Lime and Phosphoric Acid Stabilization in Missouri

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This report presents information accumulated since 1958 on a 17.37-mi chemical stabilization project in Worth and Gentry Counties in northwest Missouri. The results of field investigations and laboratory research tests, preliminary to construction in 1961-62, are reviewed. Explanation of the design procedure, based on Missouri's Flexible Pavement Thickness Chart, is included. Also reported are results of tests during construction, and those of subsequent investigations by Benkelman beam, rut gage, roughometer and core drill.

The first section of the report is limited to details of the lime stabilization on all three projects, F-297(7) and F-297(9) in Worth County and F-524(2) in Gentry County. The second section reviews the design, construction and performance of the 16 test sections composing 2 mi of Project F-524(2), Gentry County, in which various combinations of 4- and 8-in. thicknesses of rolled stone base were constructed on untreated glacial clay subgrade, on a 5-in. subbase of that soil mixed with lime or with phosphoric acid, or on a 5-in. rolled stone subbase.

•EARLY IN the 1920's, shortly after becoming Missouri's first Materials Engineer, F. V. Reagel started experimenting with methods and materials which might be used to process native soils and adapt them to more effective use in highway construction. Throughout the following years Missouri's highway engineers have diligently attacked the problem while applying different principles of road-building science to various types of soil.

Experience with lime stabilization was primarily confined to a project constructed in 1952 by maintenance forces (1). Two separate sections on Route 51, Perry County, totaling approximately 2 mi, were treated with hydrated lime, quicklime, a combination of the two materials, and a stackdust waste product. Periodic tests since that time have indicated that permanent improvement of the soil resulted from the treatment.

The most recent and most extensive study was constructed in 1961-62 in Worth and Gentry Counties, although preliminary work started as early as February 1958 when a soil survey was made and routine laboratory tests were completed.

A further step, preceding the design stage, involved extensive laboratory research on mixtures of the soil with lime and phosphoric acid. A short pilot section was constructed in another area, with phosphoric acid as the stabilizing agent, to determine the practicability of field processing such materials. With the fundamental preliminary work completed, and with due consideration for unknown influencing factors such as changes in soil or projected traffic, the thickness was determined in conformity with Missouri's method of flexible pavement design and to the standards current in 1959.

A contract was awarded to the Howard Construction Co. of Sedalia, Mo., in February 1960 for Project F-297(7), Worth Co., from the Iowa line south for 10.328 mi. Early

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in 1961, the same contractor was awarded Project F-297(9), continuing 2.497 mi south across Worth Co., and an adjacent 4.545-mi project, F-524(2), in Gentry Co. All work was completed by October 1962.

The study herewith reported consists of four principal phases:

1. Selection of a location where reasonably uniform soil conditions prevailed and the predominant soil reacted favorably when mixed with lime or phosphoric acid (and of only slightly secondary importance, a location where aggregate deposits were scarce);
2. Extensive laboratory research to investigate the effects of adding the stabilizing agents and to determine the optimum amounts to use;
3. Design and construction, guided and controlled by application of the knowledge gained in the laboratory; and
4. Studying the performance of the completed project.

LIME STABILIZATION

Purpose

This experimental project was designed and constructed to determine the performance of a road built with a 5-in. by 30-ft trenched subbase of natural soil modified by mixing 6 percent hydrated lime into the glacial clay, covering the subbase with an 8-in. compacted thickness of rolled stone base from inslope to inslope, and surfacing a 24-ft width with 3 in. of asphaltic concrete in two courses. These dimensions apply to the total length of Projects F-297(7) and F-297(9) in Worth Co. and to more than half of the 4.545 mi of Project F-524(2) in Gentry Co.

The remaining 2 mi in Gentry County were designated as the site of a complete factorial experiment of sixteen 660-ft sections. Included were duplicate sections with 4- or 8-in. thicknesses of rolled stone base over a 5-in. subbase of rolled stone or a 5-in. subbase of native soil modified by lime or phosphoric acid. Four sections had no subbase. All test sections were surfaced with 3 in. of asphaltic concrete. Three sections were treated with acid at 50 percent strength and one at 75 percent, in quantities ranging from 2.8 to 4.3 percent, on a dry soil basis. Six percent lime was used in the lime sections. Some test sections were deliberately underdesigned.

Scope

This study, as stated previously, began with an early routine soil survey in which the soils were identified and sampled, and their extents were determined. Normal procedures were followed in the laboratory testing of these samples to determine mechanical analysis, plastic index (P. I.), group index, maximum density at optimum moisture (AASHO Designation: T 99, Missouri modification), and A-group classification.

The uniform clayey soil conditions and the scarcity of aggregate deposits prompted the suggestion that lime stabilization should be applicable, beneficial, and economical in this area. Tentative approval incited further sampling and more exhaustive testing, which involved the foregoing series of tests, plus slaking and unconfined compression tests. All tests were made on specimens of Shelby soil mixed with 4, 5 and 6 percent lime.

The testing program during construction was elaborate and thorough. The subgrade, lime subbase and base were, in general, all sampled at the same place, at 2,000-ft intervals. The following tests were performed on subgrade, lime subbase and base: liquid limit (L. L.), plastic limit (P. L.), optimum moisture, maximum density, field moisture, field density, and laboratory CBR. Additional tests on subgrade and subbase were shrinkage limit, free swell, and compressive strength; on subgrade, mechanical analysis; on subbase, passing No. 200 sieve, pulverization, and thickness, and on base, sieve analysis, thickness, and L. A. abrasion.

Rutting and deflection measurements were made in conjunction with crack surveys on various sections of these projects in November 1961 and May 1962 (before opening to all traffic), June 1963, fall of 1963, and spring of 1964. In August 1963, an attempt was made to measure deflections in each component of the complete structure. Permeability tests were made on the asphaltic concrete in December 1962. Distress has twice been reported and investigated.

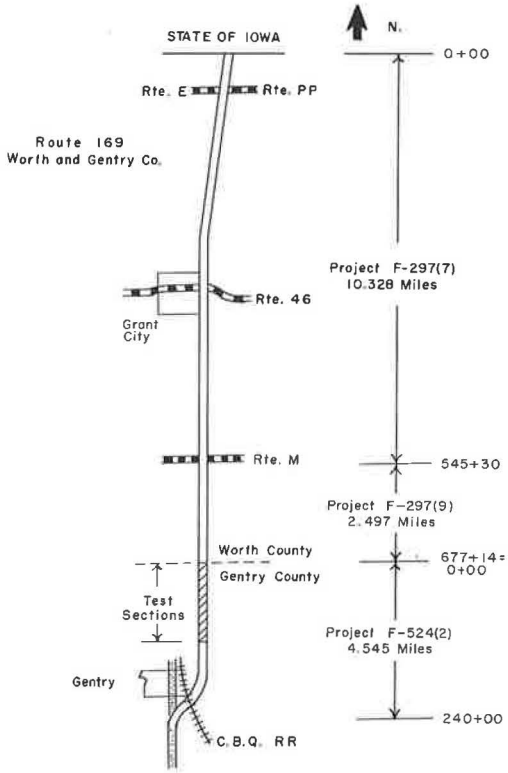


Figure 1. Study site.

Location

The experimental section is located about 60 mi northeast of St. Joseph, Mo. The length of each of the three projects is shown in Figure 1.

Topography

In general, Route 169 traverses heavily rolling country with narrow valleys and steep-sided, deeply eroded gullies. The grade consists mainly of a series of cuts and fills, of which many exceed 20 ft and a few are more than 40 ft.

Soils

The C horizon of the Shelby soil series predominates throughout this project. Glacial in origin, the C horizon is usually a yellow clay, classified A-7-6(16), and averages about 30 percent glacial sand and gravel. The clay mineral fraction is a calcium montmorillonite composed of approximately 58 percent montmorillonite, 20 percent illite, and 22 percent kaolinite.

From Station 200 (Gentry Co.) to the south end of Project F-524(2) at Station 240, the soil is Wabash clay, a black, organic, alluvial deposit classified as Group A-7-6(18-20). Since it is a derivative of the Shelby, it is considered also to be primarily montmorillonitic.

Preliminary Testing

Samples representative of the C horizon Shelby were obtained in 1958 while making a routine soil survey. Laboratory tests showed the following average results: L. L., 48; P. I., 35; S and C, 65 percent; M. D., 108 lb; O. M., 17.7 percent; and classification A-7-6(17) clay.

TABLE 1

PRELIMINARY LIME-SOIL TESTS, WORTH COUNTY

Station	Lime (%)	L. L.	P. L.[a]	P. I.[a]	S. L.[a]	Passing No. 200 Sieve (%)[a]	Silt (%)[a]	Clay < 5 μ (%)[a]	Group	Comp. Strength (psi)[b]	Slaking[b]
23 + 00	0	55	31	24	12.9	98.8	48	43	A-7-5(17)	—	—
	4	56	38	18	23.6	23.0	—	—	A-2-7(1)	39	Very slight
	5	54	36	18	23.7	23.3	—	—	A-2-7(1)	33	Slight
	6	51	39	12	27.8	20.0	—	—	A-2-7(0)	39	Minor
263 + 00	0	40	20	20	14.2	67.5	27	33	A-6(10)	—	—
	4	37	28	9	29.9	18.7	—	—	A-2-4(0)	44	None
	5	38	28	10	25.9	23.2	—	—	A-2-4(0)	44	None
	6	37	29	8	24.4	20.3	—	—	A-2-4(0)	52	None
352 + 00	0	46	18	28	9.8	75.4	27	40	A-7-6(16)	—	—
	4	39	30	9	22.7	6.8	—	—	A-2-4(0)	41	3/16 in.
	5	39	29	10	25.1	4.5	—	—	A-2-4(0)	82	1/8 in.
	6	40	32	8	26.6	2.9	—	—	A-2-4(0)	92	Small loss

[a]Made on loose material after 24-hr moist curing.
[b]Made on 4- by 4.6-in. specimens compacted at optimum moisture and to maximum density, moist cured 7 days, dried 8 hr at 140 F, and soaked for 16 hr.

To permit research into the effects of mixing lime with the soil, more samples were obtained. The material from Station 352 was considered to represent the job's predominant soil. The results of tests on this and other soils, as well as on mixtures of each with 4, 5 and 6 percent lime, are given in Table 1. Based on these results, a lime content of 6 percent by weight of dry soil was recommended because:

1. All such mixtures showed the greatest reduction in plasticity (probably even greater if cured compacted specimens rather than loose material had been tested);
2. Two of the three soils showed higher shrinkage limits, indicating less volume change;
3. The same two soils showed more agglomeration, as indicated by the lower minus No. 200 content;
4. The compressive strength was highest with two of three soils, and as good with the other soil; and
5. Slaking was negligible with all the 6 percent mixtures, but was rather severe for the 4 and 5 percent mixtures with soil from Station 352.

Design

The Missouri Flexible Pavement Thickness Design Chart was used to determine the total roadbed thickness. At the time, no equivalencies had been determined for rolled stone base vs other types of base construction. Total thickness was governed by the 20-yr anticipated heavy axle traffic and the group index of the subgrade. On this basis a 16-in. total thickness was recommended, consisting of a 5-in. lime-treated subbase, 8 in. of compacted crushed stone base, and 3 in. of asphaltic concrete, as shown in the typical section (Fig. 2).

Specifications. —The following specifications were to apply to all lime-stabilized sections covered by this section of the report:

1. Hydrated lime—to comply with requirements of ASTM C 207-49, omitting Sections 3a and 5b;
2. Subgrade—top 18-in. to be compacted to a minimum of 95 percent of T 99 maximum density (Missouri method, using four layers when P. I. exceeds 25);
3. Subbase—5-in. depth, trenched 30 ft wide, to be compacted to a minimum of 95 percent of standard T 99 density;
4. Base—8-in. compacted rolled stone base to be placed in two 4-in. lifts, each compacted to a minimum of 95 percent AASHO T 99 density;
5. Surface—3-in. of asphaltic concrete, 24 ft wide, consisting of 1¾ in. of Type B base course and 1¼ in. of Type C surface course;
6. Shoulders—9 ft wide where guardrails were specified, elsewhere 7 ft wide; on the high side of superelevated curves surfacing as on the 24-ft riding surface, elsewhere paved with Type B asphaltic concrete tapered from a thickness of about 2.5 in.

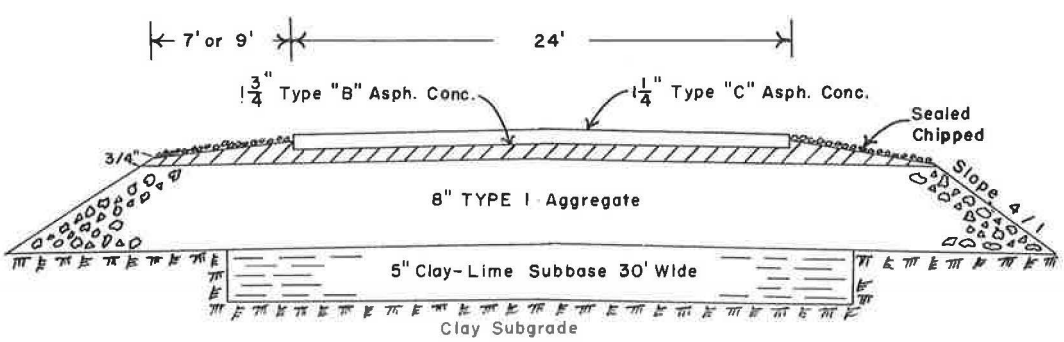


Figure 2. Typical section, lime subbase.

at the riding surface edge to ¾ in. at the edge of the shoulder, and sealed with 200-250 penetration asphalt or RC-4 cutback asphalt, and white stone chips;

7. Pulverization—on Project F-297(7), the first constructed, the only requirement was that 100 percent pass the 1½-in. sieve after lime was added. On Projects F-297(9) and F-524(2), specifications were changed to require that 100 percent pass the 1-in. sieve and at least 60 percent pass the No. 4 sieve. Moisture was to be within 2 percent of optimum at the completion of mixing and during compacting.

Construction

Equipment. —The contractor chose to use bulk lime and mix in place with the soil. For these projects the following equipment was found to be satisfactory and sufficient: one 10-ft P & H one-pass stabilizer, one Seaman lime spreader, three lime-hauling dump trucks, two water trucks (5,000 and 1,500 gal), one motor grader, and one 12-ton pneumatic roller.

Procedure. —Construction procedure involved spreading lime, processing soil, compacting the mixture, finishing and sealing.

Lime Application. —Lime, furnished by the Marblehead Lime Co. of Hannibal, Mo., was distributed on the unscarified finished subgrade by the Seaman spreader. Because of the varying air content and fluffiness of the bulk lime it was sometimes necessary to make two passes of the spreader to apply the proper quantity.

Processing the full 30-ft width in three passes with the stabilizer removed some of the subgrade crown and diluted the lime. The crown could be protected by making four passes, but progress would be considerably reduced by such a procedure. Therefore, it was decided to compensate for the additional soil by increasing the lime quantity slightly and processing in three strips. The lime spread was increased from 72 to 75 lb/lin ft, a quantity which was generally maintained throughout the remainder of the lime stabilization work on all three projects (Table 2).

Processing. —With the exception of perhaps 10 percent of the project's length, satisfactory pulverization (100 percent pass the 1½-in. sieve) was obtained on Project F-297(7) after one pass of the P & H machine at its slowest speed, 5.5 ft/min, and no rotting period or remixing was necessary. An average of screening tests indicated that all material passed the 1½-in. sieve, approximately 97 percent passed the 1-in. sieve, and the minus No. 4 content was 43 percent.

To comply with gradation specifications on Projects F-297(9) and F-524(2) it was essential to allow a seasoning period during which the initial soil-lime reaction occurred and the breakdown of clay lumps began. Although a few short sections showed satisfactory pulverization after only a 6-hr rotting period, 24 hr was usually allowed before remixing. In no instance was the specified 72-hr maximum exceeded.

Because the soil-lime mixture was much more friable after seasoning, the remixing pass of the P & H could usually be made in second gear at 10.1 ft/min. Occasional trials prove that any speed faster than that did not permit satisfactory pulverization.

TABLE 2

SUMMARY OF LIME SPREAD DATA[a]

Project	Tot. Ft. Proc.	Tot. Lime (tons)	Avg. Appl. Rate (lb/lin ft)	Tot. Time for Appl. & Proc. (days)	Avg. Daily Prod. (ft)
F-297(7)	56,773.8	2,127	74.9	45	1,261.6
F-297(9)	13,183.6	511.5	75.4	19	693.9
F-524(2)	12,488	460.37	73.7	16	780.5

[a]Detailed project data from which this summary was derived are available from the Highway Research Board, Supplement XS-5 (Highway Research Record No. 92), 29 pp.

Because the optimum moisture of the soil-lime mixture was higher than that of the natural soil, water had to be added through the P & H during the initial pass. Light rolling immediately followed to reduce evaporation and, by somewhat densifying the mix, to promote the reaction of the soil-lime-water system. During remixing it was again necessary to add water.

Spray nozzles in the P & H sometimes became plugged, causing variations in the amount of water added. Soil moisture also varied because of the frequent rains. Satisfactory moisture control was nevertheless maintained by the squeeze test method when used by competent inspectors.

Erratic glacial boulders sometimes stripped teeth off the P & H cylinder, and paddles had to be changed often because the sand and gravel quickly wore them out. When properly maintained, however, the machine did an excellent job of processing the soil and lime.

Compaction. —Compacting equipment closely followed the P & H as it mixed the subbase materials. As stated elsewhere, this was done to seal the surface partially and prevent entrance of excess water during frequent rains, as well as to force the soil and lime into more intimate contact and hasten the base exchange reaction. The initial relatively light rolling was applied to all sections, whether or not they were to be remixed.

Early tests proved that density requirements could be met by about three passes of the pneumatic roller alone, and the sheepsfoot tamper was not used after a short trial on the first day of construction.

Testing During Construction. —Construction forces performed many routine tests in the normal course of their work, the results of which are not included in this report. The experimental research nature of the project, however, demanded sampling and testing far beyond their capacity. The Materials Division had been instrumental in promoting the experiment and had recommended the design thickness and percentage lime to be used. As a result of this special interest, a testing crew of Geology and Soils personnel was assigned to the project, with a mobile laboratory to supplement the usual facilities available in the field. Some tests, impossible for the testing crew to make because of time or equipment limitations, were performed by the Jefferson City central laboratory.

Test Methods. —The procedure set forth in AASHO Designation: T 99, Method A, was followed to determine moisture-density relationships in both subgrade and subbase, with one modification. Because the P. I. of the soil exceeded 25, those specimens were compacted in four layers, as is standard procedure in Missouri. Lime-soil specimens were molded in three layers. Densities of subgrade and subbase were determined by the sand-cone method. AASHO Designation: T 88 procedure was followed to determine gradation of the soil.

Field tests to determine the minus No. 200 fraction of the lime-soil mixture were made on processed material from immediately behind the P & H. The loose mixture was first oven-dried, then thoroughly soaked and, accompanied by light rubbing, washed through the sieve.

Compressive strength tests of soil were made on unsoaked, triplicate 2- by 2-in. specimens molded at optimum moisture to maximum density and stored overnight in the humid room.

Lime-soil specimens were usually 2 by 2 in., although the dimensions of a few were 4 by 4.6 in. Triplicate test specimens were molded to maximum density at the field moisture content, wrapped in plastic or foil, and cured for 28 days at 72 F and 95 percent RH. Most were broken without soaking, although in a few instances comparative strengths were obtained by breaking soaked and unsoaked sets.

Laboratory CBR was determined on raw soil specimens molded in the laboratory under static load to T 99 maximum density and optimum moisture and tested after soaking until swelling became negligible.

The laboratory CBR test was made on lime-soil specimens molded in the field at field moisture with dynamic compaction (T 99). The specimens were moist cured for 28 days and tested after soaking until swelling had ceased.

Pulverization Tests. —The results of tests to determine the amount of pulverization obtained on each project are given in Table 3. The results for Project F-297(9) seem

TABLE 3

RESULTS OF PULVERIZATION TESTS

Project	Station	Rotting Time (hr)	Percent Passing Sieve		
			1½ In.	1 In.	No. 4
F-297(7)	350	0	100	100	48
	287	0	100	98	35
	227	0	100	100	44
	207	0	100	92	44
	114 + 50	0	100	99	53
	107	0	100	100	30
	67	0	100	90	38
	687 (Rt. 46)	0	100	98	51
Avg.			100	97	43
F-297(9)	587	6	—	100	67
	598 + 50	6	—	100	61
	625 + 10	18	—	100	61
	627 + 25	18	—	100	61
	647	48	—	100	60
	664	48	—	100	60
Avg.			—	100	62
F-524(2)	107	24	—	100	58
	127	72	—	100	69
	147	24	—	100	64
	167	24	—	100	61
	174	24	—	100	71
	204	24	—	100	56
	217	24	—	100	71
	232	24	—	100	67
Avg.			—	100	65

TABLE 4

AVERAGE EFFECT OF LIME ON SOILS, SUBGRADE AND SUBBASE[a]

Property	Amount (%)		
	F-297(7)	F-297(9)	F-524(2)
P. I., decrease	86	90	92
Shrinkage limit, increase	141	132	152
Shrinkage ratio, decrease	25	24	25
Free swell, decrease	27	45	53
Opt. moisture, increase	44	50	57
Max. density, decrease	13	14	13
Minus No. 200 sieve material, decrease	56	69	62
Compressive strength, increase	406	609	276[b]
	—	—	190[c]

[a]See footnote, Table 2.
[b]Both unsoaked.
[c]Lime soaked.

to indicate that a long rotting time has little or no effect on soil breakdown. In ten additional tests on Project F-524(2), after 24 hr rotting, the minus No. 4 content ranged from 60 to 81, with 6 of the 10 below 64 percent. This may indicate that, for this soil and equipment, a higher minus No. 4 requirement might be achieved only through additional pulverizing effort, or by using a greater percentage of lime, since the effect of tripling the rotting period was almost negligible.

Summation of Test Results. —Numerous other tests were made on both subgrade and subbase. Analysis of the results on the three projects, summarized in Table 4, permits the following statements to be made.

1. A reduction in L. L. is shown from 46 in the raw soil to 40 after mixing in lime.
2. An average P. L. of 16 in the clay was increased to 31 in those samples of the mixture on which the test could be made.
3. Average P. I. for native soil was 30. After lime processing, 18 tests averaged 9, 23 were nonplastic and the overall average was 4.
4. The shrinkage limit increased from 11 in the soil to 27 in the mixture.
5. The shrinkage ratio changed from 2.03 to 1.52.
6. Natural soil displayed a free swell of 75 percent, and the mixture showed 50 percent in this indicator test.
7. The average optimum moisture of 15.5 for soil was increased in the mixture to 22.8 percent.
8. Addition of 6 percent lime reduced maximum density from 113.3 to 98.7 pcf.
9. The average content of minus No. 200 material for untreated soil was 73 percent. Processing with lime reduced this to 30 percent.
10. The average group index of 16 for the soil was reduced to an average of 1 in the mixture.
11. Based on averages of all tests on the three projects, the classification shows a change from A-7-6(16) to A-2-4(0), a very remarkable improvement.
12. Although exhibiting rather wide variations in unconfined compression tests, the following averages are indicative of the improvement in compressive strength from untreated to treated soil: (a) Project 297(7), from 41 to 207 psi; (b) Project 297(9), from 25 to 292 psi; and (c) Project 524(2), from 49 to 184 psi. Specimens were cured for 28 days and broken without soaking. Twelve additional pairs of specimens were molded and cured in similar manner. One of each set was soaked overnight before breaking; the other was not. Average strength was reduced, by soaking, from 170 to 135 psi.

TABLE 5

Project	Subgrade		Subbase	
	From O. M. (%)	% M. D.	From O. M. (%)	% M. D.
F-297(7)	+1.1	99.5	-0.1	101.1
F-297(9)	+0.9	99.2	+0.7	98.5
F-524(2)	+5.4	93	+0.7	95.8

13. Although only four CBR test were made on soils and four on mixtures from the same locations, the limited data are indicative of the improvement caused by addition of the lime. Average CBR of the soil was 3.8, which was increased in the mixture to 66.5.

14. The average percent compaction on F-297(7) was 101.1, as compared to 98.5 on F-297(9) and 97.8 on F-524(2). All projects received approximately the same compactive effort and density tests were made on the mixture after processing and finishing. No explanation can be given for the fact that the unrotted material showed a higher percentage of compaction than the rotted sections.

15. Table 5 indicates that, except for the subgrade on F-524(2), compliance with moisture and density requirements was excellent.

16. The thickness was measured at all sampling locations, at approximately 2,000-ft intervals. Measurements were made at edges, quarter points and centerline. Averages show that the depth of subbase exceeded the 5-in. designed thickness on all projects. On F-297(7) it was 5.44 in., whereas F-297(9) and F-524(2) showed 5.45 and 5.08 in., respectively.

Finishing. —The day after compaction was completed the subbase was trimmed by blade to proper grade and cross-section, sprinkled, and finished with a pneumatic roller. A considerable quantity of material was shifted by the blade in evening up ridges and depressions left by the stabilizer and pneumatic. The probability of developing compaction planes during this phase of the work was not considered to be of critical importance at a depth of 11 or more in. below the surface.

Curing. —All lime subbase was cured by sprinkling daily until the first 4-in. layer of base material was placed. On F-297(7) a 20-day limit was specified between completing the subbase and placing the base. The difficulty of maintaining the proper moisture content on this project influenced the curing procedure on the other projects. Although no time limit was set, the contractor found it expedient to protect the subbase within a few days to avoid frequent applications of water. It is of interest to note that placing stone base was sometimes delayed after a rain because of the soft untreated shoulders, but the lime-treated subbase was in excellent condition and supported traffic without deformation of any sort.

Production. —Crushed rock for the rolled stone base was produced at a nearby local quarry from the Stanton formation of shaly Pennsylvanian limestone. Specifications applicable at that time were for Project F-297(7): pass 1½-in. sieve, 100 percent; pass 1-in. sieve, 95 to 100 percent; pass ½-in. sieve, 60 to 85 percent; pass No. sieve, 40 to 60 percent; pass No. 40 sieve, 15 to 35 percent; L. L., 25 max. and P. I., 6 max. Specifications for Projects F-297(9) and F-524(2) were pass 1-in. sieve, 100 percent; pass ½-in. sieve, 60 to 90 percent; pass No. 4 sieve, 40 to 60 percent; pass No. 40 sieve, 15 to 35 percent; L. L., 25 max., and P. I., 6 max.

The material was inspected during production and all that complied with specifications was stockpiled. The average P. I. was well below the allowable 6, having been reduced by the addition, during production, of 5 to 10 percent nonplastic fine sand when early tests indicated the possibility of P. I. trouble.

Moisture. —Water added to the pug mill mixer at the plant was usually adequate to assure satisfactory compaction. Critical points were about 0.5 percent below and 2 percent above optimum. Beyond those limits density requirements were difficult to meet without additional work to bring the moisture within that range.

Test Methods. —AASHO Designation: T 99, Method A, was used to determine the moisture-density relations in the minus No. 4 portion of the stone base. The maximum density and optimum moisture of the total base were computed from this moisture density relation, the percent passing the No. 4 sieve, and the specific gravity and absorption of the plus No. 4 material. Gradations were determined by washing.

TABLE 6

SUMMARY OF ROLLED STONE BASE TEST RESULTS[a]

Course	L. L.	P. L.	P. I.	Opt. Moist.	Max. Dens.	Field Moist.[b]	Field Dens.	Comp. (%)	Percent Passing Sieve							
									1½ In.	1 In.	¾ In.	½ In.	No. 4	No. 10	No. 40	No. 100
(a) Project F-297(7)																
1, avg.	18	14	4	7.6	138.7	7.9	136.5	98.4	100	97.7	92.6	83.4	59.3	45.2	29.2	17.3
2, avg.	19	14	5	7.3	137.8	7.4	135.7	98.5	100	96.0	88.6	77.8	54.1	43.0	38.9	16.9
Both:																
Avg.	19	14	5	7.4	138.2	7.7	136.1	98.5	100	96.8	90.5	80.5	56.6	44.1	29.0	17.1
Max.	21.8	16.5	6.6	8.3	141.0	9.2	146.3	104.5	100	100	98.4	96.2	70.2	53.0	37.5	25.5
Min.	15.1	11.6	NP	7.0	134.5	5.7	129.3	93.3	100	87.2	74.3	60.2	39.3	30.5	21.2	12.6
Specification	≤ 25	—	≤ 6	—	—	—	—	≥ 95	100	95-100	—	60-85	40-60	—	15-35	—
(b) Project F-297(9)																
1, avg.	21	14	7	7.2	137.4	8.1	138.7	100.9	—	100	96.9	87.7	64.7	50.2	30.0	18.8
2, avg.	21	15	7	7.2	137.8	8.0	142.3	103.2	—	100	95.5	86.4	63.6	49.2	31.5	20.5
Both:																
Avg.	21	14	7	7.2	137.6	8.1	140.5	102.1	—	100	96.2	87.0	64.1	49.7	30.8	19.7
Max.	23	16	9	7.2	139.5	8.7	144.9	105.6	—	100	98.4	89.7	68.4	53.2	34.2	22.5
Min.	20	13	5	7.2	136.0	7.6	134.0	97.8	—	100	86.8	85.0	60.3	43.7	27.8	16.2
Specification	≤ 25	—	≤ 6	—	—	—	—	≥ 95	—	100	—	60-90	40-60	—	15-35	—
(c) Project F-524(2)																
1, avg.	21	15	6	7.8	138.8	7.7	136.5	98.3	—	100	96.3	84.9	60.0	45.3	28.6	18.8
2, avg.	21	15	6	7.9	137.9	7.9	138.5	100.4	—	100	96.5	84.6	58.3	44.4	29.5	19.8
Both:																
Avg.	21	15	6	7.9	138.4	7.8	137.5	99.4	—	100	96.4	84.8	59.2	44.9	29.1	19.3
Max.	22	16	8	7.9	139.4	8.4	143.4	103.7	—	100	98.3	88.0	63.0	47.7	32.0	22.0
Min.	20	13	5	7.7	137.3	7.3	132.0	95.4	—	100	95.2	80.4	54.3	42.0	26.4	17.4
Specification	≤ 25	—	≤ 6	—	—	—	—	≥ 95	—	100	—	60-90	40-60	—	15-35	—

[a]See footnote, Table 2.
[b]Results for Project F-297(7) at time of compaction.

TABLE 7

BASE THICKNESS MEASUREMENTS

Project	Max.	Min.	No. Tests	Avg.
F-297(7)	9	7	250	8.02
F-297(9)	10	7¾	29	8.23
F-524(2)	8⅜	7¼	26	8.03

Construction Procedure. —The 8-in. designed thickness of crushed stone was compacted in two layers by pneumatic roller and a 12-ton tandem flat wheel to a density of 95 percent or more of the maximum weight.

Test Results. —A summary of the results of tests for P. I., moisture, density and gradation in both lifts, at about 2,000-ft intervals, is given in Table 6.

Base thickness was measured at quarter points and centerline. Results are given in Table 7.

Tests on plus No. 4 material showed average specific gravity of 2.44, 2.45 and 2.47 for the three projects in the usual order; absorption was 4.4, 3.5 and 4.3 percent.

One CBR test showed 100 percent and one Los Angeles abrasion test (Gradation B) showed a wear of 36.3 percent.

The following conclusions may be drawn from an analysis of the data.

1. Average liquid limits were from 18 to 21, and none reached specified maximum of 25.
2. The P. I. of the first lift on F-297(7) averaged four, with none higher than 6, the specified maximum. The second lift, on which the average was 5, showed 4 of 29 tests above 6. Results were not as good for Project F-297(9), where an average for both courses was 7, and 7 out of 9 tests exceeded 6. Both courses on F-524(2) averaged 6, and 6 of 14 individual tests were above 6.
3. At the time of compaction, base moisture on all projects was within less than 1 percent of optimum.
4. Densities ranged from 98.5 to 102.1 percent of the maximum.
5. As far as gradation was concerned, complying with the No. 4 sieve specification gave the only trouble. The crushed material, as produced, was in the upper register of the 40 to 60 percent specified band and, at the same time, near the maximum allowable P. I. Addition of fine sand to lower the P. I., plus some probable degradation during stockpiling, caused quite a few samples to exceed the upper limit of 60 percent minus No. 4.

Curing. —Following completion of compacting operations the base was properly shaped and cured to meet specified moisture content before priming. On Project F-297(7) it was required that the 8-in. base contain no more than 50 percent of optimum moisture at the time of application of the 0.35 gal/sq yd MC-0 prime. Due to the change in specifications referred to previously, requirements were less stringent on the other two projects, where it was necessary that the moisture in the top 2 in. of the base be no more than two-thirds of optimum.

Asphaltic Concrete

Compaction of the two courses of asphaltic concrete was accomplished with tandem steel-wheel and pneumatic rollers. Job mix limits and average results of tests made on the finished asphaltic concrete for Project F-297(7) are as follows:

Limits—Type B (%)		Limits—Type C (%)	
1½ in. -½ in.	29.1-39.1	¾ in. -No. 10	47.3-57.3
½ in. -No. 10	38.4-38.4	No. 10-No. 200	31.5-37.5
No. 10-No. 200	21.6-27.6	-No. 200	4.8-6.8
-No. 200	2.4-4.4	AC	4.7-5.7
AC	4.0-5.0		

Finished product, Type B—spec. gr. = 2.341, % voids = 3.9, % compaction = 99.9, % bitumen (by extraction) = 4.2 %.

Finished product, Type C—spec. gr. = 2.295, % voids = 5.0, % compaction = 98.2, % bitumen = 4.7 %

For Projects F-297(9) and F-524(2), these are as follows:

Limits—Type B (%)		Limits—Type C (%)	
1 in. -½ in.	21.6-31.6	½ in. -No. 10	50.3-60.3
½in. -No. 10	31.3-46.3	No. 10-No. 200	31.5-37.5
No. 10-No. 200	26.9-32.9	-No. 200	4.2-6.2
-No. 200	2.0-3.7	AC	4.5-5.5
AC	4.0-5.0		

Finished product, Type B—spec. gr. 2.355, % voids = 4.1, % compaction = 100.7, % bitumen = 4.4.

Finished product, Type C—spec. gr. 2.325, % voids = 4.9, % compaction = 99.7, % bitumen = 4.91.

Construction Costs. —Listed in the following are costs, from final plan quantities, of pavement construction on the various projects.

Project F-297(7):

2089.0 tons lime at $30	$62,670.00
10.76 mi lime processing at $2500, 5 in. × 30 ft	26,900.00
110,500 tons base stone at $2.00, 8 in. × 41 ft avg.	221,000.00
554,000 gal water at 60/100 gal for subgrade and base maintenance	3,324.00
10.76 mi at $2500 for spreading, shaping and compacting stone	26,900.00
79,908 gal primer at $0.18 for stone base	14,383.44
22,813 tons Type B asphaltic concrete at $7.02	160,147.26
10,664 tons Type C asphaltic concrete at $7.31	77,953.84
33,447 gal 200-250 penetration asphalt at $0.18 for shoulders	6,020.46
1538 tons cover aggregate at $6.00 for shoulders	9,228.00
Total	$608,527.00

Cost per mile:	
Lime subbase	$8,324.35
Stone base	23,347.96
Asphaltic concrete surface, prime, shoulder surfacing	24,882.24
Total	$56,554.55

Project F-297(9):

197,600 gal water at $0.30/100 gal for lime-soil mixture	$592.80
501.6 tons lime at $30	15,048.00
2.5 mi lime processing at $2500, 5 in. × 30 ft	6,250.00
24,833 tons base stone at $2.20, 8 in. × 41 ft	54,632.60
25,000 gal water at $0.30/100 gal for subgrade and base maintenance	75.00
2.5 mi at $2500 for spreading, shaping and compacting stone	6,250.00
19,881 gal primer at $0.17 for stone base	3,379.77
5185 tons Type B asphaltic concrete at $7.50	38,887.50
2430 tons Type C asphaltic concrete at $7.50	18,225.00
7080 gal RC-4 cutback asphalt at $0.18 for shoulders	1,274.40

Project F-297(9) (Cont'd):

249 tons cover aggregate at $6.00 for shoulders	1,494.00
Total	$146.109.07
Cost per mile:	
Lime subbase	$8,756.32
Stone base	24,383.04
Asphaltic concrete surface, prime, shoulder surfacing	25,304.26
Total	$58,443.62

Project F-524(2), Station 105+60 to 234+77, minus bridge:

154,740 gal water at $0.30/100 gal for lime-soil mixture	$464.22
451.00 tons lime at $30	13,530.00
2.36 mi lime processing at $3000, 5 in. × 30 ft	7,080.00
24,363 tons base stone at $2.20, 8 in. × 41 ft	53,598.60
24,000 gal water at $0.30/100 gal for subgrade and base maintenance	72.00
2.36 mi at $2500 for spreading, shaping, compacting stone	5,900.00
20,742 gal primer at $0.17 for stone base	3,526.14
5037 tons Type B asphaltic concrete at $7.50	37,777.50
2516 tons Type C asphaltic concrete at $7.50	18,870.00
6552 gal RC-4 cutback asphalt at $0.18 for shoulders	1,179.36
326 tons cover aggregate at $6.00 for shoulders	1,956.00
Total	$143,953.82
Cost per mile:	
Lime subbase	$8,929.76
Stone base	25,241.77
Asphaltic concrete surface, prime, shoulder surfacing	26,825.85
Total	$60,997.38

By prorating costs of the stone base, the following comparisons of costs per mile of 5 in. by 30-ft stone base vs the same thickness and width of lime subbase can be made. (Calculations are based on the assumption 5 in. by 30-ft stone base is 44.3+ percent of total base material used and, therefore, is that percentage of cost of stone base per mile.)

Project	F-297(7)	F-297(9)	F-524(2)
Lime	$8,324.35	$8,756.32	$8,929.76
Stone	$10,352.48	$10,811.43	$11,192.20
Additional costs, stone over lime	24.4%	23.5%	25.3%

These comparisons show that, under these contracts, 4 in. of rolled stone base 30 ft wide costs approximately the same as 5 in. of lime-soil.

Test of Finished Pavement

Deflection.—Table 8 is a sample sheet of results of Benkelman beam deflections made on each project at the test station locations. The deflections were obtained soon after completion of each section of pavement. The test results indicate no apparent correlation, at this time, between deflection and voids, density, moisture, etc., of the subgrade and the various components of the pavements when constructed.

TABLE 8

SAMPLE SHEET, DEFLECTION TEST RESULTS[a]
Project F-297(7), Sta. 545+30 to 308+75
Construction Completed and Traffic Started, October 1961

Station	Test Date	Deflection (in.)				Avg.
		Northbound		Southbound		
		OWP	IWP	IWP	OWP	
540+50	Nov 1961	0.018	0.013	0.013	0.015	0.015
	May 1962	0.034	0.037	0.039	0.042	0.038
	Nov 1962	0.038	0.032	0.035	0.035	0.035
	May 1963	0.037	0.042	0.049	0.037	0.041
527+00	Nov 1961	0.023	0.021	0.018	0.017	0.020
	May 1962	0.044	0.042	0.030	0.044	0.040
	Nov 1962	0.040	0.032	0.040	0.032	0.036
	May 1963	0.049	0.039	0.042	0.039	0.042
508+06	Nov 1961	0.021	0.018	0.022	0.022	0.021
	May 1962	0.034	0.025	0.025	0.037	0.030
	Nov 1962	0.038	0.021	0.027	0.035	0.030
	May 1963	0.037	0.022	0.022	0.039	0.030
487+00	Nov 1961	0.015	0.010	0.013	0.010	0.012
	May 1962	0.037	0.017	0.012	0.027	0.023
	Nov 1962	0.027	0.013	0.016	0.019	0.019
	May 1963	0.037	0.017	0.017	0.034	0.026
467+00	Nov 1961	0.021	0.021	0.021	0.022	0.021
	May 1962	0.042	0.044	0.030	0.049	0.041
	Nov 1962	0.032	0.032	0.032	0.032	0.032
	May 1963	0.052	0.052	0.037	0.044	0.046
447+00	Nov 1961	0.019	0.021	0.013	0.018	0.018
	May 1962	0.030	0.025	0.015	0.047	0.029
	Nov 1962	0.030	0.030	0.019	0.024	0.026
	May 1963	0.042	0.039	0.032	0.030	0.036
427+00	Nov 1961	0.017	0.018	0.015	0.017	0.017
	May 1962	0.030	0.030	0.022	0.054	0.034
	Nov 1962	0.024	0.016	0.021	0.032	0.023
	May 1963	0.027	0.020	0.025	0.044	0.029
407+00	Nov 1961	0.015	0.018	0.018	0.023	0.019
	May 1962	0.032	0.020	0.042	0.059	0.038
	Nov 1962	0.026	0.023	0.031	0.026	0.027
	May 1963	0.030	0.030	0.034	0.052	0.037

[a]See footnote, Table 2.

Permeability. —In December 1962, permeability tests were made of the asphalt surfacing by applying water through a meter to a 4-in. diameter area under a falling head, the maximum head being about 15 in. The test showed that the newer pavement, which had been completed about 2 months before, had a permeability ranging from 1.4 to 12.8 ml/min and averaged 5.7. The older pavement, subjected to more traffic, showed a range in permeability from 0 to 7.1 ml/min and averaged 0.2. All figures represent an extremely low permeability.

Rutting. —Rutting measurements were made at the test locations in the spring of 1963.

Station 545+30 to 308+75, Worth Co., Completed in October 1961. —Rutting was negligible in the inner wheelpaths. In the outer wheelpath of the northbound lane, ruts varied from 0 to 0.3 in. and averaged 0.1 in. In the southbound outer wheelpaths, ruts ranged from 0 to 0.6 in. and averaged 0.25 in.

Station 308+75 to 223+16, Worth Co., Completed in November 1961. —No appreciable ruts were measured at the test stations except in the outer wheelpath of the northbound lane. Here they ranged from 0 to 0.4 in. and averaged 0.25 in.

Station 223+16 to 0+00, Worth Co., Completed in June 1962. —An average rut of about 0.1 in. was measured in the outer wheelpath of the northbound lane. Other wheelpaths showed negligible rutting.

Station 545+30 to 677+14, Worth Co. and Station 105+60 to 234+77, Gentry Co., Completed in October 1962. —Rutting was negligible on both projects.

Roughness. —All the projects showed approximately 68 in./mi roughness soon after completion. This is about 7 in. less than the average roughness for newly constructed flexible pavements in Missouri. There has been very little change in roughness except for the first constructed sections from Station 545+30 to Station 223+16 in Worth Co., which are also showing greater deflection and more rutting. Through these sections the roughness has increased on the average about 11 in./mi.

Investigations of Distress

In June 1962 cracking, rutting and failure occurred in the outer wheelpath of one section. An investigation was made by excavating a trench into the subgrade through the outer 7 ft of the roadway. Results of the examination and measurements indicated that the asphaltic concrete was 0.25 in. thinner in the failed part than in adjacent unfailed areas, which was probably caused by abnormal truck traffic.

The stone base in the failed area showed greater density than that generally found during construction, which may indicate some base densification under traffic. Also there was a faint indication of slippage between the two courses of stone. Moisture content of the base was below optimum. The lime subbase was hard and showed no deformation. The moisture content was about optimum and the density was about the same as during construction. The density in the subgrade was 99.5 percent of standard and the moisture content was 2 percent above optimum. It was concluded that rutting,

which led to cracking, was caused by densification of the asphaltic concrete and stone base and possibly some lateral movement of the base. The area was repaired by the contractor.

A limited investigation of localized areas was made in April 1963. Both distressed and nearby unaffected areas were examined, tested and measured. But in March and April 1964 the latest, most thorough and productive investigation was completed on a portion of Project F-297(7). Fourteen cores were removed from the asphalt surface for laboratory tests. Thickness of the various layers was determined; density and moisture measurements were made in base, subbase and subgrade; cores were removed from the lime subbase; and field and laboratory tests were performed on samples from all layers at all locations.

The investigation was restricted largely to a section about 2 mi long where, although no failures had occurred, considerable rutting and alligator cracking had developed. For the most part this was the section which was finished late in 1961, barricaded against all traffic (including snow removal equipment), and subjected to extremely harsh treatment early the following spring when used by loaded northbound base trucks and southbound sand trucks. Failed areas of asphalt which quickly developed in the northbound lane were removed and rebuilt by the contractor in June 1962.

In January 1964, alligator cracking averaged about 5 sq ft/1,000 sq ft of 24-ft roadway, and was confined mainly to the inner wheelpath of the southbound lane. By April spot sealing had covered most of the cracked areas and the increase in cracking, if any, could not be determined. No significant distress was noted at this time in the areas which had been repaired by the contractor.

Consideration of test results, thickness measurements and general conditions at the 14 test locations indicated the following at the time of this survey and on this section.

1. Surface—Asphaltic concrete thickness varied from 2⅝ to 3¼ in., with no apparent correlation between distress and thickness. Asphalt recovered from cores 2 and 2½ yr old showed penetration anticipated, respectively, in 5- and 10-yr old asphalt. Ductility at 45 F of less than 8 is considered by some to indicate asphaltic concrete of low serviceability.

2. Base—Thickness of rolled stone base averaged 7.2 in., compared to the 8 in. specified. Density in the wheelpaths had increased 2.5 percent since construction, which could account for about 0.2-in. base settlement in those areas. In general, the prime had functioned satisfactorily, adhering to base and asphalt surfacing. Cracked areas occurred over both thin and thick base. Averages showed that moisture at this time was 1 percent below optimum and 1.6 percent less than water of saturation. Only one P. I. equaled the specified maximum of 6; none exceeded it. Gradation was somewhat finer than during production and CBR testing indicated very satisfactory base material.

3. Subbase—Lime subbase averaged 1 in. more than the 5-in. designed thickness. At various locations a thin clay-like layer was noted at the top of the course. This was believed to have resulted from repeated applications of water while curing, and was not considered to be detrimental to the performance of the subbase. Average P. I. was 5.8, which would indicate some rebound since construction if credence can be placed in the accuracy of the test. The subbase moisture, density and shrinkage limit had changed little or not at all since construction. Unconfined strengths of cores submerged before breaking averaged 190 psi and indicated a gain of about 14 percent in slightly more than 2 yr. No relation could be seen between strength and distress. Average pH of about 10.4 indicated good lime distribution. There was no indication, at any site, of displacement or deformation of the lime subbase.

4. Subgrade—Average subgrade moisture was 3.8 percent above that during construction and 4.8 percent over optimum, which might be expected to reduce the strength and stability of overlying layers. No relationship could be established between higher moisture and distressed areas.

5. Cracks—Some longitudinal shoulder cracks in both cut and fill were believed to have been caused by desiccation, shrinkage and some fill subsidence. Transverse cracks which extended half to full width, and in some places into the shoulders, are

believed to have originated in the asphalt and progressed downward. They may also have been associated with shrinkage.

6. Evaluation—The upheavals, depressions and general edge distortion which commonly signal subgrade failure were in no place evident here, nor were there signs of incipient failure in the lime subbase. By process of elimination, then, substantiated by increased base density and rutting of the asphalt in the wheelpaths, and some earlier noted signs of lateral movement between the two base courses, it was decided that the source of the distress lay in the rolled stone base. Densification of the 8-in. base under traffic demanded corresponding flexure of the surface course. Since the asphaltic concrete was apparently of inferior quality it cracked instead of deflecting.

Summary

1. The Missouri Highway Department cannot lay claim to having made any discoveries in connection with soil-lime stabilization as a result of this experimental project.

2. Many data were accumulated to support the already well-known fact that hydrated lime, when intimately mixed with the glacial clay of northern Missouri, greatly improves the engineering properties of the native soil.

3. The P. I. and group index, the volume change and the minus No. 200 content all decreased and the compressive strength increased. Optimum moisture increased and the maximum density decreased with addition of lime. The consequent bulking meant, on this project, that about 4.33 in. of compacted soil expanded with addition of 6 percent lime to give 5 in. of compacted mixture.

4. Intimately mixing hydrated lime with wet clay tends to reduce moisture in the clay. The agglomeration of clay-size particles improves the drainability. In combination, the two develop an unyielding, relatively dry working platform soon after a rain, which permits lime processing to continue in what would otherwise be adverse conditions. The importance of this quick-drying feature was emphasized on these projects when frequent rains softened the untreated shoulders and, in some locations, made it impossible for trucks to cross. Thus, the base-laying operations were sometimes delayed in certain areas when they might have been in progress.

5. Construction of lime-soil subbase was relatively trouble-free. By its quick reaction with the clay, lime aided in pulverization, and density was obtained after only a few passes of a pneumatic roller.

6. Tests made on the first project seemed to substantiate the idea that a rotting period was not essential. Only a few stretches had to be remixed to meet pulverization requirements. Yet the quality of subbase was, to all appearances, as good as that on the remixed sections, and the average density was higher. Therefore, since some initial cementing action might be destroyed by remixing, and since the lime-clay reaction is of diffusive nature, rotting and remixing might not be necessary. Pulverization requirements could then be less restrictive than were applied to the second and third projects on this job.

7. Where remixing was unnecessary, average daily progress was increased by about 40 percent.

8. It was impossible to maintain a uniform moisture content while water-curing the subbase. Because wetting-drying cycles may cause damage it is believed that application of a bituminous seal would be a better curing method.

9. During the short time this road has been in service no evidence has been adduced to indicate that a crushed stone base with P. I. slightly higher than 6 is detrimental.

10. Certain sections of this job were detrimentally affected by abnormal construction traffic at an early age. As should have been anticipated, the results of such treatment were quickly reflected by the development of more and greater distress in those areas than where the roadway carried normal traffic.

11. Even including the sections mentioned above, the lime subbase portion of these three projects is giving satisfactory performance and showing only minor distress in limited areas.

12. One peculiarity, being studied with interest but not yet correlated with any feature of design or construction, is the cracking which occurs to a greater extent in the inner wheelpaths than in the outer paths, as is usual.

13. No correlation has yet been established between deflections and the moisture or density of any increment of the structure. Nor does there appear to be any relation between the age of subbase when asphaltic concrete was placed and the performance of the road.

14. Missouri's previous experience with relatively thin asphaltic concrete surface on crushed stone bases has not always been all that could be desired. Data thus far accumulated in Worth and Gentry Co. lead to the belief that lime subbase below an 11-in. depth, as here, adequately replaces the same thickness of crushed rock. In such an area, where aggregates are scarce, replacement may effect approximately a 20 percent reduction in cost at no sacrifice in service.

15. It is believed that such distress as has already appeared can be accounted for by consideration of several factors touched on in this report and that, in general, this lime subbase experimental project has been satisfactory in every respect.

TEST SECTIONS

(Project F-524(2), Gentry Co.)

As shown in Figure 1, 16 test sections were located on U. S. 169, Gentry Co., beginning at the Worth Co. line and extending south for 2 mi. This segment of the report describes the various features of design, construction and performance of these sections.

Purpose

This complete factorial experiment was proposed to try to determine the effect on a 3-in. asphaltic concrete surface course of various combinations of subgrade, three types of subbase and two thicknesses of rolled stone base.

Scope

Two factors of the design are under study. The first factor (thickness of base) contains two levels: (a) 8-in. Type I (rolled stone) aggregate base; and (b) 4-in. Type I (rolled stone) aggregate base. The second factor (type of subbase) contains four levels: (a) 5 in. of lime-stabilized subbase; (b) 5 in. of phosphoric acid-stabilized subbase; (c) 5 in. of Type I aggregate subbase; and (d) no subbase. The subbase sections are $\frac{1}{8}$ mi in length and are replicated under each thickness of base (Table 9).

Topography

The topography is very similar to that described for the preceding projects—heavily rolling country with deeply eroded gullies and narrow valleys.

TABLE 9

SUBBASE SECTIONS

Test Section	Station Limits	A. C. (in.)	Base (in.)	Subbase
1	0+00-6+60	3	8	5-in. lime
2	6+60-13+20	3	8	5-in. acid
3	13+20-19+80	3	8	5-in. lime
4	19+80-26+40	3	8	5-in. Type I
5	26+40-33+00	3	8	None
6	33+00-39+60	3	8	5-in. Type I
7	39+60-46+20	3	8	None
8	46+20-52+80	3	8	5-in. acid
9	52+80-59+40	3	4	5-in. lime
10	59+40-66+00	3	4	5-in. Type I
11	66+00-72+60	3	4	5-in. lime
12	72+60-79+20	3	4	None
13	79+20-85+80	3	4	None
14	85+80-92+40	3	4	5-in. acid
15	92+40-99+00	3	4	5-in. acid
16	99+00-105+60	3	4	5-in. Type I

Soils

The glacial Shelby was identified as the soil comprising the subgrade throughout these 2 mi. However, borrow soil was used in three of the acid sections.

General

Construction and testing of the lime sections developed little information not already determined on adjacent projects with similar subbase. The same was true concerning rolled stone base material, whether used as base or subbase. Experiences with these two types have been reported in detail in the previous section, and, to avoid repetition, additional data will not be included here. Therefore,

TABLE 10

SOIL PROPERTIES

Property	Shelby	Grundy	Till
L. L.	48	48	55
P. L.	16	17	18
P. I.	32	31	37
Shrinkage limit	12	10	12
Vol. change at FME	51	53	42
Minus No. 200	77	82	82
Silt	30	31	30
Clay	41	42	45
Colloids	30	31	37
Max. density	113	109	108
Opt. moisture	16	17	18
Psi at O. M. - M. D.	51	45	44
Free swell (%)	75	80	85
Class.	A-7-6(17)	A-7-6(18)	A-7-6(19)
Texture	Clay	Clay	Clay

this part of the report will consist largely of information having to do with phosphoric acid stabilization.

Soil Replacement

Calcareous concentrations occur frequently in the Shelby, and tests of the soil with hydrochloric acid showed considerable effervescence on Sections 2, 14 and 15. To eliminate the need for addition of excess acid to neutralize carbonates, noneffervescing soil was imported for subbase material on those three sections. Therefore, phosphoric acid was mixed on Section 2 with B horizon Grundy, on Sections 14 and 15 with nonreactive B and C Shelby, and on Section 8 with glacial till in place. Test results given in Table 10 indicate the general similarity of the three soils.

Preliminary Testing—Acid

Investigation of the stabilizing effect of phosphoric acid was started by Missouri in 1957, and much cooperative laboratory research work was completed. Preliminary work was confined to mixtures of acid in various percentages with highly plastic Putnam soil with extremely high volume change. A test section on Putnam soil was built in Callaway Co. by maintenance personnel under the supervision of the Missouri State Highway Commission and in cooperation with research personnel of the Monsanto Chemical Co. This pilot section was too short to provide much definite and reliable information, but it did focus attention on some of the factors that would probably influence construction of longer projects.

The combined laboratory and field work indicated that the potential of phosphoric acid as a stabilizing agent for heavy clays would justify a larger scale experiment. A location for such a project, with suitable soil and traffic and fitting the scheduled work program, was selected in conjunction with the extensive study of lime stabilization reported in the previous part of this paper.

Representative samples of the predominant Shelby soil were tested in the laboratory, with results as shown in Figures 3 and 4. Based on these data and reinforced by information from other sources, the recommendation was made that 2 to 3 percent acid be added for optimum results.

Design

Because soils and traffic were very nearly the same on these 2 mi as on adjacent sections, the same total thickness of 16 in. was recommended for the test sections. The half section in Figure 5 illustrates the various combinations of subgrade, subbase and base. The sections were chosen by random selection and each combination was duplicated.

Specifications.

1. Subgrade—top 18 in. to be compacted to minimum of 95 percent of T 99 standard (Missouri modification, four layers if P. I. is 25 or more).
2. Subbase-Lime—trenched 30 ft wide, 5 in. thick, 6 percent lime (dry soil basis), compacted to 95 percent or more of T 99 standard; pulverization requirement after mixing lime with soil, 100 percent passing 1-in. sieve and at least 60 percent passing No. 4 sieve.
3. Subbase-Acid—same trenched section, 2 to 3 percent of 50 percent acid (diluted from 75 percent), compacted to at least 100 percent of T 99 standard, determined by Missouri's procedure (four layers); pulverization, 100 percent passing the 1-in. sieve and a minimum of 80 percent passing the No. 4 before acid is added.

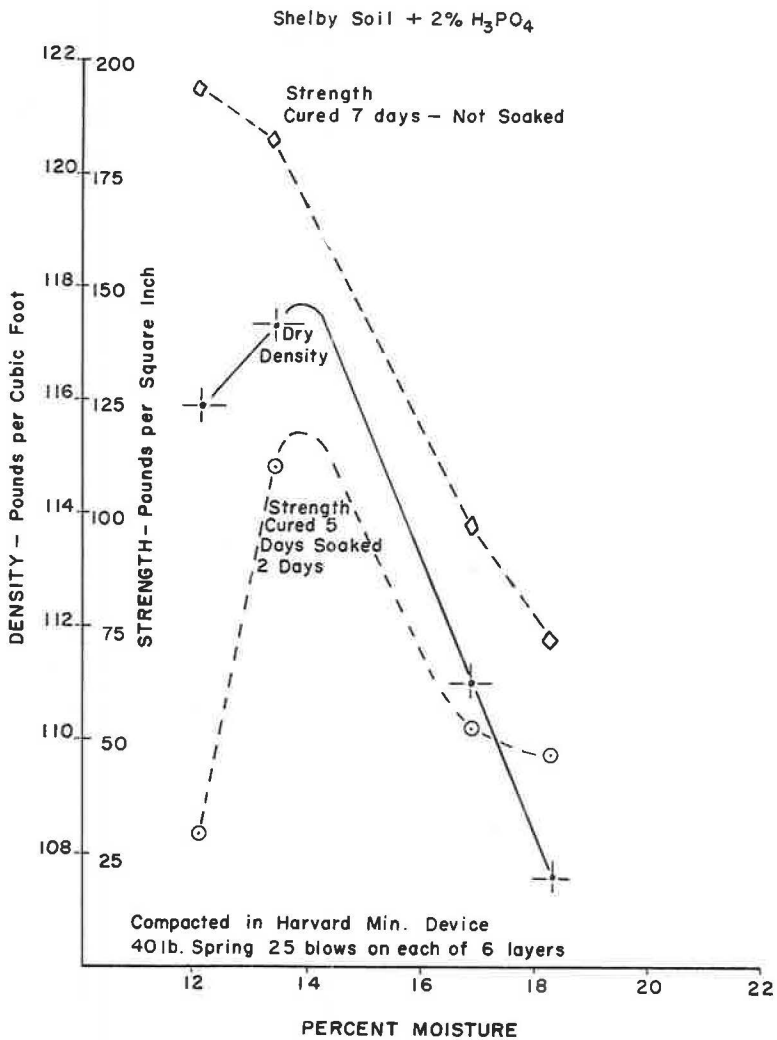


Figure 3. Shelby soil + 2 percent H_3PO_4.

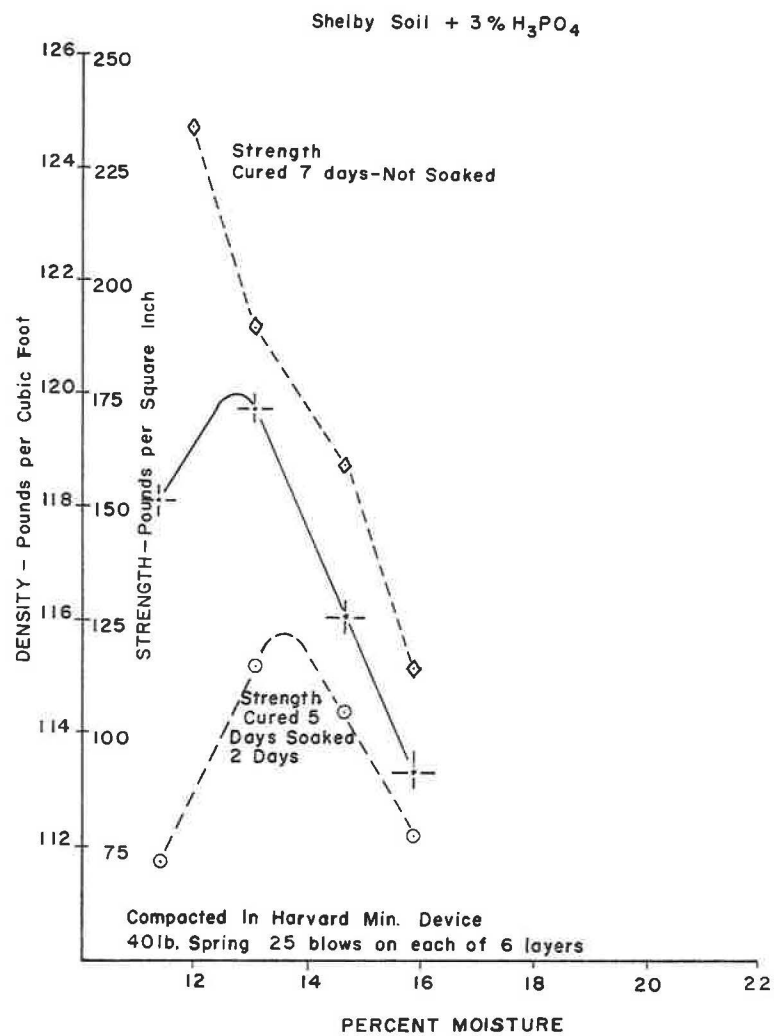
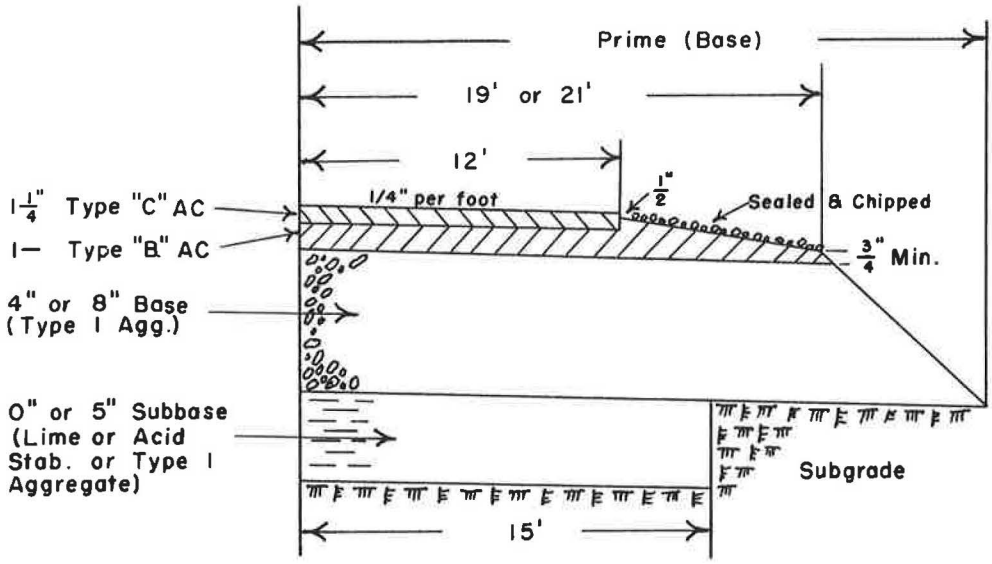


Figure 4. Shelby soil + 3 percent H_3PO_4.



Asphaltic Concrete:

1. Type "B" AC 19' wide unless guard rail is present.

Shoulders:

1. Sealed with 0.35 gal./sq. yd. of RC-4 cutback asphalt.
2. Chipped with 35 lb./sq. yd. of crushed stone.

Figure 5. Half section on tangent test sections.

4. Rolled Stone Base—dimensions shown in Figure 5; compaction to be minimum of 95 percent of standard determined on minus No. 4 fraction by T 99, and computed for total sample, P. I. to be 6 or less.

Construction—Acid Subbase

Pulverization. —It was not possible to meet the specification that 80 percent of the soil pass the No. 4 sieve before incorporating acid. Low speed passes with the P & H were supplemented by repeated processing with tiller, disc, blade and tamper. The final product only emphasized the fact that best laboratory results may be obtained by procedures that are not necessarily practicable when transferred to field operations.

In this instance, high humidity and frequent rains usually prevented proper pulverization. Continuous manipulation during the rare drying periods caused the formation of clay balls with dry hulls and wet cores. These were windrowed and sheepsfooted in one section before being processed by a tiller. At the end of this day's work, 73 to 75 percent passed the No. 4 sieve. And that night it rained again.

Prior to processing Section 14, 1 percent of the 75 percent undiluted acid was added, primarily to facilitate pulverization and secondarily to neutralize carbonates. Every care had been taken in selecting borrow soil, and the acid test and been applied frequently to avoid carbonates. Oddly enough, after repeated pulverizing passes, it was almost impossible to find a noneffervescing soil sample.

Most effective pulverization was accomplished on other sections when the moisture was reduced to near the shrinkage limit. This section could not be dried to that extent, and the average of several tests showed moisture at about optimum. As far as is known, the extra acid did neutralize most of the carbonates.

TABLE 11

RESULTS OF PULVERIZATION TESTS

Section	Station	Percent Passing Sieve			Moisture (%)	O. M.
		1 In.	½ In.	No. 4		
2	8+50	99	62	19	12.1	14.4
2	11+00	100	93	60	9.2	14.4
8	48+00	97	65	21	14.7	20.0
8	51+00	97	75	41	11.1	20.0
8	52+50	97	70	27	13.1	20.0
14	86+00[a]	95	58	19	16.3	17.6
14	87+50[a]	94	53	18	17.1	17.6
14	89+50[a]	97	45	15	18.0	17.6
15	94+50	100	84	50	11.4	18.4
15	97+00	100	87	51	11.6	18.4

[a]One percent extra acid added.

Results of pulverization tests made just before addition of the acid are given in Table 11. Little definite information can be extracted from this tabulation. Although not substantiated by all tests, the trend is toward more pulverization the further below optimum the moisture is reduced. In clay soil such a result would be expected. Wet weather, high humidity, and soil moisture cause greatest trouble in the pulverization process, but slight changes in soil characteristics may also contribute. Since neither factor is readily controllable, it seems probable that only a less stringent pulverization specification would have made phosphoric acid stabilization practicable in the conditions that prevailed in Gentry Co. in 1962.

Pulverization costs. —The cost per square yard for pulverizing varied widely for the four acid sections, indicating to some degree the time and effort devoted to this portion of the processing. All sections were 660 ft long, 30 ft wide and 5 in. thick, but costs varied from $0.39 on Section 8 to $0.80 on Section 15.

Acid. —During preliminary laboratory moisture-density tests the mixtures quickly stiffened, indicating that the acid did not act as a lubricant. The aqueous solution of phosphoric acid, as received on the project, was at 75 percent strength. To obtain more widespread and uniform dispersion, it was diluted to 50 percent and a corrosion inhibitor was added for equipment protection.

Dilute acid was used in three of the test sections with drier subgrade, in a quantity corresponding to 2.8 percent of the dry soil weight. Since subgrade moisture on Section 14 was near optimum, undiluted acid was used in it. This was also supposed to be at the rate of 2.8 percent. The P & H, however, was moved from a lime section where it had been set to cut about 4.5 in. to allow for the bulking effect. Through an oversight, no change was made in the depth of cut, which on acid was supposed to be 5 in. Consequently 75 percent acid was applied to Section 14 at the rate of 4.3 percent.

Compaction. —Densification was first attempted by use of a sheepsfoot tamper but the acid-soil mixture showed an unusual affinity for steel. At times the mixture even picked up in a solid mass and exposed the subgrade. This peculiarity was again demonstrated when a 12-ton steel wheel roller was used and the adhering mixture left a severely potholed, rough subbase surface.

Reduction of the moisture by aeration was most effective in reducing the tendency to adhere. This, and the time element between mixing and compacting was quite critical, since the mixture stiffened in a relatively short time. Therefore, when proper moisture prevented clinging, it was impossible to obtain specified density in the viscid mixture. An initial knockdown pass by a pneumatic roller densified the mixture enough that

sheepsfoot tampers penetrated only part of the layer of treated material. A relatively light steel wheel vibratory compactor was used, but densified only the top half or less of the 5-in. depth. The mixture also stuck to it. Since a supercompactor was not available, the choice of roller type was eliminated and all compacting was done with the pneumatic.

If enough moisture was added to bring the mixture to optimum, the clods were wet only on their surface, the finer material sponged up most of the water, and the pneumatic bogged down. In this latest crisis the moisture was reduced and the acid subbase was completed. In three of the four sections, the moisture was from 1.6 to 4.5 percent below optimum. The other section was 2 percent over. In none of the four was specified compaction obtained; the deficiency ranged from 3 to 13 percent.

The consensus was that everything within reason had been tried. Yet, despite all efforts, none of the four acid sections had been built to comply with specifications for pulverization, for moisture, or for density.

Finishing.—Finishing the acid subbase involved trimming to proper cross-section, sprinkling and pneumatic rolling, all within 2 hr after addition of acid. This was not always accomplished within the allotted time.

Sealing and Curing.—An MC-0 prime of 0.3 gal/sq yd was applied to the subbase, and the primed sections were barricaded for 7 days. The prime abraded severely when the sections were opened to traffic. This loosened material was probably the mixture which was trimmed and shifted during finishing operations and which did not bond to the subbase when rerolled.

During the curing period a peculiar condition developed, noticeable to some extent in all acid sections. Bulges occurring somewhat at random were, in some stretches, aligned into narrow, more or less continuous, wandering lines with a striking resemblance to mole runs. Two theories were advanced to explain their formation: (a) that CO_2 gas generated by acid reaction with carbonates had built up enough pressure to heave the primed surface, and (b) that concentrations of clay balls had swelled after absorbing moisture.

Regardless of the undetermined cause of the localized bulging, tests in the section showed a general loss in density, from 87 percent when constructed to 77 percent 8 days later. In the same period, moisture tests showed an increase from 1.6 percent below optimum to 5.5 percent above. Meanwhile, the mixture had changed in appearance from a cohesive, somewhat glutinous material to a granular type which was easily dug with a pocket knife. Density was restored to the original 87 percent by pneumatic rolling, but no improvement in cohesion was noticed. This was the condition of nearly all acid subbase when it was covered with base.

Tests.—Moisture-density tests were made in each of the four sections on typical soil and on acid-soil mixtures sampled immediately after passage of the P & H. In preliminary laboratory tests, addition of acid had increased the weight and decreased optimum moisture. Field tests followed this trend on but two of the four sections.

Tests for particle size, plasticity and shrinkage limit were made on native soil and fresh acid-soil mixtures. Similar tests were made on Shelby tube samples taken when the subbase was 280 days old and impossible to core, but still soft enough to take a push sample. Comparisons revealed that the engineering properties of the soil were improved by addition of phosphoric acid. However, the P. I. at 280 days was considerably higher than when constructed, in both the 2.8 and 4.3 percent areas.

Unconfined compressive strength was determined at various ages on 2- by 4-in. specimens molded on Sections 2, 8 and 15 from the fresh field mix and cured in the humid room at Jefferson City. Results are shown in Table 12.

A Shelby tube sample from the subbase (age 280 days) at Station 97+00 had a

TABLE 12

COMPRESSIVE STRENGTH OF 2- BY 4-IN. SPECIMENS MOLDED FROM FIELD MIX (2.8 PERCENT ACID)

Station	Test Sect.	$CaCO_3$ Equiv. (%)	Strength (psi)			
			7-Day	14-Day	28-Day	6-Mo
8+00	2	—	0	0	0	11
8+50	2	—	0	2	0	15
11+00	2	—	0	1	8	8
48+00	8	—	13	23	19	44
51+00	8	—	0	2	4	15
52+50	8	—	0	3	10	—
92+50	15	0.41	5	6	8	12
94+50	15	0.96	0	0	2	8
97+00	15	0.73	6	10	8	22

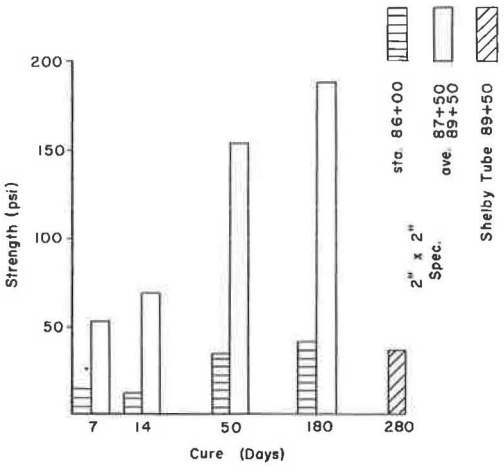


Figure 6. Soaked strength, 4.3 percent acid section.

strength of 4.5 psi. The molded specimens and Shelby tube samples from these sections showed little resistance to moisture, either falling apart during immersion or having very low strengths.

Soaked strengths of 2- by 2-in. specimens molded from Section 14 field mix are shown in Figure 6. Results from Station 86 are shown individually because the calcium carbonate equivalent in this area was 1.66 percent, and the extra 1 percent acid probably did not neutralize all the carbonates. Specimens from this station withstood immersion, but strength has shown only a slight increase with age. The specimen cured 180 days cracked in storage.

Specimens molded from two other stations have shown an increase in strength with age. The following tabulation shows the values in Figure 6 when converted to the average gain in soaked strength per day for each increment of curing age.

Days	Gain in Strength (psi per day)
0-7	7.8
7-14	2.1
14-50	2.4
50-180	0.2

Although the reason is not known, it is conjectured that the better soaked strengths of the specimens molded from this section, when compared to Sections 2, 8, and 15 may be due to the higher acid content or the higher than optimum moisture content.

The soaked CBR of the acid-soil field mix (Section 14) was 74.4, compared to 3.7 for the untreated soil. Swell of the acid-soil mix was negligible, but the untreated soil swelled 3.6 percent.

Lime Subbase

As would be expected, subbase construction procedures and results on the four lime test sections were much the same as reported in the first part of this report. The maximum density was reduced and optimum moisture increased. Bulking was calculated at about 13 percent and the P & H machine set to cut 4.5 in. The average compacted subbase measurement was a little more than 5 in., and the thickness control was generally excellent.

Test results for plasticity, gradation and shrinkage limit made on newly processed mixture showed the same general improvement in the clay soil as reported earlier. Several cores taken from the lime sections at the age of 280 days were similarly tested and, with one exception, showed continuing improvement with age. The average L. L., P. L. and P. I. all increased from that determined during construction.

Unconfined compressive strengths on field-molded moist-cured soaked specimens of various ages indicated that the rate of gain in strength was most rapid during the first 7 days. This coincides with the curing period during which no traffic was allowed to use the sections. Strengths showed an increasing trend from 75± psi at 7 days to more than 400 psi at 6 months. Soaked 280-day cores, constructed and cured under

different conditions than in the laboratory, showed almost exactly the same strength as the 28-day molded specimens. The CBR of the mixture was almost 20 times that of the untreated soil, and the swell was reduced almost to zero by addition of lime. The volume of the cores was calculated from measurements and, on the average, showed an increase in density from that at time of construction. The somewhat lower moisture content in the cores may have caused shrinkage and densification.

Rolled Stone Base and Subbase

Four of the 660-ft test sections had a 5-in. rolled stone subbase placed in one lift, and compacted to at least 95 percent. A 4-in. lift of the same base material was placed over two of the sections, while 8 in. in two lifts was constructed on the other two. The only construction difficulties or delays were caused by variations in moisture. When near optimum, pneumatic rolling produced an average density of 99.5 percent.

Base gradation was similar to that on the other projects, with a number of samples exceeding the specified maximum passing the No. 4. The P. I. at construction averaged 6, the maximum specified. In March 1963, shoulder samples averaged 8 and cores drilled from the base in May averaged 9. The latter figure may have been influenced by the inclusion of drill cuttings in the samples. Tests showed the minus No. 200 content to have been increased 7 percent by use of the drill. In both the March and May investigations, base moisture was below optimum.

Asphaltic Concrete

The two courses of asphaltic concrete were built to comply with standard Missouri specifications. About 6 wk after opening to traffic, permeability tests by two methods indicated low permeability.

Performance

Deflection. —Deflections on the test sections were measured by Benkelman beam in the fall of 1962; the spring, summer and fall of 1963; and twice in the first half of 1964. Since spring readings are most critical in Missouri, remarks will be confined largely to the results obtained in those periods. The first two sets of readings showed that the average deflection is higher in spring than in fall, regardless of design, and the weaker the design the greater is the increase. Neither would be unexpected.

Two 660-ft sections were underdesigned, with 3 in. of asphaltic concrete on a 4-in. rolled stone base on untreated subgrade. From deflection on those sections, it was evident that as the crushed stone thickness was increased to 8 in. on subgrade and to 4 and 8 in. on the 5-in. stone subbase, the percentage reduction in deflection usually increased. This would also be expected, since the stronger structure should deflect less.

Apparent incongruities have been noted, as for example in the Fall 1962 readings. Deflections indicated that the lime subbase was no better than the untreated subgrade, and the phosphoric acid subbase was actually worse than native soil. More recent measurements tend to refute the early readings, leading to the belief that subbase strength is increasing as cementation occurs with greater age. This may be especially applicable to the acid sections, which lacked density and showed an unbound, granulated structure when primed and covered with base. In May 1963, push samples of the acid subbase were quite firm and well bonded together.

The 1964 spring readings indicated that only one 3-4-0 section had an average deflection of more than 0.04 in., although maximums exceeding that figure were measured on all of the 16 sections. Signs of distress were evident on both 3-4-0 sections, perhaps portending more extensive signs of failure in the near future.

Equivalencies of Stone vs Lime. —Based on deflections alone, it was indicated that 5-in. lime subbase was equivalent to about 1 in. of rolled stone base in the sections of 3-in. asphaltic concrete—8-in. stone—5-in. lime design. But where the stone base thickness was reduced to 4 in., the 5 in. of lime subbase was equivalent to a little more than 2 in. of stone.

Rutting-Roughness. —Rutting has been negligible, and roughometer measurements have produced little reliable information. In November 1962 and in February and May 1963, roughness measurements on all the thicker sections were invariably greater in the southbound lane; there is no known explanation for this phenomenon.

Summary

The Missouri Highway Department's experience during the construction, testing and subsequent investigations of the 16 test sections justifies the following statements.

1. With some exceptions, evidence developed in the few years since construction of these projects is insufficient to warrant drawing hard and fast conclusions or making definite comparisons between different designs.
2. Various bits of information have been accumulated whose significance, if any, is at present obscure. They may, however, be relevant at some future time when all available data are assembled for a final picture of the total performance of these test sections.
3. Specifications for the acid mixture may have been unrealistic. Laboratory tests provided data for writing special provisions for the use of this experimental material. Requirements for soil pulverization and for moisture and density of the mixture could not be fulfilled despite every possible effort.
4. The designs, based on the results of preliminary laboratory research, called for a 5-in. compacted subbase, using 6 percent lime or 2.8 percent phosphoric acid, both figured on the dry weight of the soil. For comparison, a third type of subbase was also included, using 5 in. of crushed stone. The lime content was closely controlled by the lime spreader. The acid content had to be adjusted through a spraybar on the P & H stabilizer. Periodic clogging of a few of the nozzles, neutralization of some acid by unanticipated calcareous material in the soil, and forgetting to change the cutting depth caused some unknown variations in the acid quantity, which probably ranged from about 2 to 4.3 percent instead of the desired quantity. Thus, rather unintentionally, the experimental aspects of the job were somewhat amplified.
5. Both hydrated lime and phosphoric acid reacted with the glacial clay soil to improve its engineering properties, as measured by laboratory tests for plasticity, gradation and shrinkage. Examination of test results, however, reveals that neither 2.8 nor 4.3 percent acid has yet improved the soil as much as did 6 percent lime.
6. Drying and pulverizing the subgrade in the acid sections was a prolonged operation, and the results of all the work were often destroyed by rain. No such difficulties were encountered in the lime sections which quickly dried, permitting work to proceed. Rain may even have accelerated the reaction between soil and lime and expedited the breakdown of clayey clods. Therefore, what was a distinct disadvantage in the acid sections may even have been of some benefit in the lime sections.
7. Specified density was easily obtained in lime subbase with a minimum of rolling but could not be obtained in any of the four acid sections. Requirements for the latter were somewhat higher than for lime—100 percent of T 99 in four layers as compared to 95 percent by the standard method.
8. Moisture was specified to be within 2 percent of optimum on lime and 1 percent on acid subbase during compaction. In general, the lime sections complied, but none of the acid sections were within the limits.
9. Acid sections were supposed to be finished within 2 hr after the addition of acid, and lime sections within 72 hr. The tendency of the acid-soil mixture to adhere to steel compactors eliminated from use all but the pneumatic roller. A moisture content near optimum facilitated compaction but tended to bog down the roller. Lowering the moisture caused the mixture to stiffen, and proper density could not be attained. The 2-hr time limit was frequently exceeded; the 72-hr limit never was.
10. When the utmost effort failed to produce required density in the acid subbase, it was sprinkled, finished by pneumatic rolling, primed and barricaded for 7 days. During the curing period fluffing developed, and the subbase lost density and became loose and granular. The primed mixture bulged erratically but was restored to original density by rerolling.

11. Tests on Shelby tube samples have since indicated the development of a more tightly bound, cohesive material with passage of time. Strength test results are somewhat ambiguous, but little gain has been shown. A marked rebound in plasticity has been noted. Cores could not be drilled from the acid subbase, but 9-mo push samples and 6-mo molded specimens, in general, exhibited little compressive strength. Cylinders molded from material with the highest acid content did show a trend toward greater strength with increase in age. Most specimens from mixtures with lesser acid content disintegrated when submerged in preparation for compressive strength testing.

12. No test section has been subjected to abnormally heavy loads, although current volume is reported equal to that anticipated, at the time of design, for 1980. It is believed that repairs necessary on projects to the north are readily explainable by a peculiar, nonrecurring combination of several factors—pavement age, truck traffic, time of year, and lack of maintenance.

13. Construction of the overlying base and asphaltic concrete courses was normal on all sections. It has not been shown, to date, that base P. I. somewhat in excess of 6 is detrimental to the performance of this flexible pavement.

14. Maximum spring deflections have recently exceeded 0.04 in. on all of the 16 sections, regardless of design. The average, however, has been above 0.04 only on one of the two weakest sections. A full-width transverse crack and a depressed alligator-cracked area presently give evidence of weakness and may be a sign of more extensive distress to be expected. All sections are being closely observed for future developments, and further investigations will be made whenever necessary.

Summation

Since the last portion of this experimental job was completed and opened to traffic less than 2 yr ago (as of July 1964 writing), this is only an interim report to describe the preliminary research, design, construction and current performance of the three projects, including the 16 test sections. Final conclusions as to comparative performance of different materials and designs will be possible only in the future, when answers will undoubtedly replace conjectures about indications.

On these Missouri projects, both lime and phosphoric acid improved the engineering properties of the clay subgrade, reducing plasticity, volume change and percentage of fines, while increasing strength and stability.

When mixed with glacial Shelby clay, lime almost immediately reacts with the soil and improves its characteristics. When compacted, the mixture provides a firm subbase which can effectively replace some thickness of crushed stone base. In areas where stone is scarce and, therefore, expensive, or where satisfactory material must be imported, lime-stabilized subbase can also be more economical than an equal thickness of crushed stone.

Bid prices per ton on these projects were $30 for bulk lime and $125 for 75 percent phosphoric acid. By dry soil weight, 6 percent lime was used, whereas the acid percentage varied from about 2 to more than 4. A total of 79.11 tons of acid was used in four test sections of 660 ft each. On the remaining 16.87 mi, 3041.6 tons of lime were used.

On F-524(2), Gentry Co., the only project on which both lime and phosphoric acid were used, costs to the state were approximately $23,000/mi for acid and $8,900/mi for the lime. Costs to the contractor would probably present a still more unbalanced comparison, in view of the great amount of time and effort invested in fruitless attempts to meet the specifications on acid subbase for pulverization, moisture content and density.

If any future phosphoric acid stabilization should be contemplated in Missouri under specifications similar to those in effect on these sections, it is probable that manipulation costs will be higher than on this job and the cost differential between lime and acid stabilization will be even greater. Based on Missouri's experience and present knowledge, the gap in costs might be somewhat narrowed by development of more efficient pulverizing equipment or a phosphoric acid additive to facilitate pulverization, or by writing less stringent specifications.

Cost data, in combination with other factors, indicate that lime stabilization may be most efficiently used in subbase construction in areas where clay soils respond well to such treatment and satisfactory rock deposits are scarce. Acid might compete only if the cost differential can be considerably reduced and equivalent or greater benefits to the soil can be demonstrated.

At present the definite statement can be made that for more than 2 yr, all pavement composed of a 5-in. lime-soil subbase, 8-in. rolled stone base and 3-in. asphaltic concrete surface has given satisfactory service. This is true also of 15 of the 16 test sections.

Early deflection measurements indicated that, inch for inch, crushed stone was better as subbase material than either the lime-soil or acid-soil mixtures. Recent measurements indicated somewhat of a reverse trend, perhaps reflecting a gain in strength as acid and lime sections age. Future investigations will without doubt provide information to permit accurate and final evaluation of the various designs.

Based on Missouri's experience with acid stabilization, it is believed that, before any such future work, extensive laboratory testing should determine whether such rigorous specifications as were in effect here are necessary or desirable. Additional investigation should be made into the long-time effects and permanence of any improvements in soil properties induced by the acid.

On these projects the preponderance of evidence points to the superiority of lime over phosphoric acid as a stabilizing agent. Lime subbase was easy to construct to specified requirements, dried quickly after the frequent rains and did not delay progress, provided a stable foundation and excellent working platform for building the base, and has shown a consistent gain in strength. None of this can be said about acid sections.

Missouri's experience on these projects leads to the following recommendations which are believed to deserve serious consideration in the design of any future similar subbase and base for flexible pavement.

1. Extend the stabilized subbase through the shoulders instead of building a trenched section with relatively impermeable shoulders.
2. Eliminate the necessity for a rotting period and remixing pass of equipment by requiring only that 100 percent of the lime-soil mixture pass the 1½-in. sieve, thereby greatly expediting progress. Tests on one of these projects where such a specification was in effect led to the belief that the lime's stabilizing effect suffered no impairment by such a procedure.
3. Specify a bituminous prime for lime subbase instead of curing by periodic applications of water.
4. Liberalize plasticity requirements in rolled stone base.
5. Increase rolled stone base density requirements.
6. If soil and traffic conditions demand a base thickness of 8 in. or more, treat at least the top 4-in. lift with cement or bituminous material.
7. Specify asphaltic concrete thickness greater than 3 in.
8. Write and enforce a specification that will make certain that work on asphaltic concrete stops in time for pavement to be opened to traffic and receive normal maintenance during the winter. Associated with this requirement would be one exacting strict control of heavy construction traffic over new pavement.

It is believed that incorporation of some or all of these suggestions into the design will result in a much higher type road. Costs will undoubtedly be higher, but the service life and performance record of the roadway would be upgraded sufficiently to overbalance the additional costs.

REFERENCE

1. Jones, W. G. Lime-Stabilized Test Sections on Route 51, Perry County, Missouri. Highway Research Board Bull. 193, pp. 32-39, 1958.