduced over that usually required for making complete ASTM-AASHO standard tests. In addition, much-smaller soil samples are needed.

The step-by-step procedures provided reliable methods for establishing safe cement factors for 98.7 percent of the soils in the study. While the cement factors obtained were practical, they were not always the minimum or most economical that could be used to harden the soil.

It is therefore suggested that the procedures and charts be used for determining cement factors of sandy soils falling in the three groupings listed. They should be used in their present form until local test data and experience are obtained to permit revision of the charts for local conditions.

It is believed that the testing procedures suggested will find wide application and use by state, county, and city engineers, as well as by military engineers, since the procedures are easily and quickly applied and only small soil samples are needed. The charts and figures, for example, will be helpful in estimating probable cement factors to permit making job estimates before any tests are made.

The procedures can be used to advantage in conjunction with such work as establishing cement factors for major key soil series that cover wide areas.

Discussion

D. T. DAVIDSON, Associate Professor of Civil Engineering, Iowa State College — The lack of knowledge about the factors controlling the complex reactions between soil and portland cement has necessitated the development and use of empirical procedures, such as the ASTM and AASHO standard methods, for determining the cement requirement for adequately hardening soil. As pointed out by Leadabrand and Norling, the standard methods have proved to be dependable in most cases, but the time required to get the cement requirement by them is objectionably long. Because of this, procedures which will give quicker, reliable results are greatly needed.

The method for quickly determining the cement requirement for sandy soils is an important contribution and should prove to be a useful and time-saving supplement to the standard methods. It is, however, subject to the limitations of all empirical methods, principal among which is that the user does not understand the reasons for test results, whether poor or exceptionally successful. An understanding of results is necessary if we are to improve the interaction between soil and cement and to extend the usefulness of soil-cement stabilization to soils which now are considered unsuitable for this type of treatment.

Empirical methods will in all likelihood continue to be used until the frontiers of soil knowledge have been considerably advanced. The ultimate goal, however, should be a rational method based on known relationships between soil composition and cement requirements. The attainment of such a goal will require the active collaboration of all agencies working with soil-cement in all parts of the world. The Soil Research Laboratory of the Iowa Engineering Experiment Station, which is investigating these relationships for loess soils, desires to cooperate fully with all other organizations engaged in similar work.

A. B. CORNTHWAITE, Testing Engineer, Virginia Department of Highways — The problem of adequate investigation and control of highway construction materials with a minimum amount of personnel and time delay in arriving at a reasonable and acceptable solution has never before been so forcibly brought to the attention of highway engineers as at the present time. Shortages in personnel, whether competent or incompetent, is now an old story to administrators. While this report is limited in its applications to sandy soils, I am certain more is yet to come.

In laboratories already overcrowded with samples for analysis, the reduction in size from approximately 100 lb. to samples of 25 to 40 lb., and the completion of the analysis in from 7 to 10 days will be a real boon. This provides the real incentive for additional work along this line.

Recognition is made in the report that local conditions require special study and of the probable need for the development of modified curves for particular areas. Still, the foundation has been laid for a shortened procedure.

It was noted that Virginia contributed 68 samples, or 3.05 percent of the 2,229 samples of sandy soils studied. Exactly how many were included in each of the groups of soils studied is not known. However, it is felt that although percentage-wise Virginia had good representation in the correlation study, the large areas of the Midwest with large soil areas having similar characteristics may have had a cumulative effect that influenced the locating of the various curves.

The identification of soils by horizons is not feasible in the majority of our soil-cement work. With most projects being the improvement of existing roads, the determination that the present roadbed can be stabilized with cement is of major importance. In addition, the soil mantle for the A horizon generally is a matter of from 2 to 6 in., and for the B horizon from 3 to 10 in. Naturally, field operating conditions will not permit a very accurate separation of these materials from each other, or even from the C horizon. Further, it has been found in the grading and shaping of our roadways for cement stabilization that it is the practice rather than the exception to pull in a large amount of unsuitable material from the shoulders and ditches. This must be taken into account when determining the correct cement content.

Reviewing a portion of the soil-cement work done in Virginia during the past 2 years and analyzing data in the light of the report under discussion, we find that of a total of 58 projects, 30 fall within the textural classification of Group I soils, 26 carried "clay" description and 2 are classed as loams. Three could not be stabilized with cement but did carry textural classifications of clay loam, sandy clay, and fine sandy loam similar to other work which could be stabilized and located in the same area. It is believed that excessive organic matter was responsible for this situation.

The cement requirement as we determined it for these projects is considerably above the minimum requirement based on the curves presented by Leadabrand and Norling. Perhaps we have been more rigid in our specification requirements than necessary, but at the same time we have been liberal in the interpretation of our analytical data as regards the relation between compressive strength, freeze-thaw, and wet-dry tests. In all laboratory testing we have used a minimum of three different cement contents and our recommendations ranged from 10 to 14 percent cement necessary for proper stabilization. By the Portland Cement Association criteria presented today, 8 percent would have been adequate for the majority of projects.

In conclusion, I think a rational approach to a shortened testing procedure has been offered. A careful study of the data for each soil area will permit the development of curves similar to those offered and will take cognizance of special local conditions.

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Effectiveness of Various Soil Additives for Erosion Control

LOUIS J. GOODMAN, Assistant Professor of Civil Engineering,
Ohio State University

THIS paper summarizes an extensive search for soil additives that can reduce the damaging effects of rainfall on steep slopes and thereby curb erosion. Described are laboratory testing procedures developed for hydromechanical studies of soil erosion and for evaluating the soil additives studied. As a check on the laboratory work, field slopes have been set up in several sections of the country and observations on these have been quite encouraging.

Although no entirely successful material was found during these investigations, one has proved to be quite effective on certain soil types. Several other additives have shown good possibilities on one or two soil types. In order of their effectiveness based on current test data these are: (1) Monsanto CRD-189; (2) Monsanto CRD-186; (3) soil-cement aggregates; (4) Dupont Orchem DV-71; (5) Aerotil; and (6) Dupont Elchem-1089.

As a result of this study, two practical methods of application were evolved: (1) spread additive on surface uniformly and wet down and (2) bake in additive to a depth of about ½ in. and wet down.

● EROSION is one of the more-serious problems encountered by engineers and soil conservationists. Highway cut and fill sections, upstream faces of earth dams, and other types of earth slopes must be protected against erosion. Current control methods are either too expensive or detrimental to vegetation, which is the simplest means for protection of most slopes. Many fine-grained soils, which are not conducive to vegetation, are highly susceptible to erosional damage.

Navy interest in soil-erosion studies results from its control of nearly 4,500,000 acres of land in this country, ranging from barren desert to heavily timbered areas. Erosion is a particular problem at ammunition depots and airfields. Consequently, late in 1950 the current research project was inaugurated by the Bureau of Yards and Docks, U. S. Navy Department, to make hydromechanical studies of soil erosion and explore techniques for controlling construction of slopes. Important parts of the over-all objectives include: (1) design and construction of a device for simulating rainfall in the laboratory; (2) location of additives that would reduce drastically the effects of rainfall on steep slopes and be conducive to plant life; (3) determination of the practicality of promising additives for field use; and (4) establishment of a mathematical relationship between the energy of raindrops and soil loss.

Since the texture and chemical composition of soils vary over such a wide range, and since even slight changes in these properties greatly influence the susceptibility of soils to the erosional processes, the problem of curbing erosion by soil additives is extremely complex and one unlikely to be solved by a single, simple method.

It is the purpose of this paper to summarize the progress made to date and to stimulate more interest in this urgent problem.

MATERIALS TESTED

Soils

In order to test the effectiveness of prospective soil additives upon a wide range of fine-grained soils, samples

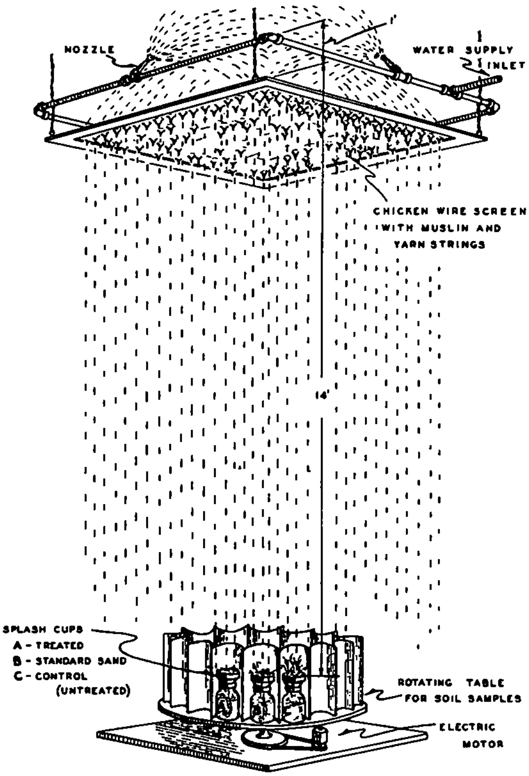

Figure 1. Artifical-rainfall applicator.

were obtained from various parts of the country. The characteristics of the soils used to date are shown in Table 1.

Additives

Six additives have been investigated as to effectiveness in curbing soil erosion at the time of this writing. A brief description and source of each additive is given in Table 2.

TESTING PROCEDURES

Since soil erosion is "a process of detachment and transportation of soil materials by erosive agents,"[1] it was believed that the primary solution to the erosion problem on steep slopes must lie in the prevention of the detachment of particles of soil. Realizing that the principal detaching agent is the raindrop, it was decided to run a sufficiently large number of splash tests to determine the effectiveness of various additives in reducing the splash loss of soils subjected to high erosional damage.[2]

For determining the splash loss of a given soil, an artifical-rainfall applicator was designed and constructed. Figure 1 shows three splash cups in place on a rotating table under the rainfall applicator. Model slopes of varying degree were also investigated under this applicator. Photographs of two typical model slopes after a splash test are shown in Figure 2. The use of a rotating table and the practice of oscillating the screen ensures identical rainfall treatment on all soil samples. A complete description of the equipment used in the splash-loss analyses and its function will be found in a previous paper[3] written by the author.

At the outset of the investigation it was realized that a simple, yet effective, method for screening the various additives must be found. Initially, to establish a trend on the effectiveness of an additive, gradation analyses were run on soil samples treated with an economical concentration of the additive (0. 2 percent of dry weight). These results were then compared to mechanical analyses on the untreated soil samples, and if the percentage of treated soil passing the No. 200 sieve was reduced by one half or more, the additive was considered promising. Due to the shortcomings of this method, it was later decided to evaluate additives in the following manner:

1. Sprinkle an economical concentration of the additive in question into a slurry of water and fine-grained soil. In most cases, if the water is taken up and the fine grains of the soil form into aggregations or clumps, the additive will be effective in curbing erosion.

2. Then, as a check on Step 1, treated soil crumbs are placed in a beaker of water. If the treated crumbs maintain their shapes indefinitely (untreated crumbs will disintegrate immediately), it is felt that the additive should be investi-

[1] W D Ellison, "Soil Detachment and Transportation," published in U S Soil Conservation Service "Soil Conservation", February 1936, Volume II, No 8

[2] These soils are principally fine sands, silts, and clays

[3] L J Goodman, "Erosion Control in Engineering Works", Agricultural Engineering (March, 1952)

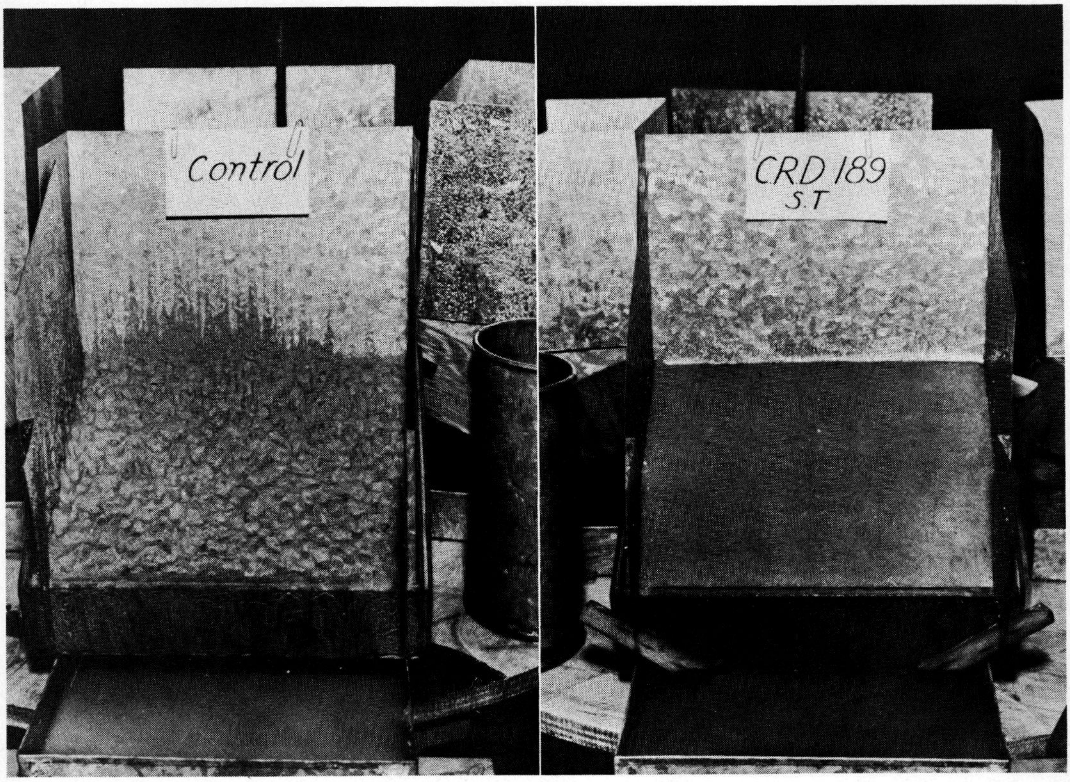

Figure 2. An untreated model slope (at left) is compared with a CRD-189 surface-treated slope (at right) after a 15-min. splash test.

TABLE 1

CHARACTERISTICS OF SOILS INVESTIGATED

	Soil	Gravel %	Sand %	Silt %	Clay %	Uniformity* Coeff.	Liquid Limit	Plasticity Index	Specific Gravity	pH	Description
A.	Shade River	0.0	81.25	16.25	2.50	6.3	(Not plastic)		2.63	6.1	Silty Sand
B.	Delaware Sand	0.0	71.1	28.9	0.0	7.8	(Not plastic)		2.73	9.4	Silty sand
C.	Boston Blue Clay	0.0	0.0	36.0	64.0	29.0	39.2	16.7	2.74	8.0	Very silty clay
D.	Mechanicsburg, Pa.	3.0	14.5	46.0	36.5	9.1	31.2	10.3	2.75	7.2	Clayey silt w/sand
E.	University Farm	2.0	50.5	22.5	25.0	29.9	39.5	15.9	2.71	7.3	Clayey silty sand
F.	Blendon Woods (Cut)	3.0	22.0	44.0	31.0	-	33.7	15.5	2.73	-	Clayey silt with appreciable sand
G.	Olentangy Sand	0.0	81.0	17.0	2.0	12.0	(Not plastic)		2.65	-	Silty sand
H.	Muskingum Sand	23.08	69.23	5.23	2.46	7.1	(Not plastic)		2.68	8.4	Gravelly sand w/ some fines
J.	Blendon Woods (Fill)	8.62	9.96	39.95	41.47	11.7	36.0	19.95	2.72	7.2	Silty clay w/ sand sizes
K.	Delaware silt	10.25	46.06	41.9	1.79	9.1	(Not plastic)		2.70	9.0	Sandy silt w/ some gravel
L.	Olentangy (East Bank)	6.60	7.96	54.25	30.74	51.1	43.1	17.27	2.76	6.8	Clayey silt w/ sand sizes
M.	New Jersey P.S. (Yellow)	0.41	84.09	13.31	2.19	5.6	(Not plastic)		2.69	5.7	Silty sand
N.	New Jersey G.S. (Red)	0.5	83.5	7.6	8.4	67.5	(Not plastic)		2.72	5.3	Sand with some fines
O.	Stonelick	5.5	8.5	66.5	19.5	7.6	33.0	6.4	2.67	5.8	Clayey silt
P.	Tusca-Meigs	0.0	3.2	37.5	59.3	0.0	51.3	24.07	2.77	-	Very silty clay
Q.	Crane Ind.	0.0	18.6	47.6	34.8	23.6	30.7	21.8	2.70	5.8	Clayey silt w/ sand
R.	New Jersey (Pier Area No. 1)	0	50.2	31.4	18.4	2.7	(Not plastic)		2.57	4.2	Silty sand w/ clay
S.	New Jersey (Pier Area No. 2)	0	51.0	26.6	22.4	14.2	(Not plastic)		2.58	3.8	Silty sand w/ clay

*Uniformity coefficient is defined by Hazen as the ratio of diameter of 60-percent size to diameter of 10-percent size.

gated in a splash-loss analysis.

In preparing for a splash test, the soil samples were compacted into the splash cups in three layers and struck off. The compactive effort was predetermined to give a density comparable to the in situ, or natural, density for each soil. The soil samples were brought to a standard condition of moisture before the test by putting water in the jars and allowing saturation to take place via cotton wicks over night (see Fig. 1). The jar served another function in collecting the water that would seep down through the soil during the test, giving a relative measure of infiltration.

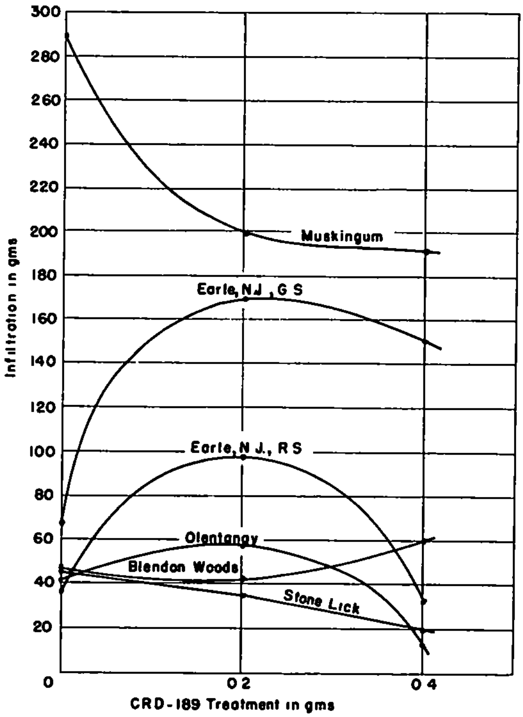

Figure 3. Treatment versus infiltration curves for various soils.

For the treated samples, the soil additives were applied either as a surface or rake-in treatment at a concentration comparable to 1 lb. per 100 sq. ft. Then both treated and untreated soil samples were surface moistened. The surface moistening served a dual purpose: (1) water soluble additives such as Monsanto's materials must be put into solution to cause the aggregation to take place. (2) All soil samples should be

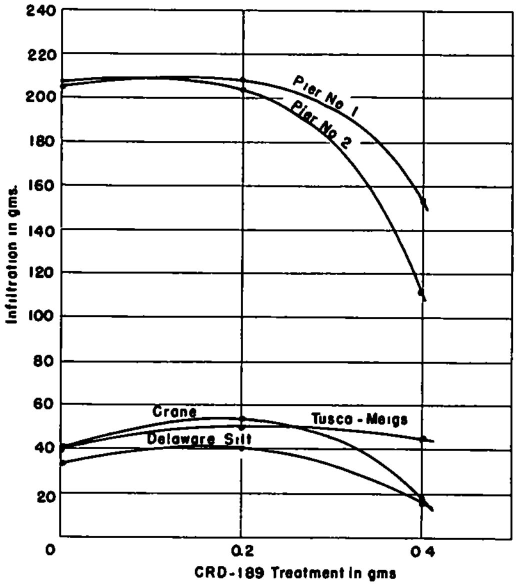

Figure 4. Treatment versus infiltration curves for various soils.

kept in a standard condition of moisture before the test.

It was found desirable to use an oil cloth large enough to cover all the samples until the rainfall applicator was functioning properly. Also, upon com-

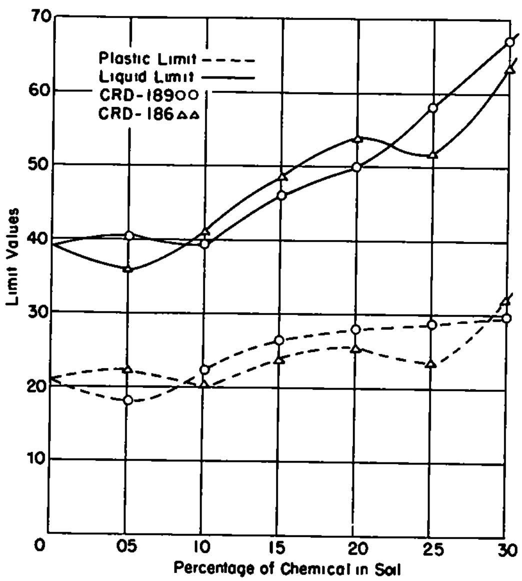

Figure 5. Liquid and plastic limits versus concentration of chemical for Boston Blue Clay.

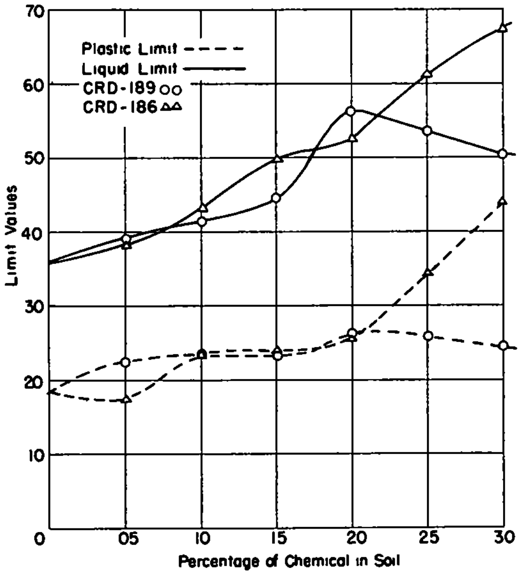

Figure 6. Liquid and plastic limits versus concentration of chemical for Blendon Woods Fill.

pletion of a test, the samples were again covered quickly until all dripping had ceased.

Rainfall intensity was measured by placing two water containers diametrically opposite each other on the turntable.

In tests to date, a raindrop diameter of 5.04 mm., which compares favorably with the size of drop encountered in erosive rainstorms, has been used. This was accomplished by employing a 3/16-in.-diameter cotton yarn with a 2-in.-mesh wire screen.

Duplicate samples of a well-rounded standard sand (60 to 70 gradation) were also investigated in each splash test to compile data for the mathematical analyses of detachability.

Splash loss is determined by obtaining the differences in oven-dry weights of each sample before and after the test.

Infiltration data were obtained with different concentrations of CRD-189. In this phase of the investigation the drip screen was lowered several inches above the splash cups to eliminate impact effects on the soil surface.

The effects of the additives on the plastic and liquid limits of certain of the soil samples were also investigated.

TEST RESULTS

The laboratory splash-loss analyses are summarized in Table 3. In studying these test results, it should be realized that most of the materials investigated were not developed primarily for erosion control and that the results obtained in this connection do not necessarily reflect the effectiveness of these materials when used for other purposes.

Infiltration data to date have been compiled on CRD-189, one of the most promising additives for erosion control used to date. The results of this study are shown in Figures 3 and 4.

The effect of various concentrations of CRD's 186 and 189 on the liquid and plastic limits of soils C and J are shown in Figures 5 and 6.

Discussion of Results

CRD-189, when applied as a surface treatment, ranged from excellent to fair in effectiveness in reducing the splash loss on all soils investigated with the exception of Soil B. As can be noted from Table 3, this resin was very effective on half of the soils on which it was tested, reducing splash loss by as much as 24 times on Soil O, a silt containing appreciable clay. It might be well to mention here that Monsanto's CRD's have remarkable effects in altering the struc-

TABLE 2

Material	Description	Source
1 Monsanto CRD-189	Sodium salt of hydrolyzed polyacrylonitrile - - powder form - known commercially as "Krilium"	Monsanto Chemical Company
2 Monsanto CRD-186	Calcium carboxylate polymer - powder form	Monsanto Chemical Company
3 Soil-cement aggregates	Aggregates made from a workable mortar of one part cement to eight parts natural soil (by wt.) Aggregates passing a 3/8-inch screen and retained on a number 8 screen used.	Ohio State University concrete laboratory
4 Dupont Orchem DV-71	-	Dupont Chemical Company
5 Aerotil	Hydrolyzed polymer of acrylonitrile - wettable flakes	American Cyanamide Company
6 Dupont Elchem 1089	Acidic vinyl polymer - powder form	Dupont Chemical Company

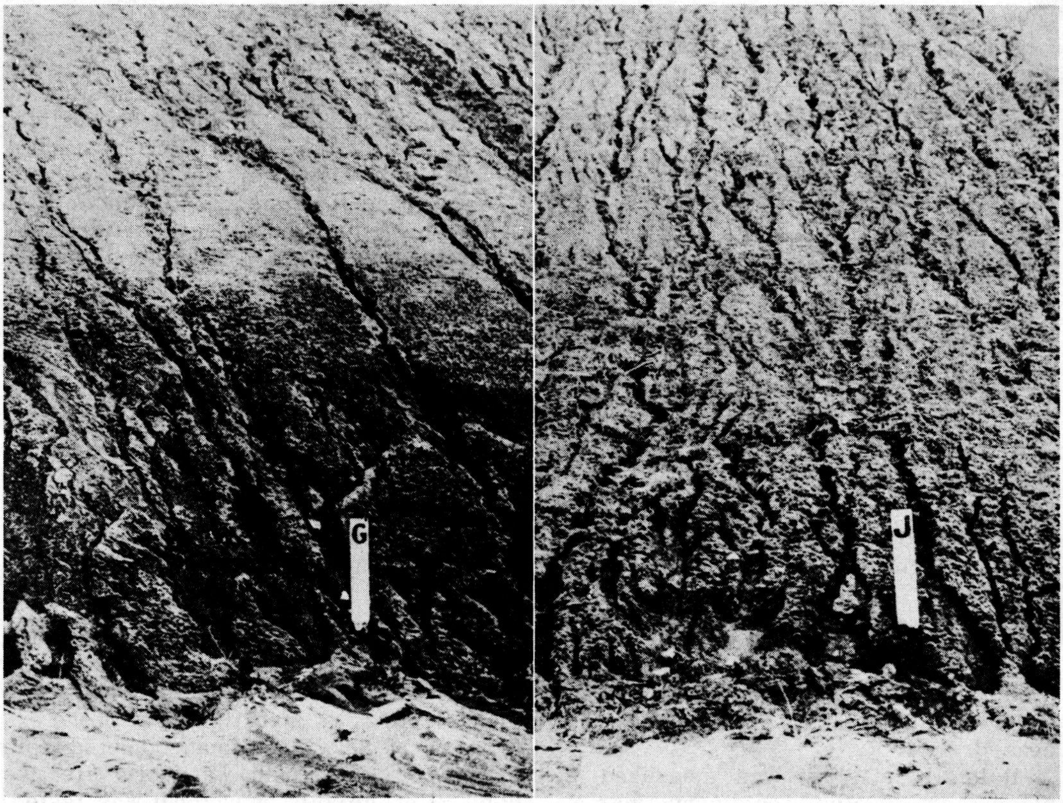

Figure 7. A CRD-189 surface-treated slope (at left) and a control slope (at right) illustrate effects of frost action on erosion.

ture of many soils containing clay particles, increasing aggregation. It can also be noted from Table 3 that CRD-189, when applied as a rake-in treatment, actually increased the splash loss. This appears to be due to the swelling effect the additive has when raked into the soil.

It can be seen in Table 2 that CRD-186 ranged from excellent to fair in effectiveness in reducing splash loss, both as a surface and a rake-in treatment, on nearly all soils investigated. Generally speaking, better results were obtained from the surface treatments, but the splash loss was reduced by approximately 16 times when this polymer was raked into Soil A, a silty sand.

Soil-cement aggregates showed good results on the silty sand from Earle, New Jersey, but a large concentration was needed. Crushing the mortar into aggregate sizes that will be effective in reducing splash loss and yet not reduce percolation has posed a problem. Adequate coverage for preventing the blasting effect of the raindrop is necessary, but the coverage must not be detrimental to vegetation.

To date, DV-71 has been investigated on several soil types. From these inconclusive results, it appears that this new additive will be quite effective in curbing soil erosion on both clayey and sandy soils.

Aerotil had no effect in reducing splash loss of clayey soils but was quite effective on a silty sand. This is a new additive and more test data are required.

Finally, it can be noted from Table 2 that Elchem 1089 showed only fair results. In some of the tests not summarized here this chemical appeared to be quite erratic, and at present it does not hold much promise as a controller of soil erosion.

It was hoped to establish a correlation

between infiltration and splash loss on the soils investigated with a CRD-189 surface treatment. On the basis of the data compiled at this writing, the results appear to be quite erratic in that splash loss reduction was effective on soils with both increased and decreased infiltration. However, it can be noted from Figures 3 and 4 that the infiltration shows a decreasing trend at the 0.4-gram concentration of CRD-189 for all soils except Soil J. This concentration is comparable to 1 lb. per 100 sq. ft. A more rigorous analysis of water percolation will be conducted at a later date in the form of falling-head permeability tests.

It is interesting to note from Figures 5 and 6 that both the liquid and plastic limits were increased at the 0.2-percent concentration of CRD-189 and CRD-186, a concentration comparable to the 1 lb. per 100 sq. ft. used in the splash-loss analyses. This increase in liquid and plastic limits indicates an increase of the strength of the soils. Since the splash loss of Soils C and J was reduced by these two chemicals, it appears that the plasticity tests might well be used in screening additives for erosion control of cohesive soils. This trend has been observed on other clayey soils and will be investigated in detail.

FIELD INVESTIGATIONS

Experimental plots have been established in several sections of the country to obtain field correlations with laboratory results. To date the only additives used in the field have been Monsanto's CRD's, but additional experimental slopes will be established in the immediate future with other promising additives, especially Dupont DV-71. The concentration of chemicals used in the field has been similar to laboratory concentrations.

At the U. S. Naval Ammunition Depot

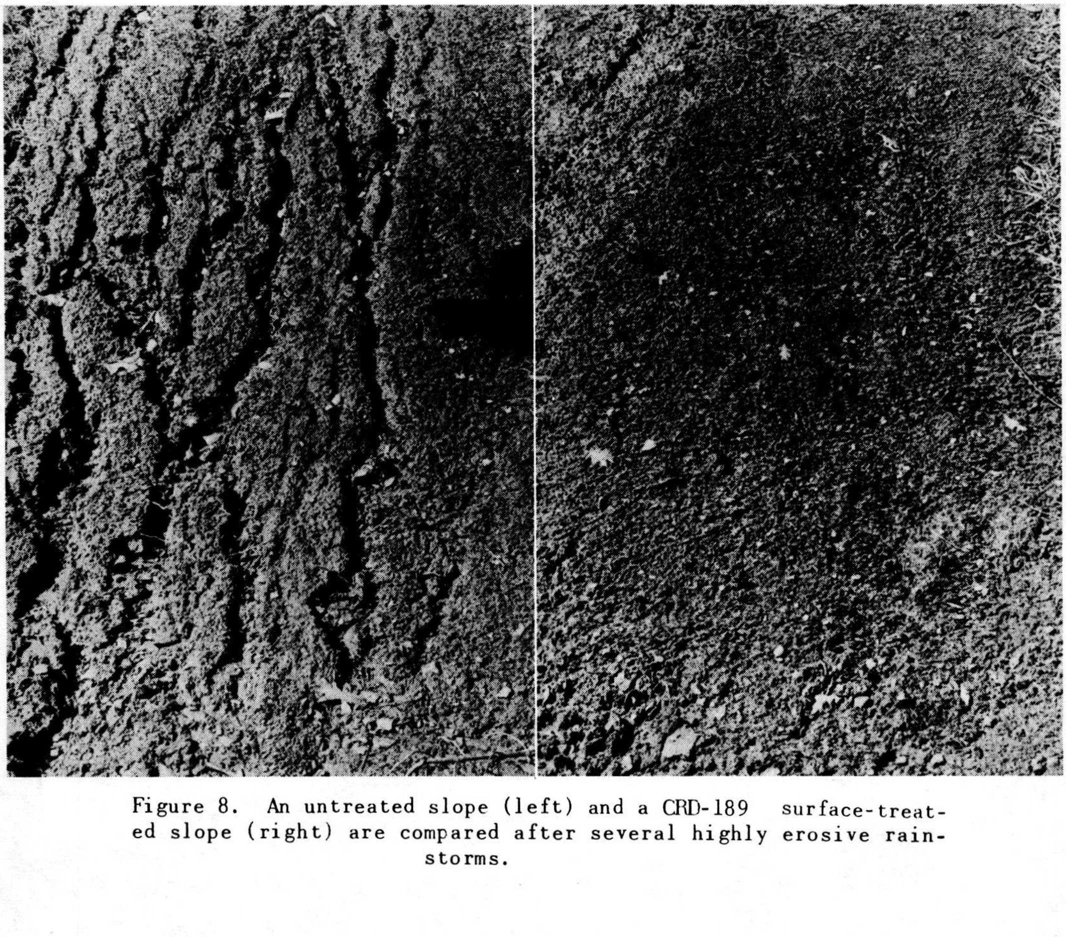

Figure 8. An untreated slope (left) and a CRD-189 surface-treated slope (right) are compared after several highly erosive rainstorms.

TABLE 3

Soil	How Investigated	Treatment	Concentration of Additive	Rainfall Intensity iph.	Duration of Test min.	Free Fall of Raindrop ft.	Average Splash Loss grams
A	Splash Cups	Control (Untreated)	1 lb/100 sq ft.	6 15/16	20	14	79 8
		CRD-186 (Rake-in)	"				4.8
		186 (Surface)	"				12.0
		189 (S)*	"				47.7
		189 (R)*	"				101 3
B	Splash Cups	Control	1 lb/100 sq. ft.	8	20	14	75.3
		CRD-186 (S)					21.0
		186 (R)					58.9
		189 (S)					86.2
		189 (R)					124.1
C	Splash Cups	Control		7 5/16	20	14	62.2
		CRD-186 (R)	1 lb/100 sq ft.				39.5
		186 (S)	"				40.0
		189 (S)	"				46.4
		Control		3	30	6	13.6
		Aerotil (S)	"				14.6
D	Splash Cups	Control		6 15/16	20	14	67 9
		CRD-189 (S)	"				6.8
		186 (S)					15.1
		186 (R)					35.3
		189 (R)					13.1
E	Splash Cups	Control		9	20	14	81.6
		CRD-186 (R)	1 lb/100 sq ft.				47.9
		186 (S)	"				68.3
		189 (S)	"				73.0
		189 (R)	"				123 4
F	Splash Cups	Control		8 1/4	20	14	86.9
		CRD-186 (R)	1 lb/100 sq ft.				17.1
		186 (S)	"				59.0
		189 (S)	"				59 7
		Control		2 3/4	30	6	14.0
		Aerotil (S)	"				13.5
G	Splash Cups	Control		10	20	14	98.7
		CRD-189 (S)	1 lb/100 sq ft				21.3
		186 (R)					91.9
H	Model Slopes	Control		3	20	9	71.5
		CRD-189 (S)	1 lb/100 sq. ft.				64.4
J	Splash Cups	Control		3	30	6	12.5
		Aerotil (R)	1 lb/100 sq ft.				19 5
		Control		1 3/8	60	6	18.6
		Elchem 1089 (S)	2 lb/100 sq ft				15.2
		Control		1 7/8	60	6	15 7
		DV-71 (S)	15 lb/100 sq ft.				7 6
	Model Slopes	Control		3 3/4	15	9	170.5
		CRD-186 (S)	1 lb/100 sq. ft.				16.4
		189 (S)	"				18.0
K	Model Slopes	Control		4 1/3	20	9	75.9
		CRD-189 (S)	1 lb/100 sq ft.				11.3
		186 (S)	"				12.9
L	Splash Cups	Control		2 3/8	60	6	26.0
		CRD-189 (S)	1 lb/100 sq. ft.				13 3
		Elchem 1089	2 " "				24.4
		Control		3			11.5
		Aerotil (S)	1 lb/100 sq. ft.				13.0
		Control		2 1/16	60	6	22 0
		CRD-189 (S)	1 lb/100 sq. ft.				10.0
		DV-71 (S)	15 lb/100 sq. ft.				15.2
	Model Slopes	Control		3 3/4	20	9	217 0
		CRD-189 (S)	1 lb/100 sq. ft.				97.8
M	Splash Cups	Control		3 2/3	30	9	37.8
		CRD-189 (S)	1 lb/100 sq ft.				3.6
		186 (S)	"				7.3
		Control		2 1/3	15	9	18.1
		Soil-Cement (1 4)	35 lb/100 sq. ft.				10.0
	Model Slopes	Control		4 1/2	20	9	272 6
		Soil Cement (1. 8)	3/4 inch depth				9 9
		Control		3 3/8	20	9	225 3
		CRD-189 (S)	1 lb/100 sq ft.				17 3
		Elchem 1089	2 lb/100 sq ft.				190 8

TABLE 3 (continued)

Soil	How Investigated	Treatment	Concentration of Additive	Rainfall Intensity iph.	Duration of Test min.	Free Fall of Raindrop ft.	Average Splash Loss grams
N	Splash Cups	Control		1 3/4	30	6	14.9
		Aerotil (S)	1 lb/100 sq. ft.				8.9
		Control		2 3/8	60	6	36.9
		DV-71 (S)	15 lb/100 sq. ft.				10.9
		Control		2 1/16	60	6	28.4
		CRD-186 (S)	1 lb/100 sq. ft.				8.6
		Elchem 1089 (S)	2 " "				16.2
O	Splash Cups	Control		2 3/4	30	6	39.2
		Aerotil (S)	1 lb/100 sq ft.				35.6
	Model Slopes	Control		4 3/4	20	9	213.3
		CRD-189 (S)	1 lb/100 sq ft.				8.7
		186 (S)	"				92.6
P	Splash Cups	Control		2 3/4	30	6	39.2
		Aerotil (S)	1 lb/100 sq ft.				35.6
	Model Slopes	Control		5	20	9	224.0
		CRD-189 (S)	1 lb/100 sq ft.				27.1
		186 (S)	"				55.2
Q	Splash Cups	Control		4 1/8	30	6	29.1
		DV-71 (S)	15 lb/100 sq ft.				10.8

* S denotes surface application
R denotes rake-in application

in Earle, New Jersey, slopes of 1 on 2 (1 vertical to 2 horizontal) and from 60 to 78 ft. long were selected for field-testing the chemicals. Generally speaking, results have been quite encouraging in this area. However, one group of plots was established in the late fall of 1951 to determine the field effects of frost action on treated slopes with no vegetation present. The results of this investigation after an average winter are shown in Figure 7. It can be seen that a surface treatment of CRD-189 at a concentration of 1 lb. per 100 sq. ft. was not effective in controlling erosion due to frost action. This may be due to soil, which is very acid (pH = 4.0), or due to the ineffectiveness of this particular chemical in controlling frost action. This matter will be investigated in detail.

Fairly steep slopes were also established on a highway fill section in the Blendon Woods Metropolitan Park area, Columbus, Ohio, (Soil J). Results from this section have been excellent. Figure 8 shows a control (untreated) slope and a CRD-189 surface-treated slope after several highly erosive rainstorms. The control slope was severely eroded, whereas the treated slope shows no sign of erosion.

Figure 9 is a photograph taken recently of the fill section at the Blendon Woods Metropolitan Park area. The slope on the right is untreated while that on the left has a CRD-189 surface treatment at a concentration of 1 lb. per 100 sq. ft. Both plots were seeded and fertilized simultaneously. Note the heavy growth of grass on the treated plot.

MATHEMATICAL RELATIONSHIPS

An attempt has been made to establish a mathematical relationship between soil loss and the energy of raindrops. The practicality of the relationship has not yet been determined, and the following discussion will concern itself solely with theoretical considerations.

The loss of soil from laboratory splash cups is in some way related to total rainfall, raindrop diameter, and the velocity of the rain. The splash-cup data for tests run on the standard sand (60 to 70 gradation) and the New Jersey yellow soil were analyzed for the mathematical relationships. Plotting of the data showed the general equation of exponential form to be satisfactory. Accordingly the following equation was used:

$$L = KV^{a}I^{b}T^{c}D^{e}$$

Where, L = Loss of soil or sand due to raindrop splash, in grams.
K = A constant of proportionality to be calculated.

Figure 9. A CRD-189 surface-treated slope (at left) and an untreated control slope are studied several months after being established.

V = The corrected velocity of a water drop falling in stagnant air, in feet per second.

I = The rainfall intensity, in inches per hour.

T = The time in minutes at which a given loss occurs.

D = Raindrop diameter in mm.

a, b, c, e = Exponents which must be calculated.

In the mathematical data compiled to date a constant raindrop diameter of 5. 04 mm. has been used, thereby eliminating this variable temporarily. The splash loss equation then becomes

$$L = K\, V^{a} I^{b} T^{c}$$

The coefficients and exponents were calculated by the method of least squares. Velocities of impact for raindrops were introduced as 25. 5 ft. per sec. for a free fall of 14 ft. and 21. 6 ft. per sec. for a free fall of 9 ft. The standard sand data were obtained over a relatively wide range of intensities and times and should be fairly reliable. Intensities ranged from 2 ½ to 10 in. per hr. and times from 0 to 60 min. However, mathematical data from tests on the New Jersey yellow soil was relatively limited, rendering the results inconclusive as of this writing.

For standard sand the following relationship resulted:

$$L = \frac{V^{0.619} I^{1.204} T^{0.891}}{10}$$

It must be emphasized here that much more data on both untreated and treated soil samples is needed for reliable mathematical relationships. It is advisable to have a minimum of three points on the velocity curve, and this is currently being accomplished by using a range of free falls from 6 to 2 ft. to supplement data and results obtained with the free falls of 14 and 9 ft. Also, varying raindrop diameters will be employed.

CONCLUSIONS

From the results presented in this paper, it is apparent that no entirely satisfactory soil additive for curbing erosion has been found. Although several materials are effective with certain soils, none has proved to be universally suitable. Based upon the data and results compiled to date, the following conclusions can be stated:

1. The testing procedures developed in this investigation, from evaluating potential effectiveness of proposed erosion control additives to conducting a splash loss analysis, appear to be satisfactory.
2. Proposed soil additives for curbing erosion should be tested with a large number of soils because the effectiveness of a particular treatment varies with the soil used. The characteristics of the natural soils are the greatest variables encountered in erosion control.
3. The surface-treatment method of applying the soil additives to soil plots appears to be the more effective and practical.
4. Based on laboratory and field results, Monsanto's CRD-189, when applied as a surface treatment, has been the most effective in curbing erosion. However, it must be cautioned here that a rake-in treatment of CRD-189 is not effective.
5. Monsanto CRD-186 has been quite effective in reducing splash loss in the laboratory, but field results using this polymer have not been too encouraging. It appears that more field data on this chemical are needed.
6. More test data on soil-cement aggregates are needed. However, it does not appear that this soil additive will be practical, since extremely heavy concentrations are necessary for effectiveness in reducing the splash loss.
7. Dupont's Orchem DV-71 has shown good promise in the laboratory investigations and will be field tested at more economical concentrations in the near future.
8. Field results show conclusively that the CRD's at economical concentrations (1 ± lb. per 100 sq. ft.) do not inhibit the growth of vegetation, the simplest means for protection of most slopes (see Fig. 9).
9. Cut and fill sections will tolerate steeper slopes when treated with effective soil additives. This may represent an important technique for reducing construction and maintenance on many miles of highway slopes.
10. It appears that there is a definite correlation between the effect of the soil additives on the liquid and plastic limits of cohesive soils and their effectiveness in curbing soil erosion. It is planned to conduct a detailed investigation of this matter.
11. Finally, it appears advisable to study the effects of the various soil additives in controlling frost action.

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