

Evaluation of Experimental Stabilized Soil Base Construction, Webster County, Iowa

J. M. HOOVER, Assistant Professor of Civil Engineering, Iowa State University

This paper presents a portion of the results of an experimental stabilized soil road base program initiated by the Iowa State Highway Commission and the Webster County Engineer's Office. The 8.058-mi long site chosen is typical of the Clarion-Nicollet-Webster soil association area materials found in hundreds of miles of farm-to-market roads in the north-central third of Iowa. The Webster series, a black, heavy-textured, poorly drained clayey soil, dominates.

Variable thickness base sections were constructed by using the in-place soil materials stabilized with Type I portland cement, lime, lime-fly ash and a combination of lime and portland cement. The surface course was a double bituminous armor coat using $\frac{3}{8}$ -in. crushed stone chips.

The experimental features of the project were divided into two primary objectives, both directly related: (a) evaluation of conventional construction procedures, existing construction-inspection specifications and techniques, and recommendations for establishment and/or changes to each of the above areas; and (b) evaluation of the constructed material by field and laboratory tests for determination of stability requirements in the development of design criteria for low-cost stabilized soil base roads. This paper deals principally with the area of the first objective.

Presented are an evaluation of the construction techniques, gradation specifications preceding introduction of the stabilizing agent(s); discussion of use of lime as a pretreating agent for reduction of plasticity and increase of friability, comparison of laboratory and field standard Proctor moisture-density relationships, in-place field density determinations using oil density and standard Proctor penetrometer methods, and variation of unconfined compressive strength of 7-day moist-cure specimens with variations in moisture and density.

THE ECONOMIC and areal lack of high-quality aggregates and soils suitable for highway bases and subbases in Iowa has prompted the use of more locally available or on-site materials. As a result, sponsored research programs at the Iowa Engineering Experiment Station, Soil Research Laboratory, have in the past 15 years evaluated innumerable types of stabilization additives for use with problem soils in highway construction. Soil additives have included inorganic and organic products, by-products, waste products and even a few so-called magic ingredients. Iowa soils are unique in that they range from old to young glacial materials, thin to thick loess deposits (150 ft and over), residual to alluvial, and even include stable sand dunes. Two major problem soils of the state, in terms of highway construction, are the relatively flat, heavy textured, poorly drained topsoils of the northern areas of the state, and the loess of western and southern counties.

Field trials of stabilized soils have been undertaken only after laboratory tests have firmly established the reliability of the newer stabilizing materials. In 1957, 6,000 ft of experimental subbase and base course were laid on Iowa Primary Highway 117 north

Paper sponsored by Department of Soils, Geology and Foundations and presented at the 43rd Annual Meeting.

of Colfax, Iowa, using lime, lime-fly ash, Arquad 2HT (Armour Industrial Chemical Co.), and lime-fly ash-activating agents in comparison with soil-cement and soil-aggregate types of stabilization (1). The major purpose of the project was to evaluate the field stability of the soil-stabilizing agent(s) mixtures. The project clearly indicated the positive effectiveness of some of the materials for use in stabilized soil base and subbase usage in Iowa.

In 1959, an additional experimental program was initiated by the Research Department of the Iowa State Highway Commission and the Webster County Engineer's office. The major purpose of the program was to produce and evaluate a low-cost stabilized base course made from the existing gravel-treated surface of the road embankment. The experimental features of the 8.058-mi long stabilized soil base project were divided into two primary objectives, both directly related: (a) evaluation of conventional construction procedures, existing construction-inspection specifications and techniques, and recommendations for establishment and/or changes to each of these areas; and (b) evaluation of the constructed material by field and laboratory tests for determination of stability requirements in the development of design criteria for low-cost stabilized soil base roads in Iowa. This paper deals only with the area of the first objective. No attempt will be made to analyze the mix design, thickness or stability of the stabilized materials.

Four types of stabilization treatment were used on the project: (a) 8 percent Type I portland cement; (b) 3 percent monohydrate dolomitic lime (used as a pretreatment for reduction of plasticity and increase of friability of the soil material) followed with 5 percent Type I portland cement; (c) 4 percent monohydrate dolomitic lime and 20 percent fly ash; and (d) 8 percent monohydrate dolomitic lime of which 3 percent was used as a pretreatment. All percentages of stabilizing agents are expressed on the basis of percent of dry soil weight at standard AASHTO density of the soil-agent mixture.

Since construction, the roadway has withstood considerably increased traffic including use as a contractor's haul road in 1961, farm implements including corn drills and spiked tooth and disc harrows, and two of the severest winters in 30 years.

MATERIALS

Soil

The test site chosen (Fig. 1) is typical of the Clarion-Nicollet-Webster soil association area materials found in the subgrades of hundreds of miles of farm-to-market roads in the north-central third of Iowa. Since the major topography is flat to very gently undulating, most farm-to-market and secondary roads of this area are basically of ditch cleanout construction. This results in subgrades which often are predominantly topsoil (high organic matter contents) or B-horizon materials (high clay contents). Gravel or crushed stone surfacing are extremely common but seldom penetrate more than 2 to 3 in., 1 to 2 in. being more common. Frost heave and boiling are common occurrences in winter and spring, as are dusty and washboard surfaces in summer and fall.

The existing gravel-treated surface of the road embankment was used for construction of the stabilized soil base course without the addition of any material other than the stabilizing agent(s). Initial inspection and sampling of the roadway showed an average thickness of gravel of about $\frac{1}{2}$ in. underlain by soil material typical of the aforementioned soil association area. As shown in Figure 2, compacted thicknesses of the stabilized base sections were 6 and 8 in. Scarification depths were approximately 5 and 7 in. with respect to compacted thicknesses, depending on the quantity and type of agent added and the density of the in-place material. Therefore, sampling of the proposed base material was made to scarification depth in 18 locations (Fig. 2, Table 1). Soil samples generally fell into classifications A-2-4(0) to A-7-6(8) over the 8-mi length. The soil material in the base had an average of 88.7 percent passing the No. 4 and 48.2 percent passing the No. 200 U. S. standard sieves. The average moisture content over the scarified depth of material was 10.7 percent at the time of sampling in August 1959. The average organic matter and carbonate contents were 1.4 and 10.2 percent, respectively, with an accompanying alkaline soil pH averaging 7.7.

The subgrade material was also sampled in each of the base sample locations, covering the thickness from bottom of scarification to a depth of 18 in. This material ranged

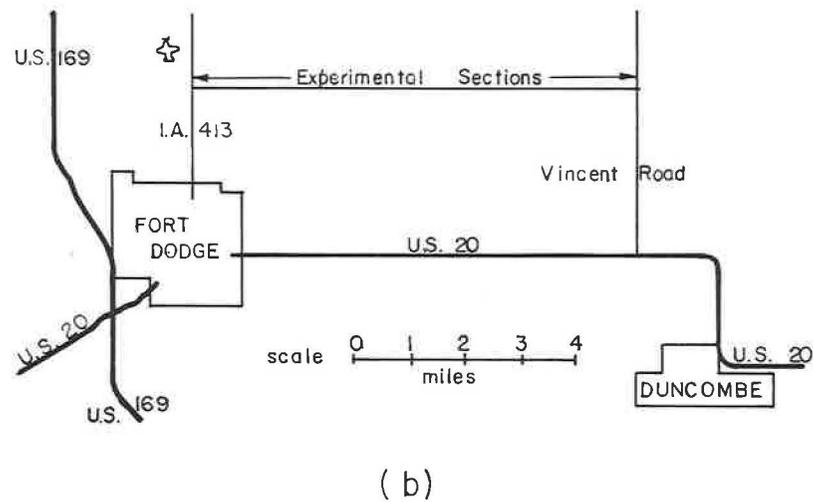
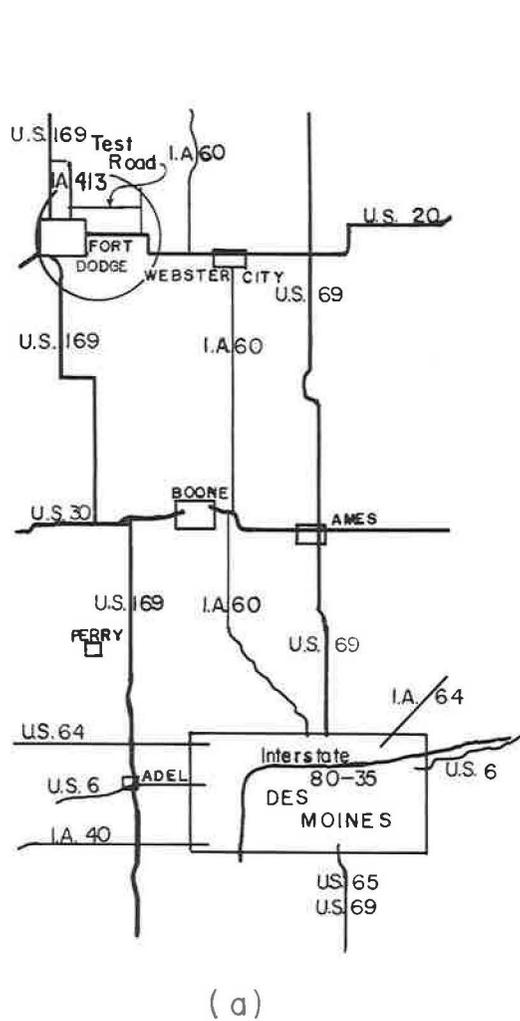


Figure 1. Site of Webster County, Iowa, experimental stabilization project in relation to (a) Des Moines and (b) Fort Dodge.

TABLE 1
 PROPERTIES OF BASE AND SUBGRADE SOILS AT FIELD SAMPLE AND TEST LOCATIONS^a

Sample Location ^b	L. L. (%)	P. I.	Moisture Content (%)	Eng. Classification	CEC (meq/100g)	Organic Matter Content (%)	Carbonate Content (%)	pH
(a) Base Materials								
1	33.1	13.9	10.7	A-6(5)	-- ^d	--	--	--
2	37.6	18.0	11.7	A-6(8)	--	--	--	--
3	38.0	15.7	14.4	A-6(7)	--	--	--	--
5	31.1	7.6	12.3	A-4(3)	26.5	2.3	4.5	7.2
6	38.6	14.5	12.7	A-6(6)	--	--	--	--
8	36.9	14.8	11.5	A-6(2)	17.0	1.3	8.7	7.1
9	41.4	18.3	16.6	A-7-6(8)	27.0	2.1	9.7	8.2
11	33.3	10.5	8.0	A-6(2)	--	--	--	--
13	38.4	16.5	13.6	A-6(8)	--	--	--	--
14	35.2	13.1	8.8	A-6(1)	16.1	1.6	11.1	7.8
15	26.6	8.0	8.5	A-2-4(0)	--	--	--	--
16	39.1	19.4	13.1	A-6(7)	--	--	--	--
17	40.4	16.6	14.1	A-4(7)	--	--	--	--
19	40.1	20.1	9.4	A-4(4)	19.9	0.9	16.7	--
20	24.2	7.4	6.0	A-2-4(0)	10.3	0.8	15.8	8.0
22	27.2	9.8	4.6	A-2-4(0)	13.2	0.7	12.3	8.0
23	31.5	12.2	7.2	A-6(2)	--	--	--	--
25	38.5	19.9	8.6	A-6(10)	22.3	1.7	2.6	7.6
(b) Subgrade Materials								
1	36.5	14.7	19.0	A-6(7)	--	--	--	--
2	41.5	17.2	22.9	A-7-6(10)	26.8	2.4	2.6	7.5
3	40.3	17.4	22.0	A-6(10)	26.5	2.3	2.4	6.9
5	37.4	14.5	14.9	A-6(4)	--	--	--	--
6	46.2	19.7	24.7	A-7-6(13)	--	--	--	--
8	37.6	13.9	18.8	A-6(9)	--	--	--	--
9	45.4	21.0	21.8	A-6(11)	26.4	3.1	8.3	7.4
11	45.8	18.1	25.1	A-7-6(12)	--	--	--	--
13	45.1	24.1	24.6	A-7-6(16)	31.2	3.4	4.2	7.6
14	42.5	17.3	24.2	A-7-6(10)	29.1	2.8	2.9	7.5
15	29.4	12.7	11.5	A-6(4)	--	--	--	--
16	47.2	22.8	22.2	A-7-6(14)	--	--	--	--
17	37.4	11.1	19.7	A-6(2)	19.5	1.9	3.4	7.4
19	47.5	26.3	18.1	A-7-6(16)	29.4	2.3	6.3	7.8
20	40.0	15.4	21.1	A-6(9)	--	--	--	--
22	45.6	17.4	25.9	A-7-6(11)	--	--	--	--
23	43.5	19.5	20.0	A-7-6(12)	--	--	--	--
25	49.1	21.8	20.5	A-7-6(12)	--	--	--	--

^aSamples taken prior to construction in August 1959.

^bSee Figure 2a for relation of stabilized section and sample location number.

^cOf dry soil weight.

^dNot determined.

TABLE 2
 REPRESENTATIVE LABORATORY ANALYSES OF
 FLY ASH SAMPLES^a

Property	Percent	
	Waterloo	Minneapolis
SiO ₂	37.70	44.52
Al ₂ O ₃	16.88	21.08
Fe ₂ O ₃	19.88	17.92
MgO	0.87	0.58
SO ₃	0.41	1.55
Loss of ignition	17.29	9.05
Passing No. 325 sieve ^b	69.2	76.3

^aSamples removed from trucks dumped and spread at site.

^bSamples sieved through a No. 20 U. S. standard sieve before test; amount given in percent by weight.

in classification from A-6(2) to A-7-6(16), with an average of 66.7 percent material passing the No. 200 U. S. standard sieve (Table 1). The average moisture content of the full depth at time of sampling in August 1959 was 20.9 percent by dry soil weight. The average organic matter and carbonate contents were 2.6 and 4.3 percent, respectively, with an accompanying alkaline soil pH averaging 7.4.

Stabilizing Agents

Figure 2 shows the various types of stabilization treatments. Type I portland cement was obtained by the contractor from two sources: Dewey Cement Co. of Linwood, Iowa, and Lehigh Cement Co. of Mason City, Iowa. These cements are hereafter referred to as Dewey and Lehigh, respectively.

Monohydrate dolomitic lime was selected for use in the project as previous investigations had shown it to be superior to other limes for soil-lime stabilization in Iowa, where the dominant clay mineral is montmorillonite (2, 9). The lime was obtained by the contractor from two sources: Dewey Cement Co. and Rockwell Lime Co. of Rockwell, Wis. These limes are hereafter referred to as Linwood and Rockwell, respectively. Samples of the bulk lime were removed from railroad cars in which they had been shipped and were tested by the Iowa State Highway Commission Materials Department. A representative analysis of the Rockwell lime indicated 0 percent retained on No. 30 sieve, 5.0 percent retained on No. 200 sieve and a magnesium oxide content of 33.3 percent. The lime was tested in accordance with ASTM designation: C 207-49.

The fly ash used in the experimental sections was obtained from the Iowa Public Service Co. in Waterloo and the Northern States Power Co. in Minneapolis, Minn. Analyses of fly ash samples (Table 2) obtained at the job site were made by the Robert W. Hunt Co. of Chicago, Ill.

CONSTRUCTION

All supervision and inspection of construction was under the direct control of the Webster County Engineer and/or his official representatives. Personnel of the Soil Research Laboratory, Iowa Engineering Experiment Station, collected and compiled the data pertinent to the objectives of the research, and acted as consultants to the Research Department, Iowa State Highway Commission, and the Webster County Engineer.

The base course construction consisted of scarification of the existing surface to the required depth(s), followed by pulverization of the scarified material with multipass mixers to produce the desired gradation as set forth in Iowa Highway Commission Standard Specifications, i. e., 80 percent of the soil material (on a total soil-moisture weight basis) to pass the No. 4 U. S. standard sieve before addition of any stabilizing agent. It was soon noted that this specification was economically impossible without lime pretreatment, due to the high amount of resistant clay aggregates of approximately $\frac{1}{2}$ - to $1\frac{1}{2}$ -in. size. The specification was therefore reduced to 65 percent passing the No. 4 sieve; this was still difficult to achieve without the assistance of the lime pretreatment.

Following pulverization, each stabilizing agent was spread by a conventional truck-pulled cement spreader and thoroughly mixed dry. Water was then added through the spray bar of the multipass mixer to bring the mix to optimum moisture content for maximum density, and the materials were thoroughly mixed again. Initial compaction was accomplished by a tapered tamping foot roller with a contact pressure of about 300 psi. Final compaction was with a segmented grid roller filled with lead dust with contact pressure of about 465 psi. Each compactor worked in conjunction with a spiked tooth harrow for removal of tractor tread and compactor marks. A vibratory steel drum roller was tried in one section but created 1- to $1\frac{1}{2}$ -in. deep surface cracks spaced a few inches apart. These were apparently due to the fineness of the soil material coupled with rolling movement of the material ahead of the drum.

A 4-in. crown was cut by a blade grader. This was followed by light rubber-tired rolling to reseal any surface hair cracking caused by the cut of the blade. Edges were cut on a 2:1 slope, as shown on the cross-section in Figure 2b. The surface of the base



a



d



b



e



c



f

Figure 3. Construction: (a) general construction sequence following pulverization of in-place soil; (b) results of one pass of multipass mixer over lime-pretreated soil; (c) appearance following third pass of multipass mixer over same location; (d) cutting of 4-in. crown and 2:1 sloped edges; (e) brooming to remove loose material from surface of finished base; and (f) spreading limestone chips for double inverted penetration type surfacing, using a medium cutback asphalt as binder.

was then broomed for removal of any loose material and sprayed with a light coat of water to assist in prevention of penetration of the bituminous prime coat. Each section was primed with MC-0 cutback asphalt and closed to traffic for at least 7 days to allow an initial set of the stabilized material. Surfacing was of a double inverted penetration type (double-chip coat) using MC-4 cutback asphalt and $\frac{3}{8}$ -in. limestone chips. Figure 3 shows some scenes of the construction.

Lime pretreatment of various sections noted in Figure 2 created an additional construction sequence. Following scarification of the roadbed, lime was applied and lightly mixed, bladed and shaped to crown, rubber-tire rolled to seal the surface lightly, and allowed to react for a minimum of 48 to 72 hr. In some of the drier sections a light application of water was mixed into the material before sealing of the surface. Following the pretreatment period, the material was again scarified and mixed with the multipass mixer until gradation requirements were met. Construction then proceeded as outlined previously.

Also following scarification and light blading of the roadbed, moistened fly ash, where used, was dumped from uncovered trucks and spread with a blade grader to a uniform loose thickness and width. Multipass mixers were then used to obtain an intimate mix of fly ash and soil before addition of lime.

Construction was started in August 1959; only $4\frac{1}{2}$ mi were completed before cold weather because of unusually wet weather conditions in September. Construction was halted in Oct. 30, 1959. Completed that year were: (a) base courses in the west $2\frac{1}{2}$ mi and east 2 mi; (b) lime pretreatment in the 6-in. thick section of lime and cement; (c) double-chip surfacing in the west $2\frac{1}{2}$ mi of the project and the east $\frac{1}{2}$ mi of the 6-in. lime section; (d) single-chip surfacing of the 8-in. lime section and the west $\frac{1}{2}$ mi of the 6-in. lime section; and (e) spreading and mixing all fly ash with the soil in the 2 mi of lime-fly ash section.

During the winter of 1959-60, Iowa had unusually severe weather. The $3\frac{1}{2}$ mi containing only the lime pretreatment and the fly ash were at times nearly impassable, even to tractors. The Webster County maintenance crews spread crushed pit-run gravel at a rate of approximately 1,000 tons/mi over the length of the previously lime-pretreated section. This provided only a light crust as a surface but assisted farmers in the area in getting to and from their farms, fields and livestock.

Construction finally resumed in the first week of August 1960 and the project was completed about mid-October. Procedures of construction were the same as used in 1959.

EVALUATION OF CONSTRUCTION CONTROL

Gradation

Originally, the project was to consist of two 1-mi long sections of 6- and 8-in. compacted thicknesses for each type of stabilizing agent(s) used. Due to the gradation difficulties during the first part of the construction, only the 8-in. section of cement-treated material was constructed, and the two sections of lime pretreatment followed by cement treatment were increased in length by $\frac{1}{2}$ -mi each. This decision was based on several factors.

1. Gradations before adding the stabilizing agent continuously ran well under the 80 percent passing the No. 4 U. S. standard sieve no matter how much the contractor aerated and pulverized with the multipass mixers. For example, one 1,200-ft section increased from 63 to 67 percent passing the No. 4 sieve during one full day of mixing and aeration. After 5 to 6 hr of mixing the following day, gradation was still at 67 percent passing the No. 4, yet the clods of material when examined were quite friable though very moist. It was also noted that each of the clods had a rounded or balled appearance after going through the mixer. The combination of moisture, high organic matter content of the soil, and the balling action of the mixer probably prevented the reduction of moisture content and decrease of surface tensile effects between individual particles.

2. A pulverization procedure of first surface air drying, then mixing by machine

and by spiked tooth harrow provided no additional percentages of gradation passing the No. 4 sieve. Appearance of the material remained similar to that noted previously.

3. Four percent cement was added as a conditioner to an 800-ft length of previously scarified, lightly mixed and pulverized base soil. Following dry mixing, the mix was brought to approximate optimum moisture content, mixed, compacted, shaped to approximate crown, and lightly rubber-tire rolled. After 7 days the material was re-scarified and pulverized with some degree of success; the maximum pulverization obtainable was about 72 percent of the material passing the No. 4 sieve. This was still not considered satisfactory.

Pulverization difficulties were further shown in later construction. A short section of base already containing fly ash would not pulverize to more than 24.1 percent passing the No. 4 sieve at a moisture content of 22.3 percent. After addition of the 4 percent lime with just over 1 hr of total mixing time, gradation increased to 67.2 percent passing the No. 4 sieve at a moisture content of 16.2 percent. A 500-ft section of base containing fly ash had 17.0 percent passing the No. 4 sieve at a moisture content of 20.6 percent following 4 hr of pulverization. The addition of 2 percent lime changed the gradation to 62.6 percent passing at 19.0 percent moisture after 2 hr additional mixing. An additional 2 percent lime further changed the gradation to 71.0 percent passing at 17.6 percent moisture. In another example, a large frost boil developed at the extreme east end of the project during the winter of 1959-60. The subgrade was removed to a depth of 2 ft and was replaced with soil with 8 percent lime added as a frost treatment. No gradation tests were possible before addition of the lime, though the soil material contained only 20.5 percent moisture. Just before recompaction, 56.1 percent of the mix passed the No. 4 sieve. No moisture content was taken at this point, however.

Thus, gradation requirements of 80 percent of the total material passing the No. 4 U. S. standard sieve were impossible to meet. Help was provided through the use of lime and cement. The average gradation passing the No. 4 sieve of all lime-pretreated sections constructed in 1959 was 73 percent following the 48- to 72-hr pretreat period. The maximum gradation obtainable without pretreatment appeared to be 60 to 70 percent passing the No. 4 sieve and also appeared dependent on the moisture content of the material.

Quantity of Admixture

With the exception of the fly ash, all stabilization agents were spread by a conventional truck-pulled cement spreader. Some difficulty was encountered in controlling the rate of feed of the spreader. Table 3 presents a comparison of the specified and spread quantities of stabilizing agents at each sample location. The calculated spread quantities ranged from 97 percent to 112.5 percent of the specified plan quantities.

Density

The effect of length of time of mixing on the moisture-density relations of cement-treated soils has long been recognized (10). Also, the effect of laboratory vs field determination of moisture-density relations has been noted by most highway materials personnel. The Webster County, Iowa, project is of interest for comparing the density effects with several stabilizing agents. Table 4 gives materials in the various mixes.

Table 5 shows the effect of mixing conditions on optimum moisture-maximum density relations and 7-day moist-cure strength of samples molded in accordance with ASTM Method D 698-58T (basically in accordance with Method C). Spot checks indicated an occasional particle larger than $\frac{3}{4}$ in., which was then removed from the mix. Samples were obtained as follows:

1. Laboratory mixed and molded—Representative samples of the various field materials were proportioned, mixed and molded in the Soil Research Laboratory, Iowa Engineering Experiment Station.
2. Field dry mixed, laboratory wet mixed, and molded—Immediately following field dry mixing and before application of water, representative samples were removed from

TABLE 3
COMPARISON OF SPECIFIED AND CALCULATED
SPREAD QUANTITIES OF STABILIZING AGENTS
AT EACH SAMPLE LOCATION

Sample Location No.	Specified % dry soil wt.	Spread % dry soil wt. ^a
1	8% portland cement	8.8
2		9.2
3		8.0
4	3% lime pretreatment + 5% portland cement	3.1 + 5.2
5		3.2 + 5.3
6		3.2 + 5.3
7		3.1 + 5.7
8	3.1 + 5.7	
9	3% lime pretreatment + 5% portland cement	3.2 + 5.5
10		3.1 + 5.6
11		3.1 + 5.2
12		3.1 + 5.4
13	3.1 + 5.7	
14	20% fly ash + 4% lime	-- ^b + 4.1
15		-- + 4.0
16		-- + 3.9
17	20% fly ash + 4% lime	-- + 4.0
18		-- + 4.0
19		-- + 3.9
20		3.1 + 5.4
21	3% lime pretreatment + 5% lime	2.9 + 5.5
22		2.9 + 5.4
23	3% lime pretreatment + 5% lime	3.2 + 5.8
24		3.1 + 5.2
25		3.1 + 5.9

^aCalculated from control densities as determined by Webster County inspectors, spread tonnage of stabilizing agent available to author, and standard plan cross-section of finished area.

^bNot determined.

TABLE 4
DESIGNATIONS OF VARIOUS LABORATORY- AND
FIELD-STABILIZED SOIL MIXTURES^a

Mix	Field In-Place Materials	Additives
A	In-place gravel-treated surface of road embankment.	8% Dewey Type I portland cement.
B	Same as A.	3% Linwood lime pretreatment plus 5% Dewey Type I portland cement.
C	Same as A but containing 3% Linwood lime as pretreatment, field mixed.	5% Lehigh Type I portland cement.
D	Same as A but containing 20% Waterloo fly ash, field mixed.	4% Rockwell lime.
E	Same as A but containing 20% Minneapolis fly ash, field mixed.	4% Rockwell lime.
F	Same as A.	3% Linwood lime pretreatment + 5% Linwood lime.

^aAll percentages by weight.

the construction site and returned to a field laboratory for wet mixing and molding.

3. Field dry and wet mixed, laboratory molded—Immediately following field wet mixing and before field compaction, samples were removed from the construction site and molded at the field compaction moisture content.

For each laboratory and field condition of mixing and molding all samples were extruded from the Proctor molds, securely wrapped, and returned to and stored in a humidity room of the Soil Research Laboratory at a constant temperature of 70 F and approximately 100 percent RH. After 7-day moist curing they were tested for unconfined compressive strength. For conditions 1 and 2, combined plots of moisture vs density and moisture (at time of molding) vs unconfined compressive strength were made. The strength results presented in Table 5 are those at optimum moisture content and maximum density obtained from the graphs.

In general, field dry mixing caused an increase in optimum moisture content and a decrease in maximum dry density from that obtained under laboratory mixing conditions. Though field dry and wet mixing showed little change in moisture content from that noted for condition 2, the dry density was decreased even further from that obtained by the field dry mixing only.

Also of interest is the general decrease in unconfined compressive strength due to the variation of mixing conditions on density; the field dry- and wet-mixed specimens had only about 50 to 60 percent of the strength of the laboratory-mixed specimens.

Table 6 presents a comparison of standard maximum density values obtained by the research personnel and those determined and used by the county inspectors at 12 sample locations constructed in 1960. Also presented are depth of base, moisture content and in-place density obtained by county inspection, and the in-place density expressed as a percentage of the values determined by county inspectors and research personnel. The values determined by research personnel were obtained on samples taken from the con-

struction site at each sample location immediately following dry mixing and before application of water. The values determined by county inspection personnel were similarly obtained or were obtained from composite representative samples of the total test sections as noted in Figure 2 and seen in the continuity of the specified densities in Table 6.

TABLE 5

EFFECT OF MIXING CONDITIONS ON OPTIMUM MOISTURE-MAXIMUM DENSITY RELATIONS AND 7-DAY MOIST-CURE UNCONFINED COMPRESSIVE STRENGTH OF MIXES USED IN FIELD TEST

Mix	Lab. Mixed and Molded ^a			Field Dry Mixed, Lab. Wet Mixed and Molded ^b			Field Dry and Wet Mixed, Lab. Molded ^b		
	Opt. Moisture Content (%)	Dry Density (pcf)	7-Day Strength (psi)	Opt. Moisture Content (%)	Dry Density (pcf)	7-Day Strength (psi)	Moisture Content (%)	Dry Density (pcf)	7-Day Strength (psi)
A	14.6	112.9	409	--	--	--	--	--	--
B	15.2	112.2	390	--	--	--	18.6	97.2	156
C	13.8	114.7	485	16.5	108.5	257	16.3	101.9	160
D	15.0	109.8	245	16.0	106.7	175	15.8	101.9	136
E	14.5	112.6	370	17.7	107.4	225	16.2	106.3	238
F	16.2	108.1	230	20.4	99.1	138	18.7	99.8	105

^aData are averages of minimum of two samples of each mix.

^bData are averages of two to six samples of each mix.

TABLE 6

COMPARISON OF MOISTURE-DENSITY DETERMINATIONS OF 12 SAMPLE LOCATIONS DURING CONSTRUCTION, 1960

Sample Location No.	Opt. Moist. - Dens. Relations, Res. Team			In-Place Dens., County Inspectors				In-Place Dens., County Inspectors, as Percent of Standard Used by:	
	Opt. Moisture Content (%)	Dry Density (pcf)	Specified Dry Dens., County Inspectors (pcf) ^a	Depth of Base (in.)		Moisture Content (%)	In-Place Density (pcf) ^b	County Inspectors	Res. Team
				Const.	Plan				
9	--	--	102.4	6 $\frac{3}{4}$	6	19.3	93.3	91.1	--
10	--	--	102.4	7	6	14.7	98.5	96.2	--
11	14.5	112.4	102.4	5 $\frac{3}{4}$	6	18.5	101.6	99.2	90.4
12	16.6	106.5	102.4	6 $\frac{3}{4}$	6	18.5	100.3	97.9	94.2
13	18.5	106.7	102.4	6	6	15.5	103.9	101.4	97.4
14	17.0	106.4	102.4	8	8	16.9	93.0	90.8	87.4
15	16.0	109.0	104.0	7 $\frac{1}{4}$	8	16.8	104.3	101.3	95.7
16	16.8	99.9	107.0	6 $\frac{3}{4}$	8	14.9	99.3	92.8	99.4
17	14.2	111.3	114.4	6	6	19.9	92.8 ^c	81.1 ^c	83.4 ^c
18	9.6	124.2	114.4	5 $\frac{1}{2}$	6	13.4	107.3	93.8	86.4
19	17.7	107.4	114.4	5 $\frac{1}{2}$	6	18.2	97.5	85.2	90.8
20	18.2	102.5	101.0	8	8	16.1	104.4	103.4	101.9

^aOptimum moisture contents for maximum dry densities used by county inspectors were not obtained by author and are not included.

^bDetermined by oil method, AASHTO Designation: T 147-54.

^cThis section was not accepted on basis of density by county inspectors and was reconstructed.

Comparison of the density values determined by the research team and those specified by the inspectors in Table 6 indicates a degree of variation of density similar to that noted from Table 5. Of significance, however, is the variation in maximum density determined at each sample location by the research personnel. A comparison of these values with the engineering classification of the base course soils of Table 1 indicates a high degree of correlation, i. e., the higher the density the better the quality of the base course soil, and vice-versa. The in-place density follows the same general pattern of higher density for better quality soil materials.

The wide variation of in-place density, expressed as a percentage of the standard used for construction control, is also felt to be related, at least in part, to variation of classification of base course soils. The in-place density expressed as a percentage of standard used for construction control by the county inspectors varied by 18.2 percent and that expressed as a percentage of the density determined by the research team varied by 15.5 percent, exclusive of sample location 17 in each case.

Thus, the variation of density with respect to engineering classification of base soil materials points up the necessity for determination of density of stabilized mixes at every suspected change of soil type. This would also produce a more uniform control of construction compaction as regards normal specifications requiring a contractor to compact a base to a minimum percentage of a standard density.

TABLE 7
RELATION OF MOISTURE-DENSITY DATA OF CORE
AND BLOCK SPECIMENS REMOVED FROM BASE
TO CONDITION 3 SPECIMENS

Mix ^a	Core and Block Samples ^b		Condition 3 Samples ^c	
	Moisture Content (%)	Dry Density (pcf)	Moisture Content (%)	Dry Density (pcf)
A	19.0	101.1	—	—
B	19.6	101.0	18.6	97.2
D	15.5	103.0	15.8	101.9
F	19.8	104.8	18.7	99.8

^aTest sections of mixes A, B and F constructed in 1959; test section of mix D constructed in 1960.

^bData presented is average of 5 to 10 core or block specimens from each mix (with the exception of mix D which is the average of only 2 specimens for sample location 17) removed from the base in October 1960.

^cData presented is average of 2 to 6 samples for each mix and taken from table 5.

The thickness of constructed base noted in Table 6 was measured during the conduct of in-place density determinations and indicates a fairly high degree of performance in construction and inspection to attain the specified compacted thickness of base section.

In October 1960, core and block specimens were removed from the base course sections which were a minimum of 1 mo old from time of construction. In addition to other tests, moisture contents and densities were determined on all specimens, the average values of which are given in Table 7. As an approximate means of comparison to the mixes as originally constructed, Table 7 also gives the moisture contents and densities of the lab-molded specimens of field dry- and wet-mixed samples (condition 3) previously shown in Table 5. It will be noted that the moisture contents and densities of the core and block samples are slightly higher than the condition 3 specimens. The reader might be led to the speculation that the high variations of in-place densities as a percent of standard, presented in Table 6, are not as critical as a first glance might indicate.

In-Place Density by Proctor Penetrometer

The Proctor penetrometer and test procedure were developed for control of field compaction of fine-grained soils in embankments and other earth structures. Curves are normally established for the moisture-density and moisture-penetration resistance relationships on the same graph. Thus, the penetrometer and the curves can be used to control moisture content and/or density.

ASTM Method D 1558-63 sets forth the standard procedure of penetrometer testing for moisture-density relationships. In the test a needle penetrates the soil at a rate of 0.5 in./sec for a distance of 3 in., and the maximum pressure exerted is read off a scale on the stem of the penetrometer handle.

Penetration resistance determinations cannot be obtained on very dry or very granular soils as even the smallest needle size ($\frac{1}{20}$ sq. in. cross-section) will not penetrate. However, it was felt that a revised penetrometer method held some promise

for determination of in-place density with stabilized materials, and the Webster County experimental project presented an opportunity to analyze the procedure.

In the standard procedure, the total length of time of test is 6 sec. In the procedure adopted, if the needle did not penetrate at a rate of 0.5 in./sec within the first 3 sec, it would be pushed to its maximum pressure reading (110 lb on the penetrometer used), held for an additional 3 sec, and the depth of penetration measured and recorded. A Penetration Resistance Factor (PRF, psi/in. or pci as assumed herein) was then calculated by the formula

$$PRF = \frac{(L)(S)}{D} \quad (1)$$

where L is load in pound, S is reciprocal of needle size in square inches, and D is depth of penetration in inches. Graphical representation of PRF for various needle cross-sections could be produced for rapid determination of PRF knowing these values.

Penetration tests were thus run on three soil-additive mixture test conditions for the sections constructed in 1960:

TABLE 8
RELATION OF MOISTURE CONTENTS, DENSITIES AND PENETRATION RESISTANCE FACTORS AT ALL SAMPLE LOCATIONS CONSTRUCTED IN 1960

Sample Location	Test Conditions ^a	Moisture Content (%)	Dry Density (pcf)	Standard Proctor Penetrometer			PRF (pci)
				Needle size (sq in.)	Avg. Penetration (in.)	Avg. Pressure (lb)	
9	A	13.8	114.7	1/20	--	--	1,650
	C	19.3	93.3	1/20	2.60	97	750
10	A	13.8	114.7	1/20	--	--	1,650
	B	13.3	106.3	1/20	0.37	110	6,200
11	C	14.7	98.5	1/20	2.57	97	760
	A	13.8	114.7	1/20	--	--	1,650
12	B	16.3	101.6	1/20	1.25	110	1,760
	C	18.5	101.6	1/20	2.73	107	790
13	A	13.8	114.7	1/20	--	--	1,650
	B	15.3	101.9	1/20	0.93	110	2,400
14	C	18.5	100.3	1/20	1.70	106	1,260
	A	13.8	114.7	1/20	--	--	1,650
15	B	18.2	98.7	1/20	1.14	110	1,940
	C	15.5	103.9	1/20	1.77	105	1,190
16	A	15.0	109.8	1/20	--	--	1,100
	B	14.0	102.3	1/20	0.29	110	7,500
17	C	16.9	93.0	1/20	1.13	110	1,960
	A	15.0	109.8	1/20	--	--	1,100
18	B	14.8	105.3	1/20	0.53	110	4,200
	C	16.8	104.3	1/20	1.23	110	1,790
19	A	15.0	109.8	1/20	--	--	1,100
	B	17.2	97.8	1/20	1.42	110	1,550
20	C	14.9	99.3	1/20	1.20	110	1,830
	A	15.0	109.8	1/20	--	--	1,100
21	B	16.7	99.3	1/20	0.43	110	5,000
	C	16.9	92.8	1/20	2.78	98	710
22	A	14.5	112.6	1/20	--	--	1,250
	B	9.4	123.1	1/20	0.43	110	5,000
23	C	13.4	107.3	1/20	1.82	109	1,200
	A	14.5	112.6	1/20	--	--	1,250
24	B	16.2	107.5	1/20	1.00	110	2,250
	C	18.2	97.5	1/20	2.63	99	760

^aA = Laboratory mixed, molded and tested on representative sample of full road test section;

PRF determined from moisture-density-PRF curves.

^bB = Field dry and wet mixed, laboratory molded and tested.

^cC = Road base tested; indicated moisture contents and in-place densities determined by Webster County inspectors at or near indicated sample locations.

- A. Standard density specimens produced under controlled laboratory conditions using representative lab samples of full test section soils and additives;
 B. Standard density specimens on field mixes removed from each sample location during construction immediately following wet mixing and before field compaction; and
 C. The finished road base immediately following rubber tire compaction.

A minimum of 5 tests per specimen were run for conditions A and B and 15 for condition C. Three penetration tests were performed 1 ft from each edge, three at each quarter point of cross-section and three at the centerline of the base in each sample location.

Moisture-density and moisture-average PRF graphs were drawn for each mix in test condition A. Average PRF values were determined for each specimen of condition B and for each sample location for condition C. All PRF values for each individual test were read from a graph produced for the $\frac{1}{20}$ -sq in. cross-section needle used throughout the tests. Table 8 presents the moisture contents, densities, and penetration results for each test condition at the sample locations of the sections constructed in 1960.

A wide variation of PRF for each test condition may be noted in Table 8. Comparison of moisture contents, densities and PRF values also indicated a widespread variation for each test condition. It was felt that one additional test condition was needed, i. e., PRF values for all specimens laboratory wet-mixed and molded from field dry-mixed samples of each location. This test condition had been considered, but due to extreme time limitations and lack of personnel for all the testing being conducted on the project it was decided to use only the values from condition B for evaluating the field mixing aspect. This decision appears justified on the basis of the moisture-density comparisons of Table 9 for mixes C and D, but does not adequately take into account the variations in field mixing-laboratory molding and laboratory mixing-molded moisture-density variations previously noted.

In general, it was concluded that the results of the testing were of little significance in the development of the practical application of the standard Proctor penetrometer test for use with those stabilized materials.

Variation of Moisture Content with Depth in Base

Continual inspection of the field mixing operations had noted discontinuities with depth of stabilization additives and moisture content. The general appearance was of pockets of unmixed additives and slightly dryer material in the lower depths of the loose mixes. The contractor was required to mix each section further until the loose material had a more uniform appearance and feel before compaction.

Though no check was run on quantity of additives vs depth, moisture contents vs depth in base were performed at sample location 15 immediately following compaction. Table 10 presents the results. It will be noted that there was an average variation between the top and bottom of the base of almost 5 percent moisture.

Results are rather conclusive that the wet mixing had been inadequate at this single sample location. Although it is not known what moisture content conditions existed at other sample locations, it may be speculated that moisture variations with depth also occurred elsewhere. It may also be speculated that the multipass mixers used for this project were not completely capable of uniform mixing procedures. This latter statement is also based on visual observations of the difficulties encountered in reaching full depth of loose base thickness.

TABLE 9
COMPARISON OF AVERAGE MOISTURE-DENSITY DETERMINATIONS OF IN-PLACE BASE MATERIAL AND SPECIMENS MOLDED FROM FIELD MIXES IMMEDIATELY BEFORE FIELD COMPACTION

Mix	In-Place Specimens		Condition 3 Specimens ^a	
	Moisture Content (%)	Dry Density (pcf)	Moisture Content (%)	Dry Density (pcf)
C	17.3 ^b	99.5 ^b	16.3	101.9
D	15.8 ^c	101.0 ^c	15.8	101.9

^aFrom Table 5.

^bIncludes sample location 9, 10, 11, 13 from Table 6.

^cIncludes sample location 14, 15, 16, 18 from Table 6.

TABLE 10
MOISTURE CONTENT VS DEPTH IN BASE IMMEDIATELY FOLLOWING CONSTRUCTION AT SAMPLE LOCATION 15

Depth Sampled (in.)	Avg. Moisture Content 4 Ft Right of Center-line (%)	Avg. Moisture Content 4 Ft Left of Center-line (%)	Avg. Moisture Content from Both Holes (%)
0-1	15.7	17.1	16.4
1-2	15.8	15.8	15.8
2-4	16.0	15.2	15.6
4-6	14.0	13.1	13.5
6-8	13.0	10.2	11.6

Adequacy of Length of Lime Pretreatment Time

As previously indicated, all sections pretreated with lime were allowed to react for a minimum of 48 to 72 hr before addition of stabilizing agent. As a means of checking the adequacy of this specification, sieve analyses, liquid limits (L. L.) and plasticity indexes (P. I.) were run on samples of lime-pretreated soil following 0 (untreated soil), 1, 6, 16, 24, 48 and 72 hr of curing. The soil used was a representative sample from full depth of base soils taken at several locations over the 6-in. section of lime pretreatment plus cement treatment immediately following field scarification and before addition of lime. With the exception of several stones, which were discarded, all the sample passed the 1-in. screen as removed from the field site. Average moisture content of the sample was 17.1 percent. The following laboratory test procedure was used in an attempt to simulate field pulverization and lime pretreatment construction conditions:

1. A 7,000-g sample was quartered from the total sample.
2. A 1,000-g portion was quartered out as the untreated sample and 3 percent lime, by dry soil weight, was then added to the remaining portion.
3. Both the untreated and treated samples were machine mixed for 10 to 5 min, respectively. This constituted zero time.
4. Sieve analysis of the untreated sample was immediately run using the 1-, $\frac{3}{4}$ -, and $\frac{1}{2}$ -in. and Nos. 4, 10, and 40 sieves; the treated sample was divided into six equal portions. Each treated portion was sealed and placed in the humidity room until time of testing.
5. At the end of each treatment period, the samples were machine mixed for an additional 5 min before sieve analysis as in step 4.
6. L. L. and P. L. were determined for all material passing the No. 40 U. S. standard sieve immediately following sieving.
7. All sieve analyses were expressed as a percentage of the weight of the total sample. All L. L., P. L. and P. I. data were expressed as a percentage of the oven-dry weight of the material.

Results of the testing are shown graphically in Figure 4. After treatment with lime, it was impossible to obtain a P. L. determination since the material would change rapidly from a thread of about $\frac{3}{8}$ -in. diameter to a friable mass of crumbs. In 1 hr, the lime treatment reduced the L. L. of the soil from 40 to 36 percent with little additional change during the period of 1 to 72 hr.

Results of the sieve analysis were somewhat more erratic. The most significant increase in particle size reduction will be noted with the No. 10 sieve. Percentage passing the No. 10 sieve increased from 33.8 percent for the untreated sample to 46.4 percent with 1 hr of lime pretreatment and averaged 43.3 percent for the remainder of the 72-hr period.

Since time, additional construction costs, and established construction sequences did not permit a similar field evaluation of length of pretreatment period, the minimum 48- to 72-hr curing was maintained throughout construction. However, the results of the laboratory study strongly indicated that a field curing period of 24 hr or less might be sufficient for future lime-soil pretreatment construction operations for reduction of plasticity and increase of pulverization-gradation requirements. It was judged that field pretreatment lasting longer than 72 hr would provide little or no additional benefits to pulverization requirements to offset any additional delay of construction. This conclusion was further shown by sections pretreated with lime in 1959 and finished in 1960 which showed an average gradation passing the No. 4 U. S. standard sieve of 74.1 percent—a rather small increase from the 73.0 percent average following the 48- to 72-hr pretreat period shown previously during 1959 construction.

Base Surface Raveling

During construction in 1960, some of the primed base course sections remained without surfacing for as long as several weeks. Though barricades were maintained, traffic still managed to drive on the base surface. Visual examination revealed a sur-

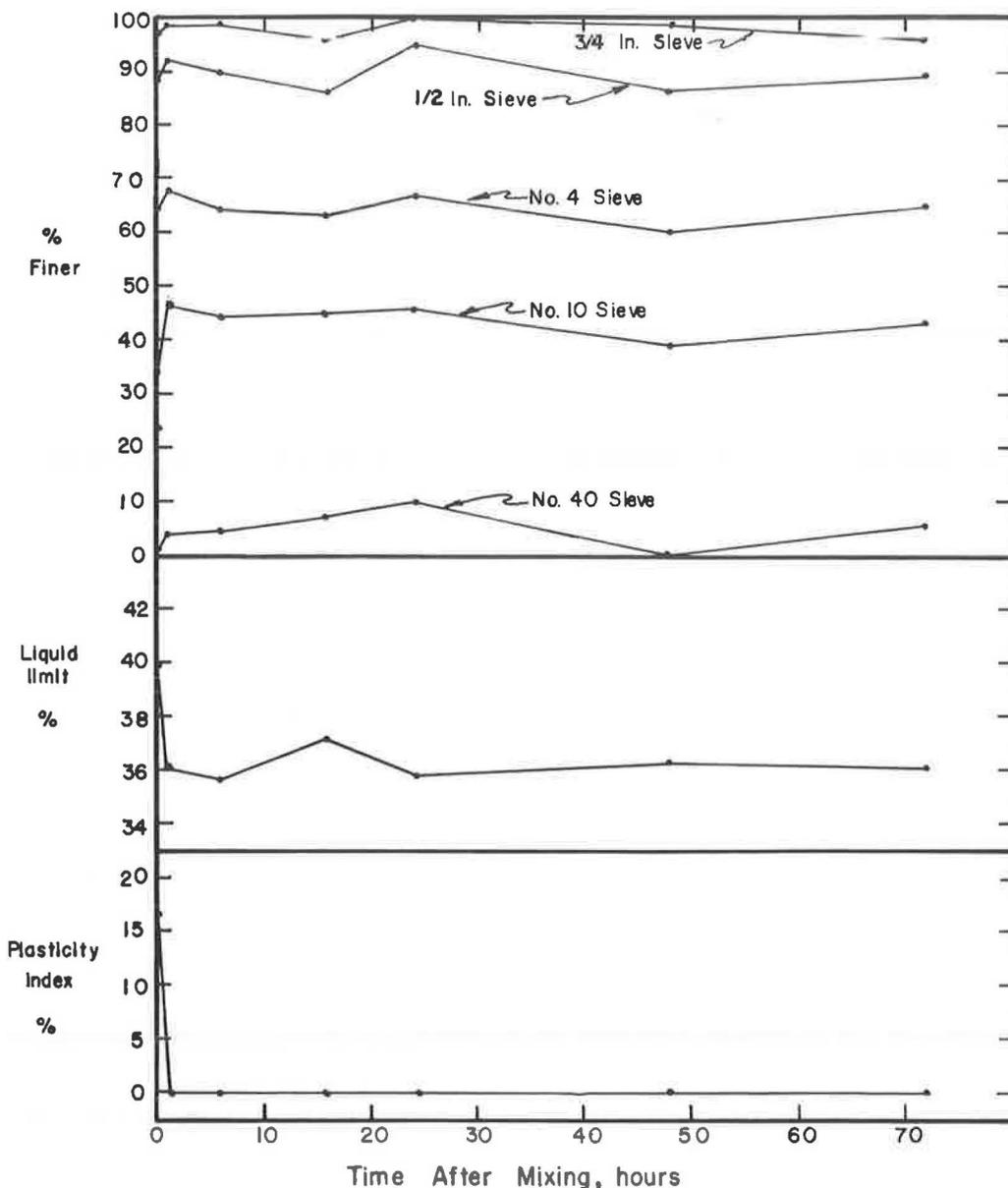


Figure 4. Effect of curing time on sieve analysis, L.L., and P.I. of laboratory-mixed lime-pretreated base course soil.

face checking, slight raveling and, in some cases, softening of the upper $\frac{1}{2}$ to $\frac{3}{4}$ in. of the base sections (Fig. 5a). It also appeared that the MC-0 prime had penetrated the base surface to a similar depth. X-ray diffraction studies of samples removed from the upper 1 in. of base revealed nothing as to the cause of the checking or softening in relation to loss or ineffectiveness of stabilization additive(s).

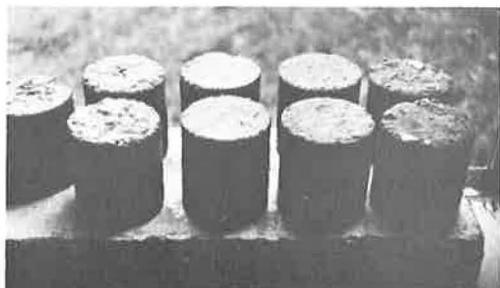
A laboratory study was undertaken in an attempt to duplicate field finishing methods of blading, moistening, priming and curing. Duplicate standard density specimens were molded from laboratory samples of representative materials below, near, and



a

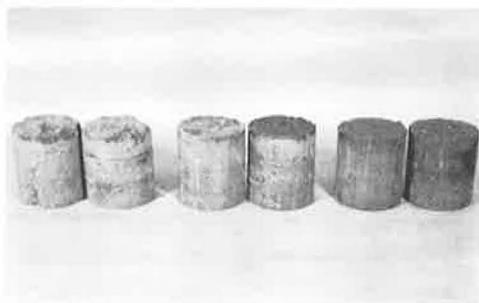


b

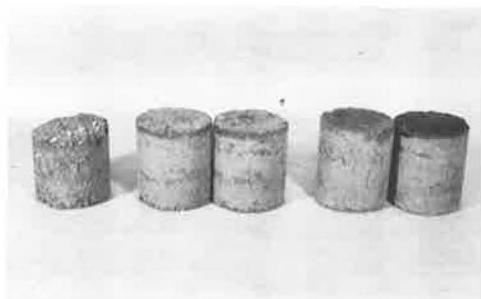


c

Figure 5. Base course raveling study: (a) primed base left without surfacing for several weeks during 1960 construction showed surface checking, raveling and some softening after limited traffic application; (b) specimens molded at varying moisture content increasing from left to right, with near optimum moisture content in two specimens second from right; each specimen shown as it appeared following designated curing period and just before removal from carton for brushing; and (c) specimens immediately following brushing.



a



b

Figure 6. Examples of specimens tested in base course raveling study following brushing: (a) specimens of mix A (specimen second from left cracked on compaction plane during brushing); and (b) specimens of mix F.

above optimum moisture content of each mix. One field-mixed sample included in the testing was removed from near location 13 in the 6-in. lime-pretreated, cement-treated section immediately following dry mixing. The procedure was as follows:

1. Duplicate standard density specimens were molded below, near, and above optimum moisture content for each laboratory mix designation. Duplicate specimens were molded from the field-mixed sample for each point of a standard moisture-density curve.

2. After compaction, the collar of the mold was removed and the specimen was trimmed even with the top of the mold with

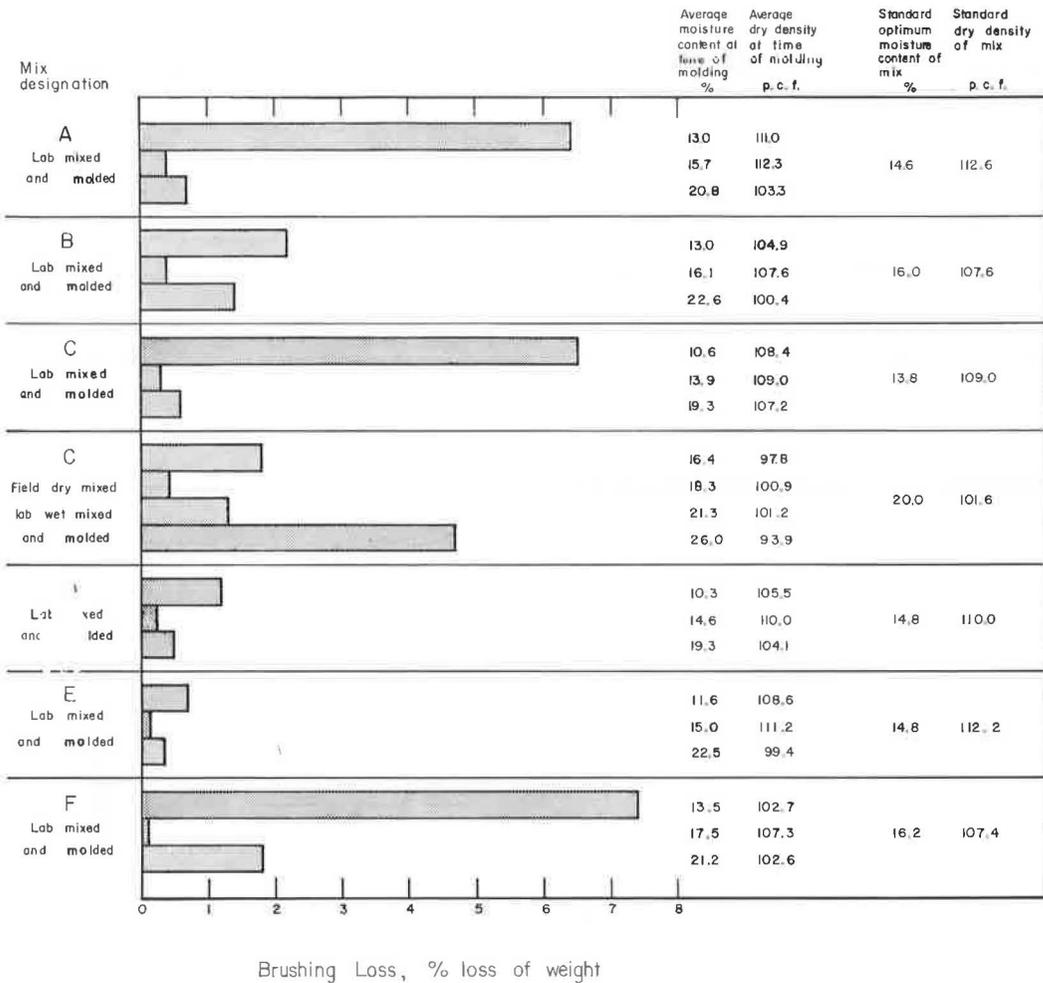


Figure 7. Effect of average moisture content and density of laboratory compacted specimens on percentage loss of weight during brushing.

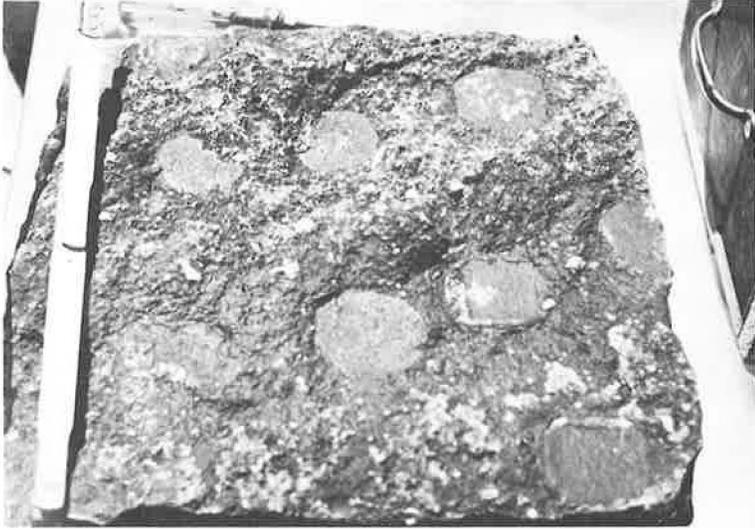
a straight edge applied in only one direction. Any holes developed in the specimen surface by removal of coarse material or surface blading cracks were not compacted or patched as in ASTM Designation: D 698-58T.

3. Each specimen was extruded from the mold by pushing from the bottom up, weighed, and placed in an ice cream carton of diameter about 1-in. larger than that of the specimen and in which paraffin had previously been placed to a depth of about $\frac{1}{2}$ in. in the bottom. Moisture tests were made on soil from the mixing bowl.

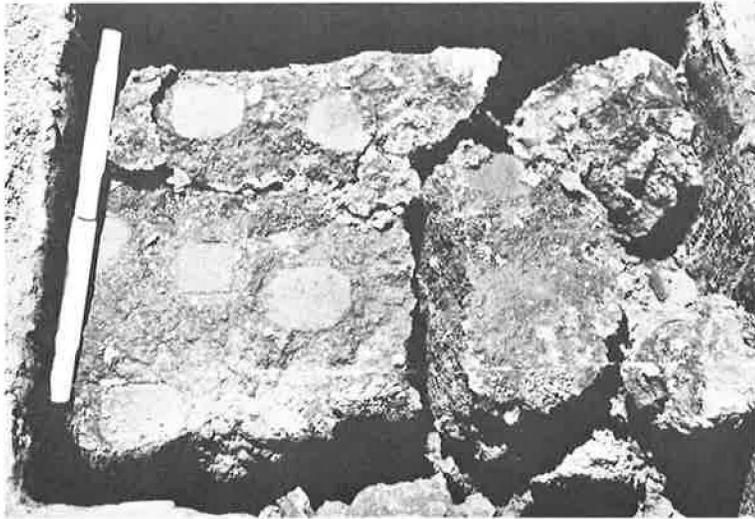
4. Melted paraffin was slowly poured in the space between carton and specimen (so that no air bubbles would collect) until level with the specimen surface. Each container was then allowed to cool to room temperature.

5. Water was added to the specimen surface at a calculated quantity of 2 percent additional moisture for the upper 1 in. as based on standard optimum moisture content at maximum density. This was assumed to be approximately the same quantity used in the field construction before priming.

6. About 15 min later, MC-0 cutback asphalt heated to 50 C was applied at a rate of 0.25 gal/sq yd, i. e., at the same rate and approximately the same time lapse as in the field construction.



a



b

Figure 8. Effect of inadequate removal of compactor footprints during compaction: (a) upper portion of a block sample removed from lime-pretreated, lime-stabilized 6-in. base section; and (b) lower portion of the same block sample. Cracking occurred when lower portion of block dropped back in hole during removal.

7. The specimens were then allowed to cure in a room atmosphere for 7 days.

8. Following curing each specimen was carefully removed, cleaned, weighed, and the top surface was firmly steel wire brushed 100 times while the specimen was constantly turned.

9. After brushing, each specimen was again weighed and the loss due to brushing was computed as a percentage of the weight immediately before brushing.

Examples of the test are shown in Figures 5b, 5c, 6a and 6b. Results of testing are shown in Figure 7.

In all mixes the brushing loss was higher at moisture contents either lower or higher than standard optimum. However, the loss at moisture contents higher than optimum were considerably less than the loss at those lower than optimum, disregarding the loss at 26.0 percent moisture in the field-mixed Mix C.

One hr after application, it was estimated that 75 to 80 percent of the prime had soaked into the samples having less than optimum moisture and maximum density. Surface checking was visible after 72 hr in all specimens on the low side of the moisture-density curve.

After 7-days curing, dry surfaces, checking and cracking were noted on all low moisture-density specimens. An estimated 50 percent or more of the prime remained on the surfaces, and there was some visible but very slight surface checking on all specimens at or near standard optimum moisture-maximum density. An estimated 50 to 100 percent of the prime remained on the surfaces, with the checking ranging from none visible to slight, on all high-moisture specimens. Following brushing, the specimens molded near and above optimum moisture content still had some asphalt adhering tightly to them. It was apparent that much of the brushing loss of these specimens could be attributed to chipping of the surface edges during brushing.

Field observations tended to corroborate the observations made on the tested specimens. Patches of the base surface seemed to soak up the primer and within a short time would appear dry and somewhat checked. Other areas of the base surface after several days from priming were still very tacky with asphalt, though some surface checking was visible.

Evaluation of the tests and laboratory and field observations made indicate that the surface checking and, in some cases, softening of the base is due at least in part to the absorption of the primer by the base material at somewhat less than optimum moisture content and maximum standard density. Traffic on the unsurfaced roadway undoubtedly aggravated the checking and created some raveling.

Removal of Compactor Feet Marks During Compaction

During construction, initial compaction was accomplished with a tamping foot roller and final compaction was done with a segmented grid roller, each working in conjunction with a spiked tooth harrow for removal of tractor tread and roller grid marks. During removal of core samples in 1959, it was noted that an occasional sample would transversely split about mid-height of its longitudinal axis either during drilling or following removal from the core barrel. Visual examination of these broken cores indicated a flat surface about the size of the tamping foot marks, but the cause was not verified.

In early spring 1960, three block samples approximately 1 ft square in cross-section were randomly cut from the full depth of the 6-in. compacted thickness lime section. During removal, one of the blocks split over its full cross-section and about midway of its full depth, exposing the pattern of the tamping foot compactor (Fig. 8). Each tamping foot mark had a shiny, almost glassy, appearance, much in contrast with the broken appearance of the areas in between.

Since 1960 an occasional sample removed by coring has also indicated evidence of breakage along a tamping foot mark; no visible evidence of grid marking has been noted to date. The spiked tooth harrow was apparently inadequate to reach deep enough for removal of tamping feet marks but was adequate in the upper area of the base.

It should not be assumed that the entire base course is in distress due to possible planes of weakness created by nonremoval of tamping feet marks, as there is much

evidence (though not a part of this paper) to show the adequacy of the supporting capacity of all the base sections. However, it may be assumed that there are random limited areas where a heavier and longer toothed drag would have assisted in removal of all compactor foot markings and produced a more durable base course.

SUMMARY AND CONCLUSIONS

The major purpose of the Webster County, Iowa, experimental project was to produce and evaluate a low-cost stabilized soil base course made from an existing gravel-treated surface of road embankment using conventional construction equipment. The road has now given more than 3 years of quality service and has sustained severe freezing and moisture conditions during two of the worst winters in over 30 years in Iowa. This in itself is a tribute to the quality of construction control measures used in the experimental project.

Following is a summary of the major evaluations regarding the construction and the control measures used to produce the stabilized soil base course:

1. Pulverization-gradation requirements of 80 percent of the total material to pass the No. 4 U. S. standard sieve before introduction of stabilizing agent were impossible to meet. Additional laboratory and field studies are needed to adequately define and specify minimum gradation requirements of Iowa soils before introduction of soil stabilizing agents. However, 60 to 70 percent passing the No. 4 sieve appears the maximum gradation obtainable without lime or cement pretreatment for the type of soil material encountered in this project.

2. Introduction of 3 percent lime as a pretreatment agent for the soil materials assisted in a reduction of plasticity, increase of friability and increase in pulverization gradations obtainable. The 48- to 72-hr lime pretreatment period appeared adequate to produce the maximum practical field benefits. Laboratory pretreatment studies indicated maximum benefits in about 1 hr. With further field evaluation it appears possible to reduce the length of pretreatment curing time to 24 hr or less.

3. Specified spread quantities of stabilizing agents were not obtained with the conventional cement spreader used during construction but in general were within a normal 10 percent tolerance.

4. The variation of control and in-place densities with respect to the engineering classification of the various base soils encountered illustrated the need for an optimum moisture-maximum density test of field-mixed stabilized materials at every suspected soil change during construction.

5. The use of a standard Proctor penetrometer as a rapid means of in-place moisture and/or density measurements appeared of little practical value without further testing and development of test procedure.

6. As a means of comparison of uniformity of dry and wet mixing of the materials, trials of additional types of mixing equipment would have been desirable. The multi-pass mixers used on the project appeared to have difficulty in obtaining a uniform mixture of base material and in reaching the loose mixing depth required in the 8-in. compacted thickness sections.

7. For prevention of surface checking and softening due to penetration of the MC-0 primer during the minimum 7-day curing period, the constructed base material should be of a moisture content at or about 1 to 2 percent above optimum moisture of the field-mixed materials.

8. Spiked tooth drags used in removal of tractor tread and compactor foot marks should follow the first pass of the compactor and be of a weight and length adequate to reach through the uncompacted material and scratch the surface of the compacted layer.

ACKNOWLEDGMENTS

Sincere appreciation is expressed to the following people and organizations: the Iowa Highway Research Board and the Iowa State Highway Commission, especially Mark Morris and Stephen Roberts for the research opportunity presented by Project HR-68; Webster County and the Webster County Engineers office and personnel, especially John Mors and Stan Mathews for the cooperation given the author during the

construction and testing phases of this program; Tom McCarthy, Earl Swane and George Heinrichson of the McCarthy Improvement Co., Davenport, Iowa, for the cooperation and interest shown during construction of the experiment; and those members of the Engineering Experiment Station Soil Research Laboratory who assisted in the testing program.

A special remembrance is also made of the late Dr. D. T. Davidson, Iowa State University, who gave so much of himself to the present-day knowledge of soil stabilization and who counseled the author in the research on this project, not only as a colleague but also as a friend.

REFERENCES

1. Hoover, J. M., Huffman, R. T., and Davidson, D. T. Soil Stabilization Field Trials, Primary Highway 117, Jasper County, Iowa. Highway Research Board Bull. 357, pp. 41-68, 1962.
2. Pietsch, Paul E., and Davidson, D. T. Effects of Lime on Plasticity and Compressive Strength of Representative Iowa Soils. Highway Research Board Bull. 335, pp. 11-30, 1962.
3. Handy, R. L., and Davidson, D. T. Cement Requirements of Selected Soil Series in Iowa. Highway Research Board Bull. 267, pp. 14-27, 1960.
4. Davidson, D. T., and Bruns, B. W. Comparison of Type I and Type III Portland Cements for Soil Stabilization. Highway Research Board Bull. 267, pp. 28-45, 1960.
5. Soil Stabilization with Portland Cement. Highway Research Board Bull. 292, 1961.
6. Goecker, W. L., Moh, Z. C., Davidson, D. T., and Chu, T. Y. Stabilization of Fine and Coarse-Grained Soils with Lime-Flyash Mixtures. Highway Research Board Bull. 129, pp. 63-82, 1956.
7. Hoover, J. M., Handy, R. L., and Davidson, D. T. Durability of Soil-Lime-Flyash Mixes Compacted Above Standard Proctor Density. Highway Research Board Bull. 193, pp. 1-11, 1958.
8. George, K. P., and Davidson, D. T. Development of a Freeze-Thaw Test for the Design of Soil-Cement. Highway Research Record No. 36, pp. 77-96, 1963.
9. Lu, L. W., Davidson, D. T., Handy, R. L., and Laguros, J. G. The Calcium-Magnesium Ratio in Soil-Lime Stabilization. Highway Research Board Proc., Vol. 36, pp. 94-805, 1957.
10. Felt, Earl J. Factors Influencing Physical Properties of Soil-Cement Mixtures. Highway Research Board Bull. 108, pp. 138-162, 1955.